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OPERATION AND CONTROL OF WATER TREATMENT PROCESSES

CHARLES R. COX

Prepared in consultation with
twenty-four specialists in
various countries

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WORLD HEALTH ORGANIZATION

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No. 49

OPERATION AND CONTROL OF WATER TREATMENT PROCESSES

OPERATION AND CONTROL OF WATER TREATMENT PROCESSES

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Prepared in consultation with
twenty-four specialists in
various countries

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CONTENTS

PREFACE

The problem of supplying adequate amounts of safe water for distribution to the public does not end with the construction of waterworks. Many waterborne epidemics of disease have been traced to defects in the operating procedures used in water treatment plants, and it is self-evident that the level of operating skill and practice is of great public health significance. It is in recognition of the need for a practical and authoritative text or guide on water treatment which will encourage better plant operation that this monograph is published by WHO.

The text deals specifically with the operation and control of water treatment processes designed to produce water for domestic purposes ; it does not deal with the treatment of water for industrial use.

Recognition is given to the limits of effectiveness of the various processes, which impose a restriction on the quality and character of raw waters capable of being treated in an effective, reliable and reasonably economical manner. Water treatment is the third aspect of the cycle comprising (a) sewage and waste treatment, (b) self-purification in streams and in ground waters, and (c) water treatment. Too much reliance cannot be placed on any one of these aspects of the cycle from sewage to potable water and then to waste water ; therefore water-supply officials should work closely with water-pollution-control authorities, to the end that the natural water resources of concern to them are not polluted to such a degree that self-purification and water treatment processes cannot produce potable water in a reliable and economical manner.

This monograph brings together a body of recent information on water treatment and control for the benefit of officials, including public health officials, who are concerned with maintaining a suitable system of controls. Its primary purpose, however, is to serve the needs of plant superintendents, operators and laboratory personnel, and the style of presentation has been adapted to those groups. Sufficient theory is presented to provide a basic understanding of the processes described, but the main emphasis is on practical operating problems. Designers will find much of value to them, but the approach used has been not to present standards of design, but rather to reveal how to get the most from a plant already built.

In addition to sources and references given in the text, the author has made extensive use of his " Water Supply Control ", " Guides to the Design of Water Treatment Plants " and " Water Analytical Procedures ", which were prepared for limited use and are not suitable for general distribution.

A preliminary text was drafted by Mr Cox in 1961 and was issued as a mimeographed document. This text was circulated to 36 experts in different parts of the world for their comments and suggestions. The replies received have greatly assisted in amending and revising the original text. A list of reviewers will be found on page 379. WHO appreciates the contributions received from these reviewers and expresses its sincere thanks to them.

INTRODUCTION

Water treatment involves physical, chemical and biological changes that transform raw waters into potable waters. The treatment processes used in any specific instance must depend on the quality and nature of the raw water. Thus many well waters are of satisfactory physical and chemical quality and require treatment only by chlorination to protect against known or potential bacteriological pollution. At the other extreme are those mineralized raw waters subject to serious pollution by sewage and industrial wastes, which require all the resources of water treatment to yield an attractive water of safe sanitary quality.

The quality of raw water supplies is affected by a multiplicity of factors, and each source of supply must be evaluated on an individual basis. It is therefore essential that basic criteria for a water supply should be established before a purification scheme is developed; only then can a system be designed that will guarantee the safety of the users.

Water treatment processes may be simple in nature, like sedimentation, or may involve very complex physicochemical changes, as with coagulation. These complexities may be further obscured by factors not disclosed by water analysis. Thus the alum dose required to coagulate a given water having a certain turbidity, colour, pH and alkalinity differs from the alum dose required for another water having the same apparent qualities. Fortunately these difficulties can be minimized by the trial-and-error procedure of the " jar test ", in which the effective coagulant dose is determined by observing results secured in jars or beakers with graduated doses of alum. Even then the most effective dose with a litre of water under the conditions surrounding the jar test will differ from that most effective on a plant scale. Experience will disclose, however, the adjustments needed to translate the results of jar tests into plant practice.

Any review of laboratory manuals, such as *International Standards for Drinking-Water* (World Health Organization) or *Standard Methods for the Examination of Water and Waste Water* (American Public Health Association), will disclose the complexity of the laboratory procedures, their limitations and the extent of the facilities involved. Those who supervise plants frequently will not have such complete facilities, or the training and experience necessary for their proper use; they must, perforce, use the more simple tests. These simplified tests are referred to in this monograph where specific processes are discussed, and the section on Laboratory Procedures describes each test for the guidance of those to whom only limited equipment is available. The selection of the simplified tests was based on the factors mentioned in the introduction to that section. It should be noted that other laboratory equipment may be used, following the specific directions for its use; but it would not be practicable in this monograph to describe all the different equipment and kits available in various countries. Specifically developed equipment of this nature often provides for convenient and simplified water analysis suitable for use in the field and for plant control.

It should be realized that the compromises involved in using simplified tests are intended to facilitate the laboratory control of water treatment plants and hence serve as tools of the operators. More technical and official control of the quality of water for public consumption should be provided by health departments having jurisdiction, as these agencies normally have the more elaborate facilities necessary, staffed by chemists, biologists and engineers. This independent appraisal of the quality of delivered water should not be interpreted as placing the operation and control of treatment plants necessarily under health departments; the basic responsibility rests with the water department.

Automation has been extended to water treatment equipment and is being improved. At first glance automation might seem to be the solution to problems of operation when trained personnel are not available. Unfortunately, automatic devices usually are costly and complex and hence cannot be maintained by unskilled personnel; furthermore, in many countries such devices must be imported. Therefore the only automatic devices discussed in this monograph are chemical feeders and chlorinators controlled by metering devices; float-operated switches; and rate controllers.

Many of the references are to practice in the USA. An effort has been made, however, to include practice followed in other countries, as, for example, in the text on filter-washing practice.

The objective of water treatment being the production of potable water, it follows that the measures of accomplishment are the numerical norms of physical, chemical and bacteriological quality of potable water. Reference is made to *International Standards of Drinking Water* (World Health Organization) for guidance as to water quality; see also Chapter 9.

Units of measurement are given in US and metric values (Annex 3 gives conversion factors). *Note that measurements in gallons refer to the US unit unless the UK gallon is specified.*

CHAPTER 1

PROTECTION OF SOURCES OF WATER SUPPLY

Water treatment plant operators usually have little or no control over the degree of pollution of sources of water supply, except when catchment areas are sufficiently small to permit their ownership and their exclusive use for water-supply purposes or when wells and springs are utilized, and even then seepage from sources of chemical pollution may flow from areas beyond the well sites. Generally speaking, therefore, it is necessary to rely on official agencies for control of water pollution. This is of prime importance in conserving water resources for economical and effective treatment.

Water-pollution control

The current trend is to place the duty of protecting water resources under a government agency concerned with all uses of the resources, including that of public water supply. This permits the establishment of policies relating to (*a*) existing sources of pollution, (*b*) the degree of treatment of sewage and industrial wastes needed to restore polluted waters to a reasonable degree of purity or to prevent significant pollution, and (*c*) the existing or anticipated legitimate uses of waters for various purposes. Usually potable water supply is given first priority, based upon public necessity.

Water treatment plant operators should be familiar with the programme of such agencies so that they may participate as needed to ensure adequate protection of their specific source of supply. Special reference is made to the *Report of Technical Commission on Pollution of Surface Water* (International Water Supply Association, 1961) for basic principles and reasonable standards.

Ownership of catchment areas

The ownership of catchment areas is feasible only when their area is restricted and their value permits purchase. Such ownership makes possible complete control over the areas and the elimination of sources of pollution. Furthermore, soil erosion may be prevented by reforestation practice, or swamps may be drained to eliminate conditions favourable to the production of the colouring matter that is characteristic of waters flowing or seeping from areas where vegetation is undergoing decomposition. Drainage of swamps will also eliminate shallow pools in which conditions are favourable to the growth of algae which, if allowed to remain, will serve to seed the reservoirs receiving the run-off. This procedure, therefore, is the first step in the control of algae in reservoirs and the related efforts to prevent tastes and odours.

Inspection of catchment areas

Many catchment areas are too large to warrant their ownership by a water authority yet small enough to permit convenient inspection. Under such circumstances, organized inspection should be undertaken with the co-operation of health officials having jurisdiction. Briefly, the endeavour should be to locate on maps of catchment areas all occupied properties and the related actual and potential sources of pollution, including the more serious sources such as any sewer discharges into tributary streams, private sewage-disposal facilities and privies, and sources of pollution of animal origin such as manure piles and barns and stables. Such basic maps will establish the corrective action needed and will also facilitate estimation of the public health significance of bacteriological pollution found in raw waters flowing from the areas. In other words, the results of sanitary surveys will help to establish the significance of any given degree of pollution of the raw waters by coliform organisms, widely used as indicators of sewage pollution but also present in animal wastes and soil drainage.

Routine inspections of private properties on catchment areas require the co-operation of the property owners, unless health officials are authorized by law to make such inspections and to require corrective action to be taken by the property owners to abate sources of pollution. Tact and a friendly approach will usually establish an awareness in property owners of their duty to maintain sanitary conditions on their property, for the protection of their families and neighbours and also of the public served by the water supply involved. The pressing problem, of course, is the elimination of direct sewage discharges, overflowing cesspools or subsurface tile fields, and insanitary privies. Operators, especially of large plants, should develop routine programmes of catchment-area inspection and examination of samples collected at significant points. Bacteriological examination of such samples will disclose the degree of pollution as well as the most likely source of pollution. Such inspections must include industrial establishments, especially those producing liquid chemical wastes. Effective programmes for waste treatment should be an integral part of the work of water-pollution-control authorities.

The protection of wells and springs presents a more localized problem, which is complicated by uncertainty as to the direction of flow of the ground water and as to the effectiveness of natural filtration as surface waters seep into the soil and become a part of ground waters. Generally speaking, the area within 30 m of a well or spring should be owned so that it can be enclosed by a protective fence. Surface waters should be diverted from such areas by suitable ditches.

The most important protective measure for wells or springs, however, is the maintenance of their structures so as to exclude surface drainage and thus protect the ground water from direct pollution. The effectiveness of such maintenance will be influenced by the design of the structures. Reference is made to specifications for deep wells developed by the American Water Works Association and available as publication A100-58, and to literature on ground waters.

CHAPTER 2

CONTROL OF RESERVOIRS

The storage of water in impounding reservoirs creates favourable conditions for self-purification of the stored water and also certain conditions for unfavourable changes in water quality. The control of these conditions by the operator, as far as possible, is the first of the series of processes discussed in this monograph and the second step in water-supply operation affecting quality.

Principles

The benefits of storage are due to the effects of sedimentation, the influence of environmental conditions unfavourable to bacteria associated with prior sewage pollution of the water, and the chemical action of the oxygen dissolved in the water. Unfavourable conditions created by storage include those associated with growths of micro-organisms, as well as the reduction and solution of iron and manganese from flooded soils and rocks containing these minerals.

Reservoir stratification and its effects

It will be noted that both oxidation and reduction phenomena are involved. This apparent paradox is due to the stratification of the stored waters when surface waters are warmer and hence of lower specific gravity than the cooler, deeper waters. When the surface waters are cooled to a temperature of 4°C, however, and have the maximum specific gravity for water, they sink to the bottom, and the replaced warmer waters are in turn cooled to this temperature and sink. This continues until the whole volume of stored water has a temperature of 4°C. Any further cooling produces water of lower specific gravity, because of water's peculiar property of being densest at 4°C rather than at 0°C. If the air temperatures are below freezing, ice will be formed of still lower specific gravity, because of expansion during the freezing process, so that the ice floats on the surface.

After the spring overturn, the waters near the surface having a temperature of 4°C will gradually be warmed by the air and sunshine. This will continue until a surface layer 5 to 10 m deep has become warmed, while the lower waters remain cool. There is then a marked difference between the two layers, characterized by rapid fall in temperature at the zone of separation. The pronounced temperature gradient in this narrow zone is known as the " thermocline ", where temperature changes are 1°C or more per metre of increased depth. The depth of the level where the thermocline is most pronounced depends upon climatic conditions, the depth and size of the reservoir, and the opportunity for surface agitation and current formation due to wind action.

When water is stored in reservoirs located where water temperatures never reach a low value of 4°C, the surface waters become heated by the air and sunlight. These warmer waters become mixed with lower, cooler water through currents induced by wind action, but the zone below the influence of these currents comprises permanently stratified cooler waters. Under these circumstances the thermocline persists throughout the year, but its depth below the surface varies with the season and meteorological conditions. Briefly, then, the waters in deeper reservoirs present three layers or zones, namely, the top (circulation) zone, the middle (transition) zone and the zone of stagnation.

Because the warmer water near the surface in the circulation zone is subject to re-aeration and mixing by wind action, it contains dissolved oxygen. Any dissolved iron or manganese that may reach the surface waters with convection currents will therefore be oxidized in time, and the resulting insoluble ferric and manganic compounds will precipitate and settle back to the lower portion of the stored water, where the minerals dissolve. Generally speaking, therefore, the surface waters do not contain significant concentrations of iron and manganese except during the periods when the spring and autumn overturns occur.

The deeper water below the thermocline in the stagnation zone will contain little or no oxygen, because of its removal by the chemical processes of oxidation of organic matter. This in turn leads to the production of carbon dioxide (carbonic acid) and to the resulting lowering of the pH of the water, favourable to the solution of iron and manganese from the flooded soil or rock. Furthermore, the absence of dissolved oxygen leads to the reduction of sulfates to odour-producing sulfides, such as hydrogen sulfide and ferrous sulfide, and also to the decomposition of organic matter, with the production of colour and substances having tastes and odours. The quiescent, stagnant waters below the thermocline, then, are characterized by absence of oxygen; higher carbon dioxide content; lower pH; presence of soluble iron, manganese and sulfide when these minerals are a problem; and presence of taste- and odour-producing substances.

The surface waters above the thermocline are favourable for the growth of algae or microscopic plants that need sunlight for the photosynthesis of carbon dioxide. These liberate oxygen in the process of securing carbon from the carbon dioxide, thus raising the pH and creating conditions still more favourable to the oxidation of iron and manganese. Conversely, the floating micro-organisms of the animal kingdom, having no photosynthetic activity, are independent of sunlight and hence migrate to the level where their food is most abundant. Algae serve as food for the animal plankton in the upper waters and they also are present in lower levels. Other forms are present on or near the bottom of reservoirs, where decaying, dead algae and other organic matter settle and are available as food.

Effects of storage

The phenomena discussed above are more pronounced in new reservoirs, where flooded vegetation and the organic content of topsoil are subject to active decomposition. The area to be flooded by a new reservoir should therefore be cleared of vegetation, which should be burned or hauled off the area. In fact, the best practice is to remove the peatlike deposits of any swampy area to be flooded; but such soil stripping ordinarily is not practised over the whole reservoir site. New reservoirs gradually become stabilized in three to five years, and the more serious conditions are replaced by those which will prevail thereafter, depending on local influences.

Storage provides conditions favourable to the sedimentation of particles responsible for turbidity. Coarse suspended solids settle rapidly and finer material progressively more slowly. The remaining very fine material is in the colloidal state and will not settle unless conditions favour natural coagulation, or until the water is coagulated at a treatment plant. Reservoirs will also equalize highly turbid floodwaters through mixing with previously settled water in the reservoir. Turbidities under 30 units are usually produced by sedimentation in reservoirs, and even lower values prevail with more prolonged storage in large reservoirs and lakes.

The effect of storage on the colour of waters is more complex, because colloidal colouring matter does not settle unless conditions are favourable for its natural coagulation. Colouring matter in the upper portion of reservoirs is subject to the bleaching action of sunlight. There is a net reduction in the concentration of colouring matter, which, however, is less pronounced than the reduction of turbidity. Nevertheless, bleaching action may reduce the colour by 50% in one month.

Another favourable influence of storage is the effect on bacterial content. This is due mainly to the dilution of more highly polluted floodwaters by the previously settled waters in reservoirs, tending to equalize water quality, and to the influence of sedimentation. The effects are those associated with (*a*) sedimentation, (*b*) chemical changes, (*c*) environmental conditions unfavourable for bacteria associated with sewage pollution, such as water temperature and absence of food, (*d*) the germicidal action of sunlight in the upper 3 m (10 ft) of waters that are of low turbidity, and (*e*) the consumption of bacteria by predatory organisms such as the free-swimming microscopic protozoa. All this requires time, so the period of storage is the controlling factor in these self-purification processes.

It is for this reason that many large lakes yield clear waters with a low degree of bacteriological pollution. For example, the passage of Mississipi River water through a chain of lakes tributary to the water supply of the city of St Paul, Minn., leads to an average reduction of coliform organisms from 1680 per 100 ml to 22 per 100 ml, the storage period in the lakes varying from three weeks to three months.

In general, therefore, reservoirs provide a complex of processes tending to alter the character as well as the quantity of the water. The problem, then, is the control of those factors to permit an operator to use the beneficial effects and to minimize the adverse effects.

Multiple-level intakes

Because of the phenomena outlined above, new reservoirs should be equipped with multiple-level gates, to permit water to be withdrawn from the most favourable level. A problem of judgement is involved in the operation of such intakes, because although the upper waters are usually free of iron, manganese, sulfides and the colour and tastes and odours produced locally by decomposition and reduction processes, they have the maximum concentration of algae. The choice may be algae control to permit the use of surface waters, as discussed in Chapter 4, or iron and manganese removal processes to permit the use of the lower waters. Fortunate is the operator who can use the cooler, lower, usually algae-free waters without special treatment because of the absence of iron and manganese in the flooded soil and rock or in the inflowing waters. The use of special treatment to permit the use of the deeper waters containing iron and manganese is discussed in Chapter 12.

Many existing reservoirs have only a single intake located near the bottom, so that surface waters cannot be withdrawn even though this might be desirable. Construction details of existing dams at times may favour the installation of a second intake pipe leading from an elevation some distance below the spillway level but above the thermocline and connected to the supply main below the shutoff valve of the main intake.

Waste drains

These drains are usually intended for emergency use when a reservoir must be dewatered for repairs, etc. Frequently they are too small in diameter for rapid dewatering of a reservoir, and their valves may not have been operated for years, making routine use impracticable. The valves should be lubricated and maintained so that the waste drain may be used during periods of high stream flow to remove unsolidified sediment incidental to the precipitation of turbidity, and also to waste the bottom waters of inferior quality rather than to waste an equal volume of fresh surface waters over the spillway. This is especially significant when iron and manganese present a problem and when an abundance of water leads to periodic flow over spillways. The drains should be used at selected periods to prevent their being clogged by silt.

Shallow coves or bays

Special supervision should be exercised over those portions of reservoirs which are sufficiently shallow to constitute a breeding place for micro-organisms, which will then be carried into deeper portions of the reservoir unless they are controlled by the means discussed in Chapter 4. During periods of drought these shallow coves become exposed, and rapid growth of brush and weeds may occur before the water level rises. The practicability of removing and burning this vegetation should be considered in the light of local conditions. Floating debris should be avoided, to minimize mosquito breeding.

Aquatic weeds develop in water up to a depth of about 10 m, when water temperatures exceed about 17°C, especially when the water has a low turbidity and contains nitrogen and phosphorus from watersheds on which there is active farming and wastes are produced. Weed control is discussed in Chapter 4.

Silting of reservoirs

The sedimentation of silt in reservoirs removes most turbidity but leads to a reduction in effective capacity. The rate of silting depends on the degree of soil erosion and on meteorological factors; it is most rapid in dry areas with scanty vegetation and where seasonal storms produce rapid runoff. Silt has collected in certain reservoirs to the extent that their capacity has been reduced by over 7% per year, whereas under favourable conditions the reduction may be only 0.1% per year.

The prevention of serious soil erosion is the best solution, but operators may have no means of encouraging soil-conservation practices on private properties on their watershed. Silt removal is very expensive but may be practical in the coves of reservoirs where tributary streams enter and where the deposits are exposed during periods of low reservoir level.

An expedient commonly used to restore reservoir capacity is the use of flash-boards on the spillway crest to increase the elevation of spillways of reservoirs in which serious silting has occurred. For instance, flash-boards 1 m (3.3 ft) high will increase the reservoir capacity 10 000 m^3 per hectare

(1.08 million US gal per acre) of reservoir area at spillway elevation. An equivalent volume of silt would need to be removed to gain the same increase in capacity. Before a decision is taken, however, to install flashboards, a careful investigation should be made of the design of the dam structure and no action should be taken which would threaten its stability.

Recreational use of reservoirs

Reservoir sites are ordinarily owned by water authorities, so that complete control is possible. Nevertheless, experience has shown that the availability of fish in the waters and the frequently very attractive nature of reservoir sites encourages fishing and other recreational use of reservoirs and surrounding areas. No hard-and-fast rules can be laid down, but such activities should be prohibited at smaller reservoirs. When personnel are available to provide supervision, restricted recreational use of large reservoirs may be satisfactory, assuming, of course, that the water is subsequently treated. For instance, supervised use of reservoirs and tributary areas has been permitted at points not closer than 300 m to intakes. It is essential, however, that supervision be routine and effective, otherwise the waters will be exposed to direct pollution near intakes, where it is not counteracted by prolonged storage before the water enters the intakes.

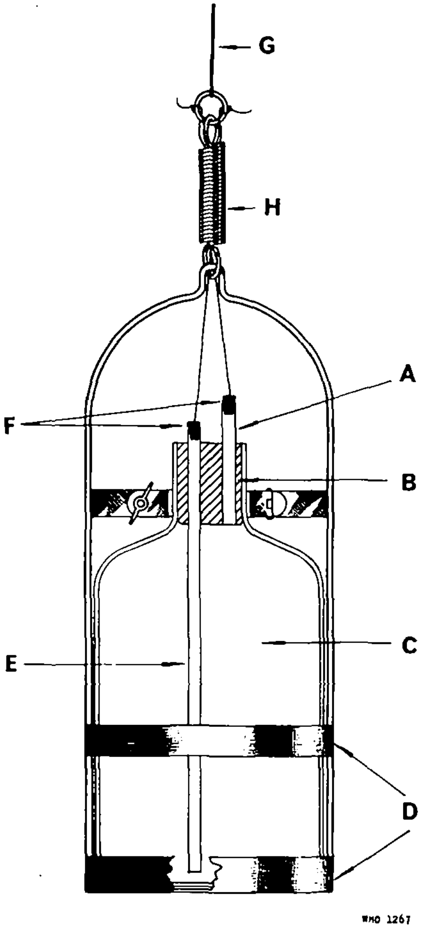
Laboratory control

In view of the above, the effective control of reservoirs by operators depends on knowledge of the conditions prevailing throughout the reservoir. The field work and laboratory procedures may not be economically justifiable for small reservoirs where the operating staff is limited. The degree of control, therefore, must be based upon the available facilities and the seriousness of the problems encountered. Emphasis, however, should be given to the routine use of the tests for pH, turbidity, chlorine demand and coliform organisms to furnish basic data as to the raw water being treated.

Deep-water sampling kits

Equipment for the collection of samples of water from reservoirs or lakes, at any selected depth, may be purchased from firms handling laboratory and scientific apparatus. The device shown in Fig. 1 may be made locally with brass strips to support a 1-litre, large-mouthed, plastic bottle with sufficient weight to sink the bottle. (The use of plastic bottles avoids breakage.) The rubber stopper is drilled with two ¼-inch holes, through which copper tubes are inserted as shown. The long tube admits water to the bottom of the bottle while air is exhausted through the short tube. The small rubber stoppers inserted in the tubes are easily removed when the lowering rope is given a quick pull to stretch the spring, thus admitting water when the bottle is at the depth where the sample is desired. The large stopper and the tubing are removed when the device is raised to the surface, and the regular stopper is placed in the bottle. A second bottle is then inserted in the device for the collection of a second sample.

FIG. 1
DEEP-WATER SAMPLING KIT



A. Copper tube
B. Rubber stopper
C. One-litre wide-mouthed plastic bottle
D. Brass strips riveted as needed
E. Copper tube
F. Rubber stoppers
G. Rope
H. Spring

Analytical determinations

The temperature of the sample should be determined and the water should be analysed for quality factors of local significance, such as turbidity, colour, pH, alkalinity, dissolved oxygen, iron and manganese; and microscopic examinations may be required to determine the content of mirco-organisms at selected points as a guide to their control. These tests will disclose the depth of the thermocline and indicate the zones of surface and quiescent bottom waters, for guidance in the use of multiple-gate intakes, as well as the wastage of stagnant waters through drains. In many instances, however, the tests may be restricted to the determination of temperature and the content of micro-organisms.

It should be emphasized that the character and quality of the water reaching treatment works is that of the water at the depth and location of the intake, and not at other points in the reservoir. Samples collected at some point at the surface of a reservoir will not be representative of deeper-lying waters. Therefore the use of the deep-water sampling kit is the only way in which the quality of water at any selected point in a reservoir may be determined. In the absence of such a kit, samples of raw water should be collected at the treatment plant, as representing the water actually being treated.

CHAPTER 3

AERATION

Purposes

Water devoid of dissolved air and hence oxygen has a " flat " taste, which is replaced by a " fresh " taste on aeration of the water. Taste- and odour-producing substances, such as hydrogen sulfide or some of the volatile substances liberated by algae growths or incidental to the decomposition of organic matter, are released from water by aeration. Aeration, however, is utilized primarily to provide oxygen from the atmosphere for the oxidation of iron and manganese and to liberate hydrogen sulfide and carbon dioxide from water; hence it serves as one aspect of corrosion prevention.

The popular notion that dissolved oxygen serves as a " purifier " of water by the " oxidation " of bacteria is erroneous, inasmuch as gaseous oxygen is not a disinfectant.

Principles

Gases are dissolved by or liberated from aerated water until equilibrium is reached between the content of each specific gas in the atmosphere and its content in the water. The concentration of each gas in aerated water at equilibrium is *directly proportional* to that portion of the atmospheric pressure on the water surface due to that specific gas. For instance, air under standard conditions and of average composition contains 20.9% oxygen by weight, so the pressure of the oxygen portion of the air is 20.9% of the atmospheric pressure of 760 mm Hg, or 159 mm. Carbon dioxide is present in the atmosphere, under the same conditions, at the low concentration of 0.03% by weight, so its pressure is only 0.23 mm Hg. These specific pressures are called " partial pressures ", as they represent that part of the total atmospheric pressure exerted by each gas.

The concentration of gases dissolved in water is *inversely proportional* to the temperature of the water, so that cold waters will dissolve a greater concentration of gases than warm waters—just the reverse of the solubility of most solids in water.

The diffusion of a gas into water is very slow unless large water surfaces are exposed, and unless the water is agitated so as to expose various portions of the water to the gases.

Gases, therefore, will be absorbed by water when their concentration in the water is below that at equilibrium. Conversely, gas will be released when the concentration of gases is higher than that at equilibrium. The gases of interest in water treatment are oxygen and carbon dioxide in the atmosphere, and these same gases plus hydrogen sulfide and volatile substances dissolved in the water.

The equilibrium values for oxygen in water are shown in Table 1 for various water temperatures noted, under standard conditions of dry atmosphere containing 20.9% oxygen at a barometric pressure of 760 mm Hg (sea level). For instance, under these conditions and at a temperature of 20°C, water will absorb oxygen up to 9.17 parts per million (p.p.m.). This is above the concentration of oxygen in stored waters below the thermocline, so aeration will increase the oxygen content of such waters. On the other hand, growth of algae may cause the oxygen content of water in the circulating zone of reservoirs to be above saturation, in which case aeration will reduce the content.

Since the normal atmosphere contains no hydrogen sulfide, aeration will remove this volatile substance, provided the atmosphere contacting the water is replaced by suitable air movement to eliminate the released hydrogen sulfide from the vicinity of the aerator. Taste- and odour-producing compounds which are only partially volatile will be removed only partially by aeration.

The situation is more complex when carbon dioxide is involved, because both physical and chemical equilibria independently are involved. *Physical equilibrium* between the very small concentration of carbon dioxide in the atmosphere (0.05% of the total weight of all atmospheric gases) and the carbon dioxide dissolved in distilled water of zero alkalinity would be about 0.5 p.p.m. Aeration of water, however, seldom reduces the carbon dioxide content below about 4.5 p.p.m., and a minimum of 10.0 p.p.m. is more common. For complex reasons discussed in Chapter 11, Corrosion Control, a content of 4.5 p.p.m. carbon dioxide is in *chemical equilibrium* with alkaline bicarbonates when the alkalinity is 100.0 p.p.m. That portion of the carbon dioxide content below 4.5 p.p.m. is non-corrosive at alkalinity 100.0 p.p.m. Aeration effective enough to secure a final concentration of 4.5 p.p.m. will therefore prevent corrosion only with waters having alkalinities in excess of 100.0 p.p.m. Less alkaline waters may have a high carbon dioxide content, and aeration will remove enough to reduce materially the dose of lime needed to reduce it further for full corrosion prevention.

The absorption of oxygen incidental to aeration introduces this more important cause of corrosion of piping, so the benefits of carbon dioxide

TABLE 1

SOLUBILITY OF OXYGEN IN FRESH WATER EXPOSED TO DRY ATMOSPHERE CONTAINING 20.9% OXYGEN AT BAROMETRIC PRESSURE OF 760 mm Hg [a]

Temperature of water, °C	Dissolved oxygen, p.p.m.
0	14.62
1	14.23
2	13.84
3	13.48
4	13.13
5	12.80
6	12.48
7	12.17
8	11.87
9	11.59
10	11.33
11	11.08
12	10.83
13	10.60
14	10.37
15	10.15
16	9.95
17	9.74
18	9.54
19	9.35
20	9.17
21	8.99
22	8.83
23	8.68
24	8.53
25	8.38
26	8.22
27	8.07
28	7.92
29	7.77
30	7.63

[a] The solubilities at lower atmospheric pressures vary approximately in proportion to the ratios of the lower pressures to the standard pressure of 760 mm Hg.

From Whipple (1948) p. 193 by permission of John Wiley & Sons, Inc., New York.

removal may be nullified by the absorption of oxygen, as often happens when well waters and waters from the deeper portions of reservoirs are aerated. This factor also is discussed in Chapter 11.

Many taste- and odour-producing substances are of low volatility, so aeration is not so effective in eliminating them as it is frequently assumed to be. Aeration alone, therefore, cannot completely control tastes and odours, but it serves to reduce the doses of chlorine or activated carbon needed for complete control.

Design of aerators

It is evident from these principles that aerators should be designed to provide the maximum area for the surface separating the atmosphere and the water, to provide agitation to aid diffusion, and to ensure free flow of the atmosphere to remove gases and volatile substances liberated from the water. Aeration practice has led to the development of two main types of aerators, those forming drops of water or thin sheets of water exposed to the atmosphere, and those forming small bubbles of air which rise in the water. Spray, waterfall and cascade aerators are of the first type, diffused-air aerators are of the second type.

Spray aerators

Possibly the most effective type of aerator is that using special nozzles to produce a fine spray, so that the total surface area of the drops in contact with the atmosphere is very large. The size of the drops depends upon the design of the nozzle and the water pressure; it may vary from a fine mist to large drops. Nozzles less than about 2.5 cm (1 in) in diameter are usually not used, as small nozzles tend to clog. Nozzles commonly used in spray aerators have diameters of 2.5-4 cm (about 1-1½ in) and discharge about 18/36 l/h (about 66-132 UK gal/min or 80-160 US gal/min) at a pressure of 0.68 atm (10 lb/in^2) but they operate at higher capacity under higher heads. Nozzle spacing is such as to provide about 0.03-0.09 m^2 of aerator area for each cubic metre per hour, or 60-180 ft^2 per million US gallons per day (m.g.d. US). As an example, using an area factor of 0.055 m^2/m^3/h (112 ft^2/m.g.d. UK; 93.5 ft^2/m.g.d. US) and a nozzle capacity of 25 m^3/h, an aerator 3.5×3.5 m, with 9 such nozzles, would be used for a flow of 225 m^3/h. (Corresponding values: For a flow of 1 m.g.d. UK, 9 nozzles discharging 77 gal/min each require an area of 10.5×10.5 ft; for a flow of 1 m.g.d. US, under the same conditions, an area of 9.66×9.66 ft is required.)

The shallow concrete basins in which the influent piping and attached nozzles are located should extend to a sufficient distance to prevent wind-blown spray from falling outside the basins, or else the basins should be fitted with louvers to contain the spray but permit circulation of the air.

In general, spray aerators remove about 75% of the content of carbon dioxide and increase the oxygen content to an extent above that required for iron removal.

Waterfall aerators

Aerators of this type consist of steps or trays arranged in a structure 1.2-3 m (4-10 ft) high, consisting of four to six steps or trays. The flow of water in thin sheets down such structures provides a large water-surface area and creates agitation. Improved performance is secured when the trays are perforated with small holes so as to secure a large number of small streams of water. The effective water surface and the period of contact of the water with the atmosphere may be increased by placing gravel or coke on the trays. This is especially desirable with aerators at iron-removal plants, as discussed in the separate chapter on that subject. Aeration beds may be enclosed in towers 2 m (about 6 ft) high, forced draft being furnished by low-head blowers, thus securing improved results with less loss of head. Experience has shown that waterfall-type aerators should have a planned area of 0.015-0.045 $m^2/m^3/h$ (30-90 ft^2/m.g.d. UK; 25-75 ft^2/m.g.d. US) capacity. Carbon dioxide removal is about 30% to 60%, and the addition of oxygen is adequate.

Cascade aerators

Turbulence may be secured when thin sheets of water flow down an incline on which numerous small baffles or obstructions are placed to divert the flow into many criss-crossing currents. Circulation of the air past the water surface may not be so effective as with the other types mentioned above. Carbon dioxide removal varies from about 25% to 50%.

Diffused-air aerators

This type of aerator consists essentially of a basin in which perforated pipes, porous tubes or plates, or patented impingement diffusers are used for the release of fine bubbles of compressed air which then rises through the water being aerated. The rate of rise of the bubbles of air is much slower than the rate of fall of drops of water in spray aerators, so the period available for the exchange of gases between water and air is correspondingly longer. To secure this effect, however, the air must be compressed to a value above the pressure of the water at the depth where the diffusers are located, and special tubes or plates with fine pores or diffusers must be used to secure the release of very small bubbles of air.

These diffusers may be located at the bottom of basins or suspended at a selected elevation. If they are located to one side of a basin, the rising air will induce circulation of the water and the retention of the bubbles of air in the water for longer periods. This circular current when combined with the horizontal flow through a basin will result in a spiral path for the water. This is facilitated when the basin width is not over 1.5 times its depth.

Usually, aerating basins provide a period of about 15 min, but periods of 8 to as much as 30 min have been used. The diffusers should be located 3-4 m (10-13 ft) below the water surface to ensure dispersion and diffusion before the bubbles reach the surface. Satisfactory results, however, have been secured with spiral-flow basins when diffusers are located at one-half the water depth, thus lowering the required air pressure. From 0.4-1.2 l of air per litre of water (0.06-0.18 ft^3/gal UK; 0.05-0.15 ft^3/gal US) is needed. This requires from about 0.3-1.3 kW of power per 100 m^3/h capacity (0.6-2.4 kW/m.g.d. UK; 0.5-2.0 kW/m.g.d. US).

Compressors with high capacity at low pressure should be used. The air to be compressed should be filtered so that dust does not clog the fine pores of the diffusers or openings in perforated piping, but this is not necessary when special diffusers are used.

This type of aeration is especially adapted for installation in existing plants, because the diffuser may be located in existing channels or basins having suitable depths and detention periods. For instance, plants with no aeration or inadequate flocculating facilities may be altered to provide both by installing a stilling wall in the inlet end of sedimentation basins, so as to isolate a portion of the basins to provide a detention period of 15 to 20 min, and by installing diffusers, connecting piping and air compressors. Furthermore, this type of aerator is especially adapted to those climates where water temperatures are below freezing in winter, because a large surface area of the water is not exposed to the cold air temperatures, as when spray or waterfall aerators are used.

Operation and maintenance

The problems of operation with various aerators are simple. They consist largely in maintaining the equipment in a clean condition and in so controlling the flow of water that maximum water surface and agitation are provided. For instance, when the volume of water being aerated is lower than the designed capacity of the unit, the reduced flow should be controlled through proportionately fewer nozzles to maintain the pressure required for fine-spray production. The cascade and waterfall types of aerators, on the other hand, will operate satisfactorily with reduced flow, because the distance of fall and the degree of agitation does not change with flow. In fact, these otherwise less efficient aerators provide a larger area separating the water and atmosphere, per volume of water aerated, when the flow is reduced.

Operators may be faced with the supervision of poorly designed aerators. This presents an opportunity for ingenuity in modifying the structure. For instance, gravel or coke may be placed on tray aerators to increase the effective area of the surface of the flowing water, and also to increase the period of flow in contact with the atmosphere. Other possibilities would be the placing of a shallow layer of coarse gravel on the horizontal steps of such aerators or the provision of altered or additional obstructions to the flow down cascade-type aerators to increase turbulence. Changes in compressor capacity or in location and area of diffusers may also be made to correct observed deficiencies of diffused-air aerators.

Slime and algae growths on the surfaces of aerators will require periodic treatment with copper sulfate, or copper sulfate and lime, to kill the growth, thus facilitating cleaning. For instance, a solution of copper sulfate may be prepared by adding 10 g per litre of water in a suitable container. To this is added 10 g hydrated lime. This mixture of a solution and a suspension, when applied to surfaces containing growths, will persist on the surfaces and provide prolonged action, even after the aerator is placed in operation. Copper sulfate will corrode galvanized iron, so the strong solution should be prepared in a wooden bucket or earthenware container. Usually at least 10 litres of solution is needed. The procedure is to by-pass the aerator and then to apply the solution with a large whitewash brush or its equivalent.

The porous plates or tubes used with diffusion aerators may become partly clogged, either from dust in the compressed air or from the collection of sediment on the outside surfaces during periods when the aerator is shut down. Such partial clogging is disclosed by an increase in the required air pressure or by unevenness in the distribution of the air. If clogging is due to oil from compressors, fused earthenware diffusers should be removed and placed in a kiln or furnace to burn the oil. If diffusers are of a type that cannot be heated, the underside of plates or the inside of tubes may be cleaned with detergent applied with a brush. If clogging is due to rust from an air pipe, acid treatment is needed. Removable diffusers may be placed in 50% nitric acid, prepared by adding concentrated acid to the same volume of water. If, however, the diffusers are grouted in place by cement mortar, an " inhibited " acid must be used. This is prepared by adding 1 g sodium dichromate to each 50 ml concentrated sulfuric acid. Use about 300 ml per diffuser. Strong acids must be used with great care. Always *add the acid to the water* when it is diluted, *never add water to the acid.* Rubber gloves and aprons should be worn, and care must be taken not to splash the acid on the face, even when goggles are worn.

Clogging of diffusers should be minimized by (*a*) maintaining air filters in effective operation; (*b*) not overlubricating air compressors or blowers; and (*c*) maintaining air pressure on the diffusers when the compressors are shut down, the valve on the compressed-air piping being closed to prevent backflow of water into the diffusers.

The aeration of filtered water exposes it to dust and air-borne bacteria, so postchlorination should *follow* any final aeration.

Laboratory control

The effectiveness of aeration is measured by the increase in the concentration of oxygen or by the decrease in the concentration of hydrogen sulfide or carbon dioxide, and also by the removal of taste- and odour-producing compounds. Generally speaking, the increase in oxygen content to oxidize iron and manganese should be sufficient to provide 50% saturation, as calculated from the data in Table 1. Hydrogen sulfide should be eliminated completely. The residual content of carbon dioxide should be no greater than 5-10 p.p.m. with very efficient aeration, but higher values must be expected in many instances. Finally, aeration will reduce tastes and odours incidental to algae growths, but the reduction may not be over 50%. An even lower degree of removal may be expected with many tastes and odours of low volatility, such as those resulting when organic matter is subject to decomposition.

These changes in water quality must be appraised in the light of other treatment processes, such as corrosion prevention or the use of activated carbon or chlorination to remove or modify tastes and odours. It is especially necessary to appraise the value of the removal of carbon dioxide from well waters devoid of oxygen, versus the deleterious influence of the dissolved oxygen introduced by the aeration for carbon dioxide removal. Reference is again made to Chapter 11 for guides in the control of corrosiveness of water.

CHAPTER 4

TASTE AND ODOUR CONTROL

Purpose

Consumers object to potable waters having noticeable tastes or odours, both because they are objectionable and because many tastes and odours are interpreted as evidence of pollution or of unwholesomeness of a water. For instance, the odour of hydrogen sulfite is associated with rotten eggs, and the fishy taste due to the protozoan, *Synura*, is associated with dead fish in a reservoir. Therefore the control of tastes and odours should not be neglected, otherwise the consumers will lose confidence in the quality of an actually safe supply and use any available substitute as long as it is attractive. Thus tastes and odours in potable waters can present a public health problem. The only real solution is the delivery of potable water that is both safe and attractive.

Principles

The control of tastes and odours involves a number of unrelated procedures and processes which must be considered separately. They may be characterized as either preventive or corrective measures. The first category includes the prevention of the growth of taste- and odour-producing organisms and the prevention of chemical changes that detract from the quality of a water. Corrective treatment includes aeration, the use of chlorine as an oxidizing agent and the use of activated carbon to adsorb taste- and odour-producing substances. These procedures may be used in combination and also as part of a broader application of treatment processes. For instance, prechlorination may be used, as discussed later, to improve disinfection and coagulation and at the same time oxidize tastes and odours and suppress the biochemical decomposition of organic matter in the sludge deposited in basins. In general, there is no more complicated group of processes than those involved in taste and odour control; yet they must be balanced and manipulated by trial and error, as the only ultimate guide is the subjective attitude towards the taste and odour of the treated water.

Even this appraisal is not final, because a satisfactory plant effluent may deteriorate in distribution systems or become less satisfactory, as when odourless cold water is heated and hidden odours are liberated. It is not surprising, then, that books have been written about this subject, or even about a single one of the processes involved, such as the control of micro-organisms.

Sources of tastes and odours

The natural sources of tastes and odours are: (*a*) the growth of micro-organisms in waters that release taste- and odour-producing substances; (*b*) the decomposition products of these same organisms when killed, or of leaves, grasses and aquatic vegetation in reservoirs; (*c*) the growth of slime organisms, moulds and fungi; and (*d*) the reduction of sulfates to sulfides in water devoid of oxygen. To these may be added the taste- and odour-producing compounds in sewage and especially in industrial wastes discharged into natural waters.

Preventive procedures

Water-pollution control

Increasing attention is being given to the treatment of sewage and industrial wastes to restore polluted water resources or to protect those waters not yet significantly polluted. The objective of water-pollution-control agencies is to maintain water resources so that normal and economical treatment of water supplies taken therefrom will produce safe and wholesome potable waters. This may require drastic action, such as limiting of phenolic wastes so that the receiving waters will not have a content over 0.005 p.p.m., because these compounds react with chlorine and produce chlorophenolic compounds having an objectionable taste even in such small concentrations. This residual concentration of phenols may be destroyed by free-residual chlorine or by chlorine-dioxide treatment. Paper-mill wastes produce a high chlorine demand and also lead to " woody " tastes, due to the action of bacteria on the waste material. These tastes are not removed by aeration or easily oxidized by free chlorine, so they must be removed by activated carbon treatment.

Water-pollution-control programmes based on broad principles of protecting water resources for " best usage " will, in general, ensure raw waters that can, by available processes, be freed of residual tastes and odours except those resulting from a few compounds such as phenols whose content is not gauged by such general measures of pollution as biological oxygen demand (B O D) or coliform index. Therefore specific limits have to be placed on industrial wastes that present this problem.

Destruction or removal by treatment is not feasible for brine wastes from oil wells, to give another instance; the wastes must be pumped back into underground brine reservoirs so as to prevent a salty taste noticed by many when the chloride ion content of potable water is over 200 p.p.m.

Prevention of chlorinous tastes

Prior to the advent of free-residual chlorination, chlorine-ammonia treatment was used to secure disinfection without chlorinous tastes, since adequate disinfection with doses of chlorine alone often caused objectionable chlorinous tastes. The chloramines produced by chlorine-ammonia treatment are poor disinfectants, however, so taste control was accompanied by less effective bactericidal action.

The chlorinous tastes produced by marginal doses are not actually tastes of chlorine itself but rather of chloro-organic compounds formed by the reaction between chlorine and organic matter (see Fig. 11, page 139). Therefore the remedy is not to *reduce* the chlorine dose but to *increase* it so that these compounds will be destroyed or altered, leaving free-residual chlorine.

The attitude of the individual to the taste and odour of free-residual chlorine depends on his subjective reactions. It is generally held that they are much less objectionable than chlorinous tastes. In any case, free-residual chlorine will produce a different degree of tastes and odours in waters of different characteristics, being somewhat more noticeable as an odour in soft waters of lower pH value. No hard-and-fast guides are available, except that residuals of 0.3 to 0.5 p.p.m. may usually be present in delivered water without serious consumer complaints.

These concentrations of free-residual chlorine are more than adequate for postchlorination, provided prechlorination has been controlled to achieve its several purposes. In the last analysis, the operator should determine the bacteriological quality of the delivered water (see Chapter 9, page 177) in the light of concentrations of free-residual chlorine and the taste and odour of the water—that is to say, safe sanitary quality must always be paramount, but operators should endeavour to produce safe waters that do not have an objectionable taste of free-residual chlorine. Emphasis should be placed on effective prechlorination and rapid sand filtration so that postchlorination may be effective with more moderate doses and residuals.

Suppression of growths of organisms

The basic sciences underlying water treatment processes include biology. The most effective and economical control of organisms in water requires some knowledge of water biology and water bacteriology. This broad subject cannot be even summarized in this monograph, but references are given to those books which are more directly applicable to waterworks practice.

Organisms of significance to water treatment plant operators are as follows:

Plants	*Animals*
bacteria	protozoa
fungi	rotifers
algae	sponges
rooted aquatic vegetation	worms
weeds and brush	crustaceans
	insect larvae

Laboratory procedures used in routine water bacteriology and in the counting and identification of free-floating algae, protozoa and a few higher forms of animal life are described in the Supplement, pages 339-373.

Bacteria

Many bacteria in water have no relationship to sanitary-quality criteria. They may be beneficial, like the organisms that modify organic matter in sewage, or they may be objectionable but still of no sanitary significance, like the bacteria that reduce sulfate to odour-producing sulfides in the absence of oxygen. This reducing action in distribution systems may be controlled either by aeration or by maintaining residual chlorine throughout the systems, as already discussed. These procedures must be supplemented by the periodic flushing of dead-end mains, so that oxygen or chlorine is present in all mains to the extent needed to prevent bacterial action. The bacteria are dormant in water below about 10°C, so control procedures may be seasonal in temperate climates. Any failure to control these organisms will be disclosed by very objectionable odours and black deposits (iron sulfide) appearing in water drawn from mains where circulation is limited.

Bacteria known as actinomycetes may grow in water mains, especially when surface water treated only by chlorination is involved. The slime coating of pipes is indicative of the presence of such growths. These may cause odours of decomposition or an odour characteristic of wood affected by dry rot. The remedy is the maintenance of free-residual chlorine throughout distribution systems. This form of bacteria may be destroyed in reservoirs by the constant application of copper sulfate to the entering water in doses of 0.1-0.8 p.p.m.

Another group of bacteria are called " iron bacteria " because they utilize iron and hence are associated with waters containing this element. The important members of this group are *Crenothrix* and *Leptothrix*. They may develop in gelatinous masses around the screens of shallow wells yielding iron-bearing water, or in distribution systems where the content of oxygen is limited. Their growth will create objectionable odours and lead to the presence of precipitated iron in the water. The growth of these organisms should be prevented by iron-removal treatment or by routine free-residual chlorination. If they develop to a troublesome degree in drilled wells, the following procedure should be used:

1. Prepare a solution of copper sulfate sufficient to treat all the water in the well, plus about 2000 l (500 gal), with a dose of 1.0 p.p.m.
2. Force this solution back into the well so that it passes back through the strainer into the sand and gravel surrounding the strainer.
3. Allow the solution to act for 24 hours.
4. Pump the well to waste.

This procedure may be improved by using the gas generated by the evaporation of Dry Ice to force the solution rapidly back into the water-bearing strata, as follows: Pump the solution into the well, then place about 4-5 kg (10 lb) Dry Ice in the well and immediately seal the top. Allow the solution to act for 24 hours, then release the gas pressure. Pump to waste until dead organisms are not observed. (Dry Ice is carbon dioxide frozen solid by compression and refrigeration. It is a convenient source of gas pressure for this purpose.)

If iron bacteria are allowed to develop in distribution systems, serious nuisances may be caused which are difficult to correct while still maintaining service. The following procedure has been followed with success:

1. Conduct a publicity campaign to inform consumers of the harmless nature of the organisms involved and of the poor physical character of the water to be delivered during the corrective period, asking for their support.
2. Install a solution feeder of the type needed to apply a dose of copper sulfate of 1.0 p.p.m. to the supply entering the system.
3. Treat the supply with this dose continuously for the duration of the procedure.
4. Organize a main-flushing programme, starting at the fire hydrants nearest the point of treatment.
5. Flush water through the hydrants progressively, after an initial 24-hour period has elapsed, to remove the organisms, until the whole system has been flushed.
6. Continue the treatment for one week to destroy any remaining organisms and prevent their " seeding " the system.
7. Flush selected hydrants to determine whether the organisms have been removed successfully.

Consumers should be advised to flush their plumbing systems *after* the mains serving their neighbourhood have been flushed.

Fungi

Fungi are plants without chlorophyll and hence can grow in the dark. They are of limited significance in water treatment, but they may grow in water pipes carrying raw waters.

Plankton, or free-floating micro-organisms

These include microscopic plants called algae, containing chlorophyll and requiring sunlight for growth, and single-celled animals known as protozoa, as well as other higher forms of free-floating or motile animal life. All free-floating organisms are called plankton. They are important to waterworks operators because they are prevalent in stored waters and create nuisances from the tastes and odours they produce, or lead to short filter runs because floc in which they are embedded does not settle readily and a portion remains in settled waters flowing to filters.

The plankton can, in general, be considered as a group in preventive measures. For the most part the algae will be found in the upper, circulating zone of reservoirs, under the influence of sunlight. Many protozoa, including the most important taste- and odour-producing forms, are found more widely distributed because they are independent of the influence of light; other protozoa are most prevalent near the bottom of reservoirs, as they use deposited organic debris as food.

Higher forms of organisms are not taste- and odour-producing but are of significance because they may be so prevalent as to shorten filter runs. Certain of these organisms are just visible to the naked eye (e.g., crustaceans, worms and insect larvae), hence their presence in unfiltered surface supplies will be objectionable to consumers, even though they are harmless.

Plankton must be controlled as a routine aspect of taste and odour prevention (see Copper Sulfate Treatment, below). The only way of preventing worms and insect larvae in distribution systems served by unfiltered supplies is not to use intakes located at or near the bottom of reservoirs; these organisms congregate in the bottom deposits and are not killed by chlorination with feasible doses when the stored water is treated as it enters distribution systems. Furthermore, they are not destroyed by the copper sulfate doses used to control plankton.

Rooted aquatic vegetation

Such vegetation is of three types, namely, emergent plants such as cat's-tails, floating leaf plants such as water lilies, and submerged, rooted aquatic plants such as seaweed. This vegetation is of significance as it contributes organic matter and colour to stored water when it dies or clogs screens of reservoir intakes when it becomes loosened and floats towards the intakes. This difficulty may be serious in large reservoirs, because submerged vegetation may grow in depths of water up to about 12 m (40 ft).

Emergent plants and floating leaf plants are difficult if not impossible to control. Where funds are available consideration may be given to the trial use of several chemicals that have the property of causing an abnormally high rate of growth and ultimate death of the plants; for instance, a dilute solution of one of these products, known as 2,4-D (2,4-dichlorophenoxyacetic acid) has been found effective when 1 kg is dissolved in a solution sprayed over an area of 1 hectare (1 lb/acre). This procedure has advantages in addition to low doses: treatment of the exposed portions of a plant kills the whole plant; rapid absorption occurs; reflooding of treated plants does not limit action; and the material is non-toxic when used as indicated.

The salts and esters of 2,4-D are soluble in water or kerosene (paraffin). Water solutions are used in concentrations of 1 part 2,4-D to 750 parts water; oil solutions are made up to about 1% strength. The oil solution seems to be preferable because the droplets of oil float and become attached to the exposed plants rather than being diluted in the water. The insoluble 2,4-D powder may be mixed with a wetting agent and then diluted in water; commercial preparations of the material and wetting agent are available ready for mixing.

The solution is applied by portable spraying equipment like that used in malaria-control practice or for the application of liquid chemicals to fruit trees. The nozzle should be adjusted to produce a fine spray or mist.

Marginal weeds

Reservoirs with steep banks and flooded areas usually do not support troublesome growths of marginal weeds, whereas those with gentle slopes and only seasonal fluctuations in water level will support weeds. Controlled fluctuation in water level, when feasible, will suppress the growth of marginal weeds. Most of these plants are killed when flooded and may create tastes and odours when they decompose. In addition, they encourage mosquito breeding.

These marginal weeds and other plants can be removed by cutting when funds are available. If they remain exposed in temperate climates and die during the winter, they may be more readily destroyed by burning, using a kerosene torch to ignite the plants. Precautions should be taken against spread of the fire to the area above the maximum flow line. Water-resistant trees such as willows may grow in areas occasionally flooded. If their trunks are cut near the ground level, regrowth of saplings will occur. This may be prevented by applying 1 pt of kerosene to each stump. Marginal weeds and other vegetation, like the rooted aquatic plants, may be destroyed by the application of 2,4-D. This chemical is most effective against broad-leaved plants, in contrast to grasses.

Copper sulfate treatment

Doses

The growth of plankton in reservoirs may be controlled by copper sulfate treatment. Generally satisfactory results have been secured, but this chemical has not always been effective as an algicide. The doses required for this purpose differ with each organism, so economy in the use of copper sulfate and in its distribution in reservoirs warrants microscopic examination of appropriate samples collected at significant locations, to determine the types of organisms and their relative numbers. It will be noted from Table 2, however, that many of the recommended doses are 0.3 p.p.m. or less, so this dose may be used in the absence of laboratory control. On the other hand, many troublesome organisms may be killed with doses of 0.12 p.p.m., indicating the economy possible when microscopic examinations can be made.

The required dose is influenced by temperature, alkalinity and carbon dioxide content of waters. Low temperatures reduce the effectiveness of copper sulfate, but usually this is of little moment because treatment of cold waters is commonly restricted to those occasions when the protozoan, *Synura*, develops in cold waters, even under ice. Use 0.3 p.p.m. to destroy this organism in cold water, rather than the somewhat lower dose shown in Table 2. High alkalinity and low carbon dioxide content of the water being treated decrease the effectiveness of copper sulfate, because insoluble copper carbonate forms and is precipitated and thus does not penetrate the cells of the organisms. Thus speed in the distribution of copper sulfate is necessary with alkaline waters.

The dose of copper sulfate does not have to be computed from the total capacity of a reservoir, because plankton usually congregate in the upper, circulating zone, that is, in the upper 4.5-9 m (15-30 ft). If the vertical distribution is not known, an easy rule of thumb to follow is to consider that the top 4.5 m (15 ft) of the water is to be treated. (This is equivalent to 45 000 m^3 per hectare of surface; about 4 million gal UK or 4.8 million gal US per acre of surface). In specific instances only the water in shallow coves may need to be treated, especially when the coves are fed by stream waters of high carbon dioxide content also containing other plant food from cultivated drainage basins.

Conversely, when protozoa are involved, or when spring and fall overturns distribute water throughout a reservoir depth, the whole reservoir content will have to be treated. This is difficult when waters have alkalinities much over 50 p.p.m., because the algicide is precipitated as copper carbonate before the copper sulfate can diffuse to lower depths. For alkaline waters the dose should therefore be based on surface area rather than volume of water. Difficulty in the treatment of alkaline waters is reduced

TABLE 2

APPROXIMATE AMOUNT OF COPPER SULFATE AND CHLORINE REQUIRED AS LETHAL DOSE FOR VARIOUS ORGANISMS[a]

Organism	Odour (taste, colour)	Cause of other trouble	Copper sulfate dosage		Chlorine dosage, p.p.m.
			p.p.m.	lb/million US gal	
Diatomaceae:					
Achnantes	—	—	—	—	0.25
Asterionella	Aromatic, geranium, fishy	—	0.12-0.20	1.0-1.7	0.5-1.6
Cyclotella	Faintly aromatic	Yes	—	—	1.0
Diatoma	Faintly aromatic	Yes	—	—	—
Fragilaria	—	Yes	0.25	2.1	—
Melosira	—	Yes	0.20	1.7	2.0
Meridion	*Aromatic*	—	—	—	—
Navicula	—	Yes	0.07	0.6	—
Nitzschia	—	—	0.50	4.2	—
Stephanodiscus	—	—	0.33	2.8	—
Synedra	Earthy	Yes	0.36-0.50	3.0-4.2	1.0
Tabellaria	Aromatic, geranium fishy	—	0.12-0.50	1.0-4.2	0.5-1.0
Chlorophyceae:					
Chara	—	Yes	0.10-0.50	0.8-4.2	—
Cladophora	—	Yes	0.50	4.2	—
Closterium	—	—	0.17	1.4	—
Coelastrum	—	Yes	0.05-0.33	0.4-2.8	1.0
Conferva	—	Yes	0.25	2.1	—
Desmidium	—	—	2.00	16.6	—
Dictyosphaerium	Grassy, nasturtium, fishy	—	—	—	0.5-1.0
Draparnaldia	—	—	0.33	2.8	—
Eudorina	Faintly fishy	—	2.00-10.00	16.6-83.0	—
Enteromorpha	—	—	0.50	4.2	—
Gloeocystis	Offensive	—	—	—	—
Hydrodictyon	*Very offensive*	Yes	0.10	0.08	—
Microspora	—	—	0.40	3.3	—
Nitella flexilis	Objectionable	Yes	0.10-0.18	0.8-1.5	—
Palmella	—	—	2.00	16.6	—
Pandorina	Faintly fishy	—	2.00-10.00	16.6-83.0	—

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[a] These are suggested doses and must be adjusted according to alkalinity and temperature. Experience will show the effective doses.

TABLE 2 *(continued)*

APPROXIMATE AMOUNT OF COPPER SULFATE AND CHLORINE REQUIRED AS LETHAL DOSE FOR VARIOUS ORGANISMS

Organism	Odour (taste, colour)	Cause of other trouble	Copper sulfate dosage		Chlorine dosage, p.p.m.
			p.p.m.	lb/million US gal	
Chlorophyceae: *(contd.)*					
Protococcus	—	—	—	—	1.0
Scenedesmus	—	—	1.00	8.3	—
Spirogyra	—	Yes	0.12	1.0	0.7-1.5
Staurastrum	Grassy	—	1.50	12.5	—
Tetrastrum	—	—	—	—	1.0
Ulothrix	—	—	0.20	1.7	—
Volvox	Fishy	—	0.25	2.1	0.3-1.0
Zygnema	—	—	0.50	4.2	—
Cyanophyceae:					
Anabaena	Mouldy, grassy, vile	—	0.12-0.48	1.0-4.0	0.5-1.0
Aphanizomenon	Mouldy, grassy, vile	—	0.12-0.50	1.0-4.2	0.5-1.0
Clathrocystis	Sweet grassy, vile	—	0.12-0.25	1.0-2.1	0.5-1.0
Coelosphaerium	Sweet grassy	—	0.20-0.33	1.7-2.8	0.5-1.0
Cylindrospermum	Grassy	—	0.12	1.0	—
Gloeocapsa	(Red colour)	—	0.24	2.0	—
Microcystis	—	—	0.20	1.7	—
Oscillatoria	—	Yes	0.20-0.50	1.7-4.2	1.1
Rivularia	Mouldy, grassy	—	—	—	—
Protozoa:					
Bursaria	Irish moss, salt marsh, fishy	—	—	—	—
Ceratium	Fishy, vile (rusty-brown colour)	Yes	0.24-0.33	2.0-2.8	0.3-1.0
Chlamydomonas	—	—	0.36-1.0	3.0-8.3	—
Cryptomonas	Candied violets	—	0.50	4.2	—
Dinobryon	Aromatic, violets, fishy	—	0.18	1.5	0.3-1.0
Endamoeba histolytica (cysts)	—	—	—	—	3.0-100.0
Euglena	—	—	0.50	4.2	—
Glenodinium	Fishy	—	0.50	4.2	—
Mallomonas	Aromatic, violets, fishy	—	0.50	4.2	—
Peridinium	Fishy, like clamshells	—	0.50-2.00	4.2-16.6	—
Stentor	—	—	0.24	2.0	—

TABLE 2 *(concluded)*

APPROXIMATE AMOUNT OF COPPER SULFATE AND CHLORINE REQUIRED AS LETHAL DOSE FOR VARIOUS ORGANISMS

Organism	Odour (taste, colour)	Cause of other trouble	Copper sulfate dosage		Chlorine dosage, p.p.m.
			p.p.m.	lb/million US gal	
Protozoa: *(contd.)*					
Synura	Cucumber, musk-melon, fishy (bitter taste)	—	0.12-0.25	1.0-2.1	0.3-1.0
Uroglenopsis (Uroglena)	Fishy, oily (cod-liver oil taste)	—	0.05-0.20	0.4-1.6	0.3-1.0
Crustacea:					
Cyclops	—	Yes	—	—	1.0-3.0
Daphnia	—	Yes	2.00	16.6	1.0-3.0
Fungi:					
Achlya	—	—	—	—	0.6
Beggiatoa	Very offensive decayed	Yes	5.00	41.5	—
Cladothrix *(Sphaerotilis dichotomus)*	—	—	0.20	1.7	—
Crenothrix	Very offensive decayed, medicinal with chlorine	Yes	0.33-0.5	2.8-4.2	0.5
Didymohelix (Gallionella, Spirophyllum)	—	Yes	—	—	0.25
Leptomitus	—	—	0.40	3.3	—
Leptothrix	—	Yes	—	—	—
Saprolegnia	—	Yes	0.18	1.5	—
Sphaerotilis natans	Very offensive decayed	Yes	0.40	3.3	—
Miscellaneous:					
Chironomus (blood-worm)	—	Yes	—	—	15.0-50.0
Chironomus (midges)	—	—	—	—	3.0
Craspedacusta (fresh-water jelly-fish)	—	—	0.30	2.5	—
Nais	—	Yes	—	—	1.0
Potamogeton	—	Yes	0.30-0.80	2.5-6.7	—

From Hale (1942).

when fine granular copper sulfate is scattered by a blower over the water surface so that the granules dissolve and contribute copper sulfate as they sink through the lower depths. In general, the rule of thumb is to use a dose of 5 kg per hectare of water area (5 lb per acre) for such waters. Waters of intermediate alkalinity may be treated on a volume basis, using the doses in Table 2 as the base and increasing the doses by 5% for each 10 p.p.m. alkalinity. For instance, a water with an alkalinity of 50 p.p.m. would be treated with the base dose for a specific organism, increased by 5×50/10, or 25%. Copper sulfate is not effective with waters having pH values over 8.5. Furthermore, organisms may develop a tolerance for copper sulfate.

In the last analysis the most effective dose for any given supply will be determined by experience. Thus values other than those suggested in Table 2 may be developed through practice under specific local conditions.

Limitations

Copper sulfate is toxic to fish. For a number of years the maximum doses deemed permissible in the United States were those shown in Table 3 for specified fishes. Note that only one fish, namely, trout, is affected by doses of less than 0.30 p.p.m. These toxic values were obtained in the laboratory with distilled water in which copper sulfate would remain soluble rather than precipitating as it does in natural waters. Experience has shown that fish-kills have been due to careless distribution of the chemical, which results in localized high doses. Furthermore, immediate

TABLE 3
MAXIMUM CONCENTRATIONS OF COPPER SULFATE SAFE FOR FISH

Fish	Safe copper sulfate concentration	
	p.p.m.	lb/million US gal
Trout	0.14	1.2
Carp	0.30	2.5
Sucker	0.30	2.5
Catfish	0.40	3.5
Pickerel	0.40	3.5
Goldfish	0.50	4.0
Perch	0.75	6.0
Sunfish	1.20	10.0
Black bass	2.10	17.0

From Kellermann (1912).

causes of fish-kills have been the clogging of the gills of the fish by dead algae or the reduction of the dissolved oxygen content of the water due to the decomposition of dead algae. These influences may be minimized by starting the application of copper sulfate at the dams of reservoirs and proceeding towards the points where tributary streams enter, so that the fish may avoid the chemical by swimming towards the untreated entering waters. The doses suggested in Table 2, then, may be applied in practice, provided that good distribution is assured.

Frequency of application

The need for copper sulfate treatment of a specific reservoir should preferably be determined by periodic microscopic examinations. Usually treatment is needed when the concentration of all organisms present exceeds 500-1000 " area standard units " per millilitre. (See page 357 for discussion of these units.) A few organisms, such as *Synura*, may cause trouble in concentrations of less than 1000 area standard units. In the absence of laboratory control, the odour test (Supplement, page 287) should be used and treatment provided when the odour has an intensity of 3—that is, " distinct "—by the simple procedure or a threshold odour number of 3-5 by the standard procedure, as described on page 289.

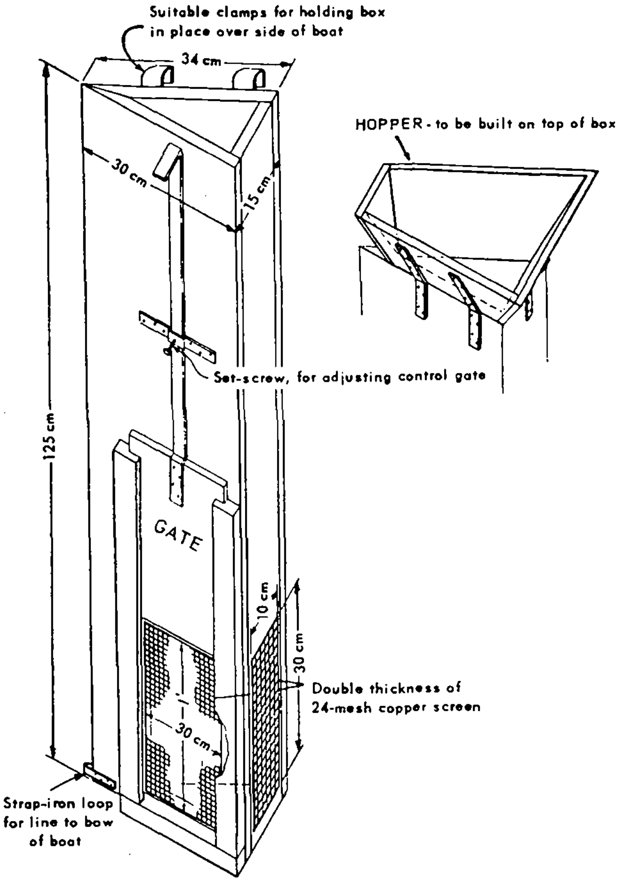
The burlap bag method

Several methods of applying copper sulfate are available. A weighed quantity of crystals usually equivalent to a dose of 0.3 p.p.m. may be placed in a cloth bag which is then dragged under the surface of the water from the stern of a row-boat or small launch. The boat is usually propelled in parallel courses about 8-15 m (25-50 ft) apart. Streaks of more concentrated solution are formed in the wake of the boat until mixing occurs by diffusion or by current action. Treatment is therefore facilitated when the wind is blowing hard enough to produce surface currents and when the boat is propelled at considerable speed over courses no more than 8 m (25 ft) apart, to prevent the solution of an undue amount of chemical at any one point. The use of row-boats equipped with outboard motors or of small launches greatly facilitates the application of copper sulfate because of the higher speed and the improved mixing due to the action of the propeller. Application is more rapid when the lumps of copper sulfate are crushed or pulverized.

The wooden box method

The crude procedure described above can be improved by replacing burlap bags with permanent boxes attached securely to each side of a small motor boat. These boxes should be wedge-shaped, with the edge pointing forward (Fig. 2). They should be so installed as to project at least 1 ft into

FIG. 2
BOX FOR CONTROLLED DISTRIBUTION
OF COPPER SULFATE SOLUTION IN LAKES OR RESERVOIRS



From Cox (1952) p. 142 by permission of the New York State Department of Health.

the water to submerge the two screened openings provided near the bottom, through which water will flow in a horizontal direction as the boat is propelled through the water. The forward opening should be fitted with a sliding cover which can be manipulated to vary the area of the opening and thus the amount of water passing through the opening. The screens retain the copper sulfate crystals. The tops of the boxes may be enlarged in the form of hoppers to facilitate the shovelling of copper sulfate crystals into the boxes. The containers should be kept full of crystals, which are continuously dissolved by the water passing through the boxes. The rate of solution of the crystals can be controlled by changing the speed of the boat or the position of the cover over the forward opening of each box, or by selecting copper sulfate crystals of various sizes. For instance, 25 kg (55 lb) of large crystals can be dissolved in about 10 min by the water passing through each box, whereas the same amount of more pulverized material can be dissolved in 6-7 min. Thus about 50 kg (110 lb) of pulverized copper sulfate can be dissolved in 6 min from two boxes, equivalent to the application of 1000 lb of copper sulfate per hour to about 15 hectares (37 acres) of water surface, with a boat speed of 10 km/h (6 miles/h) and with boat courses 15 m (50 ft) apart. This gives a dose of 500 kg (1100 lb) to about 450 000 m^3 (100 million gal UK; 120 million gal US), assuming a water depth of 3 m (10 ft), or about 1 p.p.m.—more than the required dose in most cases, so ordinarily the sliding covers have to be used to reduce the rate of solution.

The spray method

Copper sulfate solution may be sprayed over the surface of the water from the stern of a launch by conventional tree-spraying equipment, but a large volume of copper sulfate solution would have to be prepared in the tank of the spraying equipment. Home-made equipment may be readily constructed by using 1–2-hp gasoline-engine-driven, self-priming centrifugal pumps installed in the stern of a launch. The suction pipe of the pump should extend through the bottom of the boat below the water surface and be fitted with a suitable control valve; the pipe should be fitted with a tee, and a branch suction pipe, also fitted with a control valve, should lead from the tee to the bottom of one end of a wooden tank or barrel. The discharge pipe of the pump should be fitted with a valve and a branch terminating above the wooden tank or barrel; the pipe itself should terminate at a suitable hose coupling to which conventional rubber hose is attached.

The solution is prepared in the wooden tank or barrel permanently mounted in the centre of the boat to secure stability. A tank 0.60 m wide × 1.20 m long × 0.9 m deep (2 ft×4 ft×3 ft) has been used with heavy boats about 7.5-9 m (25-30 ft) long, but a barrel is adequate for smaller, outboard motor boats. A vertical baffle is used to divide the tank or barrel into two compartments. The baffle should have small perforations near the lower end to permit water to flow from one compartment to the other without disturbing the copper sulfate crystals held in one compartment. The branch discharge pipe of the pump should lead to a point directly above the first compartment, in which the copper sulfate crystals are placed. Water flowing from this pipe and trickling through the copper sulfate crystals forms a saturated solution that flows through the perforations in the baffle to the second compartment. This concentrated solution is then pumped through the branch suction pipe and forced through the hose to the water supply being treated. In this way, a relatively large volume of water mixed with a small volume of saturated copper sulfate solution is forced by the pump through the rubber hose, a small portion of the diluted solution being piped back to the solution tank to form more saturated solution. The hose may be fitted with a conventional nozzle which is manipulated back and forth to distribute the solution from each side and from the stern of the boat. It is possible, however, to provide three separate fan-shaped spray nozzles, one on each side and one at the rear of the boat, to secure more uniform distribution of the solution.

Saturated copper sulfate solution contains about 25% copper sulfate by weight, or 1 part chemical to 3 parts water. This should be diluted to secure a final strength of 0.5 to 1.0%, so that a very concentrated solution will not reach any portion of the water being treated. The adjustment of the four small valves to secure the desired dilution may be controlled by comparing the colour of the diluted solution discharged into the wooden tank with that of copper sulfate solutions of known strength in small bottles. Thus a 0.5% solution can be secured by manipulating the valves to have 1 part saturated solution mixed with about 50 parts water. The colour of this solution should be compared with that produced by adding 5 g copper sulfate to 1 litre of water in a bottle kept for comparison purposes. Similar standards for solutions of 1.0%, 1.5%, etc. can be prepared by adding 10 g, 15 g, etc. of copper sulfate to litre portions of water.

The blower method

Large quantities of copper sulfate may be distributed over large reservoirs or lakes by using a motor boat fitted with a motor-driven blower equipped with a suitable deflecting nozzle for directing the stream of air in a horizontal direction over the side of the boat. Pulverized or finely granulated copper sulfate is fed into the air entering the blower from a hopper fitted with a control valve, so that the material is blown over the surface of the water on each side of the boat.

Continuous application

Effective control of micro-organisms throughout the year is facilitated by the continuous application of copper sulfate to the water entering reservoirs; the micro-organisms are thus controlled before heavy growths occur, avoiding the necessity of periodic treatment. Furthermore, the prevention of growths obviates the subsequent destruction of large quantities of organisms, which would result in the reduction of the dissolved oxygen content of the water, and hence protects fish life because the lowering of the oxygen content is frequently responsible for fish-kills, erroneously blamed on copper sulfate. The continuous application of copper sulfate, however, reduces the available supply of food for fish, so continuous treatment should be restricted to those reservoirs used exclusively as sources of public water supply.

Copper sulfate may be continuously applied with either commercial chemical feeders or home-made equipment consisting of a solution tank and a constant-head tank with an orifice or control valve. Another simple device consists of a perforated box so supported in the water of the entering stream that the depth of submergence may be varied at will. Lumps of copper sulfate are placed in the box, to be dissolved by the water flowing through the box. The rate of solution may be controlled by raising or lowering the box, which should be kept filled to a point *above* the water level. Doses of 0.12 p.p.m. have been found effective in raw-water reservoirs, and doses as low as 0.03 p.p.m. have been used to treat filtered water before it enters open storage reservoirs.

Certain taste-producing organisms thrive in water under ice. Continuous treatment of water entering reservoirs will minimize this difficulty. In the absence of continuous treatment the following procedure is reasonably effective: Place about 5 kg (about 10 lb) copper sulfate in a weighted cloth bag attached to the end of a rope. Cut holes in the ice at intervals of 7.5-15 m (25-50 ft) and lower and raise the bag of copper sulfate several times through each of these holes. For economy in the treatment of very large reservoirs during the winter season, restrict the treatment area to points within 300 m (1000 ft) of the intake, since horizontal currents do not prevail under ice in large reservoirs.

Except with continuous application of copper sulfate, the water in the immediate vicinity of the water-supply intake should be treated first and the treatment procedure should progress towards the entering stream of the reservoir. Fish can then avoid the treatment zone by swimming towards the entering stream and remaining in untreated water, provided a small portion of the reservoir at the mouth of the stream is not treated. This method also has the advantage of giving priority to the water used for public water supply.

Corrective procedures

The previous discussion has dealt with the *prevention* of tastes and odours. Often, however, preventive procedures are not used or are not effective under specific local conditions. Therefore attention is directed here to corrective procedures.

Aeration

Removal of tastes and odours by aeration is less pronounced than some believe, as previously remarked. This is especially true of the slightly volatile odours due to the decomposition of vegetation. On the other hand, effective aeration, as with fine sprays, will release some of the more volatile odours caused by living micro-organisms and thus reduce the chlorine or activated carbon doses needed to remove the remaining odours. It is doubtful, however, whether the cost of aeration would be warranted solely for odour control, except with ground waters devoid of oxygen and containing sulfides.

Free-residual chlorination for taste and odour control[1]

A distinction should be made between free-residual chlorination to *prevent* chlorinous tastes and the procedures used to *destroy* taste and odours in waters under treatment. This destruction occurs while the reactions are taking place (see Fig. 11, page 139), so it should be completed when *only* free-residual chlorine remains. In other words, chlorine doses designed to secure rapid and effective disinfection should be adequate for taste and odour control. Experience has shown, however, that a reaction period of at least 2 h may be needed for the destruction of odorous substances; that is, the dose must be sufficient for the oxidation-like reactions during this period, so that *only* free-residual chlorine persists thereafter.

For instance, a water as tested may have a content of free-residual chlorine of, say, 0.5 p.p.m. at the end of a 30-min reaction period, but when tested at the end of a reaction period of 2 h it may have, say, 0.2 p.p.m. free-residual chlorine and 0.2 p.p.m. combined-residual chlorine, the latter due to reactions that occurred in the second interval. The combined-residual chlorine portion may have an objectionable taste if it is in the form of chloro-organic compounds rather than chloramines. The dose of chlorine should therefore be increased so that *only* free-residual chlorine is present at the end of the full 2-h period. Such prechlorination, providing only free-residual chlorine in the effluent of sedimentation basins, will ensure ample opportunity for the destruction of tastes and odours, as well as the other benefits of this treatment process discussed in Chapter 8.

Some tastes and odours cannot be sufficiently altered in character by chlorination to render the water attractive, especially the so-called " woody " tastes, or tastes due to the decay of vegetation. In fact, these tastes and odours, usually due to the action of actinomycetes, a type of bacteria, are intensified by chlorination. Free-residual chlorination must then be used for disinfecting or other purposes, and residual tastes and odours adsorbed by activated carbon; or ozone can be used for both disinfection and odour control.

Laboratory control of free-residual chlorination

The orthotolidine-arsenite test is used in routine control of chlorination for taste and odour elimination, supplemented, of course, by the tests for tastes and odours. If, however, this treatment is being started and effective doses are not known, or if the character of a water changes suddenly,

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[1] See page 137 ff. for detailed discussion of free-residual chlorination before considering this aspect of taste and odour control.

preliminary trials in the laboratory are necessary, because tastes and odours may be intensified when doses are increased or decreased.

The laboratory procedures are those associated with the chlorine-demand test, as given in the Supplement, page 323, except that the odour test is also made on portions of the sample of water, each chlorinated with a specific dose. For instance, suppose six portions of a sample of raw water have been placed in six beakers and dosed with 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 p.p.m. After a reaction period of 2 h these portions are tested by the ortho-tolidine-arsenite test and by the odour test. Assume that the dose of 2.5 p.p.m. produces at the end of 2 h a free-residual chlorine content of 0.5 p.p.m., with 0 combined-residual chlorine, and that this portion is free from objectionable odour. This, then, would be the dose to use on a plant scale, for that specific water. Since this trial treatment of portion number 5 would provide effective disinfection, the chlorinated water would be safe to place in the mouth to determine its taste. When in doubt as to the safety of swallowing a portion of water, test it only for odour.

A reaction period of 2 h is mentioned because this is the minimum time recommended for this process. In practice the period should be the same as that in the sedimentation basin for prechlorination—usually 3 h or more.

Chlorine-ammonia treatment

This treatment procedure, described in Chapter 8, page 155, was developed many years ago to produce chloramines, which are much more stable and less chemically active than free chlorine. This lesser activity reduces the rate of disinfection, as already noted, but also lessens the reactions between the chlorine and organic matter, thus minimizing chlorinous tastes and odours. In other words, chlorine-ammonia treatment prevents or minimizes tastes and odours incidental to chlorination, whereas free-residual chlorination destroys taste- and odour-producing substances. Chlorine-ammonia treatment is governed by the factors discussed in Chapter 8.

Chlorine dioxide treatment

Reference is made to Chapter 8, page 160, for a description of this special process. Experience has shown that chlorine dioxide rapidly and completely oxidizes phenol-like compounds and many other pollutants from industrial wastes but is not completely effective in eliminating " woody " and other tastes incidental to decay of vegetation. Most economical results are secured with the use of prechlorination for initial disinfection by free-residual chlorine, and postchlorination with chlorine dioxide. Chlorine dioxide doses sufficient to give an *apparent content* of 0.2 to 0.3 p.p.m. free-residual chlorine in the filtered water are adequate, this amount of chlorine dioxide being equivalent in oxidizing power to 0.5 to 0.75 p.p.m. free-residual chlorine.

Activated-carbon treatment

This treatment process is the most certain means of removing tastes and odours, provided that funds are available for the purchase of the large doses occasionally needed. As already indicated, carbon is more effective than chlorine in eliminating woody, earthy, or mouldy tastes or odours due to actinomycetes. The availability of activated carbon and market conditions in any given area will determine the practicability of the process.

Principles

Carbon is activated by controlled heating, which creates an active, or adsorptive, surface having the power to cause substances to adhere to it. This surface action is unlike absorption, in which substances enter into or are engrossed by other substances, as when water absorbs oxygen. The surface action is greatly expanded with activated carbon because of its porosity and hence very large total surface per unit of volume. For instance, 1 ft^3 of powdered activated carbon has been estimated to present a total surface of 3 million ft^2 to the water in which it is suspended.

The suspension of powdered activated carbon in water or the passage of water through beds of granular activated carbon presents conditions favourable to the adsorption of taste- and odour-producing compounds. In the first instance the powder is removed by sedimentation and filtration; in the second, granular carbon filters are used. Unfortunately the secondary granular carbon filters are so costly that the general practice is to use powdered carbon prior to sand filtration. Granular carbon filters, however, are very effective and are used successfully in the beverage industry.

Powdered material may be used at filtration plants in accordance with conventional practice, whereas activated-carbon filters can be used only to treat clear waters and hence are commonly used as secondary filters operated in series with sand filters, usually for the production of waters of a high degree of purity, as in the beverage industry. Only the use of the powdered material is therefore discussed here.

Storage and handling

The very finely powdered activated carbon is shipped in double paper bags or paper-lined wooden barrels or in bulk. Because dust is produced when the powder is handled, it is advisable to house carbon handling and feeding equipment in separate rooms, or to use dust-control equipment, or to store and apply the carbon as a slurry. The latter method has the advantage of eliminating the explosion hazard associated with the handling of very finely powdered combustible materials. Explosions have been prevented in practice by not storing and handling activated carbon near flames, sparks or heating devices and by enforcement of " no smoking " rules.

The material should be kept dry so that lumps will not form and interfere with accurate feeding of the powder.

Application at filtration plants

The finely powdered consistency of activated carbon makes it impossible to apply it with dry-chemical feeders designed to feed granular materials such as alum. Equipment that will feed powdered lime, fluoride compounds, etc. may be used, or special carbon feeders may be purchased. Because of the insolubility of the powder, some types of solution feeders cannot be used for this purpose. When, however, *constant agitation* of slurries is provided in storage tanks, as through the use of paddle agitators or compressed air, and when feeders are of a type to apply slurries without clogging, slurry feeding is feasible.

The slurry from the " solution pot " of a dry feeder or from a slurry feeder should be diluted as it is forced by a water ejector through a hose to the point of treatment. All surfaces coming in contact with *wet* activated carbon must be corrosion-resistant, because the carbon forms an electrolytic couple with metals and galvanic corrosion results.

If funds are limited and routine activated-carbon treatment cannot be practised, emergency treatment may be provided by the batch application of carbon to each individual filter influent. A weighed amount of carbon is added to about 200 l (40 gal) of water in a wooden barrel or plastic drum, and the resulting slurry is discharged into the settled water flowing into a selected filter when it is being returned to the filtering cycle just after it has been washed. The emergency dose may be 10.0 p.p.m., based on the filter output per cycle. Higher doses might seriously reduce filter runs or lead to the penetration of carbon through a filter, so if needed they should be applied by means of a home-made slurry fed to the water entering the flocculator.

The batch treatment of a filter may be illustrated by a filter with a capacity of 200 m^3/h and a prevailing filter run of 18 h. The water filtered per cycle then is 3600 m^3. A dose of 10 p.p.m. would require $3.6 \times 10 = 36$ kg per cycle. (Corresponding values: A filter with a capacity of 1 m.g.d. UK would filter 0.75 million gal per 18-h cycle. A dose of 10 p.p.m. would require $0.75 \times 10 \times 10 = 75$ lb per cycle. A filter with a capacity of 1 m.g.d. US, and with a dose of 10 p.p.m., would require $0.75 \times 10 \times 8.34 = 62.5$ lb per cycle). The amount of activated carbon needed would be mixed into a slurry in the barrel, and the slurry discharged into the filter influent during a period of at least 2 min, to secure its distribution throughout the filter area. The carbon would act throughout the filter run.

Points of application

This material must be applied prior to filtration, except when special treatment of reservoirs is being practised, as noted later. Application *prior* to coagulation favours floc formation, permits the use of higher doses without unduly affecting filter runs, and aids in removing odours from the decomposition of sludge in sedimentation basins. Application to the settled water, however, may result in economy, because any given amount of carbon is more effective when deposited on filter sand. In practice, it is feasible to apply a moderate dose of carbon to the raw water and to supplement this with additional carbon applied directly to the filters. This is especially desirable when the normal intensity of odour increases suddenly without sufficient warning to permit increase of the regular dose.

The total dose of carbon from a single feeder may be split by using a hard-rubber Y-branch and two hard-rubber stopcocks on the slurry hoses. Manipulation of the stopcocks will enable an operator to divert all the slurry to either the raw water or the settled water, or to divert, say, 80% of the total dose to the raw water and 20% to the settled water. Accurate division is not necessary; the proportions would be based on taste and odour tests.

Doses

Doses of activated carbon have varied from several p.p.m. to as high as 80.0 p.p.m. but are usually in the range of 3.0-15.0 p.p.m. Higher doses are needed only during periods of serious taste and odour difficulty. If high doses are needed as a routine and the use of carbon is essential to supplement free-residual chlorination, adequate conventional treatment procedures are not economical. The remedy may be more effective stream-pollution control, copper sulfate treatment, longer detention periods for pretreatment processes, or the like.

Inasmuch as an overdose of carbon does no harm except to shorten filter runs, the dose may be selected by trial and error on a plant scale and then modified as needed to obtain effective results at the lowest possible dose. For greater precision, the optimum dose may be determined by the laboratory procedure given in the Supplement, page 293.

Direct application to raw-water reservoirs

When unfiltered surface supplies have objectionable tastes and odours due to decomposition of aquatic vegetation and algae, copper sulfate treatment should be utilized to prevent or destroy algae growths. But because this treatment may lead temporarily to difficulty from the decomposition of the algae so destroyed, powdered activated carbon has sometimes been applied directly to the water in reservoirs to adsorb taste- and odour-producing substances.

Equipment :

Small row-boat

Water barrel with two ¼-in.-diameter copper tubes leading horizontally from near bottom of barrel to each side of boat

Small water pump to force water into barrel
Paddle or other stirring device
Measure for powdered carbon

Place the barrel in the boat so that the two tubes project over the sides. Pump water into the barrel and compress the ends of the copper tubes if the rate of flow is excessive and too-laborious pumping is required to keep the barrel nearly full. As carbon is not soluble, only a suspension is produced when the powder is added to the water in the barrel, so it must be stirred to maintain uniformity. Three men are required for the following procedure, one to row, one to operate the pump, and one to add carbon and stir the suspension of carbon in the barrel.

Procedure :

1. Row the boat in parallel courses about 7.5 m (25 ft) apart.

2. Add at short uniform intervals a measured amount of powdered carbon to the water in the barrel, stirring the suspension in the meantime and keeping the barrel nearly full of water.

3. Continue until the whole surface of the reservoir has been traversed.

Required doses. Experience with this treatment has indicated that the required carbon doses are from 0.6 to 7 p.p.m. The equipment and procedure used to determine the required dose for any specific water are given in the Supplement, page 293.

The volume of water in a reservoir usually is known, or its capacity may be estimated from its surface area and average depth. For one hectare, each metre of average depth equals 10 000 m^3. For one acre, each foot of average depth equals 0.272 million gal UK or 0.326 million gal US. A reservoir having a surface area of 3 hectares (7.4 acres) and an average depth of 6 m (20 ft) would contain $3 \times 6 \times 10\,000 = 180\,000$ m^3 ($7.4 \times 20 \times 0.272 =$ about 40 million gal UK or $7.4 \times 20 \times 0.326 =$ about 48 million gal US). A carbon dose of 1.2 p.p.m. would require $1.2 \times 180 = 216$ kg ($1.2 \times 10 \times 40$ million gal UK = 480 lb; $1.2 \times 8.34 \times 48$ million gal US = 480 lb).

Assume that the boat is rowed at the rate of 3 km/h (2 miles/h), or 0.82 m/s (2.75 ft/s). The width of each course covered is 7.5 m (25 ft), so during each second the boat effectively covers an area of $0.83 \times 7.5 =$ 6.2 m^2 (about 68 ft^2). The average depth being 6 m (20 ft), the volume of water treated per second is $6.2 \times 6 = 37.2$ m^3 (about 1360 ft^3), weighing 37 200 kg (about 82 000 lb). If the rate of application is set at 1.2 p.p.m., it is necessary to feed the carbon at the rate of $1.2 \times 37\,200/1\,000\,000 =$ 0.0445 kg/s ($1.2 \times 82\,000/1\,000\,000 = 0.098$ lb/s). Carbon may conveniently be added to the barrel every 30 seconds, or twice a minute. The amount to be added at 30-s intervals would be $30 \times 0.445 = 1.335$ kg ($30 \times 0.098 =$ 2.94 lb), more or less.

Economical treatment of large reservoirs justifies varying the amount of carbon added to the barrel to compensate for variations in depth of water being treated, provided that the change in depth is not too rapid, since some time elapses before the *average* strength of the carbon suspension in the barrel is modified by changing the amount added every 30 seconds.

Ozone

This chemical is very effective in the destruction of tastes and odours and gives more uniform, predictable results. Ozone is also a satisfactory disinfectant, as noted in Chapter 8, page 134, where the process is discussed more fully.

Ozone is being used at a number of plants in Europe where industrial wastes create serious odour problems, or where the demand for attractive water justifies the costs involved, or where it is desired to disinfect water with a chemical leaving no tastes in the water.

Since ozone is a very active oxidizing agent, it alters taste- and odour-producing compounds into substances not affecting the senses of taste and smell. The dose must be in excess of the ozone demand of the organic content of the water. Furthermore, ozone is only slightly soluble in water and hence persists in the treated water for periods up to about 30 min.

The ozone demands of the taste- and odour-producing compounds must be determined and the equipment capacity selected to provide doses in excess of anticipated maximum values, sometimes as high as 3.5-4.0 p.p.m. Ozone doses are determined on the basis of the elimination of tastes and odours. Any excess will produce residual or free ozone, which reacts with the orthotolidine reagent to produce the typical yellowish-green colour. The intensity of the colour is greater than that resulting from a corresponding concentration of residual chlorine: 0.1 p.p.m. ozone produces an intensity equal to that given by 0.15 p.p.m. residual chlorine. In the orthotolidine arsenite (OTA) test, however, the intensity is the same as that from free-residual chlorine.

Conclusion

The procedures discussed in this chapter are interrelated, and the following general suggestions are offered for dealing with problems of taste and odour:

1. Maintain records of complaints of consumers and corrective action taken, to serve as a guide for the future.

2. Make an effort to *prevent* tastes and odours from developing by routine periodic copper sulfate treatment and by careful, controlled chlorination to secure the intended results, in addition to destroying tastes and odours, so far as this is feasible, with free-residual chlorination.

3. Use activated-carbon treatment when needed to supplement routine practice.

4. Use ozone for both disinfection and taste and odour control.

5. Maintain routine programmes for flushing distribution systems, especially at hydrants served by dead-end mains.

6. Examine routinely samples of raw, settled and filtered waters, and also samples from distribution systems, for odours.

7. Be resourceful and use improvised equipment to meet unforeseen developments.

CHAPTER 5

COAGULATION AND FLOCCULATION

Purpose

Suspended solids in water range in size from coarse material, which settles readily, to very fine material, which will not settle unless the particles coalesce naturally and precipitate or unless a coagulant is used. The precipitating solids formed by coagulation are finely divided unless they are agglomerated into larger solids or well-developed floc by agitation of the water to cause the fine solids to contact and adhere to one another and form progressively larger particles. These larger particles of floc will then settle in sedimentation basins or will be removed by filtration. Flocculation, therefore, follows treatment of water by coagulants and is essential for the preparation of the water for sedimentation and filtration at economically high rates of flow through rapid or mechanical sand filters.

Principles

An understanding of the coagulation and flocculation processes requires a distinction between successive steps in the process. First a coagulating chemical is applied to the water. In order that the chemical may react uniformly it must be distributed promptly throughout the body of water. This requires *rapid agitation or mixing* of the water at the point where the coagulant is added. Second, complex chemical and physicochemical reactions and changes occur, leading to coagulation and the formation of microscopic particles. Third, much more *gentle agitation* of the water causes the agglomeration of the particles; in other words, the fine particles are flocculated into settleable floc.

In the past, flocculation was termed " mixing ", and the whole process was given that name. It is now realized, however, that mixing for the distribution of the coagulant in the water is only the first step in flocculation. Nevertheless this rapid or flash mixing is necessary, because otherwise the coagulant would diffuse through quiescent water very slowly and the initial chemical reactions would be restricted to that portion of the water in which the concentrated coagulant happened to be introduced. This would produce localized conditions quite unlike those intended, because of the marked influence of the concentration of chemicals on the resulting type of reactions. On the other hand, if flash mixing were followed by quiescent conditions, the fine precipitate would not be agglomerated into sizable floc in a reasonable period of time. Effective and economical clarification, therefore, requires the completion of coagulation and flocculation before the treated water enters sedimentation basins.

The coagulation process is quite common in nature: thus the heating of egg white causes its physical state to be changed into a gel or solid, and the lactic acid of sour milk causes the casein to coagulate as curd. Complex forces are at work in water coagulation and flocculation, which are influenced by the character and quality of the water, the type and dose of coagulant, the water temperature, the period of time and the degree of agitation. Those factors which are of chief importance and which are subject to control and supervision by an operator need to be clearly understood to secure effective results.

Colloids

When insoluble solids, such as clay, are so finely subdivided that particle sizes are intermediate between the molecular dispersion of true solutions and sizes just about visible under an ordinary microscope, the particles are said to be in the " colloidal state ". The colouring matter leached from leaves and other organic matter by water also has the dimensions of colloids; it may consist of colloidal droplets of humic or tannic acid or of complex compounds of iron and organic matter. One important property of the colloidal state is that the individual particles or colloids have electric charges. For instance, colloidal clay and colouring matter both have negative charges in natural waters. These charges cause the colloids to repel each other, so they do not coalesce into larger settleable particles but remain in suspension. The colloidal state also is characterized by the very large total surface area of the particles as compared with the same weight of undispersed material.

Adsorption

The large surface area of colloids favours the adsorption or adhesion of extremely thin layers of molecules of gases or liquids or of dissolved substances with which the colloids are in contact. Thus activated carbon will adsorb taste- and odour-producing substances. The adsorption of impurities in the water by floc is of prime importance in flocculation.

Ionization

Many substances dissolve in water, and their molecules become dispersed in the water in a manner characteristic of each substance. The study of solutions has disclosed that the dissolved substances may be subdivided into two classes, electrolytes and non-electrolytes. Electrolytes get their name from the fact that when dissolved in water they dissociate into positively and negatively charged particles. These charged portions are called " ions ", and the dissociation process is called " ionization ". The positive ions are called " cations ", the negative ions, " anions ". For instance, solid salt, or sodium chloride (NaCl), when dissolved in water, is ionized to the positive sodium cation, Na^+, and the negative chloride anion, Cl^-. Conversely non-electrolytes (sugar, for instance) do not dissociate; each sugar molecule remains unchanged in solution and has no electric charge.

Ions of specific importance in coagulation include the cations aluminium (Al^{+++}), iron (Fe^{+++}) and hydrogen (H^+) and the anions chloride (Cl^-), sulfate (SO_4^{--}) and hydroxyl (OH^-). The interaction of these ions is discussed below.

Hydrogen-ion concentration, or pH

Pure water, H_2O, actually may be visualized as HOH. A very small portion of the water is ionized into hydrogen ions H^+ and hydroxyl ion OH^-, so that water is $HOH + H^+ + OH^-$. Measurements indicate that 1 litre of neutral water contains only 1/10 000 000 g of hydrogen ions. This fraction is awkward to use, so a scale has been devised known as pH values, derived from the positive potential of the hydrogen ions. In order to develop a whole number rather than a fraction, the pH values have been selected as " the common logarithm of the reciprocal of the concentration of hydrogen ions ". Inasmuch as the hydrogen-ion concentration of *pure* neutral water at 25°C is 1/10 000 000, or 0.0000001, g per litre of water, its pH is the logarithm of 1/0.0000001, that is, the logarithm of 10 000 000, which is 7.0. Therefore pure neutral water has a pH value of 7.0. but a hydrogen-ion concentration of 0.0000001 g/l.

Table 4 shows the pH scale and the corresponding concentrations of hydrogen ions and hydroxyl ions. It will be noted that the concentration of hydrogen ions balances that of hydroxyl ions at the neutral point of pH 7.0, and that the concentration of hydrogen ions goes up as that of hydroxyl ions goes down. Without discussing technical details, it may be stated that the acid properties of acids, such as HCl or H_2SO_4, are due to their content of hydrogen ions; the basic properties of bases, such as $Ca(OH)_2$ or NaOH, are due to their content of hydroxyl ions. Neutrality, therefore, represents balanced concentrations of hydrogen ions and hydroxyl ions, as with pure neutral water, H^+OH^-. As Table 4 indicates, unbalanced

TABLE 4

RELATIONSHIP BETWEEN pH SCALE AND HYDROGEN-ION AND HYDROXYL-ION CONCENTRATION

pH value (approximate)	Grams of hydrogen ions (H^+) per litre of solution	Grams of hydroxyl ions (OH^-) per litre of solution
0.0	1.0	0.00000000000001
1.0	0.1	0.0000000000001
2.0	0.01	0.000000000001
3.0	0.001	0.00000000001
4.0	0.0001	0.0000000001
5.0	0.00001	0.000000001
6.0	0.000001	0.00000001
7.0 - - - Neutral - - -	0.0000001 - - - - - - - Neutral - - -	0.0000001
8.0	0.00000001	0.000001
9.0	0.000000001	0.00001
10.0	0.0000000001	0.0001
11.0	0.00000000001	0.001
12.0	0.000000000001	0.01
13.0	0.0000000000001	0.1
14.0	0.00000000000001	1.0

From Cox (1952) by permission of the New York State Department of Health.

higher concentrations of hydrogen ions, characteristic of acids, are shown by pH values *below* the value of 7.0 for neutrality; conversely, unbalanced higher concentrations of hydroxyl ions, characteristic of bases, are due to high concentrations of these ions and lower concentrations of hydrogen ions, shown by pH values *above* the value of 7.0.

The relationship of the pH scale to the acid or alkaline characteristics of natural waters is shown by Fig. 18, page 302. This figure illustrates an apparent paradox in that the range of natural alkalinity extends below the neutral point of pH 7.0, that is, into what may be called the " acid range ". These low alkalinities are due to bicarbonates in the presence of carbon dioxide, or carbonic acid. The pH is in the " acid range ", *but the water is " alkaline " to the methyl orange indicator*, which changes colour at pH 4.6. Any pH of water below this value of 4.6, however, would indicate the presence of mineral acids introduced by acid mine wastes or industrial wastes. The practical factor is that in the test for alkalinity the amount of acid reagent needed to change the pH from, say, 7.0 to 4.6 is very small, so methyl orange is a satisfactory indicator, except that errors occur with waters of very low alkalinity.

Process of coagulation and flocculation

The principles outlined above may be given practical significance by reference to the use of alum as a coagulant. Alum is represented thus: $Al_2(SO_4)_3 \cdot 14H_2O$, which includes the water of crystallization. Alum ionizes in water thus: $Al_2(SO_4)_3 \cdot 14H_2O \rightleftharpoons 2Al^{+++} + 3SO_4^{--} + 14H_2O$; that is, into positive aluminium ions and negative sulfate ions. (Chemically pure aluminium sulfate contains 18 molecules of water of crystallization, but filter alum is as shown.) The aluminium ions react with the hydroxyl ions incidental to the natural alkalinity of the water, which, if inadequate, must be supplemented by the application of lime, $Ca(OH)_2$, or soda ash, Na_2CO_3. In all three cases aluminium hydroxide is formed:

1. $Al_2(SO_4)_3 + 3Ca(HCO_3)_2 \rightleftharpoons 2Al(OH)_3 + 3CaSO_4 + 6CO_2$
2. $Al_2(SO_4)_3 + 3Ca(OH)_2 \rightleftharpoons 2Al(OH)_3 + 3CaSO_4$
3. $Al_2(SO_4)_3 + 3Na_2CO_3 + 3H_2O \rightleftharpoons 2Al(OH)_3 + 3Na_2SO_4 + 3CO_2$

The above reactions are theoretically correct, but they do not disclose the actual amount of aluminium hydroxide produced or the amount of alkaline substances involved. At the optimum pH for any specific water the maximum precipitation of aluminium hydroxide occurs. The problem, then, is to determine what these optimum conditions are, in the light of the following discussion.

It will be noted that reactions 1 and 3 produce carbon dioxide. This may be an advantage when coloured waters of low mineral content are being coagulated at low pH values. In any case more soda ash may be used in reaction 3 to remove the carbon dioxide. The superiority of cheaper lime for pH control is shown by these reactions.

Expressed in non-technical language, the above reactions are reversible, so the application of too much or too little alkali will cause complex acid or basic salts of aluminium to be formed, which are soluble, in contrast to insoluble aluminium hydroxide. For instance, aluminium hydroxide will ionize into soluble aluminates, $Al(OA)O$ or AlO_3, at higher pH values, and basic compounds such as $Al(OH) + 2OH$ when the pH value is in the acid range. This latter compound in turn reacts with the sulfate ion introduced with the alum to form the soluble basic sulfate, $Al(OH)SO_4$. The important point is that these compounds are much more soluble than the intended one, aluminium hydroxide.

The normal reactions, however, precipitate minute particles of aluminium hydroxide, which have a positive charge; thus they repel one another and would not coagulate were it not for the presence of the negative sulfate ions, which neutralize and lead to the precipitation and coagulation of aluminium hydroxide as alum floc. These reactions are sensitive to the pH and the mineral content of the water. Thus distilled water containing no mineral matter, when treated with alum and lime, coagulates best at a pH value of 5.5, at which point aluminium hydroxide is least soluble (about 0.3 p.p.m.); thus the degree of precipitation is at a maximum at this pH value. Natural waters, however, are influenced by the presence of negative ions, such as the sulfate SO_4, which extend the effective pH range for coagulation in the acid zone, whereas the positive ions such as calcium (Ca) extend the effective range to the alkaline or basic zone, because the charge of the colloidal aluminium hydroxide reverses at pH 7.0 from positive to negative and hence is precipitated by the positive calcium ion at higher pH values.

The negatively charged colour colloids, however, are coagulated most effectively in the acid zone of pH values from 4.4 to about 6.0, because the positive aluminium ions from the alum dose neutralize the negative colour colloids. Therefore alum floc, or aluminium hydroxide, is not involved in the coagulation of colour colloids. In fact, aluminium hydroxide is not formed at the lower pH values used in colour removal, but rather aluminium colour complexes are formed as noted above. Effective coagulation of highly coloured waters at low pH may therefore have to be followed by a second step, namely, the application of lime to the effluent of primary basins where colour complex has been flocculated and settled, so as to increase the pH and precipitate the remaining aluminium as alum floc in secondary basins. Chapter 12 should be consulted for other procedures to be followed when colour is associated with iron.

The complex interrelationships between added chemicals and the constituents of the water being treated, the influence of water temperature, and the period and degree of agitation make it virtually impossible to determine the required chemical dose from the results of analysis of the water. In practice, therefore, the jar or coagulation and flocculation test, as described in the Supplement, page 312, is used to determine by trial and error the most effective and economical dose of coagulant for the period and degree of agitation selected as most closely comparable with those of the plant being supervised.

The period available for the initial coagulation followed by flocculation is related to the required dose of coagulant and the degree of agitation, lower doses being effective with longer periods of agitation and greater agitation. Usually the period and degree of agitation are fixed by plant characteristics, so the dose is varied to meet changes in water temperature, turbidity, colour, pH, etc.

Coagulants

Details of the coagulation of water by the use of various chemicals are given here. Annex 2, which summarizes the properties of different chemicals, will facilitate comparison between them.

Alum coagulants

Filter alum

Filter alum, or aluminium sulfate, weighs about 480 kg/m^3 (30 lb/ft^3). It is available in lump form, as rice, and as granular or powdered alum. The material is an acid salt and hence corrosive to most metals. It is readily soluble in water and is easily applied as a solution or as dry material.

Reactions between alum and the natural constituents of various waters are influenced by many factors, so it is impossible to determine accurately the amount of alum that will react with a given amount of natural alkalinity or of lime or soda ash added to the water. Theoretical quantities, however, serving as a general guide, are as follows:

1 p.p.m. of alum reacts with:
  0.45 p.p.m. natural alkalinity, expressed as $CaCO_3$;
  0.30 p.p.m. 85% quicklime as CaO;
  0.35 p.p.m. 95% hydrated lime as $Ca(OH)_2$;
  0.48 p.p.m. soda ash.

The above approximate amounts of added alkali are those required with alum for the formation of aluminium hydroxide floc where the alkalinity of the treated water is *not changed*; that is, water treated with 1.0 p.p.m. alum and either 0.35 p.p.m. hydrated lime or 0.48 p.p.m. soda ash will have approximately the same alkalinity as the raw water. If no alkali is added, then the acidity of 1.0 p.p.m. alum will lower the natural alkalinity of the raw water by about 0.45 p.p.m. This lowering of the natural alkalinity is desirable in most cases (the pH range for coagulation of turbid waters being 5.7-8.0.) In fact, sulfuric acid may have to be added to assist the alum in producing the low pH value required for coagulating highly coloured water (the pH range for coagulating highly coloured waters being 4.4-6.0). The alkali required for corrosion prevention, therefore, would be added to the filtered water, the required dose being influenced but not governed by the alum dose.

Liquid alum

Water plants within trucking distance of alum-producing works may find it economical to purchase alum syrup rather than crystallized alum. This liquid alum contains 5.8%-8.5% water-soluble alumina as against 17% for crystallized alum, but is lower priced and convenient to transport, handle and store, provided that acid-proof equipment is used, such as rubber-lined or stainless-steel tanks and piping. Its specific gravity is 32.2 Baumé and its weight 10.7 lb/gal at 20°C for the 7.2% Al_2O_3-content grade. Liquid alum has little usefulness at small plants, or when trucking or transport in tank cars is not convenient and economical.

Sulfuric acid

Sulfuric acid is of interest only as an aid in the coagulation of soft coloured water with alum, where the use of acid is a more economical means of producing the required low pH value than an equivalent dose of alum. For instance, a water polluted by paper-mill wastes required alum doses of 68-85 p.p.m. to secure effective coagulation at the optimum pH value of 5.8, found by experience. The same pH value and satisfactory coagulation was secured with 40 p.p.m. alum plus 14 p.p.m. sulfuric acid. Subsequently it was found that prechlorination of this specific water raised the optimum pH value to about 6.2 and also extended the effective range of pH values, thus simplifying control. Furthermore, the cost of acid was eliminated and a reduction was secured in the dose of soda ash subsequently required for corrosion prevention. Acid coagulation was still used at intervals, however, when jar tests showed that better results were secured thereby. Another example of the benefits of coagulation with acid is a situation in which 3.4 p.p.m. sulfuric acid permitted a reduction of 8.5 p.p.m. in the alum dose added to a moderately soft coloured water.

The usual commercial strength of sulfuric acid is 66° Baumé, which is equivalent to 93%. The material is supplied in 40-litre (about 10-gal) glass carboys or in drums holding 25-250 kg (50-500 lb). The *concentrated* acid may be stored in iron tanks and conducted through wrought-iron pipe, but the *dilute* acid is very corrosive to iron and must be stored in glazed earthenware vessels, rubber-lined steel tanks, etc.

Sulfuric acid must be diluted by very slowly adding the acid to the water and *not* the water to the acid, otherwise violent sputtering will occur with danger of acid burns.

Dilute sulfuric acid may be applied by the use of corrosion-resisting solution feeders, such as those developed for the application of hypochlorite solutions.

In summary, sulfuric acid may assist alum in the coagulation of coloured waters where low pH values are required, thereby securing better results at lower total cost of chemicals.

Sodium aluminate

Sodium aluminate consists of aluminium oxide stabilized with caustic soda. This coagulant is not used alone but with alum to secure some special results. Thus the failure of some very cold waters to coagulate with alum alone may be corrected by adding about 3.4 p.p.m. sodium aluminate with the alum. Another use of this coagulant is in connection with " double coagulation " of highly coloured water, in which alum alone or alum and sulfuric acid are added to the influent of the primary basin to coagulate highly coloured water at the required low pH range of 4.4 to 5.0. This results, however, in the presence of undesirable concentrations of soluble alumina (free alum) in the presettled water; alkaline sodium aluminate is then added, and also lime if needed, to increase the pH value to, say, 6.5. This secondary treatment results in the formation of aluminium hydroxide floc, which settles in the secondary basin. A third use of sodium aluminate is in conjunction with lime-soda softening of water, in which insoluble calcium aluminate floc is formed and coagulates the calcium carbonate and magnesium hydroxide resulting from the softening reactions.

Sodium aluminate may be purchased either as a solution or as a solid and may be applied by conventional chemical feeders. The cost of the material is high but the required doses are small, so the treatment may be applied to advantage and at reasonable cost when special local conditions justify it.

Activated silica

Rapid formation of large heavy floc is facilitated by the use of activated silica with alum. Activated silica also serves alone or in conjunction with alum or ferric sulfate to coagulate calcium carbonate in softening. Activated silica is formed as needed by reactions between dilute solutions of sodium silicate and one of the following: sulfuric acid, alum solution, ammonium sulfate, chlorine, sodium bicarbonate or carbon dioxide. All these reactions must be controlled very carefully so that the alkalinity of the sodium silicate is only partially neutralized and the desired colloidal silica is formed, but without the formation of a gel (water-glass). For example, 41° Baumé[1] commercial sodium silicate, having a strength of 28% SiO_2, is diluted with water in the ratio of 1 to 20, that is, to a strength of about 0.5% SiO_2. While this dilute solution is being stirred, its alkalinity is lowered by the addition of previously diluted sulfuric acid containing 1 part concentrated acid to 170 parts water. Stirring is continued for 2 h, then the solution is diluted by the addition of water in the ratio of 2 parts solution to 5 parts water. The use of ammonium sulfate rather than sulfuric acid simplifies the procedure and permits the preparation of more concentrated solutions.

Activated silica also may be formed by the reaction between dilute solutions of alum and sodium silicate. Mix 4 parts 1% alum solution and 1 part 1% sodium silicate solution. No aging is necessary. The resulting mixture may be used alone or in limited doses to assist the regular alum solution, applied separately.

Equipment is manufactured in the USA for the controlled feeding of sodium silicate and chlorine to form activated silica, which is added separately to the raw water to aid the regular alum solution.

The dose of activated silica needed depends upon the characteristics of the raw water and the procedure followed in its preparation. A solution of

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[1] On the Baumé scale for hydrometers (for liquids heavier than water), 0° equals the reading for pure water and 15° equals the reading for 15% salt solution. Specific gravity is computed by the formula: $145/(145 - n)$, where n = Baumé reading.

activated silica may be mixed with the regular alum solution and applied as a mixture, with somewhat reduced dosage; in other instances the activated silica dose applied directly to the water may be a fraction of the separate alum dose. In any case, the most effective procedure and doses are established by jar tests, the activated silica serving as a coagulant aid to the alum.

The improved production of a tough, heavy floc when activated silica is used to assist in coagulation is especially significant when low water temperatures interfere with coagulation or when a heavy floc is needed with high surface-overflow rates, as with solids-contact basins, discussed in Chapter 6. However, the use of activated silica is complicated and usually requires technical supervision. Information regarding this process may be secured from the suppliers of sodium silicate and the manufacturers of chemical feeders and chlorinators.

Ferric coagulants

Copperas

Ferrous sulfate, ordinarily known as copperas, is a granular acid compound available in bags or barrels or in bulk. The alkalinity and pH value of natural water are too low to react with copperas to form the desired ferric hydroxide floc, because the reaction involves oxidation by the dissolved oxygen in the water, which does not occur when the pH value is less than 8.5. It is necessary, therefore, to add lime with copperas to secure coagulation. For this reason, copperas is not used in the coagulation of highly coloured water, which coagulates best at pH values less than 6.0. But copperas and lime makes a very effective coagulant in lime-soda water softening and in the coagulation of iron and manganese (see Chapter 12). The dose of lime required to react with copperas is approximately 0.27 p.p.m. to 1.0 p.p.m. of copperas. Generally the floc formed by the reaction of copperas and lime is feathery and fragile, but it has a high specific gravity.

While copperas is an acid salt, it is readily applied by either solution or dry-feed equipment, although there is a distinct tendency for hygroscopic action to render the material moist and cause " arching " in the hoppers of dry-chemical leaders.

Chlorinated copperas

The oxidation of copperas by the dissolved oxygen of water under the conditions noted above occurs only at pH values exceeding about 8.5 and preferably over 9.0. Chlorine, however, may be used to react with copperas irrespective of the pH value; ferric sulfate and ferric chloride are formed, and these in turn act as coagulants. Theoretically, 1 lb chlorine reacts with 7.8 lb copperas. Additional chlorine is added ordinarily to ensure complete reaction and also to disinfect the water being treated. Coagulation with chlorinated copperas, therefore, is especially adaptable to conditions in which prechlorination is required; and the flexibility of coagulation by ferric chloride and ferric sulfate over a wide range of pH values is an additional advantage. As copperas is more readily applied to water than the ferric coagulants, chlorinated copperas treatment may be used with conventional chemical-feeding equipment without modification.

Ferric sulfate

Ferric sulfate is available as a commercial coagulant in the form of an anhydrous material that may be transported and stored in wooden barrels. The material will dissolve most readily in a *limited* quantity of warm water, so a special " solution pot " must be used with chemical feeders, in which 1 part ferric sulfate by volume is dissolved in 2 parts water to produce a solution of about 40% strength. This is equivalent to about 3½ lb ferric sulfate per US gallon of water (416 g/l).

Ferric sulfate reacts with the natural alkalinity of the water or with the alkalizing lime added and forms ferric hydroxide floc. No intermediate oxidation reaction is involved as with copperas. The required natural alkalinity or the dose of lime varies with different waters. Theoretically, the requirements are as follows:

1 p.p.m. of ferric sulfate reacts with:

0.75 p.p.m. natural alkalinity, expressed as $CaCO_3$;

0.58 p.p.m. 95% hydrated lime.

Ferric chloride

Ferric chloride is available commercially in liquid, crystalline and anhydrous form. Liquid ferric chloride is very corrosive and must be transported in rubber-lined tank cars or in glass carboys. The strength varies from 31% to 45% ferric chloride by weight, depending on the temperature. The crystalline form of ferric chloride contains 60% ferric chloride by weight and is supplied in barrels holding about 198 kg (435 lb). Anhydrous ferric chloride is free from water of crystallization and is supplied in steel drums holding about 45 kg (100 lb). The material remains non-corrosive to the drums *as long as moisture is excluded.* The drums must therefore be left sealed until use, and their entire contents must be dissolved in water at one time.

Ferric chloride reacts with the natural alkalinity of the water or with the added lime to form ferric hydroxide floc. Here, again, the required concentration of natural alkalinity or lime varies but is approximately as follows:

1.0 p.p.m. of ferric chloride reacts with:

0.92 p.p.m. natural alkalinity, expressed as $CaCO_3$;

0.72 p.p.m. 95% hydrated lime.

Summary

In summary, chlorinated copperas, ferric chloride, and ferric sulfate give similar results when their doses are compared in terms of iron content, but ferric sulfate generally is a better coagulant than ferric chloride at the low pH values required in the coagulation of highly coloured water, because of the beneficial influence of the sulfate ion as contrasted with the chloride ion. As in coagulation with alum, the positively charged floc, in this case ferric hydroxide, adsorbs bacteria and the negatively charged particles that cause colour and turbidity. A number of practical differences between alum and ferric coagulants, however, may be briefly noted:

1. Ferric hydroxide is formed at low pH values, so that coagulation is possible with ferric sulfate at pH values as low as 4.0 and with ferric chloride at pH values as low as 5.0.
2. Ferric hydroxide is insoluble over a wider range of pH values than aluminium hydroxide except for the zone of 7.0 to 8.5. Chlorinated copperas, however, has been used with success even in this pH zone.
3. The floc formed with ferric coagulants is heavier than alum floc.
4. The ferric hydroxide floc does not redissolve at high pH values.
5. Ferric coagulants may be used in colour removal at the high pH values required for the removal of iron and manganese and in the softening of water.

Clay

Bentonite, fuller's earth and other adsorptive clays have been utilized to some extent in recent years to assist in the coagulation of water. Definite information cannot be given as to when the use of clay would be advantageous, what doses are required, etc., because of lack of data on the various clays and their reactions in different waters. It appears, however, that clays assist in coagulation of relatively clear waters by supplying suspended matter around which the floc forms. Some of the clays swell when added to water and produce a floc when used alone or with a limited dose of coagulant; their use may improve coagulation and reduce the cost of coagulants. Thus a dose of 10.0 p.p.m. bentonite and 10.0 p.p.m. alum may give better results than several times as much alum alone. It is well, therefore, to experiment with the use of clay when difficulty is being experienced in the coagulation of relatively clear waters.

Some of the clays adsorb taste- and odour-producing substances, especially oily materials, so their use may assist in the treatment of waters polluted with trade wastes.

Calcite or whiting

Powdered calcium carbonate may be available where adsorptive clays are not distributed. The coagulation of cold soft waters of low turbidity may be assisted by the application of about 20 p.p.m. calcite, near the point where alum is applied. The powder furnishes numerous nuclei for the start of floc formation. Each granule of the powder will gradually dissolve in corrosive waters, but it remains as a solid during the initial period and then contributes alkalinity as it dissolves, within the floc, thus favouring both the physical and the chemical aspects of floc formation.

Powdered calcium carbonate may be purchased as calcite or whiting, the latter term being used in the building industry. It is readily handled, stored and applied. The dose needed to secure improved flocculation, which is not related directly to the alkalinity of the water, is determined by the jar test. First find the best alum dose by the jar test and then apply this optimum dose in all the jars, adding graduated doses of calcite to each, to determine what calcite dose produces the fastest-settling floc. The dose of calcite may be eliminated when high turbidities lead to a heavy floc with alum alone; on the other hand, an increased dose of calcite may be preferable when solids-contact basins are involved, because of the relatively high concentration of floc in the sludge blanket, which should be sufficiently heavy to settle in the water rising towards the effluent weirs. Usually the calcite is added after the alum has been applied, but success has been reported when the alum solution and calcite suspension have been thoroughly mixed together before application.

Coagulant aids

Important progress is being made in the development of coagulant aids as patented products of a number of manufacturers. They are effective in aiding coagulation with alum, even with doses in the range of 0.5-4.0 p.p.m. Improved flocculation permits shorter sedimentation periods and higher rates of filtration. These products, however, are not widely marketed, and better distribution and the development of accepted practice through practical experience are needed before they may be considered for routine practice on a world-wide basis.

Alkali

Soda ash

Soda ash, or sodium carbonate, is a white powder very soluble in water, so little difficulty is experienced in introducing it into water to be treated. It is used when waters do not contain enough natural alkalinity to react with the coagulant; it is also used to reduce the non-carbonate hardness of water. Solutions of this chemical do not clog the piping or orifices of the dosing equipment, and unlike lime suspensions, do not have to be stirred after they have been made up. The reaction of alum and soda ash produces about one-half the amount of carbon dioxide formed when the natural alkalinity of water reacts with alum. No carbon dioxide is formed when an excess of soda ash is used, because the excess combines with the carbon dioxide to produce sodium bicarbonate. Soda ash, therefore, can provide effectively the artificial alkalinity needed at some small filtration plants, although the cost is about three times as great as an equivalent quantity of lime.

Soda ash is practically pure sodium carbonate but should be purchased by specification to contain at least 98% sodium carbonate. Obviously it is necessary to use about 102 lb 98% commercial soda ash to provide 100 lb sodium carbonate.

Quicklime

Quicklime, or calcium oxide, also may be used with alum or copperas to provide artificial alkalinity when necessary, and for water-softening. Quicklime varies in quality from about 75% to 99% calcium oxide, so operators of water treatment plants must know the quality of the lime they are using before the doses can be calculated. The computations in this monograph are based on the use of quicklime with a calcium oxide content of 85%. Even though it may cost more per pound, a high-calcium lime should be utilized because of the ease with which it may be slaked.

The slaking of the lime should be done carefully, as the success of the treatment depends to a great extent on this process. Special slaking tanks are necessary, and a minimum amount of water should be used to ensure high temperature during the slaking period. The slaking requires 15-30 min under optimum conditions; poor grades of lime may require much longer. The slaked lime is diluted with water and stored in solution tanks. As the calcium hydroxide formed by the slaking process is only slightly soluble, the so-called " solution " of limewater is in reality a *suspension* of the chemical. It is therefore necessary to agitate the contents of the tank continuously to maintain a uniform suspension. *The diluting water should be cold, because calcium hydroxide is more soluble in cold water than in warm water.* Owing to the insolubility of the lime suspension, the storage tanks, orifice boxes and pipes of the dosing equipment must be cleaned at frequent intervals to prevent clogging. The use of excess lime should be avoided, otherwise a water with a caustic reaction will be produced and lime will be deposited about the sand grains of the filters and in the clear well and distribution system.

Hydrated lime

Slaked or hydrated lime, also known as calcium hydroxide, is a white powder formed when quicklime is slaked in water or in moist air. This material may be obtained in paper or cloth bags in relatively small quantities. It does not deteriorate when stored, does not have to be slaked, and contains fewer impurities than most quicklimes. The material can be mixed directly in solution tanks and fed through orifice boxes into the water to be treated, or the chemical can be fed in dry form by the use of dry-feed equipment. The same precautions have to be observed when feeding suspensions of hydrated lime through solution-feed equipment as with quicklime.

Hydrated lime varies in quality between about 80% and 99% calcium hydroxide. Computations in this monograph are based on the use of hydrated lime with a calcium hydroxide content of 95%. Because hydrated lime is easily handled, its use is preferable in the smaller water-purification plants where lime is used to supply additional alkalinity to the water. Quicklime is used at many softening plants and at large filtration plants because of its lower cost.

The alkalinity ratio of pure calcium oxide (CaO) to pure calcium hydroxide [$Ca(OH)_2$] is 1 : 1.32. From this it follows that the alkalinity values of 85% CaO, or quicklime, and 95% $Ca(OH)_2$, or hydrated lime, have the ratio:

$$\frac{1}{0.85} : \frac{1.32}{0.95} = 1.18 : 1.39$$

For ready reference purposes, the alkalinity of 95% calcium hydroxide may be converted to unity:

$$1.18 : 1.39 = x : 1.00$$
$$x = 0.849$$

Therefore 0.849 part of 85% quicklime = 1.00 part of 95% hydrated lime. This means that the quantity of 85% quicklime equivalent to any given quantity of 95% hydrated lime can be computed by multiplying by the factor 0.849.

Example : Assume that it takes 2.35 p.p.m. 95% hydrated lime to react with a given dose of alum. What would be the equivalent quantity of quicklime?

Solution:

2.35 × 0.849 = 1.99 p.p.m. 85% quicklime

Similar computations based on the proportion $Ca(OH)_2 : CaO = 1.0 : 0.757$ give the ratio of 1.00 part 85% quicklime = 1.18 parts 95% hydrated lime.

Example : Assume that it takes 3.15 p.p.m. 85% quicklime to react with a given dose of alum. What would be the equivalent quantity of 95% hydrated lime?

Solution:

3.15 × 1.18 = 3.72 p.p.m. 95% hydrated lime

The properties of these alkalies are summarized in Annex 2.

Chemical handling and storage

Storage

The types of containers used in the transport and storage of chemicals used in coagulation are shown in Annex 2. Bulk shipment in railroad cars or special trucks provides economy at the larger plants, provided that storage bins are located below the railroad siding to permit direct dumping into the bins, or that conveyor belts are available to transfer the chemical to elevated bins. The most convenient procedure at large plants is the use of vacuum conveyors to drive the chemical by atmospheric pressure through hose and piping to covered elevated bins, the air withdrawn from the bins by vacuum blowers being filtered to remove dust.

The transport, handling and storage of chemicals at smaller plants is facilitated when bags holding 50 kg or 100 lb are used. These may be carried to elevated storage on small hand trucks wheeled on to freight elevators, then hoisted and unloaded on the floors of storage rooms. Storage space near chemical feeders may suffice at small plants, but even here dry storage of a truck-load of chemicals justifies the use of separate storage rooms. The location of inverted conical hoppers below rooms where bagged chemicals are stored facilitates the emptying of the bags through screened openings over the hoppers. The screens will remove large particles of foreign materials in the chemicals and thus minimize clogging of the chemical feeders. These hoppers, in turn, should be above the individual chemical feeders. Dust-proof but flexible adapters should be used to connect the bottom of the hoppers to the smaller hoppers of the chemical feeders. " Gates " at the bottom of the large hoppers permit the chemical to be admitted as desired into the hoppers of the feeders. At small plants chemicals may be carried as needed and emptied from the bags directly into the chemical feeders.

All plants should be fitted with scales for weighing chemicals. Platform scales may be used to weigh bags as they are emptied into the hoppers of chemical feeders, or else scales may be placed under the individual feeders and loss in weight recorded at stated intervals to indicate the weight of the chemicals fed during that interval. Another expedient is to have the large hoppers mounted in such a way that their weight is transmitted to special scales for measurement.

Chemical dissolving tanks

Solution feeding of alum is favoured at many small plants, but the availability of conveniently used dry-chemical feeders of low capacity is leading to their more general use. Alum is dissolved in a separate dissolving tank with a perforated tray to hold the lump or granular alum. A spray of water is directed on the alum until it is all dissolved. The solution flows by gravity to larger storage tanks. Water is added to the storage tank as needed to prepare the desired strength of solution.

Preparation of solutions

A common error in preparing a given strength of solution in terms of per cent weight is the assumption that the amount of chemical required would be that percentage of the weight of the water. (See Annex 1 for computations and tabulations for preparing solutions.)

As noted in Annex 1, the best procedure is not to prepare a specified strength of solution in terms of per cent but rather to use the weight of the chemical per unit volume of water. For instance, if 25 lb alum is dissolved in sufficient water to make 100 gal of solution, then each gallon of solution contains $^1/_4$ lb alum, and when fed by volume with a solution feeder the weight of chemical will be known without any confusion.

The table in Annex 1 is useful, however, because solubilities may be expressed in terms of per cent and many references in waterworks literature also use this form. Note, however, that the tabulation in Annex 2 gives the solubility of the various chemicals in terms of pounds per gallon, that is, weight per volume.

Solution-storage tanks

Dissolving and solution-storage tanks for the storage of acid coagulants, such as alum, must be made of suitable lumber, stainless steel, or Duriron, or have plastic, Fiberglas, rubber, lead or asphalt lining for steel or concrete tanks. The tanks should be calibrated so that the volume of concentrated solution and diluting water may be gauged. This also facilitates recording the amount of solution applied during any given period, which in turn indicates the actual dose applied by the solution feeders. At least two solution-storage tanks should be available. Their capacity should be ample for the maximum volume of solution to be applied during one shift of eight hours, so that solution does not need to be prepared too often. Drains should be of ample capacity to facilitate cleaning the tanks and flushing the insoluble residue to waste.

Chemical feeders

Chemical feeders are manufactured by many companies. Their specific characteristics and calibrations and the procedures for their operation and maintenance are covered in the instruction books furnished by the manufacturers. The equipment may be divided into two types, solution-feed and dry-feed, each of which may be manual, semi-automatic, or automatic.

Solution feeders may consist of constant-level orifice tanks, in which the solution flowing from the storage tanks through float valves is controlled by adjusting a calibrated orifice or valve. There is growing interest, however, in the use of variable-capacity chemical-solution pumps, because of their accuracy and flexibility and the pressures they make available. A third type consists of a tank in which calibrated cups revolve into the solution and remove adjustable volumes of solution. This type is especially suitable to the feeding of lime suspensions because the revolving cups and associated paddles keep the lime in suspension.

Dry-feed equipment may consist of inverted conical hoppers fitted at the bottom with a device for withdrawing the chemical, such as a variable-speed screw conveyer, a revolving disc or an oscillating mechanism. Vibration created by a magnet driven by alternating current is also used to give a controlled flow of the chemical, the intensity of vibration being modified for this purpose. Finally, there is the type in which a variable-speed belt is located below the hopper, so that controlled feed of chemicals is secured by adjusting the position and speed of the belt. One type of this equipment has one end of the belt mechanism mounted on scales, with provision for varying the adjustment automatically to apply a uniform *weight* of the chemical, as contrasted to *volume* of chemicals fed by other dry-chemical feeders. Gravimetric feeders are preferable because they feed by weight and are calibrated directly in terms of dosage. Volumetric feeders are simpler and usually less costly.

The dry chemicals, as measured by such equipment, drop into so-called " solution pots ", in which a strong solution is formed by a continuous flow of water. These pots should have ample capacity to ensure detention with adequate agitation so that the solution and any remaining solids will be flushed continuously into the outlet tubing.

Chemical solutions from solution feeders, or from solution pots of dry feeders, should be carried in rubber or plastic hose or lead piping. Hose or tubing should be supported so as to prevent sagging in loops where insoluble residue may collect, or where air may cause air-binding. One satisfactory procedure is to place the hose in 4-inch-diameter tile pipe, from which it is easily withdrawn for repairs. Every effort should be made to locate chemical feeders as close as possible to the points of application of chemicals.

Operating chemical feeders

The above general discussion does not include certain factors which require study in the light of specific local conditions. For instance, alum is hygroscopic, so the material will lump on storage in climates with high humidity. This is of little moment with solution feeders, but most dry feeders are designed to feed powdered or fine granular alum. Therefore a smaller crusher should be available when needed. The significance of this factor for other chemicals is shown in Annex 2. Damp alum also will " arch " in the hoppers of chemical feeders unless an agitator or vibrator is provided with the feeders. If this is not done, the alum in the hopper must be inspected at intervals and arching remedied with a small rod used by hand.

Dust-free operation and cleanness are essential to proper maintenance.

Feeders should be calibrated at intervals by collecting the solution or dry chemical fed in one minute and determining its volume or weight. If this volume or weight is more or less than indicated by the calibrations on the feeder, a corrective factor is determined. For instance, the scale on a dry feeder, from 0 to 100, may be read as 40 units, and this may be equivalent to a factory-calibrated capacity of 30 lb/h, whereas the actual weight applied by the machine may be 0.6 lb/min by measurement, or 36 lb/h. The error, then, is 6 lb/h. If the dose desired is 30 lb/h, the feeder must be adjusted to a value on the scale proportional to the above values, that is, $30 : 36 = x : 40$, so $x = 33$. The machine would be adjusted to the value of 33 on the scale and again checked.

Any major change in the characteristics of the chemical will alter the feed rate and hence require recalibration. In any case the chemical dose should be checked with the weight of chemical used during some selected period of time, as determined by the loss of weight shown on scales. This last value is the actual dose applied during the period covered by the loss in weight.

Chemical-solution hose should be kept free of scale or deposits. When the hose is exposed, a series of blows with a rod on the hose at intervals dislodges scale or deposits adhering to the inner walls so that they may be flushed to waste.

Flocculation equipment

Rapid mixing to distribute the coagulant throughout the water being treated is frequently called " flash mixing ". This rapid agitation may be provided in special basins with capacities equivalent to about one minute of flow, in which small propellers are driven by electric motors. Sometimes the hydraulic jump, or standing wave, is used for flash mixing, being provided by a channel with sloping and widening sections. The coagulant is added just before the water flows down the channel at high velocity to enter a level portion of the channel, where the energy of rapid flow is suddenly transformed into static head of deeper water, turbulence being produced at the wave front of the deeper water. In other instances turbulence is provided by aerators, weirs, or spiral-flow tanks, but flow in channels used to conduct the coagulant-treated water to flocculation basins is not sufficiently turbulent for flash mixing unless obstructions are placed in the channels below the point where the chemical is applied. Efficient low-lift centrifugal pumps do not provide turbulent flow and thus do not serve as flash mixers.

Flocculation basins are of various types, some using patented equipment. Good results have been reported with very simple types consisting of a weir for flash mixing, with provision for subsequent gentle agitation in the channel leading from the weirs to the sedimentation basins. Early basins were fitted with a series of baffles around the ends of which the flowing water was reversed in direction, thus causing more gentle turbulence in the channels formed between the baffles, but more violent agitation at each point of reversed flow. The same effects were secured by arranging the baffles so that the water flowed over and under them alternatively. Such basins are cheap to build, as the baffles may be of ordinary lumber placed in concrete basins. They are only moderately successful, however, because the degree of agitation is determined by the space between the baffles, the total length of the channels so produced, and the volume of flow. While the spacing of the baffles may be altered, this is a major operation and usually is restricted to the correction of initially faulty design. Furthermore, as the degree of agitation decreases with the volume of flow, the water is less effectively flocculated during the period of low flow; and serious loss of head results from turbulence at each point of change in direction together with friction due to the area of the baffles and of the bottom of each channel between the baffles in contact with the flowing water. For instance, a basin of this type with an average velocity of 0.3 m/s (1 ft/s) would require a channel length of 18 m (60 ft) for each minute of flocculation period. Therefore even periods of 5 min, provided by channels 90 m (300 ft) long, would entail serious loss of head, especially at smaller plants where the friction losses are proportionately higher due to a larger surface (" wetted perimeter ") per unit volume of water. These basins therefore seldom are large enough to provide the flocculation periods of 15-30 min favoured by modern practice.

Baffled flocculation basins are being superseded by those fitted with mechanical agitators or paddles, or using either diffused air or the jet action of the flowing water to secure controlled agitation.

Mechanical agitators, or flocculators, consist of revolving paddles with horizontal or vertical shafts, or of paddles suspended from horizontal oscillating beams and moving up and down as the beams are driven by a crank attached to a speed-reducing unit, in turn driven by an electric motor. The total area of the paddles usually is 10%-25% of the area swept by their movement, so that the paddles move at about twice the speed imparted to the water—that is, the paddles move through the water at about one half their average speed of movement, causing eddies to form around the edges of the paddles and thus imparting agitation. The speed-reducing units usually provide for variable speed, so that the degree of agitation may be regulated to secure best results. The degree of agitation is thus controlled, independent of the rate of flow of water. The maximum peripheral speed should be about 0.6 m/s (2 ft/s) so as to provide an adequate range of speeds below that value.

A less well-known type of flocculation basin is that using the jet action of the flowing water to cause agitated flow. For instance, one large basin or *a series of several smaller basins may have their inlets and outlets located* so as to cause spiral flow. A square plan for the basins creates resistance to this spiral flow, and hence agitation. The degree of agitation may be controlled by adjustable sluice gates on the inlets, to change the area and hence the velocity of the entering water. For instance, a series of four square basins, each having a capacity equivalent to 5 min flow, would provide 20 min flocculation. The sluice gate of the first basin would be adjusted to provide the desired initial degree of agitation, say, 0.45-0.60 m/s (1.5-2.0 ft/s), to secure flash mixing. The sluice gate of the second basin would be adjusted to provide a velocity of about 0.36 m/s (1.2 ft/s). The growing floc in the third and fourth basins would justify the use of lower velocities of 0.2-0.3 m/s (0.6-1.0 ft/s) so that the floc will not be broken by over-agitation.

The loss of head in these basins is much less than with baffled basins, and the degree of agitation may be selected as needed and also maintained during periods of low flow by partly closing the sluice gates. These basins provide the same flexibility as basins with mechanically operated baffles, without the cost of the mechanical equipment. Power requirements, as represented by the small pumping head required to compensate for the small loss of head, should be no greater than the power used to operate paddles.

The use of compressed air in flocculation basins, so as to combine aeration with flocculation, should follow aeration practice, but the basins should have sufficient capacity to provide the required flocculating period. Velocities as such cannot be controlled, but the degree of agitation may be controlled by the amount of air released through the diffusers.

Controlling factors

Degree of agitation

Experience has shown that flash mixing requires pronounced agitation for a brief period to mix the coagulant and favour the initial chemical reactions associated with coagulation. Subsequent flocculation, however, should be aided by controlled agitation represented by velocities of flow or of paddle speed of 0.2-0.6 m/s (0.6-2.0 ft/s). Velocities below 0.1 m/s (0.3 ft/s) permit sedimentation of the floc when it should be in suspension aiding in the adsorption of smaller floc, whereas velocities over 0.6 m/s (2.0 ft/s) will prevent the growth of readily settling floc. In general, best results are secured with intermediate degrees of flocculation secured with velocities of about 0.3-0.4 m/s (1.0-1.4 ft/s). Higher velocities are favoured for turbid waters and lower velocities for coloured waters or those of low turbidity.

Detention period

Flocculation should be continued for periods of 10-30 min, and even longer periods have been used. In general, the longer periods permit lower coagulant doses and the use of a lower degree of agitation to ensure the formation of large floc easily settled. Conversely, when detention time is short, the degree of agitation should be increased. A period of only 5 min would require agitation at a rate of 0.6 m/s (2.0 ft/s); this extreme, however, is not advocated.

Temperature of water

The reduced rate of chemical and physicochemical reactions with lower water temperatures requires that the flocculating period be longer or that larger coagulant doses be used. Any given velocity of flow or speed of paddles will cause greater agitation of colder waters, however, because of their high viscosity. Economy therefore dictates the use of flocculating periods of 15 to 30 min, with provision for a wide range in degree of agitation, for treatment of cold waters.

Correction of inadequate flocculation

Difficulties in securing effective flocculation with existing equipment may result from *(a)* wrong coagulant dose or unsuitable coagulating chemicals, or *(b)* ineffective means for flocculation. Both types of difficulty should be studied by use of the flocculation or jar test (see Supplement, page 312). A suggested procedure is as follows:

1. Make trial laboratory tests with graduated doses of alum or, if pH correction is needed, with graduated doses of alum and lime, remembering that the effective pH range with alum is 5.5 to 8.0. If these tests do not indicate an effective dose of alum, with or without lime, proceed to the second step.

2. Consider the characteristics of the raw water. If a coloured water of low alkalinity is involved, test six portions of the raw water, each dosed with 30 p.p.m. alum and with graduated doses of chlorine (hypochlorite solution), to determine whether prechlorination would aid coagulation, and if so, which of the chlorine doses is most effective.

3. If step 2 is not successful, repeat it using sulfuric acid, if the commercial grade of this chemical is available locally.

4. If, however, the water is relatively clear and of low alkalinity, repeat the tests with alum and graduated doses of powdered calcium carbonate or clay, to determine whether either of these will assist coagulation.

5. If the water is cold and small fragile floc is formed that settles poorly and is easily disrupted, conditions may warrant the use of activated silica with the alum, provided that technical staff and facilities are available for this sensitive process of aiding coagulation.

6. If the tests with alum are not successful and ferric sulfate is available, the tests should be repeated with this coagulant, especially if the water has a pH over about 8.5.

Good results are usually secured with alum provided that the optimum pH range for a specific water can be determined by these tests and that correct doses of alum, either alone or with lime or other auxiliary chemicals, are used in this range. In any case the effective chemical doses indicated by these tests would be used on a plant scale, with minor adjustments as needed to correct for plant-scale conditions. Usually chemical doses effective in the plant are somewhat lower than with laboratory-scale equipment.

The second major type of difficulty is failure to reproduce on a plant scale the good results secured with the flocculation test, showing that the degree of agitation and the periods used in the laboratory are more favourable than plant facilities provide. This is a common situation at many older plants. A suggested procedure is as follows:

1. Determine the approximate degree of agitation (for instance, by observing small pieces of paper thrown into the water) and the detention period provided by the plant flocculator.

2. Adjust the laboratory equipment to provide the same apparent degree of agitation of the raw water in the midmost jar or beaker of the series and lesser and greater degrees of agitation in the jars to the left and to the right of this middle jar. When the laboratory flocculating equipment cannot be adjusted to provide different degrees of agitation in the various beakers, repeat the test in a single beaker until all degrees of agitation have been tried.

3. Add the most effective coagulant dose, as indicated in the flocculation test, to all the jars.

4. Mix each portion at the selected varying degrees of agitation for the period of time estimated to be provided by the plant.

5. The results will indicate the influence of the degree of agitation. If they are conclusive, they will point to the needed plant changes. If step 4 does not produce conclusive results, continue with step 6.

6. Use the most effective coagulant dose in all jars and also agitate all the jars with what appeared in step 4 to be the most satisfactory degree of agitation, but for varying periods of time: the water in the first jar may be agitated for, say, 5 min, corresponding to the estimated inadequate flocculation period at the plant, and the portions of water in the other jars for 10, 15, 20, 25 and 30 min.

Suppose, for example, the portion of water flocculated for 20 min, with a degree of agitation estimated as equivalent to 0.3 m/s (1 ft/s), is effectively coagulated, with the formation of firm, easily settleable floc. If existing plant conditions provide only 5 min flocculation with inadequate agitation, the conclusion would be that plant facilities should be altered to provide 20 min flocculation at a rate of about 0.3 m/s (1 ft/s).

Under these circumstances, the plant would be studied in the light of the several procedures for providing agitation discussed previously. If funds are limited, one solution would be to construct *within the sedimentation basin*, at the inlet end, three square basins, each having a capacity equivalent to 5 min flow, thus giving with the existing facilities a period of 20 min for flocculation. These small basins could be built of ordinary lumber, as they will not be exposed to appreciable water pressure. Agitation could be by jet action of the flowing water. If funds are available to purchase mechanically operated flocculating equipment, a " stilling wall ", described in Chapter 6, would be constructed across the sedimentation basin at the point needed to provide the 15 min additional flocculation. Motor-driven paddle flocculators would be installed in this portion of the basin. Other expedients might be used, such as the installation of air piping and diffusers in a portion of the sedimentation basin created by a stilling wall.

The reduction of the available sedimentation period of 15 min by this procedure would not be serious, because the improvement in flocculation and the production of settleable floc would make the remaining portion of the sedimentation basin much more effective. For example, the usual sedimentation basin provides a period of 4 h. The stilling wall would isolate a volume equivalent to 15 min of flow, or only 6% of the total capacity of the sedimentation basin.

Aids to the control of flocculation

The routine supervision of flocculation is aided by a few simple procedures which supplement the more technical procedures, as follows:

1. The appearance of floc in water being flocculated and settled is evident when the sun shines on the water and one stands so as to observe light reflected by the floc. This, however, restricts such observations to favourable conditions of sunlight. Accordingly, an important adjunct to plant control is the placing of submerged lights near the outlets of flocculating and sedimentation basins to illuminate the floc at any time desired. The lights should have a shade or reflector to prevent the light rays from reaching the eyes of the observer except as reflected by the floc. Submergence should be 1.2-1.8 m (4-6 ft). If electric fixtures capable of being submerged are not available a shaded light may be located above water level, but this has the disadvantage that part of the light will be reflected by the water surface into the eye, making the floc less evident.

2. The degree of agitation in flocculating basins may be estimated by using handfuls of confetti which are thrown into the water to be carried with the eddy currents under observation.

3. Dyes may be used in place of confetti to show the course of eddy currents. The dye known as fluorescein or uranin is well suited to this purpose, as it creates a pale-green colour even in very small concentrations: 1 g of the powder in a litre of water produces a strongly coloured solution which will impart a noticeable green tint of an opalescent nature when added to a large volume of water under observation. After further dilution by diffusion, past the point where observations are made, the colour becomes unnoticeable to consumers ; in fact, the diluted dye is visible only when viewed through many feet of water, except at the point where it is added, before diffusion has occurred. Use only the amount needed to show the direction and intensity of currents incidental to the agitation under study.

The over-all objective of flocculation is the production of a practically clear water in which visible floc is suspended. Any remaining cloudiness of the water separating the floc particles indicates the presence of uncoagulated and unflocculated suspended solids or colour, which will not settle in the sedimentation basins unless delayed flocculation is favoured by adsorptive action of the settling floc as it passes downward through the water. The final check on flocculation, therefore, is the quality of the settled water flowing to the filters, as well as the quality of the filter effluent.

Laboratory control of flocculation

The routine control of flocculation is through plant observations supplemented by tests for turbidity, colour, pH and alkalinity. The laboratory tests, however, only apply to the water as sampled, to disclose the results currently being obtained. To determine the results that might be secured by altered practice, or to determine the effects of altered chemical doses, it is necessary to make coagulation and flocculation or jar tests when change in raw-water quality occurs, before making changes on a plant scale.

The coagulation and flocculation of water in laboratory equipment naturally is influenced by the same factors discussed above: the characteristics of the coagulant; the temperature, turbidity, colour, pH, and alkalinity of the water; and the period and degree of agitation. For simplicity the procedure given in the Supplement calls for the use of alum, with or without lime, as the coagulant. The same procedure would be used, however, with other coagulants. It may be found in the laboratory that it is desirable to add two or more chemicals in a different order from that used in the plant. For instance, alum usually is added first, followed by any lime dose needed for the coagulating reactions, which should not be confused with lime added to *filtered water* for corrosion control. But prechlorination doses may be more effective when added prior to the alum dose. The laboratory tests are simple means of checking such factors.

CHAPTER 6

SEDIMENTATION

Purpose

The purpose of sedimentation is to permit settleable floc to be deposited and thus reduce the concentration of suspended solids that must be removed by filters. The basins used for this purpose should not be considered coagulation basins, because both coagulation and flocculation should be completed in flocculation basins.

Principles

Water does not flow through basins as an undisturbed body but rather by irregular flow. Conditions in flowing-through basins, therefore, are quite unlike those prevailing in a vessel in which quiescent sedimentation occurs: the intent is to provide conditions in which the flow is as uniform as possible for a period long enough to permit the greatest practicable amount of the settleable solids to be deposited before the water reaches the effluent end of the basins.

The factors that influence sedimentation are: (*a*) size, shape and weight of the floc; (*b*) viscosity and hence temperature of the water: (*c*) effective average period available for sedimentation; (*d*) effective depth of the basins; (*e*) their areas; (*f*) surface overflow rate; (*g*) velocity of flow; and (*h*) inlet and outlet design. Each of these presents problems of design and operation. Sedimentation theory is complex and of little avail, because floc is not uniform and hence its basic sedimentation properties cannot be given quantitative values, and because the influence of eddy currents cannot be predicted. Hence the discussion of these six factors in relation to design and operation relies largely on experience.

Floc characteristics

Floc is characterized by its large bulk in proportion to its weight, although a large portion of the bulk may represent space occupied by water. Therefore floc will not settle as rapidly as a more dense solid of: he same weight but having much smaller volume and area. This is one reason why flocculation is directed to the production of dense firm floc rather than feathery floc. Properly formed floc should settle at much higher rates than are used in practice with conventional sedimentation basins. For instance, the conservative rate of 3 m (10 ft) in 4 h has been used because this provided factors of safety for periods of poor coagulation, or for the sedimentation of the finer feathery floc in cold waters. In any case, the surface overflow rate of a basin, as discussed below, should not be greater than half the rate of sedimentation; therefore the essential factor is the production of rapidly settling floc through adequate flocculation.

Viscosity of water

The viscosity of water has a marked influence on the rate of sedimentation; for instance, this rate at 30°C is 2.3 times that at 0°C. Therefore basins intended to settle cold waters must be designed to meet this situation. Such allowances may not have been made at existing plants, which explains why activated-silica treatment is used with many cold waters to produce a more dense, compact floc which will settle within the treatment limitations of existing plants.

Effective average period of sedimentation

The " detention period " in a basin is the time required to fill the basin at the normal rate of flow; for instance, a basin of 100 m^3 (22 000 UK gal; 26 400 US gal) capacity would provide a detention period of 4 h at a flow of 25 m^3/h (92 UK gal/min; 110 US gal/min). As noted previously, however, water does not move through a basin as a uniform mass but as an incoming current displacing water already present in a basin and thus producing complex currents and eddies. This may be demonstrated by adding a dye or salt for a brief period to the influent and observing the distribution of the material. The detention period provides a measure of the " flowing-through period " of a basin, that is, the average effective period; in fact, the efficiency of a basin is determined by the ratio of the observed flowing-through period and the theoretical detention period, a ratio known as the " efficiency of displacement ":

$$\text{Efficiency of displacement} = \frac{\text{flowing-through period}}{\text{detention period}} \times 100$$

This efficiency should be greater than 30%. The flowing-through period is measured as indicated later in this chapter under Laboratory Control.

Depth of basins

The area and depth of a basin determine its capacity and hence its detention period. Depth, however, is much less important than area.

Settling floc must be deposited from the flowing water before the water reaches the effluent end of the basin. Inasmuch as floc should settle at a rate greater than 0.6 m/h (2 ft/h), it could be removed theoretically in 1 hour in a basin 0.6 m (2 ft) deep. Such a basin, however, would require continuous sludge removal and would present problems of scour. Therefore in practice, basins with a 4-h detention period are given a depth of about 3 m (10 ft) over the zone intended for sludge deposits and storage. A total depth of 3.6 m (12 ft) is common for flat-bottom basins.

Area of basins

Conventional sedimentation basins with horizontal flow are most effective when the area is as great as practicable, that is, when the depth is the minimum noted above and the area and depth combined provide the necessary detention time. As noted, the depth is significant only in providing sludge storage space and maintaining the velocity of the overlying flowing water below scouring limits. The area of a basin may be related to performance in terms of the rate of flow of water through it, as expressed by the surface overflow rate, which is dependent solely on area and rate of flow.

Surface overflow rate

This is expressed in metres per hour or in unit volume per unit area per day. The established norms are based on the rate of sedimentation of floc. Well-formed dense floc settles at the rate of about 3 m/h (10 ft/h). The surface overflow rate should not exceed one-half of this value, i.e., 1.5 m/h upward velocity or 36 m^3/m^2/day (5 ft/h; 900 US gal/ft^2/day). This rate, when related to a basin depth of 3 m (10 ft), gives a detention period of 2 h.

Experience has shown that this maximum surface overflow rate is satisfactory only when coagulation and flocculation are adequately controlled and when coagulation aids are used when necessary to provide a heavy floc, especially in cold waters. In practice, therefore, factors of safety are provided by selecting lower rates. For instance, an upward velocity of 0.75 m/h or a rate of 18 m^3/m^2/day (450 gal/day/ft^2) provides a detention period of 4 h and a velocity of approach to outlet weirs low enough to avoid carrying fine floc over on to the filters, provided that the length of the outlet weir is selected accordingly.

Sedimentation-basin practice

The design of sedimentation units involves such factors as shape, 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 the 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 rectangular basins when they are long and narrow and relatively shallow, with the inlet at one end and the outlet at the other, so that the direction of flow is not reversed. The general principles given here, however, apply to basins of various shapes, including those with radial or circular flow.

Radial flow, from the centre of a basin, results in progressively larger unit volumes and hence in lower velocities of flow as the water passes outward to the outlet troughs on the periphery of the basin. This factor is beneficial, inasmuch as the velocity of flow is lowest near the outlet.

Circular flow in circular basins, however, with inlet and outlet troughs " back to back " on some selected radius, is not uniform or progressively diminishing, because of the varying lengths of the circular patterns of flow of the increments of water entering along the radial inlet troughs. Many such basins are in use, so the problem is to secure as effective sedimentation as possible. This has been accomplished by the use of radial walls extending from the centre for one-half the radius of the basin, or for the whole radius but with perforations in the outer half of the wall. Or the radial walls may be so located as to provide " over-and-then-under " flow, to minimize short-circuiting at any one level. One or more such walls have been used with improved results.

Circular structures of concrete are more costly to build and cover a larger ground area when two or more units are used. The trend is therefore towards rectangular basins; the details discussed below refer especially to this type.

Upward-flow basins of the various patented types, providing suspended-solids contact between the entering water and previously formed floc, are so different that they are discussed separately later.

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

The detention period should be at least 4 h and should be based on the maximum capacity of the plant, thus providing 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 3-4.5 m (10-15 ft) provide a reasonably short distance of travel of the settling floc so as to ensure its reaching the bottom before the flowing water reaches the effluent end of the basin. 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 already formed will be broken up. The influent pipe or flume should be proportioned to provide a velocity not greater than 0.45 m/s (1.5 ft/s) 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 0.12-0.24 m/s (0.4 to 0.8 ft/s) should be provided to minimize eddies and encourage uniform flow through the basin. These velocities necessitate restriction of the total area of the slots. 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 0.12 m/s (0.4 ft/s) when both basins are in use will prevent velocities higher than the upper value when only one basin is in use. Frequently the slots are 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 in the lower portion of basins or 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 middle two-thirds of the baffle, so that 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 1-3 m (3 to 10 ft) from the inlet end, depending on 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 3 m (10 ft) 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 desirable to provide an adequate total length of weir. Submerged weirs were favoured in the past, as they eliminated turbulence. The trend to great weir length, however, required true weir action to ensure uniform flow throughout collecting troughs or conduits. The maximum weir loading advocated in the past was 500 m^3/day per metre of length (33 000 UK gal/day or 40 000 US gal/day per foot), but values of 20%-40% of this figure are preferable for normal plant operation. More certain weir action may be secured with a series of V-notched weirs about 2½ inches deep and on centres 6-12 inches apart, rather than a great length of level weir with a very slight 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 0.15-0.30 m ($^1/_2$-1 ft) in the depth of a basin should provide adequate storage with coloured waters of moderate turbidity, but space equivalent to 25% of the volume available for settling is desirable for turbid waters.

A bottom slope of 1 in 12 towards a centre gutter, which in turn slopes to one end of the basin, facilitates the removal of sludge, but small basins may have flat bottoms. Provision 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 0.4 m/s (1.5 ft/s), and bends should be designed to avoid turbulence that would tend to destroy the floc remaining in the settled water.

Started as a means of preventing ice formation, the covering of settling basins also excludes light and thus will prevent the growth of algae on the walls.

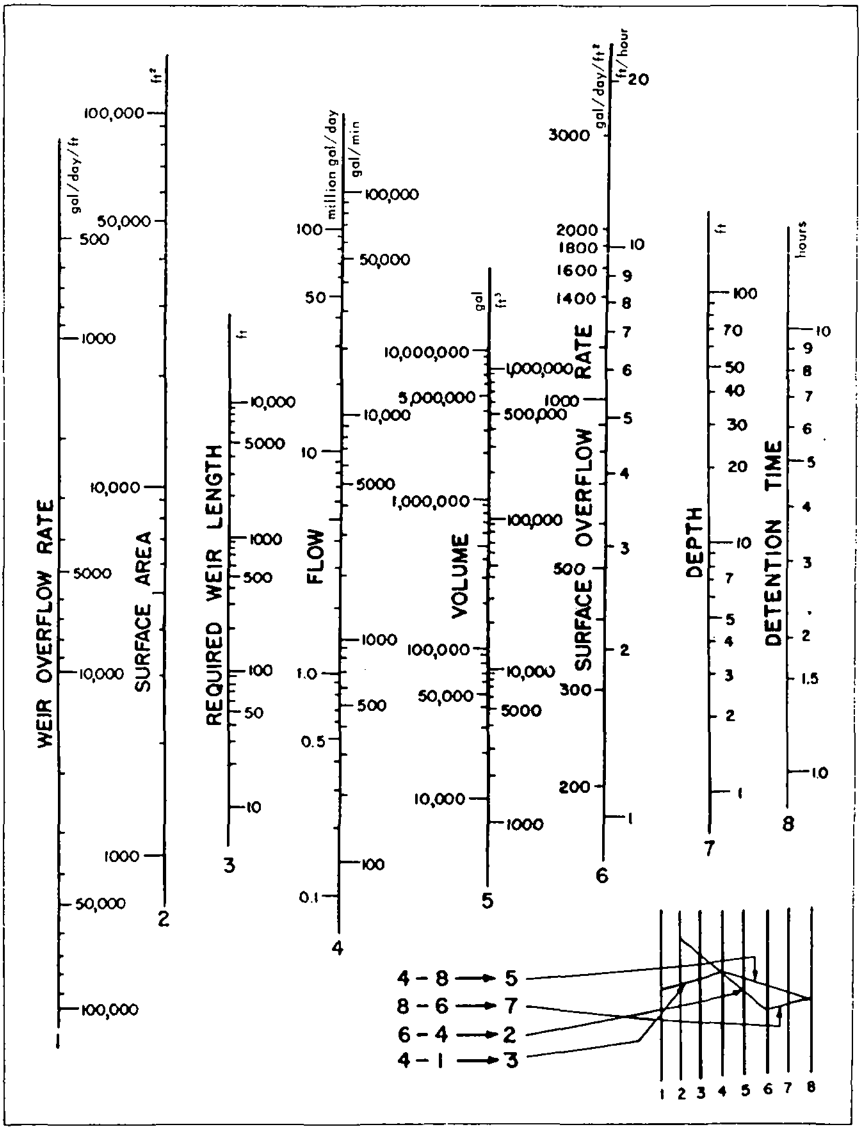
The design of a basin is based on: (*a*) the quantity of water to be treated; (*b*) the selected detention period; and (*c*) the selected surface overflow rate. Fig. 3 is convenient for determining the required dimensions of sedimentation basins. For example, suppose a filter plant of 3780 m^3/day capacity (834 000 UK gal/day; 1 000 000 US gal/day) is under design, and that a detention period of 4 h and a surface overflow rate of 18.3 m^3/day/m^2 (375 UK gal/day/ft^2; 450 US gal/day/ft^2) have been selected as the basis of design for the sedimentation basins. This gives a rate of upward flow of about 0.75 m/h. Then the nomogram indicates from the rate of flow and detention time that the volume is about 625 m^3 (165 000 US gal). The surface overflow rate and the rate of flow give a surface area of 206 m^2 (2220 ft^2). The detention time and surface overflow rate give a depth of about 3 m (10 ft) for the portion of the basins *above* the sludge storage space. The added depth of 25% for sludge storage would be about 0.6 m (2 ft), so the basins used in this example could be given a total depth of 3.6 m (12 ft). 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 206 m^2 (2220 ft^2) would require two basins of about 103 m^2 (1100 ft^2) each. A length of 16.8 m (55 ft) and a width of 6 m (20 ft) would provide 101 m^2 (1090 ft^2) and a length-to-width ratio of about 2.8 to 1. The total width of 10 m (40 ft) for the two basins gives a weir overflow rate, from the nomogram, of 12 m^3/h/m (25 000 US gal/day/ft; 20 800 UK gal/day/ft), which is acceptable. A slower velocity of approach of the water, however, may be secured by using a double-edged weir trough, located 1-1.5 m (3-5 ft) from the outlet end of the basin, thus providing twice the weir length, or 24 m (80 ft), and a weir overflow rate of 6 m^3/h/m (10 400 UK gal/day/ft); 12 500 US gal/day/ft).

Operation and supervision

The operation of sedimentation basins receiving well-flocculated water and having the normal design indicated above is quite simple. Difficulties with flocculation cannot be correct by sedimentation-basin operation, but

FIG. 3

NOMOGRAM FOR THE DESIGN OF SETTLING-TANK CAPACITIES AND DIMENSIONS



From Cox (1960) p. 17 by permission of the US Agency for International Development, Washington, D.C.

when the basins have an ample capacity and time is available for delayed flocculation in the basins, an operator has more opportunity to correct coagulant doses. The operating procedures summarized below are designed to give the information needed for corrective action.

Baffles and stilling walls

Many basins have no provision at their inlets for evenly distributing flocculated water, such as an influent channel with openings spaced across the basin. This is serious, as high entering velocities tend to destroy well-formed floc and lead to short-circuiting or turbulent flow. A transverse baffle may be installed to dissipate the energy of a concentrated inlet current, but the best procedure is to use a transverse baffle and also to install a stilling wall about 3 m (10 ft) from the inlet end of the basin. As previously described, surface currents and sludge scour may be minimized by placing the slots in the middle third of stilling walls.

If the perforations are too large the walls will not impose sufficient resistance to dissipate eddy currents and secure uniform distribution; if too small, they will create jet action and defeat the purpose of the wall. As already noted, good results have been secured with perforations having a total area such that the average rate of flow through them will be 0.12-0.24 m/s (0.4-0.8 ft/s). The lower value is suggested, as it will ensure a maximum rate of not over 0.24 m/s (0.8 ft/s) when all the flow is through one of two basins, during periods when the other basin is being cleared. An example would be the two basins referred to above, with a total flow of 4000 m^3/day (0.88 million UK gal/day; 1.06 million US gal/day), a width of 6 m (20 ft) each, and a depth of 3.6 m (12 ft) at the inlet end. The stilling walls then would be 6 m (20 ft) wide by 3.8 m (12.5 ft) deep, allowing for a freeboard of 0.15 m (0.5 ft). A flow of 4000 m^3/day equals 46.3 l/s (1.63 ft^3/s). A selected velocity of 0.12 m/s (0.4 ft/s) would require an area of perforations of 3900 cm^2 (4.2 ft^2 or 604 in^2) for this flow rate. This total area could be divided into 70 of these openings, or 35 per basin, each having an area of about 56 cm^2 (8.6 in^2) and arranged in two rows of 12 and one row of 11 in each basin. The lowest row would be one-third the wall height from the bottom, that is, 1.2 m (4 ft); the other two rows would be 1.5 and 1.8 m (5 and 6 ft), respectively, above the bottom.

Stilling walls are usually of concrete when installed in new plants, but as they operate under very low head they may be constructed of lumber. For instance, boards 30 cm (12 in) wide, 5 cm (2 in) thick and 3 m (10 ft) long could be used in the small basins mentioned above. They would be laid on edge and supported at their ends by 15- × 15-cm (6- × 6-in) uprights. The perforations would be formed by cutting rectangular notches about 5 cm (2 in) deep and 11 cm (4 in) long along the edges of selected boards. They would be spaced 0.45 m (18 in) apart, centre to centre, starting 0.3 m (1 ft) from each side of the wall.

Special baffles

Many existing basins have centrally located dividing walls around which the water flows in a 180° turn and then returns to effluent weirs at the same end as the inlet facilities. This plan has the advantage of eliminating long effluent conduits, but the reversing of flow creates eddies or cross currents and dead areas of little flow. The observation of floats will disclose the general path followed by the water in such basins. Floats are easily made by attaching two pieces of sheet metal, 120×60 cm each, bent at the centre at right angles and attached in the form of a cross, to one end of a 5- × 5-cm (2- × 2 in-) pole about 2 m (6 ft) long, to which weights are attached as needed to cause the device to float upright in the water, with about 45 cm (18 in) of the pole projecting out of the water. The float will follow the currents at the depth of the sheet-metal vanes rather than surface currents, and it will not be unduly influenced by wind action.

Such floats frequently have shown that the flowing water tends to be concentrated in a short-circuiting current around the end of the separating wall, the quiescent water at the corners of the basin not being displaced. This may be corrected by installing a special wooden baffle at right angles to the end of the central wall in the influent half of the basin. This baffle should have a length about 20% of the width of the first half of the basin. It serves the purpose of diverting the flow towards the corner of the basin, where dead areas may exist. Other arrangements will be suggested by the observed paths of the floats.

Effluent weirs

One of the most common weaknesses of existing sedimentation basins is the inadequacy of effluent weirs. Many plants have been constructed with weir rates of 20 m^3/h per metre of length (31 700 UK gal/day or 38 000 US gal/day per foot) or greater. This creates high velocities of approach of the settled water towards the weirs, so any unsettled floc is drawn upward over the weirs. Rates of 5-10 m^3/h/m (9000-18 000 UK gal/day/ft; 10 500-21 000 US gal/day/ft) may be secured in existing basins by installing wooden flumes or troughs about 1.8 m (6 ft) from the outlet of the basins, giving each edge of the trough the elevation needed to serve as a weir. This low flow requires that the weirs be level so as to ensure distribution of the water over their full length. Distribution is facilitated by using 90° V-notches about 15 cm (6 in) apart and 5 cm (2 in) in depth.

If double-edged weirs cannot be installed for structural reasons, a short existing weir may be lengthened by installing wooden troughs on each side of the basin at right angles to the old weir and discharging into existing effluent channels. The length of these weirs should not be over 20% to 30% of the basin length on each side, otherwise they may collect partly settled water.

Depth of sludge

Operators should determine the distribution of sludge in sedimentation basins and gauge its depth. This can be done conveniently by attaching a wooden board about 30 cm (12 in) square at right angles to the bottom of a calibrated rod, so that the board is horizontal when the device is lowered into the water. Experience will indicate when the board reaches soft sludge and also the firmer sludge underlying the more recently deposited material. The depth of sludge is the difference between the depth of the basin at the point under investigation and the depth of submergence of the device when it encounters the sludge.

Sludge removal

Sludge should be removed from basins when the lower portion reserved for sludge storage is filled, otherwise the upper layer of unsolidified sludge will be subject to scour by the overlying water, incidental to reduced space above. Furthermore, sludge containing algae and other organic matter will be subject to putrefaction, especially when prechlorination is not practised, and so may have to be removed before the sludge storage space is completely filled. When basins are fitted with mechanical sludge-removing equipment, as at larger plants treating waters of high turbidity, the sludge is of course removed frequently or continuously.

Sludge removal is facilitated by sloping the bottom of small basins to their inlet end, where the heaviest sediment collects, or by sloping the bottom to central collecting channels, which in turn slope to drains, at the inlet ends of basins. The opening of the plug valve on the drain of a basin will discharge the water with sufficient force to flush sludge from the vicinity of the drain. Remaining sludge should be flushed towards the drain with a hose stream. This requires the use of 2 to $2^1/_2$-in fire hose with pressure of 3.4-4.0 atm (50-60 lbf/in^2) to secure ample force with the stream of water.

Sludge containing organic matter, such as precipitated colour or algae, will decompose when water temperatures exceed about 15°C, frequently imparting objectionable tastes and odours to the overlying water. This may require the frequent removal of such sludge or the practice of prechlorination or the periodic use of activated carbon. Algae and slime growths on the walls of open basins may be minimized by coating that portion of the walls extending from above the normal flow line to a point 0.6 m (2 ft) below the water level with a mixture of copper sulfate and lime, as described in Chapter 3, page 27. Troublesome growths of algae in the water in open sedimentation basins may justify prechlorination of the raw water.

Covered sedimentation basins

Covers for sedimentation basins exclude sunlight and hence prevent algae growths and also avoid trouble from ice. Such covers complicate supervision, however, unless there are adequate manhole openings for convenience in inspection. Therefore the use of open basins in tropical and subtropical climates to minimize structural costs is warranted, provided that the precautions described above are followed.

Suspended-solids contact basins

Manufacturers of suspended-solids contact basins in a number of countries have developed patented details including proportions of the units, so designs are not standard and only the basic principles outlined above apply.

The fact that such basins provide in one unit mixing, flocculation, floc conditioning, sedimentation and manual or automatic sludge removal makes them attractive. Their special feature is flocculation in the presence of previously formed floc; the upward rise of water through the settling floc that constitutes the so-called " sludge blanket " provides contact between the settling floc, moving downward, and the smaller residual floc moving upward with the water. This favours the formation of larger floc through the agglomeration of the fine material. The purpose is to secure a floc that will settle at a rate twice that of the rate of upward flow of water, so that separation occurs and the floc does not rise much above a level 1.5 m (5 ft) below the effluent weirs. These processes are facilitated by application of the coagulant to the water being flocculated, rather than to the raw water, and in many instances by the recirculation of the flocculated water through the zone of agitation.

The development of such units for lime-soda softening, where the sludge is heavy, led to upward-flow rates of about 3 m/h (10 ft/h), which resulted in basins with a total period of about 1 h. This means that the floc produced must have a settling rate of 6 m/h (20 ft/h) to meet the norm of twice the rate of upward flow of the water. The suitability of these units therefore depends primarily upon the assurance that flocculation will be adequate for the purpose, otherwise the basically sound principles and economy of size will be negated.

Since alum floc does not settle so rapidly as calcium carbonate, a lower rate of flow may be used with coagulation—e.g., one-half the above, or 1.5 m/h (5 ft/h), to provide a factor of safety. This will provide a total period of 2 h for the units (equivalent to 750 UK gal/day/ft^2; 900 US gal/day/ft^2) or about half the period for conventional basins.

The selection of such units in preference to conventional flocculation and sedimentation basins should be based on the ability of the operating staff, variability of flow characteristics of the raw water including its fluctuations in quality, ease of flocculation, relative costs of extra chemicals to secure rapid sedimentation, and relative costs of construction and maintenance.

Operation

The control of these special basins must follow the instruction books prepared by the individual manufacturers.

The rising water below the level of clarification should have a solids content of 0.5%-2.0% by weight, thereby favouring flocculation by contact between previously formed floc and that in the water flowing into this zone. The concentration of solids is maintained by a balance between the fresh floc being produced and that which settles into the sludge-removal zone, from which it is withdrawn at frequent intervals. About 2% of the water flow is used for sludge removal.

The upper level of the suspended floc is called the " slurry level ". Its position can be determined by lowering a waterproof electric light into the water until the floc begins to obscure the light. The problem is to so control coagulant dosing and sludge removal that effective flocculation occurs and the slurry level does not rise above a safe limit, say, 1.5 m (5 ft) below the effluent weirs. This is very difficult with low water temperatures and when colour and low turbidities are being removed. Therefore clay or calcium carbonate are frequently used to weight the floc, or activated-silica treatment is followed to produce a denser floc with alum.

In general, such basins require close attention so that the coagulant doses may be changed without delay when raw-water quality or rate of flow varies or when poorly formed floc or " floc boils " are observed in the clarification zone.

Laboratory control

Sedimentation is controlled by the observations outlined above, supplemented by tests for turbidity and colour of the settled water and by the appearance of the residual floc. It is very important to determine whether poor results are due to faulty flocculation or to poor sedimentation, but usually it is the former. The basic criterion is the effectiveness of preparation of the water for filtration. This is disclosed by the length of filter runs, the ease of washing filters and the quality of the filtered water produced (see Chapter 7).

The turbidity of properly treated settled water should be under 10 p.p.m. and preferably under 5 p.p.m. Higher turbidities usually are associated with the cloudiness imparted by uncoagulated solids and not with turbidity due to unsettled floc. In fact, a good effluent should have no apparent colour and turbidity, in so far as the water between the residual floc particles is concerned, the particles floating in clear water to produce apparent turbidity. This is disclosed by submerged lights, as noted previously.

The salt test for flowing-through period

The use of dye to trace the passage of water through a basin is a simple procedure but does not give quantitative results unless a precise colorimeter is available for the comparison of samples and standard solutions containing known amounts of the dye. The procedure here described presents a picture of basin performance which is not made evident by the usual observations. The only technical weakness is the possible influence of the higher specific gravity of the salt solution as compared to that of the water, which may be disregarded in practice.

Principles

The principle of the test is based on the rapid application of salt to the influent of the basin under study and the determination of the concentration of salt in the effluent at frequent intervals. Naturally, the first appearance of salt in the effluent represents the time of arrival of a portion of the salt-treated influent. This may occur in 10-20 min, but a portion of the salt-treated influent will be shown to be leaving the basin long after the lapse of the theoretical detention period. Fluctuations in the content of salt in the effluent disclose erratic flow or short-circuiting portions of the influent. The concentration of salt is determined by measuring the content of chlorides in appropriate samples of the influent and effluent.

Procedure

1. Add the weight of salt needed to provide a concentration of *chloride* of about 300 p.p.m. in the influent flowing past the point of application in 1 min. The chloride content of salt is 60% by weight, so 500 p.p.m. of salt will give the 300 p.p.m. of chloride desired.

For instance, water flowing at the rate of 4000 m^3/day (0.88 m.g.d. UK; 1056 m.g.d. US) represents a flow of 2778 l (612 UK gal; 734 US gal) per minute. Then 500 p.p.m. in this volume requires a dose of 1.39 kg (3 lb) of salt, computed by multiplying the dose required by the weight of water passing in 1 min: $500 \times 2778/1\,000\,000 = 1.39$ kg; $500 \times 10 \times 612$ (UK)/$1\,000\,000 = 3$ lb; $500 \times 8.34 \times 734$ (US)/$1\,000\,000 = 3$ lb. This weight of salt is dissolved in about 40 l (10 gal) of water, and this solution is poured into the influent at a point where good mixing is secured, as in the influent flume.

2. Begin at once to collect a series of samples of *both* the influent and the effluent, collecting the first sample at the moment the salt is first added to the influent and timing successive collections at the suggested intervals of 5, 10, 15, 20, 30, 40, 60, 80, 100, 120, 150, 180, 210, 240, 300, 360 and 480 min.

3. Determine the chloride content of the two series of 18 samples of influent and effluent.

4. Plot the chloride content of the samples on plotting paper. The horizontal scale should represent elapsed period of time in minutes between collection of samples. The chloride content of each sample is plotted at a point corresponding to the elapsed time when that sample was collected, and these chloride values are represented by the vertical scale.

5. Connect the plotted points by two lines, one for influent samples and one for effluent samples.

6. Carefully cut out the area between the two lines with a pair of scissors.

7. Determine the centre of gravity of this area by carefully balancing the piece of paper on a level knife edge *so that the knife edge is parallel to the vertical lines marking the time periods.*[1]

8. Observe the elapsed-time period represented by the position of the knife edge.

The above is illustrated by Fig. 4, giving the results of the testing of a basin with a theoretical detention period of 4 h, or 240 min. It will be noted (line A) that the chloride content of the influent dropped from the applied dose of 300 p.p.m. to under 5 p.p.m. in 10 min; then there was a gradual increase to about 10.0 p.p.m., due to delayed diffusion of some of the salt. More important, the chloride content of the effluent (line C) reached the high value of 32 p.p.m. in about 30 min, then a series of lower values, and then another high value of 32 p.p.m. in about 80 min, followed by some fluctuations and a gradual fall to below 10.0 p.p.m. in about 460 min. These values indicate the passage of two short-circuiting currents through the basin in 30 and 80 min. The centre of gravity of the area between the two curves A and C was found, by balancing the paper representing the area between the two curves, to be equivalent to 125 min (line B), which is the desired flowing-through time. The displacement efficiency is calculated as flowing-through period/detention period: 125/240 = 0.52 or 52%. These results show a generally efficient basin but one in which improved eddy control is needed. This could be obtained by installing a perforated stilling wall.

Guides for the evaluation of displacement efficiency are as follows:

Ideal basins	63% efficient
Good basins	30%-50% efficient
Poor basins	5%-30% efficient

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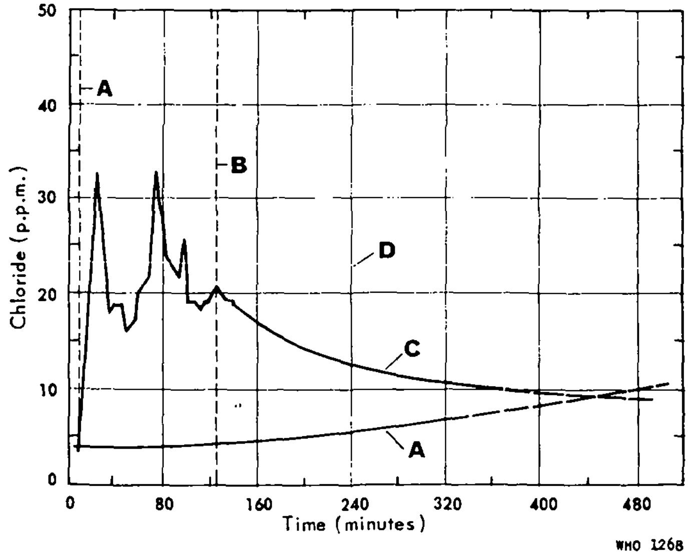
[1] The centre of gravity may be determined by computation by taking moments on the vertical axis.

FIG. 4

GRAPH FOR DETERMINING FLOW-THROUGH PERIOD IN SEDIMENTATION BASIN



A. Influent, showing initial drop in chloride content and subsequent concentration

B. Flow-through period as determined

C. Effluent

D. Theoretical detention period

Modified from American Water Works Association (1950) p. 180 by permission of the Association.

CHAPTER 7

FILTRATION

There are four types of water filters, which are sufficiently different to require separate discussion. It is essential that the selection of any one type be based on consideration of local factors as well as the advantages and disadvantages of each type.

Objectives

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

Slow sand filters should be so designed 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 adsorption of impurities in the layer and in the underlying sand is facilitated. This requires a large area and slow rates to ensure surface contact and adsorption.

Rapid sand filters should be designed to receive coagulated and settled water; in these filters colloidal material and bacteria are adsorbed on the gelatinous floc and are removed with the floc. Adequate pretreatment, 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 on the inherent characteristics of the type of filter involved.

Slow sand filters

The *advantages* of slow sand filters may be summarized as follows:

1. There is no need for coagulation facilities.
2. Equipment is simple and need not be imported.

3. Suitable sand is readily secured.
4. Supervision is simple.
5. The effluent is less corrosive and more uniform in quality than chemically treated waters.
6. They give effective bacterial removal.

The *disadvantages* as compared to rapid sand filters are as follows:

1. A large area is required, with correspondingly large structure and volume of sand and higher structural costs.
2. They have less flexibility in operation.
3. They are not economical with raw waters having turbidities over about 30 units for prolonged periods, unless preliminary plain sedimentation will secure such turbidities in the settled water.
4. They are less effective in removing colour.
5. They give poor results with water of high algal content, unless pretreatment is practised.

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.

Pretreatment

Ordinarily, slow sand filters are used without pretreatment. Plain sedimentation prior to filtration may reduce the turbidities of many raw waters below 30 units and thus permit their economical filtration without the cost and technical complexity of coagulation.

Prechlorination of water to be filtered through slow sand filters was formerly 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 prechlorination, so as to destroy these organisms, as well as to control algal growths on the surface of the sand. This is of great practical significance when filter runs are influenced more by the clogging effect of algae than by suspended solids.

Consideration should also be given to pretreatment with copper sulfate when algae are sufficiently prevalent to seriously reduce the filter's run. Inasmuch as a continuous dose can be less than that commonly used periodically, the chemical may be applied by a small solution feeder at a dose of 0.15 p.p.m. Such pretreatment is aided when a presedimentation basin is available, but application directly to the filter influent is acceptable.

Design details

At least two filters should be provided, and three 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 growth of algae, but this greatly increases costs of construction. Headroom of 1.8 m (6 ft) should be provided to facilitate scraping. The cover should be provided with enough openings to provide daylight for scraping and to facilitate removal and replacement of the sand.

Rates of filtration of 2-5 m/day (2-5 m.g.d./acre) 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 m/day (3.1 m.g.d./acre UK; 3.7 m.g.d./acre US) results in a satisfactory compromise between economy in structural costs and economy 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 these filters that manual control is feasible, provided that an orifice meter, 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, that is, a loss of head exceeding the positive head produced by the depth of water over the sand, otherwise air-binding will occur. The depth of water over the sand therefore limits the available loss of head and in turn the length of filter runs. Accordingly the depth of water should not be less than 1.2 m (4 ft), and consideration should be given to 1.8 m (6 ft) of headroom under any filter cover to provide a depth of 1.5 m (5 ft) of water and 0.3 m (1 ft) of freeboard.

Each filter should be equipped with a main drain and a sufficient number of lateral underdrains to collect the filtered water. Tile pipe with open joints is commonly used. Drains should be so spaced that the maximum velocity of flow does not exceed 0.22 m/s (0.75 ft/s). The maximum spacing of underdrains should not exceed 3 m (10 ft), and there should be none within 0.6 m (2 ft) of the side walls, so as to prevent water flowing in the space between the sand and the concrete wall and thence into the gravel without passing through the 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 is acceptable: a 15-cm (6-in) layer passing an 80-mm (3-in) screen but held on a 10-mm ($^3/_8$-in) screen: a 5-cm (2-in) layer passing a 25-mm (1-in) screen but held on a 10-mm ($^3/_8$-in) screen; a 5-cm (2-in) layer passing a 10-mm ($^3/_8$-in) screen but held on a 5-mm ($^3/_{16}$-in) screen. The gravel should be placed over the underdrains but not within 0.6 m (2 ft) from the side walls of the filter, so that only sand will rest on the filter bottom in the 0.6 m (2 ft) 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 fine sand grains. Washing may be accomplished by the use of home-made equipment or by special equipment using the jet action of a stream of water. The size of sand for slow sand filters 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 " (see page 104) should not be less than 0.2 mm or greater than 0.4 mm, and usually should be about 0.3 mm. The sand need not be so uniform as with rapid sand filters, but the uniformity coefficient should be within the range of 1.7 to 3.0, preferably no greater than 2.5.

Operation

A slow sand filtration bed, which has been scraped recently of the accumulated deposits on the surface, should be placed in operation as follows: Filtered water should be allowed to flow through the proper valves and piping from other filters into the underdrains of the scraped unit until this water rises through the sand to a depth several inches over the surface. The raw water then can be turned on to the filter without danger of the surface being scoured by the entering water. The filter is then operated to filter water at low rates until a film has collected on the surface of the sand. This requires several days, so the best procedure is to increase the rate of filtration gradually during a period of four to seven days before the maximum rate is reached. This period of " conditioning " filters will have to be lengthened when one containing new sand is first placed in use. Furthermore, the water from beds of new sand being conditioned should be wasted until the quality of the effluents indicates that the sand has developed the necessary film.

The maximum rate of filtration during normal operation should be maintained until the loss of head approaches the depth of water on the sand bed, the depth usually being about 1.2-1.8 m (4-6 ft). This loss of head should not be exceeded, otherwise the filter will be operated under a " negative head ", which is undesirable because the mat on the surface of sand is likely to be ruptured, or the filter may become air-bound through the release of air dissolved in the raw water.

Rate of filtration

Generally speaking, the effectiveness of filtration is proportional to the depth and fineness of the filter sand and inversely proportional to the rate of filtration. With a sand depth of 0.9 m (3 ft) the rate of filtration should not exceed about 3.7 m/day (3.3 m.g.d./acre UK; 4 m.g.d./acre US), otherwise some of the bacteria will penetrate through the filter and be present in the effluent. Lower rates, therefore, should be used with filters of lesser depth. The best criteria as to permissible rates for specific filters, however, are the results of the bacteriological examination of samples of the filtered water. If the rates of filtration cannot be reduced because of demand for water, the only recourse is the construction of additional filters in order to produce a final effluent of safe sanitary quality, following final chlorination.

Scraping filters

When the accumulation of material in the sand is such that the loss of head reaches the permissible maximum, filtration should be stopped to permit the filter to be scraped. The filter may also be cleaned by the use of special sand-washing machines, which move over the surface of the sand on caterpillar treads or on tracks provided for the purpose, although in most cases the filters are scraped by hand. The filter should be drained to a point sufficiently far below the surface to permit walking on the sand. The amount of sand that has to be removed from the surface of the filter depends on the depth to which the majority of the suspended solids penetrate. Ordinarily from 0.5-2.5 cm ($^1/_4$ in to 1 in) of the surface sand is scraped by the use of flat shovels and placed in the hoppers of water ejectors, which force the mixture of sand and water to the sand-washing equipment. Occasionally the dirty sand is thrown into conical piles on the sand bed until after the filter is cleaned several times, when the sand in the piles is removed for washing. This latter procedure is not advocated, because it decreases the filtering area and thus may lead to surface clogging owing to high rates of filtration in the restricted area. The usual procedure is to continue to scrape the filters until the remaining depth of sand is about 0.8 m (2.5 ft), after which about 15 cm (6 in) of washed sand is replaced in the filters to restore the normal depth.

If the water applied to slow sand filters contains relatively large volumes of fine suspended solids, or if these filters are operated at excessive rates of filtration, subsurface clogging may result. This should be avoided in every way possible, because such clogging necessitates the removal of most, if not all, of the sand for cleaning, in order to restore normal operating conditions. Partial clogging may necessitate the removal of only the upper portion of the sand. Clogging is disclosed by a progressive increase in the loss of head when a scraped filter is returned to operation.

Washing sand

The dirty sand removed from the surface of slow sand filters may be discarded and replaced with fresh, clean sand, or it may be washed and reclaimed in special sand-washing machines. Economy dictates the use of such equipment. The sand is agitated by jets of water to remove the adhering material, the muddy washing water being discharged to waste. Sand ejectors, previously mentioned, and sand-washing equipment may be purchased ready for use. The effective washing of sand requires the use of a sufficient volume of water under pressure to ensure the maximum degree of agitation without loss of the fine sand grains. If the process is not carefully supervised, however, the fine material may be removed and the average size of the sand increased above the desired point.

Laboratory control

Laboratory control of slow sand filtration plants consists in examination of the sand at intervals and determination of turbidity and colour and of content of bacteria and micro-organisms in the applied and filtered water. Samples of sand should be secured at intervals from various depths, dried, and a mechanical analysis made to indicate the percentage of fine material accumulating in the bed due to possible subsurface clogging. Ineffective bacterial removal or subsurface clogging is not readily apparent and may occur even when the effluent is clear. These plants should not be neglected just because the effluent seems to be satisfactory.

Rapid sand filters

Filters designed to operate at a much higher rate than slow sand filters are called " rapid sand filters ". Another name is " mechanical filter ", because the original units had mechanically operated rakes to assist in agitating the sand during the washing process.

Filters of this type are usually open structures and water flows through the sand by gravity, hence another name, " gravity filters ". Other filter units are enclosed in steel tanks and may be operated under pressure, hence the name " pressure filters ". Diatomaceous-earth filters are in a separate category. Each will be discussed separately, the text following being devoted to gravity filters.

The various units of rapid sand filters should be designed so as to be co-ordinated 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 one another. 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.

Rate of filtration

The normal rate of filtration is 4.8 m/h (16 ft/h; 2 US gal/min/ft^2) of filter area. Any proposed rate in excess of this should be considered in the light of all aspects of the plant. Rates range from 2.4 to 12.0 m/h (8-40 ft/h; 1-5 US gal/min/ft^2). The lower rates provide factors of safety but naturally result in the need for larger filters. The choice of the design rate thus requires the use of judgement, weighing the factors noted below.

In general, filters should be designed so as to provide means for removing residual floc from properly pretreated water, and also to serve at times of poor or delayed coagulation as contact beds to ensure 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 pretreated waters. Lower rates of filtration, finer sand and greater depth of sand favour 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 factors should be considered in connexion with filter rates, in the light of local conditions and their interrelationship:

(1) the training and reliability of the operator and the resulting reliability of the pretreatment in assuring an effectively coagulated and settled water rrespective of changes in the quality, temperature, etc., of the raw water;

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

(3) the designed capacity of the plant;

(4) 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;

(5) the quality of the raw water and the degree and rapidity of the fluctuation in quality;

(6) the ease of coagulating, settling and filtering the supply;

(7) the size and depth of filter sand;

(8) the effectiveness and reliability of disinfection.

High rates of filtration are discussed in this monograph where appropriate, but always with reservations based on the above factors.

At least two filter units should be provided, and consideration should be given to designing each unit to take the full load at twice the normal rate 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 1.2 m (4 ft) of water over the sand so as to discourage air-binding and ensure reasonable length of filter runs. The minimum total depth of the filter structure should be at least 2.5 m (8.5 ft) and preferably 3 m (10 ft).

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 that the water does not impinge against a concrete wall and thus break up the residual floc.

General

The filter structure should be designed as a unit, with due consideration of the interrelationships of the quality of the raw water, its range in temperature, the selected rate of filtration, the depth and size of sand, the rate of flow of wash water required to secure 50% sand expansion, and the height of the wash-water troughs above the sand. Fig. 5 presents a nomogram for co-ordinating dimensions of filters with rates of filtration, and Fig. 6 a nomogram for relating wash-water requirements. These factors are discussed separately below. The procedure may be summarized 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 designed rate of filtration.

3. Determine from Fig. 6 the required rate of flow of wash water, based on 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 1 min. To prevent this distance being exceeded, select a wider but shallower trough, with its lower surface just above the expanded-sand level.

Deep filter basins are recommended so as to ensure long filter runs without the development of serious negative heads.

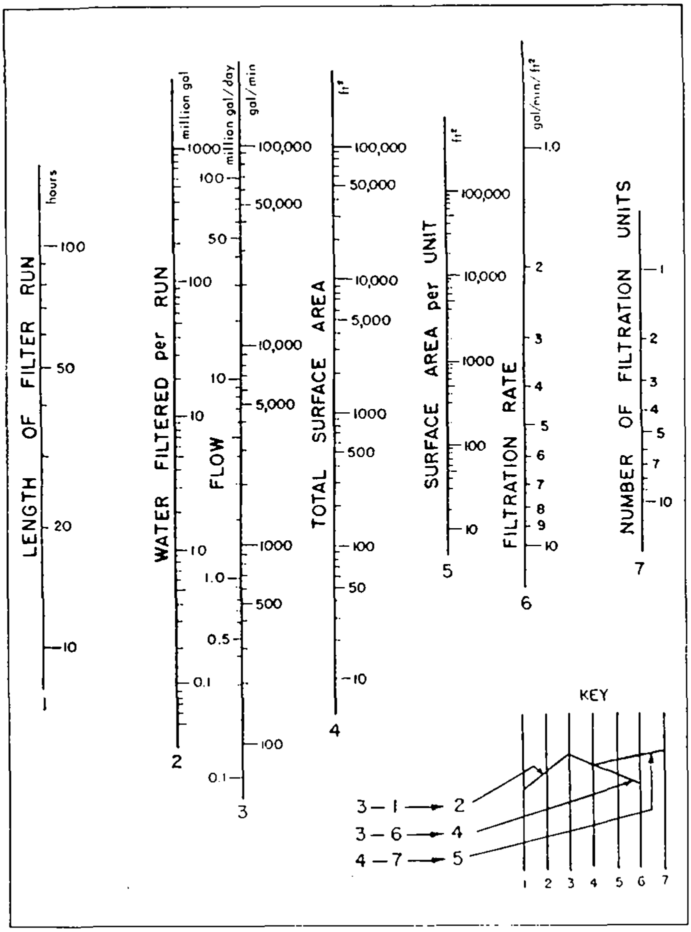
The filter bed

Specifications for sand

Filter sand should be selected with care, with due regard to the anticipated effectiveness of pretreatment, the rate of filtration and the depth of sand. When conditions warrant the selection of a unit rate of filtration of

FIG. 5

NOMOGRAM FOR DIMENSIONS OF RAPID SAND FILTERS



From Cox (1960) p. 33 by permission of the US Agency for International Development, Washington, D.C.

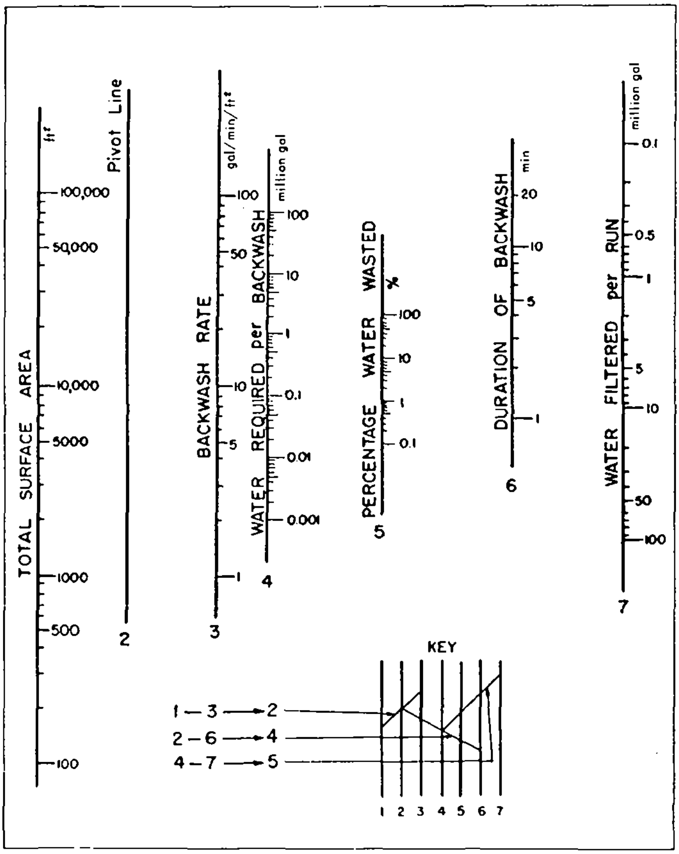
4.8 m/h (8 ft/h; 2 US gal/min/ft^2), the sand may have an effective size in the range of 0.40-0.50 mm to secure reliable filtration with reasonable filter runs. If higher rates of filtration than noted above are deemed feasible and effective pretreatment is assured, the sand should be coarser, having 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 0.75-0.90 m (30-36 in). The minimum depth of sand should not be less than 0.6 m (24 in). The sand should be as uniform as obtainable, the uniformity coefficient being no greater than 1.8.

The filter sand should preferably be supported by a 7.5-cm (3-in) layer of coarse sand, having an effective size between 0.80 and 2.0 mm, placed over the gravel.

The sand bed is the heart of the filtration plant and thus the sand must be selected with great care. Ordinary " run-of-bank " sand should never be used for filter beds, as it almost invariably contains a mixture of grains of various sizes and frequently contains large quantities of foreign material and silt. The sand should be free from clay, dust, roots, and other impurities. It should contain less than 2% lime and magnesium calculated as carbonates and soluble in dilute hydrochloric acid in 24 h at a temperature of 70°F.

FIG. 6

NOMOGRAM FOR WASH-WATER REQUIREMENTS FOR RAPID SAND FILTERS



From Cox (1960) p. 35 by permission of the US Agency for International Development, Washington, D.C.

The efficiency of filtration through sand depends to a great extent on the size of the grains. Although the finer sands are more effective in the filtration of water, they have a higher frictional resistance to the passage of water and thus cannot be used economically except in slow sand filters, where the rate of filtration per unit area is about one-fortieth that obtained in rapid sand filters. Moderately fine sand beds may be used in rapid sand filters but must be washed more frequently than coarse sand beds, and even then they are more difficult to maintain free from mud balls and clogged areas.

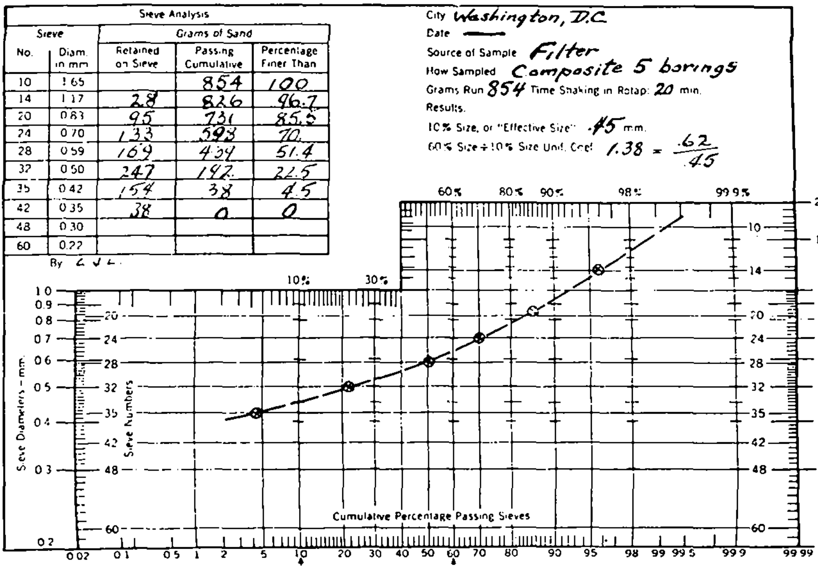
The various sizes of sand grains in a given sample are determined by passing a weighed quantity of sand through graded sieves of successively smaller mesh sizes and then weighing the portions of the original sample which pass through each successive sieve. Special Tyler sieves are available in the USA for this purpose, each having a successively smaller mesh. These sieves are so shaped that they will fit together and thus form a single enclosed vessel. The sample of sand is placed in the top compartment over the largest mesh screen, and the group of screens is then agitated in a special machine or by hand until the various sand grains pass through as many sieves as their size will permit. The weight of the portions of the sample passing any given screen is then calculated in percentages of the weight of the sample used in the test. The information so secured is plotted on special forms, and the resulting curve furnishes accurate information as to the sample. The firms selling filter sand make this test of their product, and they are in a position to furnish sand according to specifications.

" *Effective size* " *and* " *uniformity coefficient* ". The finer sand grains are of special importance in filtration. In any given sample, the " effective size " is that size of grain such that 10% of the grains by weight are smaller and 90% are larger, as determined by test. Fig. 7 is a reproduction of an actual sieve analysis of 854 g of sand, the results of which are shown in the table in the upper-left-hand corner. The right-hand column gives the computation of the " percentage finer than ", that is, the percentage of the sample which passes through each sieve because it is finer than the sieve openings. These percentage values are plotted on the special double-log paper, as shown. It will be noted that the curve crosses the 10% mark on the " Cumulative percentage passing sieves " at a point corresponding to a sieve size of 0.45 mm. This, then, is the effective size of the sample of sand.

Another characteristic of sand which is considered of great importance in water filtration is the uniformity of the size of the grains; it is considered desirable to have filter sand more uniform in size than most natural sand. Such uniformity is indicated by the " uniformity coefficient ", which is defined as the ratio of the sieve size passing 60% of the sand to the sieve size passing 10% of it. Thus in Fig. 7, since 60% of the sample is finer than 0.62 mm and the effective size is 0.45 mm, the uniformity coefficient is 0.62/0.45, or 1.38.

FIG. 7

TYPICAL SIEVE ANALYSIS



Sieve Analysis

Sieve		Grams of Sand		
No.	Diam. in mm	Retained on Sieve	Passing Cumulative	Percentage Finer Than
10	1.65		854	100
14	1.17	28	826	96.7
20	0.83	95	731	85.5
24	0.70	133	598	70.
28	0.59	169	439	51.4
32	0.50	247	192	22.5
35	0.42	154	38	4.5
42	0.35	38	0	0
48	0.30			
60	0.22			

By

City Washington, D.C.

Date —

Source of Sample Filter

How Sampled Composite 5 borings

Grams Run 854 Time Shaking in Rotap: 20 min.

Results.

10% Size, or "Effective Size" .45 mm.

60% Size ÷ 10% Size Unif. Coef. $1.38 = \frac{.62}{.45}$

From American Water Works Association (1950) p. 258 by permission of the Association.

Standard Specification for Filtering Material, B-100-53, 1950, of the American Water Works Association, gives another method of specifying sand size in which all sizes of grains are appraised and not just the single " effective size ". This procedure specifies the percentage of the sand which shall be finer than stated grain sizes. Therefore, if the percentages are selected in connexion with the separation sizes of the standard sieves, the results of the sand-screening test can be used without plotting.

The specification mentioned above includes Table 5, showing typical ranges in sand grain sizes for fine, medium and coarse sands, based on the above procedure for specifying sand.

Many prefer to specify sand in terms of percentage by weight that is *retained* on a specific numbered sieve. For instance, a medium sand with good uniformity would be specified by the values shown in Table 6. The sand having these characteristics would on an average have an " effective size " of about 0.50 mm and a uniformity coefficient of 1.45, but the important point is that the stated limits are in sieve sizes and not in values secured by plotting.

British practice describes both particle size and uniformity in terms of the sieve meshes that will pass and retain the total weight of sand in the sample undergoing examination. A common British specification is that a filter sand shall pass a 16-mesh and be retained on a 30-mesh sieve, the

TABLE 5

TYPICAL GRAIN-SIZE DISTRIBUTION BY PERCENTAGE PASSING VARIOUS SIZES OF SIEVE

Sieve number		Sieve opening, mm	Effective size—% passing sieve		
Tyler	US		Fine (0.35-0.45 mm)	Medium (0.45-0.55 mm)	Coarse (0.55-0.65 mm)
14	16	1.19	94-100	84-99	68-93
20	20	0.84	71-97	49-84	30-71
28	30	0.59	31-73	14-39	6-31
35	40	0.42	6-25	2-6	0-1
48	50-	0.30	0-3	0-1	0

From *J. Amer. Water Works Ass.*, **48**, No. 7, p. 806, July 1956, by permission of the Association.

meshes used being in accordance with British Standard 410, 1943. This would be approximately equivalent to an effective size of 0.6 mm and a uniformity coefficient of 1.2.

There are many areas where sand cannot be purchased by specification. Under these circumstances the available sand should be screened through a coarse screen to remove foreign matter, gravel, etc. It is then placed in the filter and backwashed thoroughly. Very fine unwanted sand is thereby washed to waste, and the coarser grains collect over the gravel. It may be necessary to remove the fine sand which has not been flushed to waste but which is finer than desired. This is done by scraping the fine sand from the

TABLE 6

SPECIFICATIONS GIVEN IN PERCENTAGE RETAINED ON VARIOUS SIZES OF SIEVE

Sieve number		Sieve opening, mm	% retained on sieve	
Tyler	US		Minimum	Maximum
14	16	1.19	0	1
20	20	0.84	0	9
28	30	0.59	40	60
35	40	0.42	40	60
48	50	0.30	0	9
65	70	0.21	0	1

Adapted from Riddick, T. M. (1940) *J. Amer. Water Works Ass.*, **32**, 121, by permission of the Association.

upper few inches of the bed with a shovel. Backwashing at higher than the intended normal rate for the plant will remove most of this fine sand, reducing the amount to be removed by shovel.

Specifications for 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 38-60 cm (15-24 in), 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 that directly support the sand.

The following ranges in sizes and corresponding depths of gravel are suggested for rapid sand filters:

Range in size			*Range in depth*	
in	*mm*		*n*	*cm*
2½-1½	65-38		5-8	13-20
1½-¾	38-20		3-5	8-13
¾-½	20-12		3-5	8-13
½-$^3/_{16}$	12-5		2-3	5-8
$^3/_{16}$-$^3/_{32}$	5-2		2-3	5-8
		Total depth	15-24	39-62

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 should preferably be about 0.003 to 1.0, except with special bottoms. For instance, a filter with an area of 100 ft^2 would be provided with underdrain openings having a total area of 0.3 ft^2. The total cross-sectional 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 are also acceptable. These may be of brass, cast iron or asbestos cement, the latter, with brass nozzles to prevent erosion of the openings, being very satisfactory.

False bottoms of concrete with porcelain, plastic or metal strainers spaced about 20 cm (8 in) apart simplify the distribution of wash water. An alternative is to use a porous-plate filter bottom, i.e., one made 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. Clogged plates may be cleaned with 2% caustic soda solution.

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 wash water to enter the lower end of the strainer when agitation with air is followed by the water wash.

Filter-washing facilities

The earliest procedures for washing rapid sand filters consisted of one of two methods. In the first, 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. In the second, compressed air was used to secure agitation. High-velocity wash systems were developed later followed by surface-wash equipment, which provides agitation without an unduly high rate of flow.

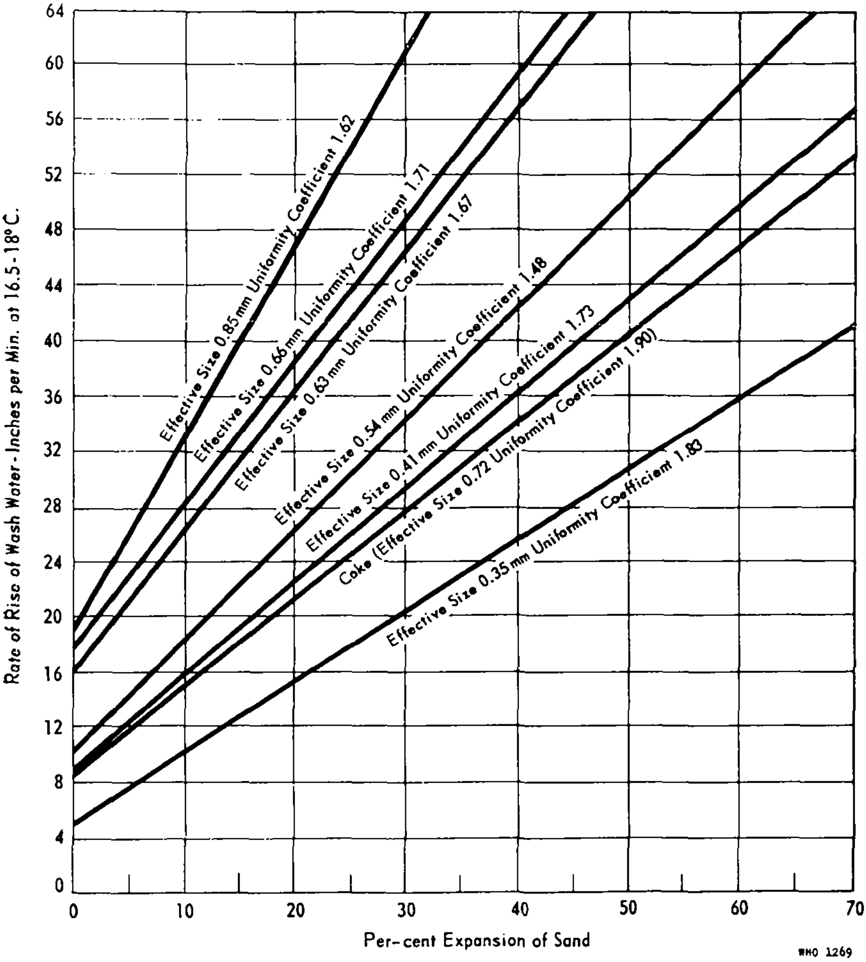
High-velocity wash system

Unfortunately the old practice of using wash-water rates of 36 m/h (15 gal/min/ft^2), or 24 in per minute of rise, without supplementary air, 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 use air-water wash systems, or to base the hydraulics of wash-water tank and piping on 50% sand expansion, so that there will be a factor of safety. The trend is also towards the use of coarser sand. As a result, 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.

Fig. 8 shows the relationship between sand expansion, sand size and required rate of rise of wash water. This basis gives 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 45 m/h (30 in/min). The rate for 50% expansion and sand of 0.54-mm effective size is about 70 m/h (47 in/min), these values being needed with water temperatures of about 20°C. In summary, modern practice is

FIG. 8

PER CENT EXPANSION OF SAND



From Cox (1960) p. 29 by permission of the US Agency for International Development, Washington, D.C.

to design wash-water systems to produce rates up to 56 m/h (36 in/min). (Allowances for high water temperatures in the tropics justify the use of maximum rates up to 72 m/h (48 in/min).

A wash-water pump, or the more economically operated wash-water tank together with a small pump capable of filling the tank in less than 8 h, should be provided, with sufficient capacity to wash one filter for at least 10 min at the designed rate of flow of wash water. Larger pumps or tanks are needed when more than two filter 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 trough to provide the head needed to produce the maximum rate of flow of wash water used in the design. A wash-water flow regulator or manually controlled butterfly valve should be installed on the main wash-water pipe, so that 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 securing 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.

Surface-wash system

Some of the more recently constructed water-purification plants have been fitted with the so-called " surface wash " developed by Baylis, in addition to the conventional wash-water system. The pipe grid through which the surface wash is applied consists of horizontal header pipes suspended on the wash-water troughs or filter superstructure. The horizontal headers are connected to secondary pipes extending vertically downward to within about 10 cm (4 in) of the surface of the *quiescent* sand bed. The bottom ends of the vertical pipes are perforated with holes about 2.5 mm ($^3/_{32}$ in) in diameter at an angle 30° below the horizontal. The vertical pipes are spaced about 60-75 cm (24-30 in) centre to centre. Pressures of 0.7-2 atm (10-30 lbf/in^2) are used. This equipment may be installed in existing filter beds and affords an inexpensive method of improving defective washing facilities.

Filter-bed agitators

Another method of improving the degree of agitation of the sand grains during the washing process is through the use of the Palmer filter-bed agitator, developed for this specific purpose. This agitator consists of a revolving pipe, suspended at its centre by a suitable bearing, to which a number of nozzles are attached. One or more of these units are located in filter beds about 2.5 cm (1 in) above the normal surface of the sand. Wash water under a pressure of at least 2.7 atm (40 lbf/in^2) is forced through the nozzles during the washing process. The nozzles are directed downward about 30° from the horizontal, the resulting jet action causing the arms to revolve up to 18 r.p.m., depending on the length of the pipe. In this way circular areas somewhat greater in diameter than the length of the arms are traversed and the sand is subjected to agitation by the water jets. These agitators may be installed at existing filtration plants without modifying the original construction and thus provide a means of securing adequate agitation of the sand beds when structural limitations prevent most other corrective measures being used.

Air-wash system

The operations with the air-wash system are similar to the above except that compressed air is used to secure effective scrubbing action with a lower volume of wash water. The air may be forced through the underdrains before the wash water is introduced, or, preferably, the air may be introduced through a separate piping system placed between the gravel and the sand layers. With the former procedure about 0.9-1.5 m^3 of free air per minute per square metre of filter surface (3-5 ft^3/ft^2) is forced through the underdrains until the sand is thoroughly agitated, after which wash water is introduced through the same underdrains at a rate of 19-29 m/h (63-96 ft/h; 8-12 US gal/min/ft^2). With the latter procedure, air is forced, in about the same volume, through separate piping at the *same time* water is forced through the underdrains. A perforated pipe grid for distributing air may be installed above the gravel in existing plants where higher-velocity wash with water alone cannot be provided because of hydraulic limitations or because the wash-water troughs are less than 0.6 m (2 ft) above the sand beds, and sand would be lost if greater agitation were produced solely by increasing the rate of rise of wash water rather than by use of water and air. There is a trend away from air-wash systems because of the poor results secured with a limited volume of wash water. This may be remedied by increasing the capacity of wash-water pumps to provide a flow of water sufficient to remove the material loosened from the sand grains by the air. The air-wash system permits the use of less gravel than noted for high-velocity water wash, so that a graded bed 0.3 m (12 in) in depth is adequate.

Mechanical rakes

Circular filters equipped with mechanically revolving rakes are washed in the usual manner, but at the same time revolving rakes or agitators are forced through the sand bed. The speed of the rakes should be increased until the normal rate of 10-12 r.p.m. is reached. This rate should be continued until large clear areas begin to appear in the rising wash water, at which time the motion of the agitator should be reversed and the agitator raised above the surface of the bed while the wash water is still flowing. The wash-water valve is then closed. The modern practice is to use the high-velocity wash without mechanical rakes, but long years of use have demonstrated the effectiveness of rakes in preventing clogged areas, mud balls, etc., even with wash-water rates as low as 24 m/h (80 ft/h; 10 US gal min/ft^2).

Wash-water troughs

Wash-water troughs should be spaced so that each serves the same unit of filter area. The maximum *horizontal* travel of wash water preferably should not be over 0.9 m (3 ft), that is, the edge of a trough next to a wall should be spaced 0.9 m (3 ft) from the wall, and adjacent troughs should be 1.8 m (6 ft) 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 0.25 m (30 in) is becoming standard, the lower surface of the trough should be not less than 0.43 m (17 in) above the sand surface to permit 50% sand expansion without having the sand grains hit the troughs.

The width and depth of a trough is determined by the capacity required 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 ensure free fall of water into the whole length of the trough. The rate of flow of wash water is determined as noted below.

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 1 min. At 20 US gal/min/ft^2, this is 0.8 m (2.7 ft). Thus an elevation of the wash-water trough of 0.9 m (3 ft) above the sand will provide freeboard for washing with water of lower temperature than 18°C, or will permit washing at rates greater than 48 m/h (160 ft/h) with warm waters.

Rate controllers

The rate of filtration may 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 are usually fitted with rate indicators and even recorders. Such complete units, however, are expensive and are difficult to repair. Their use is therefore usually 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 (Venturi) flume must be used on the raw-water piping, unless reasonably constant-capacity raw-water pumps are to be used, or unless pump capacity is measured by the flume and is subject to manual control by a butterfly valve. Furthermore, settled water must be equally distributed to each filter, irrespective of the elevation of the water surface in each filter, by using weirs or orifices on the filter-influent piping.

The operation of filters at self-controlled decreasing rates is being advocated by some, thus dispensing with rate controllers. This is referred to below.

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 plants. 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.

Valves and piping

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 are generally preferable for plants where large-size valves are not needed. Quick-opening plug valves are available for wash-water drains of 8 to 12 in.

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. Such facilities, however, aid in chemical treatment of the sand.

In general, filtered-water and wash-water piping should be designed to provide a velocity of flow not greater than 0.9-1.8 and 2.4-3.6 m/s (3-6 and 8-12 ft/s), 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.

Operation

Rate of filtration

The factors controlling the rate of filtration were reviewed above as problems of design. Operators, however, must consider these same factors in terms of the plant they must operate to secure the most economical and satisfactory results.

The rate of filtration should be selected to provide the required volume of water with the lowest possible rate of filtration. This should be accomplished by adjusting the rate controllers of all the filters, rather than by shutting off one or more filters, when the consumption decreases.

Rate controllers should be given protective maintenance so that they function as intended. Certain types are fitted with diaphragms, which respond to pressures from Venturi meters and in turn control valves. These diaphragms develop leaks at times and then must be replaced. Follow the instruction manual accompanying the equipment.

If for some reason rate controllers are not available or cannot be made to function properly, three courses are open to the operator:

First, float-operated butterfly valves may be installed, provided that the rate of flow of the influent is constant and the flow can be equalized to all the filters.

Second, the rate of flow of the influent may be equalized to all the filters, but without the use of float-operated butterfly valves. In this way the rate of filtration equals the rate of entrance of water, but, inasmuch as a recently cleaned bed will filter at a rate in excess of the inflow, the water level drops to a point where the lowered head is equal to the loss of head. As the latter increases through partial clogging of a filter, the head of water will increase likewise, until it reaches the maximum level for the plant. Thereafter the rate of filtration will decrease as the filter becomes more clogged, until washing is necessary.

Third, a recently advocated procedure may be followed, allowing the rate of filtration to be controlled solely by the loss of head, without any restrictions on the rate of flow of filter influents. This means that filters will operate at a high rate when first washed, and that the rate will gradually decrease as the loss of head increases. Rates of filtration between 12 m/h (40 ft/h; 5 US gal/min/ft^2) and less than 4.8 m (16 ft/h; 2 US gal/min/ft^2) result, giving maximum rates higher than would be desirable at many plants, but correspondingly higher average rates. The important consideration with this third procedure is that flocculation must be satisfactory at all times. If it is, then floc penetration during high rates of filtration will be only into the upper portions of clean filters, and progressively lower rates will prevail as the penetration increases. The filters must be washed before floc " break-through " occurs. This procedure can be used only under those conditions where high-rate filtration is feasible.

If flocculation is not sufficiently effective and reliable to justify the third procedure, a compromise between the last two procedures is feasible. The influent valve is used to throttle the rate of flow of the influent to a filter, as guided by the rate-of-filtration indicator, so as to avoid excessive rates of filtration through recently washed filters. Eventually the influent valve will be open, and then the third procedure is followed. This compromise has the weakness of requiring detailed attention by operators, which may not be practicable when a number of filters are involved.

The second procedure is recommended when rate controllers are not available, provided that operators are able to install influent weirs to equalize flow to all filters when the total flow is reasonably uniform, as when low-lift pumps are used. One procedure is to install a 90° bend on influent pipes to serve as circular weirs, so that the level of water on the several filters may differ without diverting more water to the filters with lower levels.

In general, the use of rate controllers is advocated, as this permits complete control of the rates, within the hydraulic limitations of a plant.

The rate of filtration may be measured by closing the influent valve temporarily and then measuring the distance the water surface falls in 1 min. This measurement can be converted into the volume of water filtered in the selected time per square foot of filter area.

Washing filters

Filters should be washed when the loss of head reaches 2.4-3 m (8-10 ft). Longer filter runs may be obtained when washing is delayed until the rate of filtration begins to decrease, showing that the rate controller is fully open and that the loss of head has reached a point where it is decreasing the flow below the normal rate. This depends on the depth of water over the sand, that is, the positive head.

Washing filters requires more attention than just operating certain valves; the time during which the valves are opened or closed and the rate of operation are important. The following steps have been found to give the best results:

1. Close the influent valve but continue the filter in operation until the water level drops to a point about 15 cm (6 in) above the sand. This conserves the settled water that would otherwise be discharged into the drain and also permits inspection of the filter sand.

2. Close the effluent valve.

3. Inspect the filter for mud balls, cracks, mounds, or evidence that the sand is drawing away from the side walls.

4. Open the drain valve. This may be done any time after the water on the filter is below the wash-water gutters.

5. *Open the wash-water valve slowly*, that is, open it gradually during a period of 45 to 60 s to the point where sand expansion is about 15%, or flow is about 24 m/h (80 ft/h; 10 US gal/min/ft^2). This prevents wash water from entering so fast that the compacted surface sand is lifted as a body until a portion cracks and causes high horizontal flow towards the crack, this flow carrying the cleaner bottom sand and pea gravel with it.

6. Wash at the low rate for about 3 min, to provide ample time for the sand grains to impinge on one another and dislodge the floc.

7. Open the wash-water valve to the point that gives the maximum sand expansion desired, such as 40%, and continue for about 1 min or until the wash water appears to be relatively clear and the agitated sand can be observed.

8. Close the wash-water valve.

9. Close the drain valve.

10. Open the influent valve slowly, so as to avoid undue turbulence and disturbance of the sand.

11. Open the influent valve wide when the water level reaches normal.

12. Open the effluent valve.

The surface-wash system is used as follows: Open the surface-wash-water valve to permit the flow of 5-20 m/h (16-64 ft/h; 2-8 US gal/min per square foot of filter area). Water pressures of 0.7-2.1 atm (10-30 lbf/in^2) are needed. Open the main wash-water valve to give a rate of 24 m/h (80 ft/h; 10 US gal/min/ft^2).

The jets of water of the surface wash impinge on the agitated sand and greatly increase scrubbing action without increasing the total flow of wash water to a point where sand would enter the troughs. Close the surface-wash valve before the main wash-water valve is closed.

Filter-bed agitators are operated as follows: Wash water is introduced through the underdrains in reduced amounts to secure a sand expansion of only 15% to 20%. Water is then forced through the nozzles of the agitators, causing them to revolve through the expanded sand and creating effective agitation, again without the danger of loss of sand.

Rate of flow. Washing causes the sand grains to impinge on one another and thus dislodge adhering floc, the rising wash water carrying the material into the gutters. An inadequate rate of flow will not secure either effect. On the other hand, an unduly high rate of flow will cause more expansion than needed, so that the sand grains will be separated further and scrubbing action will be decreased. Surface-wash and filter-bed agitators secure increased agitation by horizontal movement of the sand grains, so the vertical movement is not sufficient to cause loss of sand.

The practice of " bumping " a filter bed by opening the wash-water valve for a few moments, to loosen the packed material on the surface and thus prolong filter runs, may be necessary during periods of emergency when temporary difficulty in coagulation is experienced, but this practice is objectionable.

It may be difficult to wash filter beds effectively with the available rate of flow of wash water. This difficulty may be due to a number of causes, such as the use of finer sand than desirable, imperfect preliminary treatment, high organic content of the raw water, the growth of slime organisms in the sand, the improper distribution of the wash water (especially around the edges of the beds), or insufficient velocity of wash water. In addition washing may be more difficult in summer than in winter. In the past this has been attributed to the presence of algae or to the higher organic content of the raw water during the summer, but it appears that the significant factor is the decreased viscosity of the water in summer, which results in less agitation of the sand grains for any given rate of wash-water flow, in contrast to the more viscous cold water of winter.

There are probably many existing filtration plants where it would be possible to increase the rate of flow of wash water in summer without washing sand into the troughs. Many new filtration plants are being constructed with rate-of-flow indicators connected to the wash-water lines and with capacities equal to a maximum of 72 m/h (240 ft/h, 30 US gal/min per square foot of filter area, or a 48-inch vertical rise of wash water per minute). With such facilities it is possible to develop a schedule of washing based on the seasonal temperature of the wash water.

The adequacy of wash water used in amounts of 1 % to $2^1/_2$ % of the total amount of water filtered each day indicates effective pretreatment and filter operation. Amounts in excess of $2^1/_2$% indicate the need to investigate for corrective action.

Sand expansion. As explained, the criterion for selecting the rate of wash is sand expansion, that is, the increase in volume of the sand bed resulting from a passage of wash water upward through the sand. Experience has shown that effective washing occurs when this expansion is about 40% of the depth of the quiescent bed. A conventional sand depth of 0.75 m (30 in) is equivalent to a depth of 1.05-1.12 m (42-45 in) during the washing operation. Undue sand expansion is detrimental because the sand grains are so far apart as not to impinge on one another to the degree required for effective scrubbing action. The actual expansion during the washing process may be readily measured through the use of a special tool, made as follows:

Attach about seven small metal cups, approximately 25 mm (1 in) in diameter and 6 mm (¼ in) deep, to a rod. These cups should be spaced at intervals of 50 mm (2 in). A cross-piece should be attached at right angles to the rod at such a point that when the rod is lowered vertically and the cross-piece rests on the top edge of a wash-water trough, the cups are held 15-45 cm (6-18 in) above the level of the quiescent bed; that is, the cups are held 15-45 cm (6-18 in) below the cross-piece for conventional filters having troughs 60 cm (24 in) above the sand beds.

This rod is used by lowering it into the bed being washed until the cross-piece rests on the top of one of the wash-water troughs. The rod is then carefully withdrawn, while the washing is still in progress, to observe the highest cup in which sand has collected. The relative elevation of this cup indicates the maximum elevation reached by the sand grains during the washing process, from which the percentage of expansion may be computed. For instance, if the cup 35 cm (14 in) above the normal sand level contains sand and the cup 40 cm (16 in) above does not, the expansion is 37.5 cm (15 in), or 50% of the normal depth of sand.

The upper elevation reached by the sand grains during the washing process can also be observed by lowering a board painted white into the rising wash water during the latter part of the washing process when the waste wash water is relatively clear and the upper limit of the agitated sand may be observed directly. This board should be calibrated in centimetres or inches for convenient use.

It should be remembered that the degree of expansion of sand beds being washed depends on the size of the sand grains as well as on the viscosity of the water and the rate of flow of the wash water (Fig. 8, page 109).

The rate of flow of wash water may be measured, in the absence of meters, by timing the rate of rise of the wash water from a point 15 cm (6 in) below the edge of a gutter until it overflows into the gutter.

Care of sand in filter beds

The following details must be borne in mind in the care and supervision of the beds, because there are occasions when the usual washing of the beds must be supplemented by other procedures. Careful note should be made of the appearance of the sand beds by lowering the water level sufficiently for the purpose before washing. Also note the initial loss of head when a washed filter is first placed in operation. The distribution and appearance of the wash water should be observed during the washing period to ascertain whether washing is effective throughout the total area of each bed, and also whether the aggregations of solids called " mud balls " are being thrown into suspension during the washing operation.

The presence of mounds and craters in sand beds indicates that the underdrains or strainers are broken, thus permitting rapid rates of filtration and washing over a restricted area and penetration of sand into the gravel or upheaval of the gravel into the sand. Clogged air lines, underdrains, or strainers lead to dead areas in the filters, the presence of which is indicated by a quiescent area when the bed is being washed. Broken or clogged underdrain systems should be repaired at once, otherwise the bed will not function correctly and eventually more costly repairs will be required.

Never permit water to drop from the wash-water troughs directly on to the sand surface. If for any reason the water has been drawn below the sand surface, introduce filtered wash water through the underdrains until the water stands well above the sand surface before permitting the unfiltered water to flow on to the bed.

Mud balls. The presence of mud balls in the sand indicates incomplete washing. This may be due to insufficient rate of flow of wash water or to the use of too short a washing period. The mud balls may be removed by scraping the surface of the sand with shovels, or at times by the use of a piece of 6-mm (¼-in) wire mesh about 30 cm (1 ft) square, properly supported and attached to a long handle. This device is passed through the upper portion of the sand bed being washed, to skim or strain the mud balls from the finer sand. The very small balls will not be removed by this device, nor is the procedure as certain as the scraping of the bed with shovels. After the removal of mud balls, the beds should be washed more effectively to prevent their recurrence.

As previously pointed out, it may be possible to improve existing washing facilities by the installation of surface-wash systems or by installing separate air-wash piping between the gravel and the sand beds. Where this is impracticable, the removal of mud balls may be facilitated by the use of a high-pressure hose stream. The nozzle is attached to a long pipeline which is used as a handle so that the jet can be moved throughout all portions of the sand while the bed is being washed in the usual manner.

Mechanical or hydraulic agitation of the sand bed during the washing process may disintegrate the mud balls; thus they are seldom found when high-velocity wash, agitators, or rakes are in use. Where mechanical agitators or high-velocity wash are not provided and the filter beds are not large, it is feasible to agitate the sand during the washing period by the use of hand rakes, which are passed back and forth through the sand after the wash water has been turned on. Some operators have reported successful results with the use of electric drills actuating a long shaft to which one or two small boat propellers are attached, the shaft being moved back and forth through the sand bed during the washing process.

Unchecked formation of mud balls will eventually result in large balls sinking through the agitated sand during the washing process and then collecting between the gravel and the sand. Freedom from mud balls in the filter bed is a practical measure of the effectiveness of pretreatment and the maintenance of the sand bed. A method developed by Baylis for measuring the percentage of mud balls in a sand bed is described below.

BAYLIS METHOD OF MEASURING VOLUME OF MUD BALLS

Equipment:

Wire sieve having meshes 2.5 mm square (10 meshes/in)

Sampling device consisting of a sheet-metal tube 75 mm (3 in) in diameter and 15 cm (6 in) long, one end of which is closed by a wooden base to which a short handle is attached

Procedure :

1. Wash the sand bed in usual manner.
2. Lower the water level to a point at least 30 cm (12 in) below the sand surface.
3. Collect at least four samples of sand from different portions of the sand bed by inserting the sampling device vertically into the sand to a depth of 15 cm (6 in) to fill the tube. The handle is then moved from the vertical to the horizontal position and the sand is transferred to a portable container.
4. Place a portion of the sand in the sieve, which is then raised and lowered gently in a vessel of water so that the sand grains are washed from the mud balls which are retained on the sieve.
5. Repeat this process with the remaining portions of the sand until all the mud balls have been separated from the sand.
6. Determine the volume of mud balls by transferring them carefully to a graduated cylinder partly filled with water and noting the increase in the level of the water in the graduate due to addition of the mud balls.

The percentage of mud balls in the sand can then be computed by determining the ratio of the volume of the mud balls to the total volume of sand examined. For example, let us assume that the total volume of the mud balls was found to be 15 cm^3 and that four samples of sand were collected and examined. A tube 75 mm (3 in) in diameter and 15 cm (6 in) long has a capacity of 689.9 cm^3 (42.1 in^3). Four samples, therefore, would represent 4×689.9, or 2759.6 cm^3 of sand examined. The percentage of mud balls would therefore be $(15/2759.6) \times 100 = 0.55\%$.

It has been found that most mud balls remain near the surface of the sand bed, hence if only a small percentage of mud balls is found in the top 15 cm (6 in), the volume of mud balls will probably be low throughout the entire depth of the sand bed. Filter beds tested by this method are classified by Baylis as follows:

Percentage of mud balls by volume	*Condition of filter bed*
0.0 to 0.1	Excellent
0.1 to 0.2	Very good
0.2 to 0.5	Good
0.5 to 1.0	Fair
1.0 to 2.5	Fairly bad
2.5 to 5.0	Bad
Over 5	Very bad

CRACKS IN SAND BEDS. Cracks appear in most sand beds when the water is lowered below the surface of the sand. The existence of cracks in a sand bed under water, however, indicates that the sand grains are being cemented together by some material in the applied water. These cracks are most likely to occur at the filter walls, where the sand is drawn away from the walls. The rate of flow of water increases through such cracks, and thus a heavier deposition of solids occurs at these points, which in turn intensifies the forces compacting the sand until a dense mass is formed. The degree of agitation of this dense mass during washing may be very limited and thus a " dead area " will develop, the existence of which will be evidenced by unequal distribution of the wash water. Such an area should be destroyed by the use of a hand rake or by draining the bed and removing the clogged sand.

CHEMICAL TREATMENT OF FILTER SAND:

Caustic soda. Organic matter, fine clay, oil, and the very gelatinous floc formed at times when micro-organisms are coagulated or developed in the beds have been found to cement the sand grains and thus cause shrinkage of the sand beds. The gelatinous film on the sand grains may be removed by treatment with alkali. Caustic soda (lye) will dissolve alum floc and organic matter coating sand grains. The procedure is as follows:

1. Wash the bed as thoroughly as possible, leaving about 30 cm (12 in) of water over the sand.
2. Dissolve 5-10 kg caustic soda in this water for each square metre of filter area (1-2 lb/ft^2).
3. Open the filter-to-waste valve[1] to draw the solution into the sand until a depth of about 25 mm (1 in) of solution stands on the bed.
4. Allow the solution to act for 6-12 h.
5. Wash the bed very thoroughly.

This procedure gives an application of 1%-2% caustic soda solution to sand beds, which strength has been found effective in the removal of alum floc and organic matter.

Chlorine. Another method of removing organic matter from sand grains is by the use of chlorine compounds. The sand bed to be cleaned is thrown out of use and the water surface is lowered until it is about 30 cm (12 in) above the sand level. About 4 kg (8 lb) chloride of lime or about 2 kg (4 lb) calcium hypochlorite are dissolved in this water for each 10 m^2 (100 ft^2) of filter area. The water level is lowered by filtering to waste until it stands about 25 mm (1 in) above the sand level. The bed is allowed to stand 12-24 h and is thoroughly washed. This treatment with chlorine compounds is especially adapted for the removal of slime-producing organisms or algae from sand beds, but the alkali treatment described above should be used when the sand is coated with an aluminium hydroxide film resulting from the passage of abnormal quantities of floc into the sand.

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[1] If filter-to-waste piping is not available, filter at rate of 2 gal/min/ft^2 for about 1 min so that the solution penetrates about 20 cm (8 in) into the sand, allowing 40% for the volume of the voids in the sand.

Chlorine in liquid or gaseous form, rather than chlorine compounds, may also be used to clean filter beds. Liquid or gaseous chlorine is applied most conveniently to the wash water, which is then forced upward through the sand bed being treated. This is accomplished by inserting a chlorine diffuser in the wash-water piping so that gaseous chlorine can be introduced directly from a cylinder of chlorine through the necessary metal tubing. Another procedure is to connect a solution line from a solution-feed chlorinator to wash-water piping. The dose of chlorine added at periodic intervals in this manner need not be accurately controlled so long as an adequate dose is utilized, because an overdose is of little significance. It is suggested that the dose be adjusted so that the chlorinated wash water flowing to waste will contain at least 1 p.p.m. residual chlorine. This latter procedure provides a convenient and permanent means of treating sand of any selected filter. It is especially suitable for use at plants where filter-to-waste valves and piping are not available.

Salt. Common salt has been used for the removal of slime growths from rapid sand filter beds. The bed to be cleaned is washed as usual and the water is lowered to the level of the sand bed. Then the salt is distributed evenly over the surface of the sand, using 7 kg per square metre of filter area (1½ lb/ft^2), after which the wash-water valve is opened until water rises about 15 cm (6 in) above the sand level. Allow the water to remain about 2 h to dissolve the salt, then open the filter-to-waste valve to lower the solution to the level of the sand. Allow the salt solution to remain in contact with the sand for 24 h, then wash the filter thoroughly and place it in service. This procedure is not effective in removing mineral deposits or surplus floc from sand beds.

Where the above procedures are not effective in cleaning the filter sand, it will be necessary either to replace the material with new sand of suitable effective size and uniformity coefficient or to remove the sand with an ejector for washing with special equipment providing violent agitation.

REMOVING SAND AND GRAVEL FROM FILTERS. The sand beds of filtration plants should not be disturbed as long as they are functioning properly. Whenever sand and gravel have to be removed from a filter bed to repair the underdrains, the gravel should be regraded before being replaced. Sand for replacement should be mixed with the old sand which is to be re-used, and the mixture should be analysed to ascertain its effective size and quality. The replaced sand should be washed at increasing rates until the sand has been stratified *vertically* by the wash water, which carries various sizes of sand to different levels.

Air-binding

Difficulty is sometimes experienced from the release and accumulation of air in the filter bed and underdrains to such an extent that the rate of filtration is materially reduced. This is known as " air-binding ". This trouble occurs more frequently during the winter season when the dissolved-air content of the water is at a maximum, the air being released whenever the water temperature is increased as it flows through the plant or when the water is subject to negative head in the lower portion of the bed. High negative heads, therefore, and intermittent operation of filter beds increase the release of dissolved air, so washing at lower loss-of-head values than normal may be necessary at times to avoid air-binding. Ordinarily the accumulated air may be released by shutting off the filter for a few moments to eliminate the negative head, or by washing for one or two minutes at the conventional rate.

Filtering to waste

Many filters are fitted with a valve and piping that permits the filtered water to be wasted into the drain for a short period after a filter is washed. The water so wasted, however, is not currently filtered water, but rather previously filtered water that was in the filter when the wash-water valve was closed during the washing process. The somewhat higher turbidity of this water is due not to faulty filtration, but to the floc dislodged from the sand but not completely removed by the wash water. It may take 30-60 min for the effluent to have a turbidity below the goal of 0.5 units. Only accurate turbidimeters will disclose this phenomenon. Filtering to waste, therefore, is not necessary but facilitates sand maintenance through the use of chemicals.

Care of gravel

The upward flow of wash water is not supposed to disturb the gravel and the thin layer of torpedo sand separating the sand and gravel. Experience has shown, however, that serious disturbances in the gravel do occur. At least three factors influence the stability and effectiveness of the gravel in distributing the upward flow of wash water throughout the filter area. First, mud balls sink to the torpedo sand and gravel and collect there when the sand is expanded. Eventually this will partly clog portions of the gravel, causing uneven flow of wash water, with sand boils in the free area and downward eddies over the partly clogged areas; this in turn will cause more mud balls to fall and increase the degree of clogging. The remedy here is to maintain effective washing of the sand to prevent the formation of mud balls. Second, underdrains may not distribute the wash water evenly, owing to clogging or corrosions of openings in strainers or in underdrain piping, or to the dislodgement of strainers and the resultant passage of a jet of wash water up from the relatively larger opening so formed. This jet action may be sufficient to affect the gravel. Third, the gravel does not in fact uniformly distribute wash water, but inevitably forms a multitude of currents, some of which may be sufficiently strong to be in the nature of jets. Operators, therefore, must be observant during the washing period to note areas where jet action is sufficient to cause visible boils of rising sand, or localized areas where the wash water first becomes less turbid near the end of the washing period, showing a greater flow of clear wash water into such areas.

The elevation of the top of the gravel will be altered by these factors until it is no longer level. Eventually the gravel may be so displaced as to require removal of sand and gravel, inspection and repair of underdrains and replacement of the sand and gravel. The top of the gravel at any point may be located through the use of a rod about 20 mm (¾ in) in diameter, which is used as a probe. The rod is calibrated so that the depth of the gravel below the water surface during the washing process is shown for each point at which the rod is used. The water level during washing is only a short distance above the edge of the wash-water trough and hence serves as a satisfactory measuring datum while the rod is lowered at selected points in a filter until it encounters the gravel. Clean gravel will provide a " hard " contact with the end of the rod, whereas gravel partly clogged with mud balls will produce a " soft " contact.

Disruption of the gravel may be minimized by care in opening wash-water valves, so that the rate of flow is gradually increased over a period of 1 min until the desired maximum rate is reached.

Because of the factors noted above, operators must be prepared to renovate filters when necessary to maintain effective operation. Experience has shown that this may be required in periods from less than a year to longer than ten years. This should be considered as part of normal maintenance.

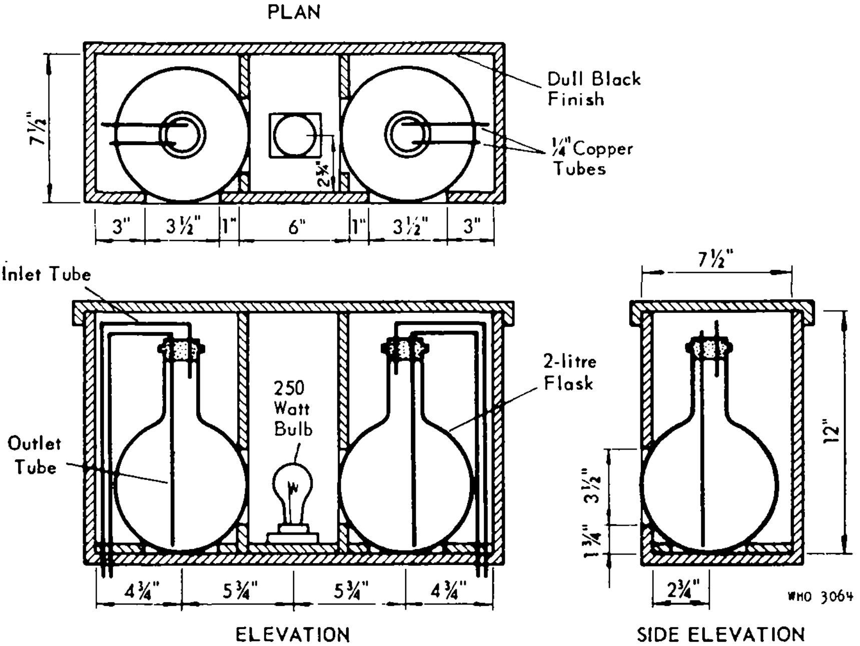
Observation of filtered water

Filtered water may be clear and yet contain discrete particles of floc that pass through the sand. In other words, floc penetration may have progressed downward to the full depth of the sand, rather than to some intermediate level as intended. If appreciable quantities of floc pass through filters, its presence will be disclosed by a floc detector or by the appearance of the filtered water illuminated by the light in the clear well. A cotton-plug filter (see below) may be home-made to disclose the *weight* of floc in the filtered water, from which turbidity units may be computed. This gives a quantitative measurement of the concentration of floc particles in otherwise clear water, showing floc penetration through the filters rather than poor coagulation.

Floc detector. Construct a wooden box as shown in Fig. 9, large enough to hold two 2-litre flasks through which filtered water passes continuously, thus permitting the observation of the effluent of two filters, or the mixed effluent from two selected sampling points. The box is painted dull black on the inside to minimize reflected light from the box walls.

FIG. 9

DETAILS OF HOME-MADE FLOC DETECTOR



From Cox (1959) p. 199 by permission of the Reuben H. Donnelley Corporation.

Cotton-plug filters. The cotton-plug filter was developed by Baylis as a means of collecting floc particles that have passed through filters, so that their weight can be determined and expressed as turbidity units in terms of the volume of filtered water allowed to pass through the cotton plug during a specific time, such as seven days. At the end of this period the plug is removed and then ignited to destroy the cotton; the unburnt ash, made up almost entirely of floc, is then weighed. This represents the weight of floc present in the total volume of water passing through the cotton plug in the selected period.

The rate of flow of water passing through a 1-g plug of cotton should be about 40 ml/min, but a rate of 10 to 20 ml/min may have to be used if the weight of floc is higher than should be the case. For example, assume that the weight of the ash—that is, the weight of the collected floc—is 100 mg and that water has been allowed to flow at the rate of 40 ml/min for seven days, or 10 080 min. Then the 100 mg of floc was present in 403 200 ml, that is, 403 l, of water, so the weight is 100/403 = 0.25 mg/l. This is equivalent to 0.25 units of turbidity, a small and acceptable value, indicating that the filter, as sampled by the cotton-plug filter, is functioning normally. The values for turbidity determined in this way should be under 1.0 unit and preferably under 0.5 unit.

A home-made version of the Baylis cotton-plug filter is shown in Fig. 10. Water is piped from the effluent pipe of a filter through ¼-inch copper or rubber tubing to the top of a laboratory filter tube 25 mm in diameter, whence it passes down through the plug of cotton and through rubber tubing to the point where the float, when it rises, exerts pressure through its lever to pinch off the flow. The tube then goes to the small float chamber. An outlet tube leads from the float chamber to the orifice. Note that the rubber stopper of the filter tube is held in place by a clamp because the device is under pressure. The float chamber should have an area as small as feasible so that the water level will respond more accurately to changes in the rate of flow. The glass orifice tube should be selected to give a flow of about 40 ml/min. Lower flows may be secured by adjusting the screw clamp on the rubber hose connecting the chamber to the orifice.

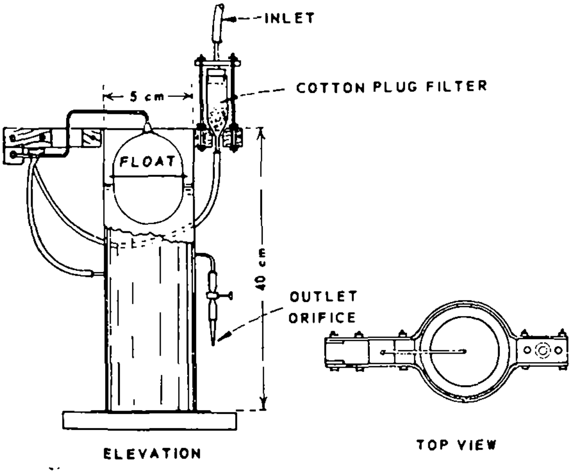
One unit is needed for each filter. If this is impracticable, provision should be made to tap each filter effluent pipe so that the cotton-plug filter may be connected to the filter under study. Its operation is as follows:

1. Insert with a pair of tweezers about 1 g cotton into the lower portion of the glass tube.

2. Open the pinchcock so that the orifice will control the flow to 40 ml/min, or adjust the pinchcock to provide some other selected value between 25 and 40 ml/min.

FIG. 10

BAYLIS COTTON-PLUG FILTER



From Baylis (1940) by permission of the *Journal of the New England Water Works Association.*

3. Continue in operation for seven days.

4. Shut off the flow of water, remove the clamp and remove the cotton plug with tweezers.

5. Place the cotton plug in an evaporating dish, supported over a Bunsen burner, and heat until the plug is dried, ignited and burned completely, leaving no black residue.

6. Weigh the ash.

7. Compute the units of turbidity from the weight of the ash and the total volume of water that has passed through the cotton plug.

Pressure filters

Pressure filters are identical in principle to gravity filters, described above; but their underdrain system, gravel and sand bed are placed in horizontal or vertical steel cylinders that are designed to withstand water pressures up to about 10 atm (150 lbf/in^2). In this way water under pressure may be subjected to filtration without the pressure being dissipated, and thus double pumping is avoided. This advantage is more than counteracted by the increased difficulty of introducing chemicals under pressure and in providing mixing and sedimentation facilities to operate under pressure. Therefore few pressure-filter installations have reliable chemical-dosing units or mixing and sedimentation basins. It is possible, however, to provide mechanical agitation in pressure tanks through the use of motor-driven paddles actuated by suitable gearing and a stuffing box. The effluent of such mixing basins should be discharged into cylindrical sedimentation tanks. These tanks may be considerably smaller than conventional gravity-sedimentation basins, but their use is essential to effective filter operation. A sedimentation period of at least 1 h is desirable.

Recently developed hypochlorinators and other chemical pumps may be used to introduce coagulants, soda ash, hypochlorite solutions, etc. Several individual units may be used for this purpose (as many as four units may be driven by a single electric motor). Meter-paced units also are available, permitting accurate chemical treatment of water flowing at varying rates. Difficulty is likely to be experienced in feeding activated carbon and lime suspension in such units, but the solutions mentioned may be applied if the precautions noted elsewhere are followed.

Vertical pressure filters are inherently more satisfactory than horizontal units, because the area of the space above the sand, is the same as that of the sand, gravel and underdrains. Usually, however, the upper space is restricted, so that effective location of wash-water-collecting piping is not feasible. Furthermore, this piping is not so effective as equivalent wash-water troughs of gravity filters. Horizontal pressure filters are less effective because of their circular vertical section. This means that the area of the underdrains is less than the area of the sand surface, which in turn is less than the area of the sand bed at the mid-point of the unit. Therefore the upward flow of wash water is not uniform over the sand area, and the agitated sand is directed upward and then towards the centre from each side.

One essential phase of filter operation is the inspection of the sand and the flow of water during washing. This is not possible with pressure filters. The inherent weaknesses of pressure filters may be summarized as follows:

1. The treatment of water under pressure seriously complicates effective mixing, coagulation, and sedimentation of the water to be filtered; many pressure filters are thus installed without adequate coagulation facilities.

2. It is more difficult to apply chemicals to water under pressure.

3. The appearance of the water being filtered and of the sand bed is not under observation, nor is it possible to observe the effectiveness of wash water or the degree of agitation during the washing process.

4. Because of the shape of pressure filters it is difficult to provide effectively designed wash-water piping to ensure that material washed from the sand is discharged to waste and not flushed back to other portions of the sand bed.

5. It is difficult to inspect, clean, and replace the sand, gravel and underdrains of pressure filters.

6. The operation of such units under pressure encourages the pumping or forcing of water through the beds at excessive rates.

The advantages of pressure filters, however, are such that they will continue to be used for smaller supplies, because they may be assembled complete in the factory and shipped for convenient installation. The best procedure to follow is to have the raw water flow by gravity, or be pumped by low-lift pumps,into gravity flocculating and sedimentation basins, where prechlorination also is provided, and to pump the settled water through four pressure filters and thence into the distribution system. The use of four smaller filters facilitates shipment, and more important, it reduces the rate of flow of wash water to that needed for each smaller unit. The filters should be washed with filtered water. The practice of washing with raw or settled water should be avoided, especially as the high-lift pump would provide a wash-water flow rate of only four times the rate of filtration, or 20 m/h (64 ft/h; 8 US gal/min/ft^2), which is inadequate. Note that the rate of filtration should not be that frequently used in practice, namely, 7.2 m/h (24 ft/h; 3 US gal/min/ft^2), just because this volume of water can be forced through the filters. The inherent weaknesses of these filters require their use with due precautions, including a filtration rate of 4.8 m/h (16 ft/h; 2 US gal/min/ft^2).

Diatomaceous-earth filters

Advances in the clarification of liquids in the chemical industry through the use of powdered filtering material have led to the development of filters in which diatomaceous earth or diatomite is used as the filtering medium. These units are frequently used for the clarification of water being recirculated through swimming pools, because the relatively clear water does not require coagulation prior to filtration through the finely powdered material.

Another characteristic of these filters is their large filtering area in comparison with their over-all size and weight. This led to their being used during World War II as portable units for military forces, and subsequently for emergency use where their small size and weight outweighed certain limitations, which will be discussed later.

Diatomaceous-earth filters are manufactured by a number of firms. They consist essentially of (*a*) strainer elements, equivalent in purpose to filter underdrains, but with very fine openings so as to retain the powder; (*b*) a thin coating of the powder on the elements; (*c*) open or enclosed tanks to contain the elements and collect the filtered water; and (*d*) facilities for applying the powder to the elements after each washing operation. Details of each commercial type are given in the catalogues of the manufacturers. Filters of this type are available having areas from 56 cm^2 (0.06 ft^2) to over 18 m^2 (200 ft^2).

Pressure or closed units have internal partitions to which small cylindrical elements are attached, or arrangements are made for supporting elements consisting of two flat strainer surfaces, which are separated a short distance to form spaces for the collection of filtered water. Open units use both types of elements, but they are attached to false bottoms of open tanks or to special collecting piping.

Pressure units may be operated at pressures of 2-3.4 atm (30-50 lbf/in^2) so as to secure longer filter runs before the clogged material has to be removed and replaced by fresh material. Open units operate under atmospheric pressure, plus the head of water on the elements, plus the negative pressure created by the pumps connected to filtered-water-collecting piping. Open units have the advantage of exposed elements, so operators can observe operating details and also use a hose stream to clean elements when it is necessary to supplement the usual washing procedure. This is possible with certain pressure units, which have provisions for opening one end of the horizontal cylindrical tanks.

This type of filter is especially adapted to the clarification of waters of low turbidity, when suspended solids and micro-organisms such as algae may be removed without coagulation. Conversely, residual floc in coagulated and settled water will clog these filters quite rapidly. Therefore the suitability of their use for the routine treatment of any specific supply should be determined by local conditions and the quality of the raw water. Small-size units, loaned by manufacturers, may be used as pilot plants to determine their suitability.

Rates of filtration have been as high as 14.4 m/h (48 ft/h; 6 US gal/min/ft^2), but this leads to short filter runs and the need to replace the coating of powder at frequent intervals. The most economical rate appears to be 2.4-4.8 m/h (8-16 ft/h; 1-2 US gal/min/ft^2); this represents a compromise between the first cost of larger units to provide the lower rates, and the operating costs, represented largely by the quantity of powder used.

Operation

1. A filter of this type is placed in operation by applying a slurry of the powdered diatomaceous earth to the influent for a short period to coat the elements, hence the name " precoat ". From 0.5-2.5 kg/m^2 (0.1-0.5 lb/ft^2)—usually about 0.75 kg/m^2 (0.15 lb/ft^2)—of the powder is added for this purpose. The filtered water is recirculated back to the influent while the slurry is being applied and until the effluent becomes clear.

2. The filtered water is then valved into the effluent piping. At the same time the chemical feeder designed to apply a measured dose of the powder is started, to apply the so-called " body feed ". This is intended to maintain a more porous mixture of powder and solids removed from the water than would result were these solids to collect on the surface of the precoat. The dose of body feed depends upon whether inorganic silt (turbidity) is being removed or whether algae and slime-forming organisms are involved. The dose with silt should be equal to the turbidity, whereas the dose should be increased to two or three times the weight of slime organisms removed. This latter dose cannot be computed in practice, so must be determined by experience. When in doubt apply 60-90 g/h/per square metre of area (0.2-0.3 oz/h/ft^2) until trial runs demonstrate the most economical and yet effective body feed.

3. When the loss of head increases, through partial clogging of the coating, to values of 2-3.4 atm (30-50 lbf/in^2) with pressure units, or to about 1 atm (15 lb/in^2) negative pressure on the effluent suction piping of open units, the filter is thrown out of operation for replacement of the precoat or filter cake.

4. Filtered water is then forced back through the elements to wash the precoat to waste. Wash water should be applied at rates of 14.4-24 m/h (48-80 ft/h; 6-10 US gal/min per square foot of area) to clean the elements. Several types of these filters have provisions for using compressed air to force wash water through the elements at a very high rate for a few seconds of time. In any case the volume of wash water is much less than with sand filters.

5. The precoat is then applied and the next cycle of operation is started.

Precautions

A diatomaceous-earth filter requires special care by operators in seeing that water is recirculated through the unit while the precoat is applied in step 1, to be certain that all portions of the elements have been coated properly before the filtered water is diverted to the effluent piping. Another precaution is to see that the flow is continuous after the precoat is applied, otherwise the precoat will fall from the elements when the flow stops. Therefore this type of filter cannot be used with intermittent flow.

Advantages

The advantages of the diatomaceous-earth filter may be summarized as follows:

1. There is a large filter area per unit of over-all dimensions.

2. Their weight is low compared with pressure sand filters of the same capacity, favouring their use as portable units.

3. Waters with turbidities less than about 30 units may be effectively clarified without coagulation.

4. The fine powder forms as effective a filter medium for the removal of bacteria, and especially the cysts of dysenteric amoebae, as sand filters coated with alum floc.

5. These filters may be operated with one pumping operation, as contrasted with raw- and filtered-water pumps of gravity sand filters.

Disadvantages

The disadvantages of these filters are as follows:

1. The units are effective only so long as each precoat is properly applied and the flow is continuous, so they are more dependent on effective operation than are rapid sand filters.

2. The loss of head is much higher than with sand filters.

3. Diatomaceous earth is not generally available throughout the world.

4. Difficulty with clogging of the fine openings of elements presents problems, especially if uncoagulated iron is present in raw waters or if raw water is forced through the elements before an effective precoat is formed.

5. These filters have short filter runs if coagulated and settled water is applied, because of clogging by residual alum floc.

Summary

Diatomaceous-earth filters appear to be best suited for the clarification of waters of low turbidity, where the absence of coagulation may compensate for the cost of the diatomaceous earth used. Their compact size and relatively low weight, as manufactured in complete, self-contained units, make them portable and thus suited for use in emergencies or for temporary installations where permanent filter structures are not justified.

Laboratory control

The operation of filters should be controlled by tests for turbidity and colour and by bacteriological examinations. As stated previously, the turbidity of filtered water should be less than 1 unit. These low turbidities can be determined only by special turbidimeters, described in the Supplement, page 283, which measure turbidities as low as 0.2 unit. The objective should be the production of water with a turbidity less than 0.5 units.

In the absence of such equipment the operator should make regular observations of the clarity of filtered water with submerged lights in the clear wells or filtered-water reservoirs. If waterproof light fixtures are not available, an ordinary fixture may be installed above the maximum water level, at a point where the reflected light may be observed through a manhole or the like. The light should be offset to the side of the opening so that light cannot be reflected directly from the water surface to the eye. Immediately below the opening, at the bottom of the basin, a plate about 30 cm (1 ft) in diameter should be placed to serve as the object to be viewed when observing low turbidity of the intervening water. A black plate should be selected, because light reflected from small floc particles or from small concentrations of uncoagulated suspended solids is best observed against a black background.

The degree of removal of bacteria by filtration alone is difficult to appraise, because it depends so much on the effectiveness of flocculation and sedimentation prior to filtration. Measured in terms of coliform index, as discussed in Chapter 9, page 177, and the Supplement, page 344, the degree of removal of coliform organisms by the combination of flocculation, sedimentation, filtration and postchlorination should be 99.98%. Thus if the raw water has an index of 5000/100 ml, the filtered chlorinated water should have an index of 1/100 ml or less. If the raw water has an index above 5000/100 ml, prechlorination must also be practised, as discussed in Chapter 8.

The plate count, described in the Supplement, page 341, is less significant and more difficult to use, because many types of bacteria will develop in filters, so that the filtered water may contain a higher number of the miscellaneous bacteria than the settled water applied to the filters. The harmless bacteria may be slime-forming and hence may lead to shorter filter runs. The remedy is continuous prechlorination, or periodic chlorination of the wash water. In the absence of such growths, filters should remove about 90%–98% of the bacteria present in the applied water.

Microstrainers

Miscrostrainers are not true filters in the usual meaning of the word, but they are much finer in mesh than screens used in the past. Recently strainers made of stainless-steel wire, with very fine mesh forming openings of 0.025 mm (0.001 in), have been developed in England for the removal of micro-organisms and coarse suspended solids from raw waters prior to filtration, thereby prolonging filter runs. They have been used prior to chlorination when colour and turbidity are not a problem but when micro-organisms have to be removed to produce an acceptable water. They may remove 50%-95% of plankton.

Microstrainers vary in size from 0.75 m (2½ ft) in diameter and 0.6 m (2 ft) long to 3 m (10 ft) diameter and 3 m (10 ft) long. The latter size will handle 15-45 m^3/day (3.3-10 m.g.d. UK; 4-12 m.g.d. US) of water, with a power consumption of about 4 hp. The head loss is only about 15 cm (6 in) with unit capacities of 7.2-12 m^3/h per square metre of strainer (2.5-4.2 UK gal/min/ft^2; 3-5 US gal/min/ft^2), the capacity being associated with the content of micro-organisms in the raw water.

The strainers are washed by a continuous spray of water under pressures of 0.34-0.68 atm (5-10 lbf/in^2), although pressures as high as 2 atm (30 lbf/in^2) have been used. The volume of wash water has varied from 1% to 3% of the volume of water strained.

The cost of operation, comprising power and the value of the wash water used, is about US$ 1.00 to $1.50 per million gallons treated. Operation is simple and requires no control, except the microscopic examination of the water before and after straining, and supervision to see that washing is effective. The observed loss of head is a measure of the cleanliness of the strainer.

The employment of microstrainers would depend on many local factors. They would appear to be most useful when the plankton content of raw waters is such that existing slow sand filters have short filter runs, so that microstrainers would be much cheaper than any increase in filter area or frequent scraping of filters.

CHAPTER 8

CHLORINATION AND OTHER MEANS OF DISINFECTION

Available disinfectants

The disinfection of potable water is almost universally accomplished by the use of gaseous chlorine or chlorine compounds, because of the limitations of other procedures. Thus although bromine, for instance, has been used to a very limited extent in the disinfection of water recirculated through swimming pools, there is no wide distribution of liquid bromine or special equipment for its safe storage and application. Chlorine and chlorine compounds, such as chloride of lime or calcium hypochlorite, are relatively cheap and available and provide a disinfectant that has prolonged action. Only disinfection by chlorination is discussed in detail in this chapter, but brief consideration is first given to ozone and a few other disinfectants that are in limited use for public water supplies.

Ozone

Ozone has been used for many years in Europe as a disinfectant and has also been very effective in the removal of tastes and odours from water. Inasmuch as ozone has to be generated as needed by passing thoroughly filtered and dried air through tubes or between plates where high-voltage electric discharge occurs, changing part of the oxygen of the air to ozone (O_2 to O_3), its cost is greater than when purchased chemicals are manufactured on a wholesale basis, as with chlorine.

Equipment installed at each treatment plant consists of air filters, blowers, dryers, ozone generators and ozone absorption facilities. The equipment recently installed at Philadelphia, Pa., is illustrative. Briefly, the plant has a capacity to produce 1250 lb of ozone per day, or 9400 ft^3/day at atmospheric pressure, sufficient to treat 36 m.g.d. US with a dose of up to 4.0 p.p.m. (This is equivalent to 568 kg or 266 m^3 of ozone at atmospheric pressure, sufficient to treat 136 260 m^3 of water with a dose of 4.0 p.p.m.)

Electrostatic air filters are used to remove dust from the air. Two refrigeration dryers, followed by four activated alumina dryers, are used to remove moisture. Fifty ozone generators, each with a capacity of 25 lb/day, produce a mixture of air and ozone (0.5%–1.0% oxygen changed to ozone), which is forced through porous diffuser plates on the bottom of three tanks, 18 ft in depth and providing 10 min detention time. Settled water is ozonized prior to filtration.

The ozone generators operate on an electrical potential of 15 000 V. From 8-9 kWh of current is used per pound (0.45 kg) of ozone, and about one-third this amount to operate auxiliary equipment. The cost of ozone so produced may be computed from local cost of electric power. The cost of the equipment described above was about US $600 000, or US $15 000 per m.g.d. capacity.

The ozone-air mixture is added to the water at existing plants in three ways: first, by spraying the water into an enclosure in which the ozone-air mixture is present; second, by forcing the ozone-air mixture into the bottom of scrubbers in which the water falls in a counter-current manner; and third, by forcing the mixture through diffusers on the bottom of deep basins.

Control is through the use of special equipment indicating the concentration of ozone in the treated water, or by the orthotolidine test. The latter indicates the presence of 0.1 p.p.m. ozone when the reagent colour is equivalent to 0.15 p.p.m. residual chlorine. An excess of ozone is maintained to ensure effective disinfection and taste and odour control. The ozone dose is controlled by modifying the voltage on the generators, within the limitations of the equipment. No ozone remains in the treated water for more than about 30 min, as the ozone reverts to oxygen. Therefore there are no secondary tastes or odours from ozonation. An added benefit is the reduction of the natural colour of the water by about 50%, due to the bleaching action of ozone.

In short, ozonation is a very attractive method of disinfecting water and of taste and odour control. Unfortunately, the cost of the equipment is much higher than that of equivalent chlorinators, and the cost of electric current to generate ozone is greater than the cost of equivalent doses of liquid chlorine. The equipment must be well maintained and operated, especially the air-filtering and drying apparatus.

Ultraviolet light

Ultraviolet light has been used at a few small installations, but the water to be sterilized must be clear and the lamps must be kept clean at all times. Furthermore, even a low concentration of soluble iron in the water will absorb ultraviolet light, even though the water is clear to visible light. The equipment is costly.

Purpose of chlorination

The object of chlorination of potable waters is the destruction of bacteria through the germicidal effects of chlorine. There are several important secondary uses for chlorination, such as oxidation of iron, manganese and hydrogen sulfide; destruction of some taste- and odour-producing compounds; control of algae and slime organisms in treatment plants; and as an aid to coagulation. Since these processes are all controlled as part of the chlorination process, they are discussed in this chapter.

Principles

Chlorine gas or chlorine compounds are used, but in all cases the active disinfectant is chlorine.

The problem of effective chlorination is to ensure: (*a*) uniform application of chlorine to all portions of the water being treated; (*b*) uninterrupted application of chlorine; (*c*) selection of the dose of chlorine to meet the current needs of the specific water being treated; (*d*) control of chlorination so as to produce a safe potable water that is at the same time of attractive character.

Natural waters represent complex solutions of many substances, most of which can be ignored but some of which influence chlorination, as follows:

1. Suspended solids may shield bacteria from the action of chlorine.
2. Organic matter reacts with chlorine so that it has only weak disinfecting properties or none.
3. Ammonia reacts with free chlorine to form chloramines or combined-residual chlorine having much lower disinfecting qualities than free-residual chlorine.
4. Waters of low alkalinity and pH value—that is, waters having pH values less than about 7.2—are more easily disinfected than those having pH values above 7.6.
5. Nitrites react with and remove free chlorine and may also produce a false colour with orthotolidine reagent unless the orthotolidine-arsenite test is used to correct for such errors.
6. Manganese forms a false colour with the older orthotolidine test, although the orthotolidine-arsenite test corrects for such errors.
7. Iron, when present in a concentration above about 1 p.p.m., causes errors with the older orthotolidine test that are corrected by using the orthotolidine-arsenite test. Iron and manganese also react with free chlorine when in the reduced state and hence add to the amount of chlorine required for disinfecting purposes.

Temperature

The rapidity of disinfection with chlorine is proportional to the temperature of the water, so that, other things being equal, chlorination is more effective with high water temperatures. On the other hand, chlorine is more stable in cold water and hence will remain in the cold water for a longer period of time, thus compensating to some extent for the lower *rate* of disinfection in cold water.

Period of contact

The period available for the interaction between chlorine and constituents of the water is one of the most important aspects of chlorination practice. The minimum period should be 10-15 min and preferably several hours so that effective disinfection may be ensured without an undesirably high concentration of residual chlorine in the water reaching the consumer.

Summary

The requirements for effective chlorination are met when the chlorine dose is sufficient to react with the organic matter, ammonia, iron, manganese and other reduced substances and at the same time leave sufficient excess or residual chlorine for disinfecting purposes, and when the concentration of residual chlorine is selected so as to compensate for the deleterious influence of any prevailing high alkalinity, low water temperature or short period of contact.

Superchlorination, break-point chlorination, or free-residual chlorination

After the development of the orthotolidine test in 1918 it became the practice to add a dose of chlorine sufficient to give a concentration of residual chlorine of 0.1-0.2 p.p.m. after an arbitrary reaction period of 10 min. This proved generally satifactory with many waters, but it was subsequently found that the small doses of chlorine used resulted in the formation of combined-residual chlorine, that is, the chlorine combined with substances like ammonia to form chloramines, which have a slow disinfecting action. On the other hand, the combined-residual chlorine was found to be quite stable and did not react with some taste- and odour-producing compounds, so the practice developed about 1928 of deliberately adding ammonia to the water with the chlorine to form combined-residual chlorine, or chloramines. It was subsequently found that the poor disinfecting property of chloramines was a serious disadvantage, unless very prolonged reaction periods were available and unless waters of low alkalinity were being treated, so heavy chlorination, without the application of ammonia, was adopted. The term " superchlorination " was formerly used to signify heavier than usual chlorine doses, but this has now been replaced by the terms " break-point chlorination " or " free-residual chlorination ". Chlorine-ammonia treatment is now being superseded by free-residual chlorination.

Principles

The principle of free-residual chlorination is to add sufficient chlorine to oxidize all the organic matter, iron, manganese, and other reduced substances in the water being treated and also to oxidize free ammonia in the raw water, so that the remaining residual chlorine will be present as active free-residual chlorine rather than the less active combined-residual chlorine or chloramines. This purpose is achieved when the chlorine dose is increased by trial and error until the orthotolidine-arsenite test shows that the residual chlorine is present as free-residual chlorine rather than as combined-residual chlorine. The " break point ", which is noted with waters containing ammonia, is the point at which there is a sudden reduction in the remaining concentration of residual chlorine as the result of the more active chemical reactions leading to the oxidation of the ammonia. A smaller concentration of active free-residual chlorine, therefore, replaces a proportionately larger concentration of combined-residual chlorine or chloramines.

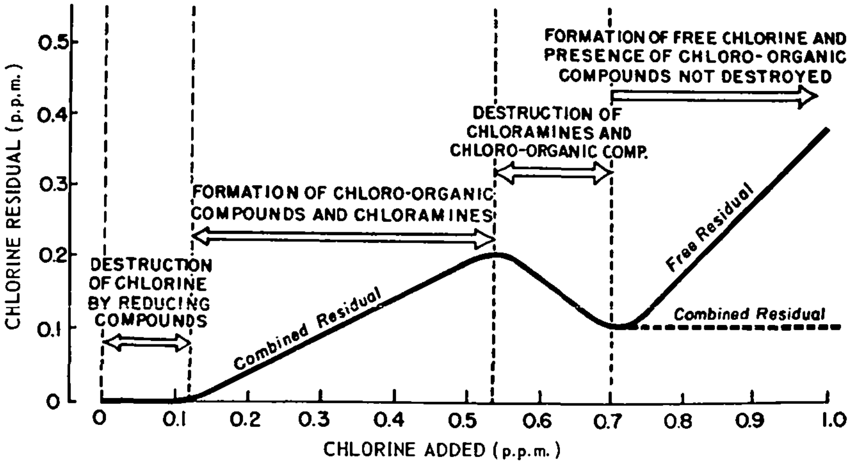
The above may be clarified by reference to Fig. 11, showing a typical " break-point curve " secured when a series of portions of a sample of water containing ammonia and organic matter is chlorinated in the laboratory with progressively larger doses of chlorine. Fig. 11 also illustrates the " chlorine demand " of the specific water with progressively larger doses, the chlorine demand being the difference between the dose and the resulting residual-chlorine value for that dose.

It will be noted that there are four zones in Fig. 11, distinguishing four ranges in chlorine doses in which different conditions prevail and different reactions occur. The first zone, with the low doses of 0 to about 0.12 p.p.m., represents a condition in which all the chlorine is removed by reaction with organic matter so that no residual chlorine remains. No disinfection would occur in the absence of residual chlorine.

The second zone, with doses of 0.12-0.53 p.p.m., represents the results with moderate doses of chlorine, which form chloramines with the ammonia in the water and also chloro-organic compounds. The chloramines and a portion of the latter react with the orthotolidine reagent as combined-residual chlorine, which then is present in increasing concentrations as higher doses are added. This zone represents earlier chlorination practice,

FIG. 11

REACTIONS OF CHLORINE IN WATER



From *Manual of instructions for water treatment plant operators*, p. 94, by permission of the New York State Department of Health.

in which residual chlorine of 0.1-0.2 p.p.m. was considered ample, regardless of whether the residual was combined chlorine or free chlorine.

The third zone, with doses of 0.53-0.7 p.p.m., illustrates the " break point ", at which these higher doses of chlorine lead to increased chemical activity, which in turn leads to the oxidation or alteration of the chloramines and chloro-organic compounds; this activity utilizes the chlorine without producing compounds that react with orthotolidine reagent. In other words, the increased chemical activity of the higher chlorine doses in this zone leads to more profound chemical changes, which increase the removal of chlorine, or chlorine demand, so that *higher* doses produce *lower* concentrations of residual chlorine.

Finally, the fourth zone represents the conditions that prevail when the reactions in the third zone have been completed; any further increase in the dose of chlorine then produces free-residual chlorine, the concentration of which increases as the dose is increased. A small concentration of combined-residual chlorine also may remain, as compounds not destroyed by the free-residual chlorine.

The purpose of free-residual chlorination is to use the dose of chlorine sufficient to produce free-residual chlorine. The typical water used as an example in Fig. 11 would be chlorinated with a dose of 0.8 p.p.m. chlorine to produce 0.2 p.p.m. free-residual chlorine. Contrast that with the effects of the dose of 0.53 p.p.m. that produced 0.2 p.p.m. combined-residual chlorine.

Free-residual chlorine is not molecular chlorine, " free " to react as a dissolved gas, except at pH values of 5.0 or below, when part of the chlorine remains in water as molecular chlorine. Waters having pH values of 5.0-6.0 react with the chlorine to form hypochlorous acid and hydrochloric acid as follows:

$$Cl_2 + H_2O \rightleftarrows HOCl + HCl$$

The hydrochloric acid is neutralized by the alkalinity of the water, and the hypochlorous acid ionizes in part to form hydrogen ions and hypochlorite ions, as follows:

$$HOCl \rightleftarrows H^+ + OCl$$

This reversible reaction depends for its equilibrium on the pH (hydrogen ion concentration) of the chlorinated water. *Both* hypochlorous acid and hypochlorite ions are present in water with pH values above 6.0, but the concentration of the former becomes less as that of the latter increases, until at a pH of 8.5 about 90% of the hypochlorous acid has been ionized to hypochlorite ion. Thus free-residual chlorine is actually a mixture of hypochlorous acid and hypochlorite ion, the two being in equilibrium with the hydrogen ions; their ratio depends on the pH of the water.

This is important, because hypochlorous acid is a more active disinfectant than the hypochlorite ions. Fortunately, most waters have pH values of 6.0-7.5, so 50%-95% of the free-residual chlorine is present as hypochlorous acid. Conversely, softened waters of high pH must be disinfected with higher doses, to compensate for the lower disinfecting characteristics of the hypochlorite ion.

Properties of free-residual chlorine versus combined-residual chlorine

These properties of free-residual chlorine as compared with combined-residual chlorine are illustrated in the experimental results summarized below.

The effect of pH value of the water

Free-residual chlorine in concentrations of 0.05 p.p.m. was found to be effective in the experimental disinfection of water, with a reaction period of 10 min, throughout the pH range of 7.0-8.5, and equally as effective, with a reaction period of 20 min, throughout the pH range of 7.0-9.8. Combined-residual chlorine, to be equally effective in 10 min, had to be present in concentrations of 1.50 p.p.m., at pH 6.5, and this concentration was not effective at high pH values. The concentration of combined-residual chlorine necessary to be equally effective with a reaction period of 20 min ranged from 0.90 p.p.m. at pH 6.5 to 1.80 p.p.m. at pH 8.5.

Thus the concentration of combined-residual chlorine must be doubled to compensate for the adverse effects of a pH of 8.5.

The effect of water temperature

The same experiments showed that 0.05 p.p.m. free-residual chlorine was effective in the disinfection of water in the pH range of 7.0-8.5 with water temperatures of 5°C and 22°C, that is, temperatures near freezing and near room temperature. Disinfection, however, was appreciably slower at the lower water temperature with pH values above 8.5. Conversely, water temperatures had a marked effect on the disinfecting action of combined-residual chlorine. For instance, 0.60 p.p.m. combined-residual chlorine was as effective at pH 7.0 with a water temperature of 22°C as 1.50 p.p.m. combined-residual chlorine at the same pH value but with a water temperature of 5°C. In other words, the dose had to be more than doubled to compensate for the slower action of combined-residual chlorine at lower water temperatures, whereas this increase was not necessary with free-residual chlorine at pH values under 8.5.

The effect of reaction periods

The same experiments also showed that at room temperature free-residual chlorine in concentrations as low as 0.05 p.p.m. was equally effective with reaction periods of 10, 20 or 60 min, in the pH range of 7.0-8.5, but that combined-residual chlorine was not effective with a reaction period of 10 min even with very high concentrations. In fact, concentrations of 0.90-1.80 p.p.m. were needed with a period of 20 min for the pH range of 6.5-8.5, and concentrations of 0.30-1.50 p.p.m. for the same pH range with a period of 40 min. In other words, the dose of combined-residual chlorine, at any given pH value, had to be much greater for a period of 20 min than for 40 min to give equal results.

Advantages

Utilizing the above values as a guide, and allowing a factor of safety for unforeseen adverse factors such as sudden changes in the temperature and quality of the water, Table 7 has been prepared to indicate desirable *minimum* concentrations of free-residual chlorine for a disinfecting period of 10 min, as compared with minimum concentrations of combined-residual chlorine with a disinfecting period of 60 min, at specified pH values.

In addition to its other properties, free-residual chlorine is stable, i.e., it persists in water, because the organic matter that otherwise would react and remove the chlorine has been oxidized by the previous heavy dose of chlorine. Furthermore, free-residual chlorine has little or no taste when

TABLE 7

RECOMMENDED MINIMUM CONCENTRATIONS OF FREE-RESIDUAL CHLORINE VERSUS COMBINED-RESIDUAL CHLORINE TO ENSURE EFFECTIVE DISINFECTION

pH value	Minimum concentration of free-residual chlorine, p.p.m. (disinfecting period at least 10 min)	Minimum concentration of combined-residual chlorine, p.p.m. (disinfecting period at least 60 min)
6.0 — 7.0	0.2	1.0
7.0 — 8.0	0.2	1.5
8.0 — 9.0	0.4	1.8
9.0 — 10.0	0.8	Not recommended
10.0 +	0.8 + (with longer contact)	Not recommended

From *Manual of instruction for water treatment plant operators*, by permission of the New York State Department of Health.

present in the concentrations used in the disinfection of water, though it may have a slight chlorine odour when water is first drawn from a faucet.

Water leaving the point of chlorination may contain nothing but free-residual chlorine because of the prior oxidation of the organic matter and ammonia in the water. This free-residual chlorine, however, will react with any pipe slime or coating on the inside of water mains, so the progressive oxidation of this coating must occur before free-residual chlorine will be present in the water throughout a distribution system. It may take from three to six months for the pipe coating to be oxidized by chlorine, and in the meantime objectionable tastes or odours may occur, especially in water drawn from dead-end mains. Once this transition period has elapsed, however, a small concentration of free-residual chlorine can be maintained in the delivered water throughout the distribution system.

The maintenance of free-residual chlorine throughout a system has other advantages besides prolonging the disinfecting period. When water remains in dead-end mains, it prevents the reduction of sulfates in hard waters to sulfides that would produce tastes and odours. Free-residual chlorine also prevents the growth of organisms that tend to decrease the capacity of mains. The trend, therefore, is to adopt free-residual chlorination so as to secure all these benefits.

Free-residual chlorination can be controlled through the use of the orthotolidine-arsenite test. This test distinguishes between active free-residual chlorine and the much less active combined-residual chlorine or chloramines, and also compensates for any errors that might otherwise result from the presence of nitrites, manganese, iron, etc. This test, therefore, is used to ensure the presence in the treated water of free-residual chlorine and not combined-residual chlorine, and also to indicate when any free-residual chlorine present at the point of treatment may be reacting with substances in the distribution system to form combined-residual chlorine.

Limitations

The limitations of treatment by *chlorination alone* should be considered, to avoid undue reliance on this process alone when other types of treatment also are required. Generally speaking, treatment by chlorination without filtration is effective and adequate only when:

(1) the degree of bacteriological pollution is moderate and reasonably uniform, and the bacteria to be destroyed are not shielded from the chlorine by being bedded in suspended solids or within the bodies of worms, for example;

(2) the turbidity and colour of the water do not exceed 5–10 units;

(3) the content of iron or manganese (or both) in the water does not exceed 0.3 p.p.m.

(4) the chlorine demand of the water does not fluctuate so rapidly as to prevent proper adjustment of the chlorine dose;

(5) taste- and odour-producing substances are absent or do not interfere with the selection of adequate chlorine doses through the production of chlorine tastes; and

(6) there is a contact period of at least 15 min between the point of chlorination and the house connexion of the consumer first supplied with water.

Chlorination practice

Liquid chlorine

Liquid chlorine is chlorine gas compressed to a point where it liquefies. The vapour pressure at which liquefaction occurs is 2.66 atm (39 lbf/in^2) at 0°C (32°F) and 41 atm (602 lbf/in^2) at 100°C (212°F). Cylinders are so filled that liquid chlorine occupies about 80% of the cylinder when the temperature is about 65°C (150°F). Higher temperatures would result in excessive confinement pressures and create a danger of rupturing the cylinder if relief were not provided; the cylinders should not be stored near steam pipes, radiators, stoves or boilers. The chlorine cylinders are fitted with fuse plugs for the release of the gas when the temperatures become greater than about 65°C (150°F).

Dry chlorine gas is not corrosive and so may be transported in iron cylinders or piped through iron tubing. When moist, as when leaks occur into moist air or when the chlorine is added to water in solution chlorinators, chlorine is very corrosive, however.

Liquid chlorine is now practically free from impurities, so the former practice of expelling a small quantity of gas from the tank being placed in use before connecting it to the chlorinator no longer need be followed.

Liquid chlorine may be purchased in standard steel cylinders of various sizes. Common metric sizes contain net weights of 50 or 100 kg, and British and US cylinders contain net weights of 100 or 150 lb. Larger containers with net capacity of 1 ton are also used, and tank cars with capacities up to 30 tons are in service in some countries. The ton containers are shipped on a multi-unit car holding 15 containers. The size of containers varies from country to country. Chlorine cylinders should be accurately weighed so that the quantity used each day may be known. The scales should be so placed in relation to the heating unit of the enclosure in which the chlorinator is located that the cylinder will be cooler than the chlorinator, so that gaseous chlorine flowing from the cylinder to the chlorinator will not be condensed in a cooler chlorinator. This may be accomplished by the use of an insulated screen between the heating unit and the chlorine cylinder.

The convenient operation of the chlorination equipment is greatly facilitated by properly designed unloading platforms and storage room for chlorine cylinders. Special equipment is needed for handling the 1-ton container; information on this can be obtained from the manufacturers of liquid chlorine.

Emergency precautions

Ventilation. Chlorine gas is 2½ times heavier than air and hence will sink to the floor when leakage occurs. Rooms used for storing chlorine cylinders and the small enclosures surrounding chlorinators should have sealed walls and should have the door opening outward to the outside of the building. The rooms should be fitted with discharge ducts at the floor level and fresh-air vents at the top of the room. In larger installations it is very desirable that a fresh-air fan be provided near the ceiling to force fresh air into the chlorinator enclosure, or that a chlorine-resistant fan be installed in the discharge duct at the floor level, to facilitate the discharge of chlorine gas in case of leakage. A very satisfactory arrangement is the use of steam-eduction ejectors in the lower opening, because they not only expel the chlorine gas but also render it less objectionable through the union of the chlorine and the steam to form hypochlorous and hydrochloric acids in the condensed steam, which should drain into a sewer. Dilution water should be discharged into the same drain or sewer.

Gas masks. Chlorine gas is an irritant, so special care should be exercised not to inhale the gas when repairing a leak in the chlorination apparatus.

Gas masks are obtainable for the use of operators and should be available for immediate use. They should be stored near the chlorinator but outside the chlorinator room or enclosure. It is usually possible to close the cylinder valve in a second or so while holding the breath, after which the chlorine room should be allowed to ventilate thoroughly to disperse the gas that has leaked into it. Repairs can then be made at leisure without any discomfort.

First aid.[1] In case of serious exposure to chlorine gas a physician should be called and the patient should be removed to hospital. While waiting for the physician the patient should be kept quiet in warm fresh air and should be covered with a blanket. A mild stimulant such as hot coffee (or a teaspoonful of brandy in warm water) is beneficial. It is advisable that the antidote, antichlor, be available for emergency use in treating the nose and throat. This may be prepared by any pharmacist according to the following formula:

Antichlor (2 litres)

Water	1485 ml
Sugar	45 g
Comp. lavender tr.	37 ml
Sp. ammonia aromatic	56 ml
Alcohol (ethyl)	333 ml
Oil of peppermint	37 ml
Sp. chloroform	55 ml

Add oil of peppermint to the alcohol, then the sp. chloroform,[2] then sp. ammonia and comp. lavender tr. in that order. Stir after each addition. Next add this mixture to the water in which the sugar has been dissolved.

Label: " ANTICHLOR. Shake well before using ".

Dose: 1 tablespoonful swallowed every 15 min until relief is obtained or until 4 tablespoonfuls have been swallowed.

Chlorine gas is irritating to the eyes. Boric acid will usually relieve such irritation. The skin is burned by chlorine gas in sufficient concentration; such burns should be treated with a weak solution of sodium bicarbonate followed by a loose dressing of carron oil. Irritation of the nose and throat is benefited by the use of antichlor, but if this is not available the patient should breathe the vapours of a warm menthol salve and should drink cooled cream or milk to soothe the throat.

Operation of gas-feeding chlorinators

Equipment for applying gaseous chlorine to water supplies is manufactured by a number of companies. Catalogues and descriptive information

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[1] These notes on first aid are based on L. L. Hedgepeth (1934) Handling chlorine to avoid trouble, *J. Amer. Water Works Ass.*, 26, No. 11, Nov. 1934 and the Committee Report (1935) Chemical hazards in water works plants, *J. Amer. Water Works Ass.*, 27, No. 9, Sept. 1935.

[2] To make spirit of chloroform, add 60 ml chloroform to 940 ml grain alcohol.

relative to the equipment may be secured from the manufacturers. Every effort should be made to select equipment that will be satisfactory under the specific local conditions existing at any particular supply.

Chlorinators consist of pressure-reducing valves actuated by metal diaphragms or hydraulically operated floats, orifices for measuring the rate of flow of gas after it has been reduced to a uniformly low pressure, and devices for injecting the gas into the supply. Chlorine gas may be piped in tubes through a check valve directly into the water to be treated, in which case the tubing is led either to a perforated solid-silver diffuser inserted in the water pipe through a special stuffing box and corporation cock, or to a carborundum diffuser inserted in the water pipe in the same way. These dry-feed units are designed for use where water under pressure is not available for operating chlorine injectors; they can be used only at points where the water pressure is less than 1.7 atm (25 lbf/in^2). Dry-feed equipment may give difficulty during cold weather because of the formation of " chlorine ice " at the diffuser where the gas comes in contact with the cold water. Heated diffusers are available to minimize this difficulty. In general, the solution-feed equipment is preferable because of the difficulty of injecting chlorine gas into the water being treated; the use of small motor or hydraulically driven pumps to operate chlorine ejectors is the best procedure when water under pressure is not available. When water under pressure is used, the pressure should be three times that of the water being chlorinated to ensure effective injector action.

Liquid-chlorine apparatus requires care and attention. Manufacturers' instructions for operating chlorinating apparatus should be always available to the operator, who should become thoroughly familiar with the mechanism and be competent to adjust it and make minor repairs when necessary.

Weighing the chlorine. The chlorine cylinders attached to the apparatus should be placed on accurate scales of proper range, and the weights of the cylinders should be recorded at regular intervals, at least once a day, in order to ascertain the actual quantity of gas being used. These recorded weights assist the operators in detecting stoppages or other difficulties, and also in determining when the chlorine cylinders are nearly empty. Recording scales are available for charting the actual rate of flow of the chlorine, thus furnishing a permanent record of the dose of chlorine applied. The smallest of these scales records on a chart calibrated from 0-5 kg (0-10 lb). Such scales are not applicable to small plants where chlorine doses are less than 0.5 kg (1 lb) per day.

Amount of chlorine to be fed from one cylinder. No 50-kg (100-lb) cylinder should be required to furnish to the apparatus more than 16 kg (35 lb) chlorine per day, otherwise the excessive drop in temperature in the cylinder will prevent evaporation. If more than this amount of chlorine needs to be fed, an additional cylinder for each additional 16 kg (35 lb) or part thereof should be attached to the apparatus.

Reserve supply of chlorine. An ample supply of chlorine should be kept on hand to tide over any delays in delivery or in transit.

Duplicate apparatus. Precautions should be taken against any interruption, through breakdowns or repairs to the apparatus, in the application of chlorine to the water delivered to consumers. For this reason duplicate apparatus should be provided. Duplicate parts such as gas meter tubes, control valves, and gaskets should be available in order that repairs may be made immediately. Auxiliary equipment, such as hydraulic pumps, should be in duplicate, or an electric motor-driven pump should be available as stand-by equipment. Chlorinators or auxiliary pumps driven by electric motors require a reliable source of current, otherwise the chlorinators may fail during sleet or electric storms. One alternative is to use a pneumatic-pressure storage tank for water with which to operate the chlorine injector during short periods of current interruption.

Temperature of apparatus. The room housing the chlorinator must be kept warm. The temperature of the chlorine cylinders and apparatus must not drop below 10°C (50°F). The operating panel must always be at the same temperature as the cylinders or higher, but never lower, otherwise the warmer gas will condense when it enters the colder apparatus and cause erratic operation.

Detecting leaks and preventing corrosion. Corrosion of the apparatus may be prevented by covering all exterior parts with some suitable oil or grease, such as petroleum jelly mixed with gasoline, and by immediately stopping all leaks that develop in the apparatus. The location of a leak can be readily detected by removing the cork of a bottle of ammonia and holding the neck of the bottle near the various parts of the apparatus. If chlorine is escaping, white fumes of ammonium chloride will be formed.

Repairs and maintenance. The supply of gas to a chlorinator should be shut off *at the tank* and the equipment drained of chlorine through the vent before repairs are attempted. All joints should be made tight with new gaskets and then tested for leakage. Permanent joints should be made by applying a thin paste of litharge and glycerin to the inner surfaces of the joints.

Apparatus not in service should be disconnected, thoroughly cleaned, oiled and kept in a dry place, and all openings into the apparatus should be plugged to exclude moisture from the interior.

Cleaning. Parts of the apparatus that are not readily accessible for cleaning may be reached by using an ordinary tobacco-pipe cleaner. The liquid in the manometer tube is usually carbon tetrachloride (CCl_4), a cleaning fluid that is also available under the trade name Carbona. This liquid may be used for cleansing purposes. However, concentrated sulfuric acid is used in some manometers.

Application of chlorine gas or chlorine solution. Chlorine should be applied to the water at a point where all the water being treated comes in contact with the chlorine gas or solution. Chlorine gas should not be applied at a depth of less than 1.2 m (4 ft) in open conduits or flumes, as it will not be completely absorbed.

Adjusting the apparatus. Operate all valves *slowly and carefully*, exerting only slight pressure on them, since undue stress will injure them. The auxiliary-tank valve should be used in stopping and starting the flow of gas. Changes in the rate of flow should be made by manipulating the control valve of the apparatus.

Water operating the injectors should be turned on first when starting all types of solution-feed apparatus. The control valve of the apparatus should be closed. Then the main-tank valve and auxiliary-tank valve should be opened, and lastly the control valve until the desired flow of gas is secured, as measured by the meter or manometer of the apparatus. Solution-feed apparatus should be shut down by stopping the flow of gas to the apparatus before the flow of water operating the injector is stopped, in order to remove all the gas in the apparatus at the time.

Summary of operation of gas-feeding chlorinators. The instruction manual furnished by the manufacturer should be followed for protective maintenance and operation. An operator must have confidence in his ability to follow instructions and to understand the functioning of gas-feeding chlorinators, otherwise repairs and major adjustments will be neglected. The best procedure is for the operator to be instructed by a representative of the manufacturer. The character of chlorine must be borne in mind, but there is no need to be unduly alarmed over its handling. Many prefer the use of hypochlorite-solution feeders so as to avoid the complexities of gas chlorinators, except for larger supplies, where the relative costs of liquid chlorine and hypochlorite compounds warrants the use of the much cheaper liquid chlorine. Local market conditions will determine the choice.

The following points should be kept in mind when operating gas chlorinators:

Metal caps over the cylinder valve should always be in place before the cylinders are moved.

Cylinders should be stored at temperatures above 10°C (50°F) but kept away from direct heat of steam pipes, radiators, etc.

Chlorine-gas leaks should not be traced by the sense of smell; aqua ammonia should be used to produce a visible cloud of ammonium chloride at the point of leakage, as described above.

Cylinders and chlorinators preferably should be installed in separate rooms located above ground level, and ventilation should be provided. The best ventilation is provided by inlet fans located in the walls near ceiling height and by outlets located at floor level and leading to the outdoors, so that the fans and their motors will not be exposed to gas fumes driven from the rooms through the lower outlets.

Both units of duplicate chlorinators should be in operating condition, except, of course, when one is being repaired. Spare repair parts should be available to permit prompt repairs. The best procedure is to alternate the use of the two chlorinators at weekly intervals.

Hypochlorite compounds

Sodium hypochlorite

Sodium hypochlorite solution may be purchased in many countries and is marketed under a variety of trade names which do not always indicate the nature of the solution. These solutions usually contain 3%-15% available chlorine by weight and are reasonably stable because of their high alkalinity. The solutions are free from suspended solids, and thus their application to water supplies is simplified because of the absence of clogging.

The long-established name, Javelle water, or eau de Javelle, refers to sodium hypochlorite. In countries under French influence this solution is rated in chlorometric degrees, with each degree representing an oxidizing power equivalent to 3.17 g chlorine per kilogram of solution.

Sodium hypochlorite solution may be prepared by the application of gaseous chlorine to a solution of sodium hydroxide, in accordance with the patented Laval process. The solution can also be prepared by adding chloride of lime and soda ash in accordance with directions given below. Sodium hypochlorite solution should be stored in cool, dark enclosures until needed. The solution can then be diluted as needed to a strength of 0.5% or 1.0% for more convenient application with solution-feed equipment.

Chloride of lime

Chloride of lime, or bleach, may be purchased in 12-oz and 10-lb cans and in 100- and 300-lb drums, but container sizes vary from country to country. This powdered disinfectant is unstable and hence should be purchased in relatively small quantities at frequent intervals. The cans or drums should not be opened until the material is to be utilized, and they should be stored in a cool, dry place. Commercial chloride of lime varies in strength from about 25% to 37% available chlorine by weight. It is usually safe to assume that the freshly opened material has a strength of

33% available chlorine, unless more definite information is obtainable as to how long the material has been stored, justifying a reduction in the assumed strength.

Chloride of lime contains excess lime which is insoluble, and thus solutions of this material contain suspended solids. It is necessary to dissolve chloride of lime in a separate solution tank where thorough mixing can be provided and where the insoluble solids may be allowed to settle. The supernatant clear liquid can then be siphoned into the storage tank and diluted to the desired strength with water. Ordinarily a solution containing 1% available chlorine is satisfactory. Careful studies have indicated that solutions so prepared lose about 7% of their available chlorine during a storage period of 14 days when exposed to light at a room temperature of about 21°C (70°F). The loss is about 3% when the solution is protected from light. It is evident, therefore, that storage tanks may be sufficiently large to enable solutions to be prepared for use over a period of at least two weeks without serious loss.

High-strength calcium hypochlorites

Several commercial brands of calcium hypochlorite are available that are considerably superior to chloride of lime. The fine granular material is much more stable than chloride of lime, so it deteriorates much less on storage, especially under tropical conditions. These products also are quite soluble, so relatively clear solutions may be prepared. The content of chlorine is 65%-70% by weight.

High-strength calcium hypochlorite products are also available in the form of tablets, each weighing 1/100 lb. This greatly facilitates the use of small amounts of the material, as weighing is not necessary.

These compounds should be stored in the original containers to exclude moisture. The storage place should be as cool as possible and protected from the sun and other heat sources, otherwise chlorine may be liberated in sufficient quantity to burst the container. Care must also be taken to prevent oil or other organic matter from coming in contact with these chemicals, otherwise explosions may occur. Solutions prepared from high-strength calcium hypochlorite may be stored in the dark for a period of at least two weeks without serious loss of chlorine.

Preparation of stable solutions

The solutions of chloride of lime or high-strength calcium hypochlorite may be stabilized by the addition of soda ash, whereby sodium hypochlorite is formed and calcium carbonate is deposited as a sludge. The clear, stable solution so produced may be stored for a period of four to five weeks at room temperature, when protected from the light, without serious loss of available chlorine, and can be applied with little or no difficulty from stoppage of equipment by suspended solids.

To prepare 114 l (25 UK gal; 30 US gal) sodium hypochlorite solution containing about 1% available chlorine by weight, dissolve the contents of a 1.7-kg (3.75-lb) can of high-strength calcium hypochlorite in 114 l (25 UK gal; 30 US gal) water, then add 1.36 kg (3 lb) soda ash to the solution, mixing thoroughly. (An empty can *loosely* filled with soda ash will contain about 3 lb.) The inert calcium carbonate formed by the reaction should be allowed to settle, and the clear supernatant sodium hypochlorite solution should be siphoned or decanted into the storage tank to which the solution-feed chlorinator is attached.

The net weight of the cans of these disinfectants is approximately 3.75 lb. The content of 70% available chlorine by weight would give 2.6 lb chlorine, which, added to 30 US gal or 250 lb of water, would produce 258 lb solution of 1.03% available chlorine, rather than 1%. Any slight excess in strength of solution, however, need not be compensated for, because any minor differences in dose are controlled by the use of the orthotolidine test to indicate whether more or less solution per unit of water treated is needed. When less than a full can of the powder is used to prepare smaller amounts than 30 gal of solution, the powder should be weighed to secure the correct amount. For instance, 14 g powder would be used for each litre of 1% solution (1.65 oz/UK gal; 1.38 oz/US gal).

The freezing point of hypochlorite solutions may be lowered very economically through the use of calcium chloride, which does not modify the disinfecting properties of the solution. For instance, the freezing point can be lowered to −12°C (10°F) through the use of sufficient calcium chloride to produce a 16% solution. As indicated in Annex 1, this requires 16 parts (by weight) of the chemical to 84 parts of water to make 100 parts of solution. This requires 190 g of calcium chloride for each litre of 16% solution (1.90 lb/UK gal; 1.58 lb/US gal).

Soft water should be used in the preparation and dilution of calcium or sodium hypochlorite solution when available so that the deposition of solids will be reduced to a minimum, thus ensuring more reliable operation of the chlorinator. Rain water provides a very satisfactory supply of soft water for this purpose and should be collected when only hard water is otherwise available.

A method of avoiding a deposit of sludge when preparing solutions of high-strength hypochlorite with hard waters is to add a special chemical that has the effect of preventing the precipitation of calcium compounds. Tetrasodium pyrophosphate may be used for this purpose. The procedure is to add 0.66 g of the compound to each 100 l of solution being prepared for each p.p.m. of hardness of the water used to make the solution (0.11 oz/100 UK gal/p.p.m.; 0.09 oz/100 US gal/p.p.m.). For instance, suppose the diluting water has a hardness of 150 p.p.m. and that 120 l solution is being prepared. Then $0.66 \times 150 \times 120/100 = 119$ g. (Corresponding values: to prepare 30 UK gal solution, use $0.11 \times 150 \times 30/100 = 5$ oz; to prepare

30 US gal solution, use $0.09 \times 150 \times 30/100 = 4$ oz.) Therefore 119 g compound would be added to the dilution water together with the required amount of chlorine compound to prepare 120 l stabilized hypochlorite solution.

Equipment for feeding hypochlorite solutions

Hypochlorite solutions (as well as " chlorine water " from gas chlorinators) are very corrosive, so only selected materials may be used in feeders, such as ceramics, glass, Fiberglas, plastics or special rubber. Commercial equipment is generally available where a market has been established. Commercial equipment should be selected in the light of local hydraulic conditions, points of application, the type and quality of the equipment and the availability of service from the supplier.

In general, the equipment may comprise: (*a*) motor-driven diaphragm pumps with variable stroke and in some cases with variable speed; (*b*) diaphragm pumps driven by larger-area water-operated diaphragms for service where electricity is not available; (*c*) constant-level calibrated orifice feeders; or (*d*) variable-head orifice feeders. The diaphragm-pump feeders may be manually controlled for those situations in which the rate of flow of water being treated does not vary to a significant extent, or they may be associated with metering devices to provide automatic compensation for changes in the rate of flow of water being treated. Electric motor-driven units should not be used where power interruption occurs and where the water being treated will continue to flow while the current is interrupted.

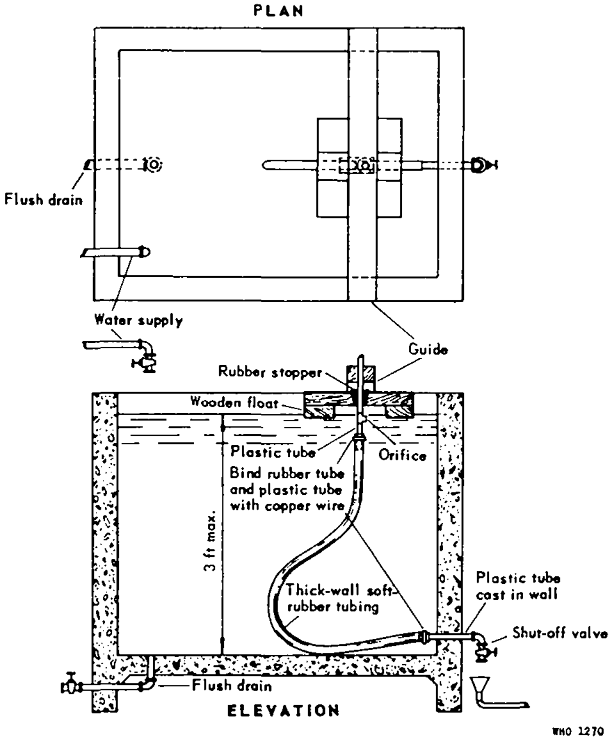
Home-made equipment. For emergency use or where funds are not available for commercial equipment, a number of home-made devices have been developed. Two very simple designs are described below for use where manual control is adequate, but unfortunately no simple device is available for the automatic control needed with variable flow of water.

A home-made unit, capable of close control and low rate of feed, uses a bottle containing about 40 l (10 gal) the stopper of which 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 10 cm (4 in) 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. Hard plastic tubes may be used instead of glass tubing. 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 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 or by controlling the rate of admission of air. The 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.

FIG. 12

MERGENCY EQUIPMENT FOR FEEDING HYPOCHLORITE SOLUTION



Modified from Cox (1952) p. 11 by permission of the New York State Department of Health.

A large-capacity home-made unit for preparing and also feeding hypochlorite solution is shown in Fig. 12. The concrete tank may be given any desired capacity, or a wooden barrel or plastic tank may be used. For instance, a tank with an effective size of 1.2 × 1.2 × 1.2 m (4 × 4 × 4 ft), filled with 1.8 m^3 (400 UK gal; 480 US gal) water in which 31.2 kg (68.6 lb) 70% calcium hypochlorite is dissolved would give 21.8 kg (48 lb) chlorine. This solution would chlorinate 20 700 m^3 (4.8 million UK gal; 5.75 million

US gal) water with a dose of 1.0 p.p.m. or would provide a reserve capacity for a maximum dose of 3.0 p.p.m. for one-third this amount of water. The inner surface of the concrete tank should be coated with bituminous enamel or plastic. The drain should be hard-rubber or plastic tubing with rubber or plastic stopcock. Tubing supported by the float should be plastic or hard rubber, although copper or brass pipe will give reasonable service. 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 1.3 cm ($^1/_2$ in) below the surface of the solution and 3 mm ($^1/_8$ in) in diameter will have a capacity of about 220 l/day (48 UK gal/day; 58 US gal/day). A head of 10.5 cm ($4^1/_8$ in) will give a flow of slightly less than 1.82 m^3 (400 UK gal; 480 US gal) per day; the actual flow may be calibrated by timing the period required to fill a liquid measure held above the funnel. The flow is 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.

Another home-made device consists of a corrosion-resistant solution tank, tubing from the bottom of this tank to a float valve in another tank at a lower elevation, and a control valve on tubing leading from this lower, constant-head tank. Valves of Bakelite or hard rubber should be used.

Chlorination procedures

Marginal chlorination

This may be defined as chlorinating with doses sufficient to provide a concentration of residual chlorine of 0.1-0.2 p.p.m. after 10 min contact, regardless of whether this is free-residual or combined-residual chlorine. This is the old practice which prevailed for some years after the development of the orthotolidine test in 1918 and which may still be followed in some places. Usually the dose is too low to oxidize free ammonia present in the raw waters, so combined-residual chlorine results. The low residuals noted, however, are not sufficient to ensure effective disinfection in the short period of 10 min, especially with the more alkaline waters or those of low temperature. This obsolete practice should therefore be replaced by free-residual chlorination.

Prechlorination

This is chlorination *prior* to filtration, which has many functions and advantages and one disadvantage, namely, the higher chlorine demand of raw water compared to filtered water, so that higher and more costly doses of chlorine are required than with postchlorination. The advantages of prechlorination may be summarized as follows:

The prolonged detention time in sedimentation basins permits the maintenance of high concentrations of free-residual chlorine for hours to ensure the very effective disinfection needed for the treatment of heavily polluted water. Doses required for this purpose may have to be over 5.0 p.p.m. to meet the chlorine demand of the water, oxidize free ammonia, etc. and leave 0.2-0.5 p.p.m. free-residual chlorine in the settled water.

The chemically active free-residual chlorine maintained for very effective disinfection will also oxidize iron and manganese, bleach colouring matter (frequently containing iron) and improve coagulation. The same higher concentrations of free-residual chlorine may be maintained for the secondary benefits even when bacteriological pollution is not serious, but in such cases the required doses may be significantly less, because the chlorine demand of the waters may be less and the free ammonia content lower. It should be emphasized that *free-residual* chlorine is required for effective disinfection and for the chemical reactions noted.

Prechlorination also will prevent the growth of algae on basin walls and will assist in the removal of algae by coagulation and sedimentation, because the dead cells of these organisms are more readily coagulated. The chlorine in the settled water will also destroy slime organisms on the filter sand and thus prolong filter runs and facilitate filter washing.

Postchlorination

This is chlorination after filtration. Chlorine at filter plants was originally applied to the filter effluent, and this procedure is still applicable when only moderately polluted waters are filtered and chlorinated. The advantages of prechlorination, even at the expense of increase in chlorine dose, have led to its more general adoption, with postchlorination with relatively small doses to provide a factor of safety and facilitate the maintenance of residual chlorine in distribution systems. The chlorine should be added to the filtered water entering clear wells or filtered-water reservoirs, so that the detention period may be as long as possible. Wooden baffles have been used to prevent short-circuiting of chlorinated water across clear wells to pump suction pipes.

Chlorine-ammonia treatment

This treatment process was developed many years ago to give a more persistent type of residual chlorine, with less tendency to produce chlorinous tastes. The reaction between chlorine and ammonia produces residuals in the form of chloramines, which are mild disinfectants because they are less active chemically, and hence less likely to form chlorinous tastes. The slower disinfecting action of chloramines, however, more than nullifies their favourable non-taste-forming characteristics. Chlorine-ammonia treatment has therefore not been used to any great extent since the development of free-residual chlorination. Another factor influencing this trend is the cost of chemicals and of equipment, similar to gas chlorinators, for applying anhydrous ammonia gas or equipment to feed solutions of ammonium sulfate. The elimination of these costs provides more than enough funds to pay for the higher doses of chlorine usually needed for free-residual chlorination, with all the added benefits of this newer chlorination procedure.

Nevertheless, there are many water supplies that have low pH values and long periods for contact between the disinfectant and bacteria and for which the technical control procedures needed for the maintenance of free-residual chlorine throughout distribution systems are not feasible. In such circumstances chlorine-ammonia treatment should be practised, bearing in mind, however, the slower disinfecting power of chloramines, as noted in Table 7, page 142. Minimum doses to produce residuals of 1.0 p.p.m. should be used when the reaction period is only 1 hour, and the procedure should not be used with supplies having pH values over 8.5-9.0.

Concentrations of combined-residual chlorine or chloramines of 1.0 p.p.m. or more at the treatment plant should ensure concentrations of 0.3-0.6 p.p.m. throughout distribution systems. Here again the *rate of disinfection* will be lower than with free-residual chlorine, so secondary bacterial pollution introduced into distribution systems will not be subject to rapid destruction.

Equipment

Compressed ammonia gas, aqua ammonia, ammonium chloride, or ammonium sulfate may be utilized as the source of ammonia. Ammonia gas purchased in iron cylinders is applied to water by dry-feed equipment identical in principle with dry-feed chlorinators. Chlorinators cannot be used because of the action of ammonia on their silver fittings. Ammonium salts may be applied to water by either dry- or solution-feed equipment manufactured to apply coagulants or hypochlorite solutions to water.

Anhydrous ammonia

Anhydrous ammonia is a colourless gas with a penetrating odour. When cooled and compressed the gas condenses into a colourless liquid which evaporates when the pressure is released. The legally loaded cylinder becomes completely full at a temperature of 63°C (145°F), and temperatures in excess of this will cause rupture of the steel cylinder. The pressure varies from 0 at −29°C (−20°F) to 26.5 atm (390 lbf/in^2) at 63°C (145°F). For this reason cylinders of anhydrous ammonia must be stored in a cool place and not exposed to temperatures exceeding 43°C (110°F). Three sizes of ammonia cylinders used in the US hold 50, 100, and 150 lb. The cylinder valves are fitted with right-hand threads. Only extra-heavy steel piping and a special ammonia valve should be used to connect the cylinders with the ammoniators, not copper or brass. The valves should always point upward when the cylinder is placed in a horizontal position on the scales at the time tubing is connected for feeding the gas to control equipment.

The presence of even small quantities of ammonia leaking from equipment may be detected by the appearance of a white cloud near the mouth of a bottle of hydrochloric acid when the stopper is removed and the bottle is held at points where leaks are suspected. Leaks should be searched for and repairs made before an appreciable quantity of ammonia gas is discharged into the atmosphere, since large concentrations are dangerous.

Emergency precautions. Special gas masks, made to absorb ammonia gas from the air being inhaled, should be available for emergency use. Ammonia gas leaking into the atmosphere rises, and anyone going to the rescue of a person overcome by this gas should keep near the floor, placing a wet sponge or cloth over the mouth and nostrils in the absence of a gas mask; the water will absorb the gas and prevent its inhalation to a considerable extent. A physician should be called immediately and the exposed person taken to a hospital.

Points of application of chlorine and ammonia

The points at which chlorine and ammonia are applied to water depend on local conditions and the purpose of treatment. The chemicals should be applied to permit the longest disinfecting period feasible, to compensate for the slower rate of disinfection with chloramines. Periods of two hours or more are preferable, as when the raw water is treated at filtration plants or when treatment is applied to water entering reservoirs or tanks. When used to prevent objectionable tastes, the ammonia must be added and thoroughly mixed before the chlorine is applied, otherwise some of the chlorine will react with taste-producing substances prior to the application of the ammonia. The chlorine should be added at a point when prior mixing of the ammonia is assured but before sufficient time has elapsed for the ammonia to be subject to biochemical oxidation with the production of nitrites before the chlorine is added.

Ratio of ammonia to chlorine doses

This ratio is not a fixed value but should be governed by local conditions. Ordinarily an ammonia dose one-third the chlorine dose is satisfactory for the disinfection of waters of low pH value, or when moderate concentrations of taste-producing compounds are present. When, however, phenolic compounds are creating taste difficulties, the dose of ammonia should be increased, and in extreme cases might have to be twice the chlorine dose. In other words, there must be an excess of ammonia present to react at once with the chlorine so that the chlorine will not react with the phenolic compounds to form chlorophenols.

A ratio of ammonia to chlorine exceeding 1:4 represents an excess of ammonia, so, as previously stated, the prevention of chlorinous tastes by the use of ammonia inevitably reduces the effectiveness of chlorination, because the resulting chloramines are less active than chlorine alone and also because the ammonia is a reducing agent and thus lowers the oxidation potential of the treated water. The ammonia dose should therefore be as low as is consistent with taste control and the maintenance of stable residual chlorine throughout distribution systems. If these desirable results cannot be secured without seriously curtailing the effectiveness of disinfection, because of high pH, high organic content, low temperatures, or short reaction periods, then the treatment procedure should be modified by the addition of chlorine before the ammonia.[1] The increase in effectiveness of disinfection secured in this way may not be accompanied by adequate taste control, in which case the chlorine dose should be increased to destroy the taste-producing substances by free-residual chlorination, or some supplementary taste-control procedure should be followed. In any case, the ammonia dose should be kept at a minimum.[2]

Free-residual chlorination

The principles of this procedure were discussed in detail above, inasmuch as they are the basic factors underlying control and operation. The several applications are here discussed separately.

Rapid disinfection

Free-residual chlorine, and chlorine dioxide as already discussed, are the only chlorine compounds that will disinfect water effectively in a 10-min period, although somewhat higher residuals must be maintained at pH values above 8.0. Free-residual chlorine, being relatively stable, persists in sedimentation basins and provides the benefits of prechlorination practice; its effect persists also in filtered-water reservoirs when a portion remains in filtered water or when it is added to the effluent in postchlorination.

Maintaining free-residual chlorine throughout distribution systems

Organisms of the coliform group may be introduced into distribution systems from open storage reservoirs, or when mains are laid or repaired, or as a result of back-siphonage of polluted waters from buildings served

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[1] When this procedure is followed, the free-residual chlorine is converted to chloramines and the dose of ammonia should be one-fourth the concentration of residual chlorine present when the ammonia is added.

[2] With this procedure, the free-residual chlorine is removed by dechlorination with the formation of nitrogen ($2\ NH_3 + 3\ Cl_2 = N_2 + 6\ HCl$), which requires an ammonia dose about one-eighth the content of residual chlorine.

by a system. This latter source of pollution can be minimized only by the enforcement of suitable plumbing codes, designed to prevent sewage or other deleterious liquids from being drawn into water piping from fixtures during periods when the water piping is under negative or zero pressure, as when valves on distribution systems are closed and polluted water may siphon back through fixtures. This problem is most acute when intermittent service prevails or when consumers pump water from mains in quantities greater than the supply, producing low or negative pressures. This is one of the important sources of pollution and stresses the basic need for continuous and adequate supplies of water.

Coliform organisms from these sources can be avoided by (*a*) covering storage reservoirs, (*b*) practising water-main disinfection, and (*c*) enforcing modern plumbing practice. But even when these measures are taken, experience has shown that the quality of filter effluents is superior to that of the water served to the consumers unless postchlorination is practised and free-residual chlorine is maintained throughout distribution systems. A note of caution is necessary here, because it is not practicable to maintain a sufficiently high concentration of residual chlorine in all portions of any given system to equal the chlorine demand of any heavy secondary pollution that may enter a limited portion of the distribution system, as with serious back-siphonage. If, however, the routine practice is to test for free-residual chlorine at selected sampling points on a system, the *absence* of residual chlorine at any one sampling point will indicate the existence of some nearby source of pollution that is reacting with the chlorine. This calls for investigation and corrective action.

The whole problem points up the importance of water-pollution control, effective filtration, and supervision of reservoirs and distribution systems, and emphasizes again that too much reliance should not be placed on chlorination alone.

The usual practice is to maintain from 0.2 to 0.4 p.p.m. free-residual chlorine throughout distribution systems. An initial period of adjustment is necessary, because the free chlorine in the filter effluent will react with any slime growths or deposits in the mains until they have been oxidized by the chlorine. Three transition periods will be encountered: the first while the chlorine demand of the organic material in the mains is removing all the residual chlorine (a period that may be shortened by systematic flushing of the system, starting at the treatment works and progressing to outlying mains); the second after the initial chlorine demand of the organic matter has been satisfied and while *combined-residual chlorine* is present owing to reactions with free ammonia; and the third after organic matter and ammonia have been oxidized by the chlorine, leaving *free-residual chlorine* as desired. The duration of these periods may be as long as one to six months, and chlorinous tastes may be produced. This difficulty may be minimized by prior flushing of mains and by starting free-residual chlorination, with gradual increment in the doses. For instance, the dose may be increased, to replace the previously maintained combined-residual chlorine, by 0.1 p.p.m. free-residual chlorine for one week and then further increased by 0.1 p.p.m. each week until *no* combined-residual chlorine remains at the treatment works and the desired full concentration of free-residual chlorine is reached.

To illustrate, assume that previous practice had been to maintain 0.2 p.p.m. combined-residual chlorine in the effluent of the treatment plant, without attempting to control the residuals in the distribution system. Then the following transition may be anticipated, although more than six weeks may be involved:

Period	*Residual chlorine at treatment plant, p.p.m.*	*Residual chlorine in water in centre of distribution system, p.p.m.*
Previous to start	0.2 combined-residual	None
Start	0.1 free-residual + 0.1 combined-residual	None
End of first week	0.2 free-residual	Trace of combined-residual
End of second week	0.3 free-residual	0.1 combined-residual
End of third week	0.4 free-residual	0.2 combined-residual
End of fourth week	0.4 free-residual	0.2 combined-residual + 0.1 free-residual
End of fifth week	0.4 free-residual	0.1 combined-residual + 0.2 free-residual
End of sixth week	0.4 free-residual	0.3 free-residual

The improved bactericidal results secured by maintaining free-residual chlorine throughout distribution systems will suppress biochemical reduction of sulfates to sulfides when waters are devoid of oxygen.

Chlorinators to be used for free-residual chlorination should be selected to have maximum capacities about 2.0 p.p.m. in excess of the maximum anticipated dose, that is, in excess of the chlorine demand as determined during periods of maximum degree of pollution and maximum temperature of a raw water, likely to occur during droughts, when there is a minimum of water to dilute the organic matter introduced with pollution. Unless this allowance is made, the maximum dose feasible may be less than that needed to oxidize the organic matter, destroy the free ammonia and produce the desired concentration of free-residual chlorine.

Chlorine dioxide treatment

Chlorine dioxide treatment was developed in 1940 to provide a very active oxidizing agent that would destroy phenols and other taste-producing substances introduced by pollution from industrial wastes. Chlorine dioxide is an active disinfectant and is superior to an equivalent concentration of free chlorine with alkaline waters having pH values in excess of about 7.5, and especially with the more alkaline waters produced by lime-soda softening. The disadvantage of this treatment process is the high cost of the sodium chlorite used in producing chlorine dioxide as needed—about US $0.50 per pound f.o.b. factory.

Chlorine dioxide is formed by adding a solution of sodium chlorite to the *concentrated* chlorine solution flowing in the outlet hose of a gas chlorinator. The ratio of the chlorine dose to the sodium chlorite dose theoretically is 1.0 to 2.5, but in practice proportionately more chlorine is added, giving a ratio of 1 to 2 or 1 to 1. The two solutions react in a special chamber made of glass and filled with ceramic spheres to produce mixing. The chlorine dioxide formed has a pale-yellow colour. If this colour is not evident by the time the solution is leaving the chamber, the chlorine dose should be increased until the colour appears. The concentrated chlorine solution has the pH of less than 3.5 required for this reaction, which is as follows:

$$2\ NaClO_2 + Cl_2 \rightarrow 2\ ClO_2 + 2\ NaCl$$

The chlorine dioxide reacts with orthotolidine solution to form the characteristic greenish-yellow colour, but the intensity of the colour is not a true measure of the concentration of chlorine dioxide but underestimates the concentration. In practice, this treatment process is effectively controlled when the orthotolidine test indicates an apparent concentration of 0.2-0.3 p.p.m. chlorine dioxide, which is equivalent in oxidizing power to about twice this concentration of free-residual chlorine.

Precise control is possible through the use of the "Amperometric Titrator" available from some laboratory supply houses or from manufacturers of chlorinators.

Disinfection of new and repaired mains

The disinfection of the inner surfaces of new and repaired mains is much more difficult than the disinfection of polluted water, because the chlorine must penetrate through organic matter coating the surfaces. Cleaning and flushing of the mains before disinfection are therefore essential for good results. Jute, yarn or other jointing materials should be disinfected as used by being placed in vessels containing a chlorine solution with a strength of 50 p.p.m. for a period of at least 30 min. Chlorine doses must be much higher (10-200 p.p.m.) than those normally used in water-chlorination practice. The lower limit of 10 p.p.m. is recommended for use when the chlorine remains in contact with the main for a period of 12 to 24 h, but the higher doses should be used when the mains are known to contain organic matter that cannot be removed by flushing, or when the available disinfection period must of necessity be short. The minimum disinfection period should be 30 to 60 min.

Several methods of disinfecting mains are available. Best results are secured by introducing chlorine gas or a hypochlorite solution in known amounts, by means of a portable chlorinator, into water flowing through the new or repaired section of the main, and then flushing to waste through an open hydrant at a known or estimated rate of flow.

Dry calcium hypochlorite compounds

Weighed quantities of chloride of lime or high-strength calcium hypochlorite may be placed in new mains, so that when water is introduced later, strong concentrations of chlorine will be present for disinfecting purposes. The difficulty, however, is that the powder will be flushed to the far ends of a main when water is introduced, hence there may be little or no disinfection in many portions of the main. There are times, however, when emergency repairs justify the use of the following procedure:

1. As the pipe is laid, place in it the appropriate amount of calcium hypochlorite, as determined from Table 8. For example, place 3.0 oz chloride of lime or 1.5 oz calcium hypochlorite in every three lengths of 8-inch-diameter pipe.

2. Open the end valve and farthermost hydrant on the new pipeline, and open the gate valve on the water main to which the new pipe is attached a few turns to admit water slowly to fill the new piping with water. Then close the hydrant and endmost valve on the line and allow the chlorinated water to stand in the pipe for 12 h.

TABLE 8

QUANTITY OF DRY DISINFECTANT REQUIRED TO PROVIDE DOSE OF ABOUT 50 P.P.M.

Diameter of pipe		Number of lengths of pipe per application	Chloride of lime (25 % available chlorine) per dose		High-strength calcium hypochlorite (70 % available chlorine) per dose	
mm	inches		grams	ounces	grams	ounces
100	4	4	30	1.0	15	0.5
150	6	4	75	2.5	30	1.0
200	8	3	90	3.0	45	1.5
250	10	4	210	7.0	75	2.5
300	12	3	225	7.5	90	3.0
400	16 [a]	2	270	9.0	105	3.5
450	18	3	380	12.5	150	5.0
500	20	2	315	10.5	120	4.0
600	24	2	450	15.0	180	6.0
750	30	2	705	23.5	290	9.0

[a] Cast-iron pipes less than 16 inches in diameter have lengths of 16 ft, larger pipes have lengths of 12 ft. Pipes of other materials have other lengths.

From Cox (1952) p. 126 by permission of the New York State Department of Health.

TABLE 9

AMOUNT OF CHLORINE GAS REQUIRED FOR DISCHARGE RATES NOTED
(dose about 50 p.p.m.)

Discharge rate			Chlorine gas	
l/s	UK gal/min	US gal/min	kg/24 h	lb/24 h
0.63	8.3	10	2.73	6
1.26	16.7	20	5.45	12
2.20	29.2	35	9.55	21
3.15	41.7	50	13.6	30
4.82	62.6	75	20.5	45
6.30	83.4	100	27.2	60
15.8	209.0	250	68.2	150

From Cox (1952) p. 131 by permission of the New York State Department of Health.

3. Then open successively all the hydrants on the line, beginning with that nearest to the supply end of the pipe, and flush the main until the water runs clear and is free from chlorine odour.

4. In case of dead-end pipelines, instruct occupants of houses connected to the pipe to open service taps until the chlorine odour disappears from the water.

Liquid chlorine

Liquid or gaseous chlorine may be admitted to new or repaired mains through the use of portable chlorinators. Special diffusers and silver tubing are needed when the gas is introduced into the mains through the use of dry-feed chlorinators. Special rubber hose, a plug valve, and a silver-tube diffuser are needed when a solution-feed chlorinator is used. This equipment should be used so as to apply gas at the rate shown in Table 9 for the estimated flow. Do not maintain full water pressure in the main, but throttle the valve to maintain a pressure of 1.7 atm (25 lbf/in^2) or less.

Procedure :

1. Install a special plug valve with stuffing and with a special gas diffuser or silver tube through which gaseous chlorine or chlorine solution is introduced into the mains.

2. Flush thoroughly the new pipeline to be disinfected.

3. Open the hydrant below the section being disinfected until the water flows to waste at a selected estimated rate. The valve on the *influent end* of the main admitting water should not be completely open, because the water pressure must be kept below 1.7 atm (25 lbf) when dry-feed chlori-

nators are used. A booster pump may have to be used to provide pressure of at least three times that in the mains to operate the injector of solution-feed equipment. Open the valve on the chlorine cylinder and adjust the chlorinator so that chlorine gas will be applied at the rate noted in Table 9.

4. Collect a sample of water from the hydrant overflow and continue the treatment until heavily chlorinated water appears, as indicated by a deep-red colour when the orthotolidine reagent is added to a sample of the water or by the results of the special orthotolidine test, that is, the drop-dilution method, described in the Supplement (page 322).

5. Stop the flow of water and chlorine gas by closing the appropriate valves.

6. Allow the water so treated to stand in the mains preferably for 12-24 h, after which the main should be thoroughly flushed until the water runs clear and has no odour of chlorine.

During the period when mains are being disinfected, taps in buildings connected to the section being chlorinated should be open for a short time to allow water to flow until the odour of chlorine is noticed, so that the service connexions will be thoroughly disinfected.

If water is to be supplied to consumers through a section of new mains before construction work on the whole system is completed, it is possible to disinfect each portion as laid by installing a special plug fitted with a rubber gasket that can be clamped in the bell or, if necessary, on the spigot end of the pipe. This plug should be fitted with a short nipple and stopcock, by which the air in the main can be released as the water is turned on and through which a sample of water can be collected for making the orthotolidine test.

Disinfecting reservoirs, elevated tanks and other waterworks structures

The disinfection of water stored in tanks or reservoirs to destroy bacterial pollution of the water is discussed elsewhere. The following directions apply specifically to the disinfection of reservoirs, standpipes and tanks following their construction or repair, to destroy any bacterial pollution incidental to such work. The walls and bottoms of such structures should be thoroughly cleansed when possible, to remove all dirt and loose material; this will greatly improve subsequent disinfection. One of three procedures may be followed in the disinfection of tanks, standpipes and reservoirs.

The first procedure is the direct application of a strong chlorine solution to the inner surfaces of the structures. The strong chlorine solution is prepared by dissolving 120 g (4 oz) chloride of lime or 45 g (1.5 oz) calcium hypochlorite to each 150 l (33 UK gal; 40 US gal) water. The powder should be made into a paste and then added to the water. If liquid bleach or sodium hypochlorite is used, add 600 ml (20 oz) 5% bleach or 200 ml (7 oz) 15%

(full strength) bleach to each 150 l (33 UK gal; 40 US gal) water. These proportions give a solution of 200 p.p.m. available chlorine. This strong solution is sprayed over the inner surface of the empty structure, using fruit-tree-spraying equipment, or is applied with whitewash brushes. The surface disinfected should remain in contact with the strong solution at least 30 min before the structure is filled with water.

The second procedure is the application of chloride of lime or calcium hypochlorite directly to the water used to fill the new or repaired structure. A dose of 50 p.p.m. should be used. The powdered disinfectant should be made into a paste and then thoroughly mixed with the water. For instance, 250 g chloride of lime or 85 g calcium hypochlorite per cubic metre of water is made into a paste and mixed with the water needed to fill the structure. This is equivalent to 1¾ lb chloride of lime or 10 oz calcium hypochlorite per 1000 UK gal, or 1½ lb chloride of lime or ½ lb calcium hypochlorite per 1000 US gal. Best results are secured when the paste is added in batches as the structure is filled. The chlorinated water should be allowed to remain in the structure for at least 30 min and preferably for 12 h before being replaced with potable water.

The third procedure is chlorination of the water entering the empty structure by the use of a portable chlorinator adjusted to apply chlorine or hypochlorite solution at the rate of 50 p.p.m. If the rate of flow of the water is unknown and the dose cannot be computed accurately, a heavy dose should be used so as to result in a concentration of residual chlorine sufficient to give a *deep-red to brown colour* when the orthotolidine reagent is added to a sample of the water, or a residual of 50.0 p.p.m. as measured by the drop-dilution method described in the Supplement (page 322). Some may prefer to use potassium iodide reagent for the determination of high concentrations of residual chlorine. The heavily chlorinated water should be allowed to stand in the structure for at least 30 min and preferably for 12 h before being replaced with potable water.

Emergency chlorination

Motor-driven diaphragm-pump hypochlorinators and special portable gas chlorinators designed for the disinfection of water mains serve as convenient means of providing chlorination during emergencies. During a period of drought, for instance, the regular supply may have to be augmented by the use of a stream remote from the main source of supply, so that the emergency supply cannot be given complete treatment in the filtration plant. If, however, an emergency supply is chlorinated and pumped directly into a conveniently located water main, the period for the action of the chlorine may be so short that very high chlorine doses are needed to secure rapid disinfection. This in turn will result in complaints of chlorinous tastes from consumers. The best procedure under these circumstances is to provide a contact period of at least 30 min and to practise free-residual chlorination to provide a residual of 0.5-1.0 p.p.m. in the water as it enters the distribution system.

Three very important factors must be borne in mind with emergency chlorination. First, effective disinfection is imperative, and this should not be made secondary to the production of a water free from chlorinous tastes. Second, emergency sources of supply may be subject to sewage pollution when suspended solids shield embedded bacteria from the action of the chlorine. Third, the concentration of residual chlorine in heavily chlorinated water, when mixed with the main supply, will be reduced immediately by dilution, so continued disinfecting action will be at a slower rate.

In view of the above, emergency supplies should be selected with great care and provisions should be made whenever feasible for contact in a tank or basin of some type. For instance, in an emergency due to drought, a small diversion dam may be constructed across a small stream and chlorine applied to the water flowing through the channel above the pool so impounded, with the pump suction being located at the dam. Another possibility is to isolate a portion of the bed of a larger stream with sandbags and to chlorinate the water entering the isolated basin, at a point as far as feasible from the location of the suction pipe of the pump.

When a seriously polluted stream must be used with treatment by chlorination alone, a safety factor may be provided by the practice of very heavy chlorination with doses associated with the disinfection of sewage, followed by detention and then by dechlorination to remove the portion of the residual chlorine in excess of 0.5-1.0 p.p.m. For instance, the water flowing into a detention basin may be chlorinated so as to provide a residual of 2.0 p.p.m. at the point where the pump suction pipe is located. The water as pumped then can be dechlorinated by adding sodium sulfite by means of the home-made solution-feeder described previously for hypochlorite solutions. The required dose is about 3.6 p.p.m. sodium sulfite for each 1.0 p.p.m. residual chlorine being removed. In the above example, to reduce the residual from 2.0 to 1.0 p.p.m., the required dose would be 3.6 p.p.m.

It is evident from the above that every effort should be made to locate an emergency source of supply that may be used to augment the flow of *raw water*, so that full treatment by filtration and chlorination can be provided during an emergency. Auxiliary sources of supply could perhaps be located under normal conditions, so that their use during emergencies may be planned and provision made for prompt action without panic.

Waterworks officials should keep in close contact with health officials serving their community, so that joint action may be taken when necessary to protect the health of consumers during an emergency. For instance, should effective chlorination fail because of breakdown of equipment that cannot be remedied at once, the health officials should be asked to warn the public to boil or disinfect all water used for drinking and cooking until notified that the emergency has passed (see Disinfection of Water in the Home, below). This notification will be difficult in areas without local radio service, so plans should be developed in advance for procedures in each community to facilitate prompt action when necessary.

Laboratory control

The most important test of all those used in the control of water-treatment processes is the orthotolidine test, and especially the modified orthotolidine-arsenite test, as described in the Supplement (page 317), because effective disinfection is basic to the delivery of water of safe sanitary quality.

Inasmuch as the temperatures and pH of the water affect the rapidity of the chlorination process, their determination should be routine.

Experience with any given supply will disclose what precautions should be taken in making the tests for residual chlorine. For example, if manganese or nitrites are not present to form a false colour with the reagent, and if combined-residual chlorination is being practised, then the conventional test is adequate. In many instances, however, manganese creates errors, the magnitude of which cannot be determined unless the complete orthotolidine-arsenite test is performed. Errors due to nitrites are seldom serious, except for the water of swimming pools with recirculation, where nitrites are formed by the biochemical oxidation of ammonia introduced with pollution by the bathers.

The second field of usefulness of the orthotolidine-arsenite test is in the determination of the chemical character of the residual chlorine: combined-residual chlorine (chloramines), free-residual chlorines, or a mixture of both.

Sampling points should be selected with due regard to the processes under control. For example, prechlorination usually is best controlled by maintaining a selected concentration of residual chlorine in the effluent of sedimentation basins. Postchlorination is best controlled by collecting samples from the effluents of clear wells or reservoirs, to disclose the content of residual chlorine after the detention period providde by these storage facilities—that is, in the water as it enters the distribution system. Sampling points on distribution systems should be selected so as to disclose the overall picture. Samples should not be collected as a routine from dead-end mains, but only from mains with free circulation of water. A good practice is to select sampling points in public buildings, such as schools, police stations and fire stations, where there is free access to selected taps.

The collectors of samples must bear in mind that the content of residual chlorine *in the main at the time of sampling* is being tested. Therefore the sampling taps should be opened and the water allowed to flow long enough to displace the water in the local piping. Furthermore, a more representative sample is secured when the house connexions used are short and when water after flushing is allowed to flow at a reduced rate while a clean litre bottle is filled with the chlorinated water flowing through the tap in, say, one minute, rather than the few seconds required to fill a small bottle. Portions of this sample would be tested.

The tests should be made at the sampling point, because there may be appreciable reduction in the content of residual chlorine or change in the relative content of free- and combined-residual chlorine in the samples before they can be transported to the laboratory or to the treatment plant for testing.

The routine bacteriological examination of samples of water collected at various selected points at treatment works will disclose the degree of removal of bacteria by the series of treatment processes. The most important samples are those of raw waters, filter effluents, postchlorinated waters and waters as distributed. The objective should be to have definite information on the content of coliform organisms in the raw water, so that treatment processes, especially chlorination, may be adjusted to produce a final effluent of safe sanitary quality. This is of such basic importance that it is discussed in detail in Chapter 9.

The drop-dilution method, described in the Supplement, page 322, as a variation of the orthotolidine test for use when high concentrations of residual chlorine are being measured, should be used when mains, tanks and reservoirs are being disinfected.

Disinfection of water in the home [1]

There are three general methods for the purification of water on an individual or domestic scale: (*a*) boiling; (*b*) chemical disinfection; and (*c*) filtration. These three can be used singly or in combination. (Only the first two are dealt with here.)

Boiling

Boiling is a satisfactory method for destroying disease organisms in water, and it is equally effective whether the water is clear or cloudy, relatively pure or heavily contaminated with organic matter. Boiling destroys all forms of disease organisms usually encountered in water—bacteria, spores, cercariae, cysts, and ova.

The amount of fuel required to boil water varies with the type of fire, stove and vessel. Under the conditions usually associated with the boiling of drinking water, it requires about 1 kg of wood to boil 1 litre of water, or 2 lb per quart.

To be safe, water must be brought to a " rolling " boil. The appearance of bubbles (simmering) is sometimes confused with boiling, as is the appear-

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[1] Clarke, R. N., in Wagner & Lanoix (1959), pp. 186-188.

ance of mist or steam over the water. None of these signs is sufficient indication that water has reached a boiling temperature. It is good practice to boil water in the same container in which it is to be cooled and stored and to use this container for no other purpose.

Boiling alters the taste of water because it drives out dissolved gases, particularly carbon dioxide. The frequent admonition to aerate water by stirring or by pouring from one container into another is badly founded, as this practice makes for a serious risk of recontamination in handling. Water left for a period of several hours, up to a day, in a partly filled container, where there is a good air surface exposed, even though the mouth of the container is covered, will lose most of the boiled taste. In any case, the taste is harmless, and much to be preferred to the risk of drinking polluted water.

Chemical disinfection

Chlorine

Chlorine is a useful disinfectant for drinking water and is effective against the bacteria commonly associated with water-borne disease. In the usual doses it is not effective against certain cysts and ova or against organisms embedded in solid particles. Chlorine enters almost instantaneously into chemical combination with organic matter in water, and in such combined form it is of no use for disinfection. Sufficient chlorine must therefore be added to satisfy the " chlorine demand " of water in addition to the amount required for bactericidal action. This combination with other substances may give rise to a chlorine taste, but the presence of such a taste is not a sufficient indication that a free chlorine residual exists. In general, cloudy water or highly polluted water containing large quantities of organic matter is not suited for chlorination. Turbid water can be filtered, and when clear it can be successfully chlorinated.

Chlorine is easiest to apply in the form of a solution. A good stock solution to use in treating water contains about 1% available chlorine, and this is about the strength of antiseptic solutions such as Zonite, Milton, or Javelle water.

Laundry bleaches, available as liquids under a variety of trade names, usually contain about 3%-5% available chlorine and can be diluted easily to 1%. Dakin's solution contains about 0.5% available chlorine.

Bleaching powder, or chlorinated lime, is a white powder which contains about 30% available chlorine when freshly made. However, the strength of this powder rapidly vanishes after the can is opened; storage over a long period also results in loss of strength. In using chlorinated lime, it is best to use the whole can at once, immediately after opening, in making up a stock solution. The inert lime will settle in a few hours, leaving the active chlorine in the clear solution.

Another type of powder is high-test hypochlorite, which contains about 70% available chlorine. Cans of this powder should be kept as cool as possible; there are instances of cans bursting when left in the hot sun or in hot storage places. The powder can be used to make a stock solution in the same way as chlorinated lime. High-test hypochlorite is more stable than chlorinated lime and will keep its strength better after the can is opened, so it is not necessary to use a whole can at once. However, even this material will lose its strength in a month or two after a can has been opened.

Solutions containing much over 10% available chlorine are unstable in warm climates. Chlorine solutions should be kept in brown or green bottles and stored in dark places.

To make 1 litre (1 quart) of about 1% stock solution, add the requisite amount of water to any of the following:

250 ml (1 cup) of liquid laundry bleach, or
40 g (2½ tablespoons) of chlorinated lime, or
15 g (1 tablespoon) of high-test hypochlorite.

Smaller quantities of stock solution can be made by using proportionate amounts. Zonite or Javelle water can be used without dilution.

To chlorinate water, add three drops of 1% solution to each litre or quart of water, or 1 fl. oz. to 32 UK gal. If the water is clear but highly coloured, like very weak tea, or if it has a noticeable sulfur odour, the dosage should be doubled. After being dosed, the water should be thoroughly mixed and left to stand for 20 min or longer before being used. Sufficient mixing can be obtained by pouring water from the container in which it is chlorinated into a storage vessel.

Chlorine can be secured in tablet form. Some commercial tablets available are those known as Halazone, Chlor-dechlor, and Hydrochlorazone. Directions given on the package should be followed carefully. For example, Chlor-dechlor has a double action. This tablet first disinfects the water with a special chlorine compound, then, after the outer portion of the tablet dissolves, the inner core neutralizes the remaining chlorine to reduce the taste. If more than one tablet is to be used, the whole amount must be put in at one time. Tablets added after the dechlorinating core of even one of these pills is dissolved are completely ineffective.

Iodine

Iodine is a first-class disinfecting agent. Tincture of iodine can be used to disinfect water; normally, two drops of 2% tincture of iodine are sufficient for 1 litre of water. Water that is cloudy or muddy, or water having noticeable colour, even when clear, is not well suited for disinfection with iodine. Turbid water can be filtered, and the clear filtrate can then be treated. If the water is heavily polluted, the dose should be doubled. There is no harm in using the higher dosage, but it will produce a medicinal taste.

Iodine compounds such as tetraglycine potassium tri-iodide have been made into tablets that are effective against amoeba cysts, cercariae, leptospira, and some viruses as well. Some of the commercial names are Globaline, Potable Aqua, and Individual Water Purification Tablets. These tablets appear to be among the most useful disinfecting devices developed to date.

Potassium permanganate

Potassium permanganate has frequently been used for the disinfection of water. It is a powerful oxidizing agent, hence its action is rapidly spent in waters containing organic material. The commonly used dosage is 1 part in 2000, or 0.5 g/l. Potassium permanganate may possibly be effective against the cholera vibrio but is of little use against other disease organisms. Water treated with potassium permanganate in time produces a dark-brown precipitate, which is noticeable as a coating on glass or porcelain vessels and which is difficult to remove without scouring. Potassium permanganate is not satisfactory and is not recommended for water disinfection.

CHAPTER 9

STANDARDS OF POTABLE WATER QUALITY AND WATER-BORNE DISEASES

This chapter is intended to summarize the objectives of water treatment for the control of water-borne diseases and hence deals primarily with the removal of pathogenic organisms from water. On the other hand, potability is also determined by physical and chemical factors and by the content of toxic substances. Reference is made to the WHO monograph, *International Standards for Drinking-Water*, for a detailed discussion of all factors influencing water quality and the laboratory procedures for testing water quality.

Quality standards for treated water

Toxic substances

Table 10 gives the maximum allowable concentrations of toxic substances according to the International Standards.

Lead is of significance when lead tubing is used to conduct corrosive waters. Selenium content is not usually determined in routine water analysis, but it may be of significance in food grown in areas where the soil includes this mineral. In such instances the waters from the area may contain selenium, hence the inclusion in the tabulation. Arsenic is present in some waters from areas where volcanic action has occurred or where arsenical insect sprays are used. There are no known reports of arsenical poisoning from a public water supply. Chromium and cyanide (also cadmium) are present in potable waters only as a result of pollution by wastes from the metal-plating industry.

Substances that may affect health

In addition to the substances listed in Table 10, there are other substances that may be injurious to health under certain conditions if present in high concentrations. Some of these substances are regarded as essential

TABLE 10
ALLOWABLE CONCENTRATIONS OF TOXIC SUBSTANCES

Toxic substance	Maximum allowable concentration, mg/l
Lead (as Pb)	0.05
Selenium (as Se)	0.01
Arsenic (as As)	0.05
Chromium (as Cr hexavalent)	0.05
Cyanide (as CN)	0.2
Cadmium	0.01
Barium	1.0

From World Health Organization (1963) International *Standards for Drinking-Water*, p. 27.

constituents of drinking-water, and too low a concentration of them also may have an adverse effect on health.

Fluorides, although beneficial when present in concentrations of 0.8-1.0 p.p.m., cause mottled enamel of the teeth when present in potable waters in concentrations in excess of 1.0-1.5 p.p.m. (see Chapter 10).

Nitrates, which may be present naturally in potable waters or may result from seepage from cesspools, sewers, etc. or from the use of fertilizer, are harmful to some infants but do not affect growing children or adults. Nitrates present in water consumed directly or indirectly in prepared food by infants may be reduced to nitrites in the intestinal tract, leading to infantile methaemoglobinaemia. This may occur when the nitrate content exceeds 45 mg/l, although there is a trend to consider a content over 10.0 p.p.m. as significant. These concentrations of nitrates occur only in some ground waters subject to seepage from near-by-sources of pollution. The remedy is to use water from another source in the feeding of infants.

Substances affecting the potability of water

The content of many chemical substances in potable waters varies widely in different parts of the world, and rigid standards of quality cannot be established. However, Table 11 gives maximum concentrations generally acceptable to consumers, and also maximum allowable concentrations beyond which potability is seriously impaired.

It will be noted that Table 11 lists many substances that are involved in water-treatment procedures, hence this tabulation serves as a summary of objectives in the production of potable waters.

TABLE 11

ACCEPTABLE AND ALLOWABLE CONCENTRATIONS OF CHEMICAL SUBSTANCES AND PROPERTIES AFFECTING POTABILITY

Substance or property	Maximum acceptable	Maximum allowable
Total solids	500 mg/l	1500 mg/l
Colour	5 units [a]	50 units [a]
Turbidity	5 units [b]	25 units [b]
Taste	unobjectionable	—
Odour	unobjectionable	—
Iron (Fe)	0.3 mg/l	1.0 mg/l
Manganese (Mn)	0.1 mg/l	0.5 mg/l
Copper (Cu)	1.0 mg/l	1.5 mg/l
Zinc (Zn)	5.0 mg/l	15 mg/l
Calcium (Ca)	75 mg/l	200 mg/l
Magnesium (Mg)	50 mg/l	150 mg/l
Sulfate (SO_4)	200 mg/l	400 mg/l
Chloride (Cl)	200 mg/l	600 mg/l
pH range	7.0-8.5	Less than 6.5 or greater than 9.2
Magnesium + sodium sulfate	500 mg/l	1000 mg/l
Phenolic substances (as phenol)	0.001 mg/l	0.002 mg/l
Carbon chloroform extract (CCE; organic pollutants)	0.2 mg/l	0.5 mg/l [c]
Alkyl benzyl sulfonates (ABS; surfactants) . . .	0.5 mg/l	1.0 mg/l

[a] Platinum-cobalt scale.

[b] Turbidity units.

[c] Concentrations greater than 0.2 mg/l indicate the necessity for further analyses to determine the causative agent.

From World Health Organization (1963) *International Standards for Drinking-Water*, p. 29.

Water-borne diseases

The first significant development in environmental sanitation was the establishment of the role of water in the transmission of typhoid fever and cholera, and the effectiveness of filtration and later chlorination in the control of these two intestinal diseases. This knowledge has been used so effectively in many countries that the incidence of such diseases has been greatly reduced, although much needs to be done in other countries, where water-borne diseases still are very prevalent. The challenge, therefore, is the more widespread use of established procedures for the collection, treatment and distribution of public water supplies, as discussed in this monograph.

Pathogenic bacteria when present in potable waters cause typhoid fever, bacillary dysentery and cholera, also the less important paratyphoid fever. A heterogeneous group of bacteria cause non-specific diarrhoeal diseases, which may be controlled by the procedures used for typhoid fever or cholera. The only *protozoan* associated with water-borne diseases is *Endoamoeba histolytica*, causing amoebic dysentery. A *virus* is known to be the cause of infectious hepatitis, and other, unknown, viruses are implicated as causes of diarrhoeal diseases. (See Miller, 1962, for a comprehensive discussion of this subject.)

The above diseases may be water-borne when waters are contaminated with the intestinal discharges of persons who have such a disease or who are carriers of the pathogenic organisms if the contaminated waters are not treated to remove or destroy the organisms. Control procedures, therefore, are directed to: (*a*) disinfection in the home of contaminated stools of those suffering from one of the diseases; (*b*) treatment of sewage; (*c*) utilization of natural self-purification processes; and (*d*) treatment of potable-water supplies when necessary.

The control of cholera, paratyphoid fever and diarrhoeal diseases is governed by the factors discussed below for typhoid fever. *Endamoeba histolytica*, however, is very resistant to the action of disinfectants, requiring concentrations of residual chlorine of 8-10 p.p.m. for its destruction. This organism, fortunately, is removed by effective filtration. In any case, all the evidence indicates that very heavy pollution must occur for amoebic dysentery to be water-borne, as when the potable supply in a building is directly polluted through a cross-connexion with plumbing fixtures used by those harbouring the organism. There is no record of a community-wide epidemic of water-borne amoebic dysentery.

The same gross degree of pollution of potable waters appears to be necessary before the virus disease, infectious hepatitis, is transmitted by water. Studies have indicated that effective coagulation, sedimentation and filtration will remove about 90%-99% of the virus with the floc, although the very minute virus would otherwise pass through filters. These studies also have demonstrated that effective destruction occurs with prechlorination doses producing 0.3-0.4 p.p.m. free-residual chlorine, followed by contact in sedimentation basins, filtration and then postchlorination with doses producing 0.3 to 0.4 p.p.m. free-residual chlorine in the final effluent.

Free-residual chlorination will also destroy other viruses of intestinal origin and those associated with diarrhoeal diseases where known pathogenic bacteria have not been found in stool specimens. More work is needed to establish limits for free-residual chlorine, but the above values for the virus of infectious hepatitis appear to be satisfactory as a guide.

The history of epidemics of water-borne diseases clearly establishes their association with (*a*) unforeseen pollution of hitherto safe supplies; (*b*) the use of polluted supplies without treatment; (*c*) failure of treatment processes due to faulty equipment, improper or unskilled operation or outright neglect by operators or inadequacies of the water-treatment processes; (*d*) secondary pollution of the water being distributed. All these factors are subject to control and correction. In the last analysis, corrective action rests with local water officials and operators and requires the use of trained personnel, supported by adequate funds. If funds are limited, as frequently is the case, the local officials have the more difficult task of selecting the important steps they are in a position to take to ensure the delivery of a water of safe sanitary quality, neglecting if necessary the factors affecting only potability. This compromise usually involves stressing effective disinfection.

Degree of treatment and bacteriological quality

Many ground waters are of safe sanitary quality and free from objectionable mineralization, so they may be used without treatment, provided that the wells or springs are properly located, developed and otherwise protected. Surface waters, however, are exposed to direct pollution and usually do not meet the standards of quality for potable waters. Treatment of surface waters, therefore, is necessary. The degree and type of treatment depends on the character of the water and the degree of pollution of public health significance.

The objective of treatment is to ensure the delivery of potable water to the public that is of safe sanitary quality, free from significant concentrations of toxic substances, of attractive nature in so far as tastes and odours are concerned, reasonably soft and yet non-corrosive, and with a very low content of iron and manganese so that the water is non-staining to laundry and plumbing fixtures. More recently this list has been supplemented by the objective of having a sufficient concentration of fluorides in the water to ensure the growth of sound, decay-resistant teeth among the children consuming the water.

The treatment processes required to transform raw water into potable water meeting standards of quality are discussed separately in this monograph. It is desirable, however, to appraise the combined influence of several of these processes in the removal of bacteriological pollution, because significant public health principles are involved that affect the potability of water and also the reliability of that potability. Other methods of treatment may fail without serious public health significance, but the removal of pathogenic organisms must not fail if the public is to be adequately protected.

There is a common misconception that effective water treatment will remove *all* bacteriological pollution, that is, that filtration or chlorination if properly applied will produce satisfactorily treated water. In actual fact the content of bacteria in a treated water is proportional to the content in the specific raw water. This means that the *degree* of removal of bacteria by any given process will be reasonably constant for any specific plant and water as long as effective operation is maintained. This degree of removal usually is expressed as the percentage of the content of selected bacteria in raw waters that is removed by treatment. Increases in the degree of pollution of raw waters are accompanied by a corresponding percentage increase in the bacterial content of the treated waters. Hence the establishment of any maximum limits for the permissible number of selected bacteria *remaining* in treated waters imposes a restriction on the maximum degree of pollution of the raw waters.

The pathogenic bacteria that cause water-borne diseases cannot be readily isolated from water as a routine procedure, so there is no absolute criterion for distinguishing between safe and unsafe waters, that is, between those containing no pathogenic organisms and those containing such organisms. The generally accepted procedure, therefore, is to use a group of bacteria normally present in the intestinal tract as indicators of pollution of intestinal origin. The likelihood of intestinal pathogens being present naturally increases with the degree of pollution, as shown by the number of " indicator bacteria " present, and also with the prevalence of water-borne diseases in the population contributing pollution to specific supply. The indicator bacteria selected for the purpose are those of the coliform group, that is, the bacteria associated with the colon, or large intestine, of animals and man. Faecal streptococci also have been used as indicator organisms, especially in Europe, but quantitative norms have not been standardized. (See section, Bacteriological Examinations of Water, in the Supplement.)

The pathogenic organism of chief importance in the control of water-borne diseases is *Salmonella typhosa*, which causes typhoid fever. Studies have shown that the *ratio of coliform organisms to S. typhosa* remains approximately the same when (*a*) infectious sewage is subject to treatment, (*b*) self-purification occurs in a stream, and (*c*) polluted water is treated. The magnitude of this ratio naturally depends on the incidence of typhoid fever among the population contributing sewage. This relationship, shown in Table 12, indicates that great precautions must be taken to ensure the treatment of waters in areas where water-borne diseases are prevalent—which may be the very places where water-treatment facilities are deficient and where funds are not available for sewage treatment and stream-pollution control.

Bacteriological quality standards

Experience has established that water in which the number of coliform organisms is below a certain range of values will not contain pathogenic bacteria. This range is specified in drinking-water standards such as the *International Standards for Drinking-Water.* The number of coliform organisms found to be present in samples of water is expressed statistically as

TABLE 12

INCIDENCE OF TYPHOID FEVER ON A WATERSHED IN RELATION TO THE *SALMONELLA TYPHOSA—ESCHERICHIA COLI* RATIO IN SEWAGE OR POLLUTED WATER

Annual typhoid fever case rate per 100 000 pop.	No. of *S. typhosa* per million *E. coli*
1	3
5	6
10	9
50	19
100	26

From American Water Works Association (1950) p. 41 by permission of the Association.

the " most probable number " of coliform organisms per 100 ml of sample, termed " MPN index ". For treated water, the *International Standards* (page 21) sets the following requirements:

> " In 90 per cent. of the samples examined throughout any year, coliform bacteria shall not be detected or the MPN index of coliform micro-organisms shall be less than 1.0. None of the samples shall have an MPN index of coliform bacteria in excess of 10.
>
> " An MPN index of 8-10 should not occur in consecutive samples. With the examination of five 10-ml portions of a sample, this would preclude three of the five 10-ml portions (an MPN index of 9.2) being positive in consecutive samples.
>
> " In any instance in which two consecutive samples show an MPN index of coliform bacteria in excess of 8, an additional sample or samples from the same sampling point should be examined without delay. This is the minimum action that should be taken. It may also be desirable to examine samples from several points in the distribution system and to supplement these with samples collected from sources, reservoirs, pumping stations and treatment points. In addition, the operation of all treatment processes should be investigated immediately."

For the purpose of applying the day-by-day results of the routine bacteriological examination of a series of samples, the tabulations given in the *International Standards* should be used to determine the MPN index. For instance, if five 10-ml portions of each sample were examined as a routine, the possible combinations of positive and negative results would give the following MPN indices:

Results of examination of five 10-ml portions of each sample for coliform organisms	*MPN index*
0+, 5–	<2.2
1+, 4–	2.2
2+, 3–	5.1
3+, 2–	9.2
4+, 1–	16.0
5+, 0–	>16.0

The practical meaning of the standards, therefore, is that 90% of the series of daily samples from a given supply should have *no* 10-ml portions positive, that is, that they should have MPN indices of less than 2.2.

The second aspect of the standards, serving as an alternate, is that 90% of a series of samples should have MPN indices of less than 1.0. This index of 1.0 cannot be determined by the examination of five 10-ml portions of each sample; indices of less than 1.0 can be determined only if five 100-ml portions of each sample are examined or if more than 100 ml of each sample is examined by the membrane-filter technique. The examination of five 100-ml portions of each sample would give the results shown in the following series:

Results of examination of five 100-ml portions of each sample for coliform organisms	*MPN index*
0+, 5−	<0.22
1+, 4−	0.22
2+, 3−	0.51
3+, 2−	0.92
4+, 1−	1.60
5+, 0−	>1.60

It is evident that any sample examined in this manner would meet the second aspect of the standards if not more than three in five of the 100-ml portions are positive for coliform organisms. In the same way, the examination of 500 ml of each sample by the membrane-filter technique would disclose that samples containing less than 5 coliform organisms would meet the standard of MPN indices less than 1.0, that is, less than 1.0 per 100 ml.

The third aspect of the standards is more complex in meaning. In substance, it implies that any series of samples having MPN indices less than 1.0 could include *by chance* an occasional sample having indices of 2.2, 5.1 or 9.2 but less than 10, because bacteria are not distributed uniformly in samples. If, however, consecutive samples were to show three or more 10-ml portions positive—that is, have indices of 9.2, 16.0 or higher—then the prevailing supply, as samples, would have a degree of pollution well above the norm of an index less than 1.0, so the pollution would be real and not due to chance or to the sampling error. Such results require immediate action in collecting an additional sample from the same sampling point and in corrective measures as to treatment, especially chlorination.

Application of the standards to the results of examination by the membrane-filter technique would require that 90% of the samples of water as delivered to consumers should contain less than 1 coliform organism per 100 ml, and none of the series of samples should contain more than 10 coliform organisms per 100 ml.

The United States Public Health Service *Drinking Water Standards* (1962) expresses the results with the membrane-filter technique in a manner comparable to those with the fermentation-tube procedure, as follows:

> " The arithmetic mean density of all standard samples examined per month by the membrane filter procedure shall not exceed one per one hundred milliliters (1.0/100 ml). Coliform colonies per standard sample shall not exceed 3/50 ml, 4/100 ml, 7/200 ml, or 13/500 ml in:
>
> (*a*) two consecutive samples;
>
> (*b*) more than one standard sample when less than twenty are examined per month; or
>
> (*c*) more than 5% of the standard samples when twenty or more are examined per month.
>
> " *Note :* The phrase ' standard sample ' means each of the series of daily samples used each month in appraising a supply."

It should be noted that (*a*) the supervision of a water supply should be aimed at the delivery of water *individual samples* of which from day to day do not contain coliform organisms; (*b*) as pointed out earlier, a monthly average of 1/100 ml cannot be computed unless portions larger than 100 ml are examined, to give realistic numbers below 1/100 ml.

Quality standards for water sources

The above standards for the quality of drinking-water delivered to consumers impose restrictions on the permissible degree of pollution of the raw water. The following standards (*International Standards for Drinking-Water*, 1963) provide, for waters to be used as sources of public supply, measures of the degree of pollution or indications of the need and suitability of the waters for treatment:

Physical quality

The limiting value for colour should be set at 300 units, on the basis that a value of less than 300 units indicates an acceptable quality for treatment and anything over 300 units indicates that special treatment may be needed to provide water meeting the drinking-water standards.

With regard to turbidity, no specific figures are given, since the problem of turbidity and the treatment needed is one that has to be decided for each individual case and cannot be subject to a general limit.

Chemical quality

	Maximum allowable
Substances affecting potability:	
Total dissolved solids	1500 mg/l
Iron	50 mg/l
Manganese (assuming that the ammonia content is less than 0.5 mg/l)	5 mg/l
Copper	1.5 mg/l
Zinc	1.5 mg/l
Magnesium plus sodium sulfate	1000 mg/l
Alkyl benzyl sulfonates (ABS; surfactants) [a]	0.5 mg/l

Substances affecting health:	*Maximum allowable*
Nitrate as NO_3	45 mg/l
Fluoride	1.5 mg/l
Toxic substances:	
Phenolic substances	0.002 mg/l
Arsenic	0.05 mg/l
Cadmium	0.01 mg/l
Chromium	0.05 mg/l
Cyanide	0.2 mg/l
Lead	0.05 mg/l
Selenium	0.01 mg/l
Radionuclides (gross beta activity)	1000 $\mu\mu c$/l

Chemical indicators of pollution:	*Minimum indicative of pollution*
Chemical oxygen demand (COD)	10 mg/l
Biochemical oxygen demand (BOD)	6 mg/l
Total nitrogen exclusive of NO_3	1 mg/l
NH_3	0.5 mg/l
Carbon chloroform extract (CCE; organic pollutants)[b]	0.5 mg/l
Grease	1 mg/l

[a] This value has been established on the basis of the maximum sensitivity of the presently accepted analytical procedures.

[b] Any amount greater than 0.2 indicates the necessity for further analytical determinations of the causative material.

Bacteriological quality standards

Classification	*MPN/100 ml coliform bacteria*[a]
I. Bacterial quality applicable to disinfection treatment only	0-50
II. Bacterial quality requiring conventional methods of treatment (coagulation, filtration, disinfection)	50-5000
III. Heavy pollution requiring extensive types of treatment	5000-50 000
IV. Very heavy pollution, unacceptable unless special treatments designed for such water are used; source to be used only when unavoidable	> 50 000

[a] When more than 40% of the number of coliform bacteria represented by the MPN index are found to be of the faecal coliform group, the water source should be considered to fall into the next higher category with respect to the treatment required.

The publication, *Effectiveness of Water Treatment Processes* (United States Public Health Service, 1961) offers a comprehensive discussion of this subject.

Radioactivity and water quality

The pollution of surface water supplies by radioactive material is an increasingly serious problem where reactors are in operation; where radioactive isotopes are used in medicine, industry and other civilian pursuits; and where " fall-out " of such material occurs from the atmosphere contaminated by nuclear weapons tests. Every effort is being made to prevent concentrated wastes from entering surface waters, but cooling waters from reactors contribute small amounts of radioactive material. Use of isotopes is subject to supervision by agencies supplying the material. Detailed efforts are being directed, where the problem exists, to (*a*) prevention of pollution of surface waters; (*b*) routine radiological examination of samples of selected raw waters; and (*c*) research on the removal of radioactive materials from potable water supplies.

It is premature to draw final conclusions from these studies, especially as there are so many factors that must be considered, such as the characteristics of specific isotopes involved in a given water. It may be stated, however, that so far the total radioactivity of raw surface waters examined, including naturally occurring radioactivity as well as " fall-out " and wastes from reactors and from civilian use of radioactive isotopes, has not reached limits considered to be significant.

International Standards for Drinking-Water has tentatively established the following amounts as a guide to the maximum acceptable limits of radioactivity in drinking-water used by consumers over a lifetime in large populations:

	Maximum concentration, μμc/litre
Strontium-90	30
Radium-226	10
Gross beta concentration (in absence of strontium-90 and alpha-emitters)	1000

Conventional water-treatment plants will remove 70%-90% of radioactive materials from surface waters. Radioactive material deposited on the ground and percolating into the soil with rain water is removed to an extent exceeding 99%. This is fortunate, as it means that ground-water supplies, at least from deep wells, will be essentially free from this type of pollution. In fact, civil-defence procedures include the planned use of ground-water supplies during possible emergencies involving the use of nuclear bombs, when the degree of pollution of surface waters would greatly exceed safe limits.

Waterworks officials therefore should ascertain which authorities in their country are in a position to provide guidance and technical help of a specialized nature.

CHAPTER 10

FLUORIDATION

Purpose

The fluoridation of public water supplies is designed to supplement the natural fluoride content of potable waters so as to provide the proper amount of this nutritional element in the water consumed by children, thus aiding in the development of sound teeth with a high resistance to decay. Fluoridation is therefore a dental health measure, which would be instigated and supervised by health agencies. The active assistance and co-operation of waterworks officials are essential, because they must be responsible for the selection, installation and operation of equipment for applying fluorides. In order that treatment-plant operators may have some information as to the principles and purposes of fluoridation, this chapter offers a summary of the subject.

It should be noted that fluoridation, in spite of its desirable purpose, should not be attempted where funds are so limited that the basic function of supplying ample quantities of water of safe sanitary quality is not being met.

Support

This dental health measure has received the official support of the World Health Organization and other health, scientific and technical organizations. The scope of the support is shown by the broad range of supporting organizations in the USA: National Research Council, American Public Health Association, American Dental Association, American Medical Association, and American Water Works Association. The policy of fluoridating public water supplies to secure these public health benefits also has the support of medical and dental associations in Australia, Brazil, Canada, Czechoslovakia, Egypt, Finland, the German Democratic Republic, Great Britain, Greece, Italy, New Zealand, Norway, Sweden, South Africa, and Yugoslavia. It is doubtful if any other health or water-treatment programme has received such widespread support.

This very important development in public health should receive the co-operation of local water-supply officials when action is taken by health officials to start the treatment in a community.

Conventional water-treatment procedures are used with conventional laboratory control. Water-fluoridation research was started in 1945 and was continued for 10 years in three communities in the USA. Since then a mass of practical experience has accumulated through the fluoridation of an increasing number of supplies. By the end of 1959, 1049 supplies serving a total population of 36 000 000 residing in 1878 communities in the USA, and 63 communities with a total population of 1 200 000 in Canada, were being fluoridated. It is estimated that about 40 more supplies throughout the rest of the world are so treated. The result has been a well-established lowering of the incidence of dental decay by 60%-70% among the children consuming the treated water.

The cost of the procedure is discussed below, but it should be emphasized here that no other public health measure or programme is so economical as a means of reaching the children of all homes, irrespective of their economic status, customs or habits.

History

In 1906 it was noted that some then unknown constituent of the public water supplies of various communities was responsible for the presence in certain communities of so-called mottled enamel of the teeth, while it was absent in neighbouring cities. In 1931 it was found that this constituent was fluorine, present in waters as fluorides. The study of mottled enamel in a number of communities led in 1938 to the discovery that mottled teeth, while stained, were relatively free from decay. Finally it was found that this relative freedom from decay could be conferred by the presence of about 1.0 p.p.m. fluoride ion in the water supply, and that only concentrations over 1.5 p.p.m. fluoride ion caused mottled enamel. The conclusions from these early investigations were that the required amount of fluorides needed for sound development of teeth was not present in many potable waters; that a concentration of about 1.0 p.p.m. was desirable in potable waters; and that concentrations should be less than 1.5 p.p.m. to prevent mottled enamel.

Pilot studies were started in 1945 in Newburgh, N.Y., Grand Rapids, Mich., and Brantford, Ont., and were continued for 10 years. The condition of the teeth and the general health of all children of school age were investigated in all three communities. Briefly, the results showed a reduction of tooth decay of 60%-70% as compared with the incidence of tooth decay in the same communities before treatment was started and as compared with three neighbouring control communities whose public water supplies contained no fluorides. The treatment procedure was found to be safe, effective, practical and economical and could be adequately controlled and supervised.

Objections to fluoridation

The fluoridation of water supplies has been objected to for several reasons, which should be examined rather than evaded. Some have held that the practice represents " mass medication ", which they object to for reasons of religion or custom. This practice, however, has nothing to do with medicine, but rather restores when necessary a deficiency in the content of a nutritional element that is present in favourable concentrations in other waters.

Others have objected that the practice is wasteful, in that the whole water supply of a city is treated in order that the small portion actually consumed by growing children will contain the needed element. This might be valid were the cost *per child* more than that noted later.

Still others contend that fluorides are " dangerous " substances that should not be added to potable waters, or that other procedures of supplying fluorides to children should be followed. These latter two objections warrant analysis, so that waterworks operators will be in a position to meet the arguments.

Acute morbidity is produced in man by the consumption of about 0.25 g sodium fluoride. This amount, if present in one glass of water, would be equivalent to about 1000 p.p.m. or 4 tons per million gallons of water. By contrast, the recommended dosage of sodium fluoride is 2.3 p.p.m. (19 lb/million US gal) to provide the 1.0 p.p.m. fluoride ion used in fluoridation practice. No chemical feeder selected for the required dose could be operated so incorrectly as to apply an amount over 400 times greater than intended. Mottled enamel is not produced by the consumption by children of water containing 1.5 p.p.m. fluoride or more *unless such water is consumed for an estimated period of 3 months or more.* Practical experience has shown that while doses may be in the zone of 1.0-1.5 p.p.m. for short periods of time, the usual doses average less than the intended doses. In any case, continuous errors of more than 50% for periods of 3 months are most unlikely with the control procedure to be discussed.

The addition of sodium fluoride to table salt, rather than to potable water, has been advocated, because of the well-established practice of iodizing salt to prevent goitre. Iodine needed for this purpose is only about 0.01 mg per capita per day, and uniformity of intake of the iodine is not essential, so the facts that salt intake is variable in adults and that infants receive little or none do not limit the value of iodized salt. But obviously salt could not be relied on to provide uniform consumption of fluorides by children. The same is true of milk, because its consumption is not regular or universal. Furthermore, the multitude of sources of milk prevent suitable supervision of milk fluoridation by authorities. Treatment of limited amounts of potable water in individual homes would be possible, through the use of tablets or of solutions placed in medicine-dropper bottles for accurate measurement, but it would rely too much on the ability, resources and reliability of householders for it to be a practical means of benefiting all the children in a community. In fact, one important advantage of the fluoridation of public water supplies is that it is an economical and controllable means of benefiting all children, irrespective of conditions in the individual homes, schools or other places in a community where children consume water.

Fluorides in natural waters

Fluorides are present as fluorspar in sedimentary rocks (limestone, sandstone) and as cryolite in igneous rocks (granite). These fluoride minerals are nearly insoluble in water, so fluorides will be present in ground waters only when conditions favour their solution. The maximum fluoride ion content encountered in natural waters is about 14 p.p.m. except under special conditions when the evaporation of ground water during periods of drought may result in an increase in the fluoride content of the remaining water. Shallow wells under these conditions have yielded water containing over 30 p.p.m. fluorides, enough to produce a bitter taste.

There are no comprehensive data as to the world-wide distribution of fluoride-bearing waters; it is known, however, that most natural waters have a content less than 0.3 p.p.m. The augmenting of this low natural content by fluoridation is therefore a widespread need. In the USA, natural water containing approximately the right amount of fluorides is available to only about 3 million people. Waters with a fluoride content of 1.5 p.p.m. or more tend to occur in certain areas but are even more restricted in number. Only about 1 million people were served by such waters in the USA out of a total of 118 million served by public water-supply systems at the end of 1959. Accordingly, mottled enamel of the teeth is not a general problem. In fact, one community may have a number of wells yielding waters of widely differing fluoride content, so that mixing the waters will result in an average content less than 1.5 p.p.m.

Removal of excess fluorides from water

There is no economical means of removing fluorides by the usual treatment procedures, except when softening of waters containing magnesium is practised. When magnesium is not present in hard waters, dolomitic lime should be used in the softening process to furnish the magnesium needed to precipitate the fluorides.

A limited number of fluoride-removal plants have been constructed using beds of tricalcium phosphate as a base-exchange material. These beds are regenerated by flushing with a 1%-1.5% sodium hydroxide solution, followed by a dilute hydrochloric acid wash, or by the use of carbon dioxide to neutralize the excess alkalinity resulting from the initial wash. Each cubic metre of such beds will remove by base exchange 240-530 g fluoride ion per cycle. Regeneration requires 0.6-1.4 kg sodium hydroxide per 17 000 mg of fluoride ion removed. Using average values of 385 g per cubic metre of bed and 1 kg sodium hydroxide per 17 000 mg fluoride ion removed, the following general values are obtained:

Each cubic metre of the bed will remove 1 mg/l from 385 m^3 of water. For example, the removal of *all* the fluorides from four-fifths of a supply containing 5 p.p.m. fluorides and the mixture of this with one-fifth of the raw water will produce a mixed effluent containing 1 p.p.m. fluorides. Then for every 385 m^3 of raw water, 308 m^3 would be treated and mixed with 77 m^3 of the raw water to produce a mixed effluent containing 1 p.p.m. fluorides. Regeneration would be with a solution containing 1 kg per 100 litres of solution. Inasmuch as 385 g fluoride has been removed per cubic metre, the amount of sodium hydroxide needed is 385/17 of 1 kg, or 22.6 kg. Then 22.6 × 100 litres, or 2260 litres, of the dilute solution would be used per cubic metre.

These special plants are costly to operate, so it is generally better, if possible, to abandon a well yielding water of high fluoride content than to provide treatment.

Practice of water fluoridation

The procedures used in applying fluorides to public water supplies are similar to those used in the coagulation of water. The most important difference is the need for more accurate control of chemical doses, but this is greatly facilitated by the fact that the dose is constant, irrespective of the temperature or other characteristics of the water being treated. Complex processes are not involved, and laboratory control requires only a single test.

Fluoride chemicals

Commercial sodium fluorides, sodium silicofluoride, fluosilicic acid and ammonium silicofluoride have been used in the USA for fluoridation; hydrofluoric acid is so active and corrosive that it has been used at only one plant and since abandoned. Fluosilicic acid is a concentrated liquid that must be transported in glass containers or rubber-lined drums. Ammonium silicofluoride is available only in limited quantities in the USA. All these chemicals are toxic, and precautions for their handling are indicated below.

Sodium silicofluoride, in comparison with sodium fluoride, has the advantages of cheaper cost and greater content of fluorine, and the disadvantages of its dust-producing fine-powder form and its much lower solubility. The properties of the two chemicals are summarized in Table 13.

Fluosilicic acid is available as a by-product in the manufacture of phosphate rock, Freon gas, etc., and when neutralized becomes sodinm silicofluoride. Its properties are shown in Table 14.

TABLE 13

DATA FOR SODIUM FLUORIDE AND SODIUM SILICOFLUORIDE

Property		Sodium fluoride	Sodium silicofluoride
Formula		NaF	Na_2SiF_6
Molecular weight		42	188
Form		Fine powder or granular	Fine powder
Purity: as % F compound		95	99
as % F ion		43	60
Bulk density	kg/m^3	1200 [a]	1150 [b]
	lb/ft^3	75 [a]	72 [b]
Weight of compound for each p.p.m. F ion	kg/1000^3	2.32	1.67
	lb/million UK gal.	23.2	16.7
	lb/million US gal.	19.3	14.0
Solubility: lb/100 lb at 0°C (32°F)		4.0	0.43
lb/100 lb at 15°C (60°F)		4.19	0.62
Weight of commercial compound per unit volume of saturated solution at 15°C (60°F)	g/l	44.1	6.2
	lb/UK gal	0.44	0.062
	lb/US gal	0.37	0.052
Volume of saturated solution for each p.p.m. F ion, l/1000 m^3; gal per million US gal		50.4	269
Weight of compound per unit volume of 1 % solution	g/l	10.5	above saturation
	lb/UK gal	0.104	above saturation
	lb/US gal	0.087	above saturation
Volume of 1 % solution or slurry for each p.p.m. F ion, l/1000 m^3; gal per million US gal		232	167
pH of 1 % solution		6.5	—
pH of saturated solution		6.0	3.5

[a] Grades of sodium fluoride are available ranging from 610-1520 kg/m^3 (38 to 95 lb/ft^3).

[b] Sodium silicofluoride is available in two grades: the regular, weighing 1150 kg/m^3 (72 lb/ft^3) and the fluffy, weighing 680 kg/m^3 (55 lb/ft^3).

Fluosilicic acid is delivered in rubber-lined steel drums or glass bottles. The acid should be either undiluted or diluted with 20 parts or more water to 1 part acid, otherwise a precipitate will form. It is therefore best fed directly from the 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 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 lower cost of sodium silicofluoride justifies its use when the required dose exceeds 110 g/h (4 oz/h) the reliable minimum capacity of dry-chemical feeders. Fluosilicic acid is suitable for supplies of all sizes, and its use depends largely on 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.

TABLE 14

DATA FOR FLUOSILICIC ACID (30% SOLUTION)[a]

Formula: H_2SiF_6

Molecular weight: 144.1

Fluoride ion concentration: 23.7%

Specific gravity: 1.27

Weight of 1 UK gal: 12.7 lb

Weight of 1 US gal: 10.6 lb

Weight of 1 m^3: 1270 kg

Weight of F ion in 1 UK gal: 3.01 lb

Weight of F ion in 1 US gal: 2.51 lb

Weight of F ion in 1 m^3: 300 kg

4.22 units by weight, = 1 unit by weight F ion

Weight of solution required to yield 1 p.p.m. F ion: 4.22 kg/m^3 (42.2 lb/million UK gal; 35.1 lb/million US gal)

pH: 1.2

[a] Commercial fluosilicic acid solution varies in strength from 23% to 30% of the acid *by weight*. The values tabulated here should be reduced proportionally when acid of less than 30% strength is purchased.

Modified from Cox (1960) p. 65 by permission of the US Agency for International Development, Washington, D.C.

Chemical feeders

Chemical feeders for applying fluoride compounds should be selected with consideration of the factors discussed in Chapter 5 (see p. 70). 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 provision is made to discourage 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, equipment at present available provides the ranges shown in Table 15; it should be remembered that it is not good practice to select a feeder that will have to be operated at its minimum or maximum capacity.

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

TABLE 15

RANGE IN CAPACITY OF CHEMICAL FEEDERS FOR FLUORIDES

Type of feeder	Chemical and strength of solution	Range in capacity per minute	Range in volume of water treated (1.0 p.p.m.) per second
Diaphragm pump [a] (hypochlorinator)	Sodium fluoride 4.0% (saturated)	2.5-500 ml	0.7-143.0 l [b]
Diaphragm pump	Sodium silicofluoride 0.4% (saturated)	2.5-500 ml	0.07-14.0 l [b]
Diaphragm pump	Fluosilicic acid 30% (23.7% F)	2.5-500 ml	4.2-850.0 l
30-l hypochlorinator with variable-head control	Sodium fluoride 2.0%	2-20 ml	0.5-5.6 l
30-l hypochlorinator	Fluosilicic acid 30% (23.7 F)	2-20 ml	3.4-34.0 l
Volumetric dry feeder	Sodium fluoride	0.46 g-36.6 kg	3.5-16.6 l
Volumetric dry feeder	Sodium silicofluoride	0.46 g-36.6 kg	4.7-22.3 l
Gravimetric dry feeder [c]	Sodium fluoride	75.0 g-36.6 kg	0.5-29.4 m^3
Gravimetric dry feeder [c]	Sodium silicofluoride	75.0 g-36.6 kg	0.7 to 41.0 m^3

[a] Diaphragm pump and saturator permit use of 4% saturated solution.
[b] Dilute solution will treat even less.
[c] Gravimetric feeders are preferable for large suppliers.

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be fitted with a spring-loaded valve when it is connected to a suction pipe or a conduit that may be under negative pressure, so as to prevent siphonage of the solution into the pipe. The feed water flowing to sodium fluoride saturators should be softened by a small zeolite unit when its hardness exceeds 100 p.p.m.

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 min when sodium fluoride is to be applied and preferably 15 min when sodium silicofluoride is to be applied. Provision should be made to supply at least 100 l of water per kilogram of sodium fluoride being dissolved in the solution pot (10 UK gal/lb; 12 US gal/lb) or at least five times this amount for sodium silicofluoride. The water should be admitted to the solution pot in such a manner as to prevent back-siphonage into the water supply, and provision should be made to prevent the contents of the pot from 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 on facilities for preventing the release of dust.

Dust-control facilities

Provision should be made for the disposal of empty bags, drums or barrels either by burning or by some other means that will minimize exposure to dust. A metal wheelbarrow or tray should be available for the temporary handling of leaking or damaged bags.

The following alternative equipment 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 of large enough capacity to provide an air flow of at least 60 m/min (200 ft/min) through the opening in the cover through which the chemical is dumped;

(3) an enclosure forming a part of the feeder, in which a bag or small drum of the chemical may be placed before the container is inverted and emptied;

(4) a home-made adapter for 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 connexion 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.

Operation

Doses

Inasmuch as the objective of fluoridation practice is to provide 1 mg fluoride ion per capita per day, the desirable content in the water is related to the consumption of water as a beverage, which in turn is influenced by climate. Water consumption in temperate climates is about 1.5 l/day, so a concentration of 1.0 p.p.m., or 1.0 mg/l, would provide an intake of 1.5 mg per capita per day. A dose of 0.8 p.p.m. has been used in Latin America, where the water consumption is estimated to be about 2.0 l per capita per day, thus giving an intake of about 1.6 mg per capita per day. The following range in doses has been suggested:

Range in annual average of daily maximum temperature		*Dose of fluoride ion, p.p.m.*
°C	°F	
10-12	50-54	1.2
12-14	54-58	1.1
14-17	58-64	1.0
17-21	64-70	0.9
21-26	70-79	0.8
26-32	79-90	0.7

Obviously, the actual dose would be adjusted to allow for any natural fluoride content in the filtered water, so the above are *total values.* Also, lime used in treating a supply should be checked for possible fluoride content and allowances made accordingly.

Calibration of chemical feeders

Dry-chemical feeders may have been calibrated at the factory for the feeding of alum and lime but not for specific fluoride compounds. Therefore units available for fluoridation should be calibrated, using during the trial period the specific chemical to be applied.

Point of treatment

Fluorides are removed to some extent by coagulation and to a marked extent by lime-soda softening, so fluoride treatment should be applied to filtered waters, preferably as they enter clear wells or reservoirs, so that storage will tend to equalize any irregularities in dosing.

Sampling

When fluoridation is started, some of the fluorides are absorbed as the waters flow through reservoirs and distribution systems, but after a matter of weeks the content of fluorides throughout a system approaches that in the plant effluent. The routine procedure should be to supplement the samples collected daily at the plant with representative samples collected at regular intervals from the system. Samples collected for the control of fluoride application should include at least one sample each day of filtered water prior to treatment and another sample of the final effluent. In many instances supplies do not contain measurable amounts of natural fluorides, so periodic checks to determine any change in this situation are sufficient.

Laboratory control

Inasmuch as the fluoridation of water supplies is a health measure, local health agencies will wish to participate, especially in the independent appraisal of the content of fluorides in the water delivered to the public. Arrangements should be made for the exchange of records between water and health officials.

In many instances local health departments are better able to carry out precise laboratory procedures than water departments. Under these circumstances, arrangements should be made for plant control to be exercised by plant control tests, but *official* tests of fluoride content in the supply should be made by health officials following standard procedures. The results with specific kits, such as the Taylor Analyzer described in the Supplement, pages 278 and 331 ff., should be compared with those secured by standard procedures, to check the kit and reagents and also the operator's interpretation of colour values when using the kit. It may be found that the apparent concentration of fluorides in a sample, as determined with a kit, is lower *on the average* than the *same sample* gives when it is examined by standard procedures. Thus a kit may give values about 0.2 p.p.m. less than those given with standard procedures, which will guide the operator to use the *apparent value* of 0.8 p.p.m. to secure the desired 1.0 p.p.m. fluoride content.

The weaknesses in the test for fluorides and precautions to be followed to allow for the influence of interfering substances are discussed in the Supplement (see pages 331 and 333), in which information is given as to the use of kits. Most of the precautions must be taken also when standard laboratory procedures are followed. Constant research is being conducted with a view to providing an even more accurate method of testing for the fluoride content of waters.

Costs

The cost of the fluoridation of public water supplies depends on the local cost of chemicals and equipment, the number of individual sources of supply that have to be treated in any one community, and whether fluoridation is the sole treatment process or only one of a number of processes supervised by the operating staff. Usually fluoridation is added to treatment by chlorination or filtration. Untreated well supplies, however, may be fluoridated without greatly increasing the duties of pump operators. In general, therefore, there is little or no increase in cost of supervision when a supply is fluoridated.

Small communities may be served by a number of isolated wells, whereas very large communities may have only one or two sources of supply and treatment plants. This affects the capital charges, so that comparisons are difficult. For instance, chemical feeders suitable for small supplies may cost US$ 500 to $1000, whereas large-capacity units may cost only US$ 5000, with total cost of complete installation of chemical storage facilities and auxiliary equipment under US$ 50 000. The capital charges may be spread over at least ten years.

Costs of chemicals vary in different areas. The following table is based on costs in the USA, and other unit costs of chemicals could readily be compared.

Chemical	*Amount required for 1.0 p.p.m. F, lb/million gal*	*Cost per lb,* US $	*Cost per million gal,* US $	*Per capita annual cost,*[a] US $
Sodium fluoride	19.3	0.12	2.32	0.084
Sodium silicofluoride	14.0	0.09	1.26	0.045
Fluosilicic acid	35.1	0.09	3.16	0.115

[a] Based on water consumption of 100 gal/day per capita.

In general, the total costs have varied from as low as US$ 0.03 per capita a year for large cities to about US$ 0.25 for some small communities. In England the average cost is 5/4 per capita per year.

The indirect savings from reduced need for dental services are difficult to estimate. Only children would benefit at first, but gradually they would grow to adults and replace the older people who have not benefited, so that eventually all age-groups would be included. This means that a substantial reduction in the extent of dental decay would eventually be achieved for all. The savings resulting from these benefits may best be appraised by comparison with the conventional dental-care programme of England, which costs

13/6 per capita per year, to which must be added the cost of the dental programme in schools. Cost data from New Zealand disclose that complete dental care of children in the age-group of 11-15 years costs £5 per year per child.

The above costs of water fluoridation are moderate in comparison with those of the collection, treatment and distribution of water, and well below the cost of dental care otherwise needed.

CHAPTER 11

CORROSION CONTROL

Corrosive water may be defined as that which has the property of dissolving metals to an appreciable degree. The term " corrosion " as applied to a water-distribution system also includes corrosion of piping by the surrounding moist soil, but excludes the corrosion by electric currents known as " electrolysis ".

Few realize the economic losses incidental to the corrosion of distribution systems and plumbing fixtures or the serious reduction in carrying capacity of mains that may occur even when corrosion is not active enough to produce " red water ". For this reason the need for corrosion control is not always apparent.

Unfortunately corrosiveness of natural waters cannot be completely eliminated through available treatment procedures, but it can be controlled to a marked extent, at a cost much less than the value of the piping subject to the ravages of corrosion.

Theory

Fundamentally, corrosion is due to the solution of the exposed metal by a physicochemical action related to the character of the water and the metal. The rate of solution of the metal is inversely proportional to the alkalinity, hardness (calcium content), and pH value of the water and is directly proportional to the content of dissolved oxygen and carbon dioxide and the temperature and velocity of flow of the water. There are many other factors, however, that complicate the problem, some of which are not clearly understood. Many natural waters are less corrosive than waters of the same apparent quality after treatment. This is especially the case when softening, which modifies the mineral content of the water, is practised. There is growing evidence that the silica content of water exercises a marked protective influence. Furthermore, corrosion is aggravated when sulfate-reducing organisms are active, as when water stands in dead-end mains. In fact, the trend is to give greater weight to the deleterious influence of biochemical changes associated with the growth of " iron bacteria ", that is, *Crenothrix* and *Leptothrix*, and with sulfate-reducing organisms. While more information is needed as to the role of bacteria and higher organisms in the field of corrosion, there is definite evidence that the control of " pipe growths " by chlorination will materially reduce corrosion in many waters.

The metal dissolves in the form of positive metallic ions, the electric charge being taken from the hydrogen ions of the water, thus indicating the significance of pH and alkalinity. This action leads to the deposition of a polarizing film of gaseous hydrogen, thus stopping electrochemical action unless the hydrogen is removed by the scouring action of the water or is oxidized by the dissolved oxygen in the water:

$$\underset{\text{Iron}}{Fe} + \underset{\text{hydrogen ions}}{2H^+} = \underset{\text{iron ions}}{Fe^{++}} + \underset{\substack{\text{molecular}\\ \text{hydrogen}}}{H_2}$$

In the presence of dissolved oxygen the hydrogen is removed:

$$2H_2 + O_2 = 2H_2O$$

This simplified statement does not account for the effects of localized cathodic areas but gives a practical basis for the treatment procedures to be discussed. The corrosion of exposed metal, therefore, may be prevented by coating the pipe surface with protective paints or cement, or by treating the water to make it less corrosive, or even to make it deposit a protective mineral film on the pipe surface.

With the oxygen ordinarily present in water, corrosion by waters of low pH values containing carbon dioxide is as follows:

The ferrous or iron ion mentioned above reacts with water to form an intermediate product, ferrous hydroxide:

$$Fe + 2H_2O = Fe(OH)_2 + H_2$$

As quickly as the ferrous hydroxide is formed, it reacts with the carbon dioxide in the water (carbonic acid, H_2CO_3) to form *soluble* ferrous bicarbonate:

$$Fe(OH)_2 + 2H_2CO_3 = Fe(HCO_3)_2 + 2H_2O$$

The product so formed, being soluble, is distributed in the water and does not remain attached to the pipe surface. The soluble ferrous bicarbonate is oxidized by the dissolved oxygen in the water to form ferric hydroxide and to liberate carbonic acid:

$$4Fe(HCO_3)_2 + 10H_2O + 0_2 = 4Fe(OH)_3 + 8H_2CO_3$$

This reaction, as mentioned above, occurs throughout the body of water and not on the pipe surface, so that the ferric hydroxide is precipitated as suspended particles in the water (red water) and does not form a protective film.

Conversely, the reactions which occur in alkaline waters of high pH value in the absence of carbon dioxide are as follows: The ferrous ion at the moment of liberation from the pipe surface reacts with the hydroxyl ion (OH) present in alkaline waters to form an *insoluble film* of ferrous hydroxide:

$$Fe^{--} + 2(OH) = Fe(OH)_2$$

This may be indicated as ferrous oxide and water:

$$Fe(OH)_2 = FeO + H_2O$$

The dissolved oxygen in the water will oxydize this film of ferrous hydroxide to form another insoluble film of ferric hydroxide:

$$4FeO + 6H_2O + O_2 = 2Fe_2O_3 + 6H_2O = 4Fe(OH)_3$$

Thus, under suitable alkalinity and pH value of the water, insoluble films are formed on the pipe surface *whether or not dissolved oxygen is present.* The conditions favourable for these reactions are closely related to the conditions governing the precipitation of calcium carbonate, although silica appears to have a greater influence than generally realized. In fact, the ferrous and ferric hydroxide films are somewhat porous and do not fully protect the pipe surface unless calcium carbonate also is precipitated and forms part of the film. The simple " marble test ", described elsewhere (see page 310), is used to determine the alkalinity and pH value of a given water at calcium carbonate equilibrium. The results are used to control the application of alkali to precipitate the calcium carbonate.

Indications of corrosion

Serious corrosion of distribution systems is indicated by red water, or rust in water. Tuberculation of mains may occur, however, in the absence of red-water difficulty. Pits and tubercles are formed under the rust coating when dissolved oxygen penetrates the rust in spots and replaces the chloride and sulfate ions that corrode the iron, to form more ferrous chloride and ferrous sulfate, thus beginning another cycle.

Serious corrosion is indicated by the decrease in dissolved oxygen content and increase in pH value and iron content of water flowing through the mains. Probably the best measure of the degree of corrosion is the increase in the frictional loss in pipelines caused by the corrosion of the pipe surface and the accumulation of tubercles. This may be determined by accurate pitometer surveys, to disclose the loss of pressure due to pipe friction. Organic matter will increase corrosion when decomposition occurs near dead-end mains. This may be prevented by treatment. Under other conditions, organic matter may form a protective film.

Influence of corrosion on the carrying capacity of water mains

Corrosion will seriously decrease the capacity of ferrous water mains even when the rate of corrosion is less than that required to produce appreciable concentrations of iron in the water, owing to the development of a rough, tuberculated surface on the inside of the mains. Pitometer surveys in the US have indicated that the average reduction in the carrying capacity of mains through the corrosive action of water of varying pH values is as high as 85% during a period of 30 years. This information is summarized in Table 16, which represents average conditions.

TABLE 16

REDUCTION IN CAPACITY OF CAST-IRON MAINS OWING TO CORROSION AND TUBERCULATION

pH value of water	Loss in capacity during a period of 30 years, %
6.0	85
6.5	60
7.0	45
7.5	35
8.0	30

From Cox (1952) p. 182 by permission of New York State Department of Health.

Methods of controlling corrosion

Applying the principles outlined, the practical methods of controlling corrosion may be divided into five classifications: (*a*) the removal of dissolved oxygen; (*b*) the removal of free carbon dioxide; (*c*) the deposition of a protective coating; (*d*) chlorination to prevent biochemical action that influences corrosion; and (*e*) the use of paint, enamel, tar compounds, cement linings, etc.

Removal of dissolved oxygen

Deaerators

Oxygen, carbon dioxide, and other gases may be removed by exposing the water to a partial vacuum. This procedure originally was restricted to the treatment of hot water serving single buildings and steam boilers, but deaerators have been used recently in removing oxygen from several large industrial water-supply systems. Water is exposed to a partial vacuum of about 725 mm Hg (28.5 in), which removes all but 5% of the dissolved oxygen and traces of carbon dioxide and other gases. A very small quantity of sodium sulfite is then added to remove the residual oxygen, thus rendering the water non-corrosive. This treatment is effective when the pH exceeds about 7.0-7.5; supplementary alkali treatment might be needed with very soft water. So-called acid corrosion occurs at lower pH values even in the absence of oxygen.

Deactivators

Dissolved oxygen in water may be removed by forcing the water through beds of cheap iron filings, so that the oxygen exerts its corrosive influence on them rather than on the piping system. The iron content of the water resulting from this treatment is removed by filters. This procedure has been restricted to use in the treatment of hot-water systems of buildings.

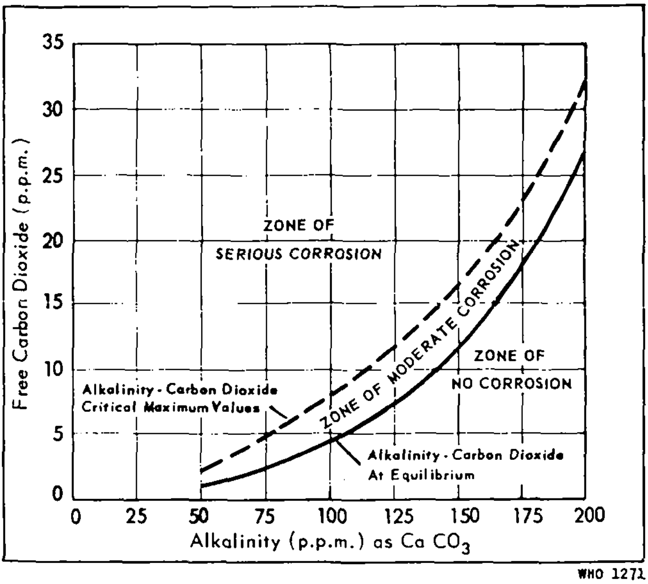
Removal of free carbon dioxide

Undue weight is given to the role of carbon dioxide in the corrosiveness of water. As stated above, the removal of carbon dioxide is not the important aspect of corrosion prevention. The deposition of a mineral film on pipes, however, requires the removal of aggressive carbon dioxide, which otherwise would prevent the deposition or would dissolve mineral film previously formed and expose the pipe to continued corrosion.

Carbon dioxide in the atmosphere is in equilibrium with about 0.5 p.p.m. carbon dioxide in water coming in contact with the atmosphere, so that theoretically aeration should result in the reduction of carbon dioxide to this value. Effective aeration does not, however, lower the concentration of carbon dioxide below 3.0 to 5.0 p.p.m. This concentration of carbon dioxide is aggressive, that is, it will dissolve calcium carbonate at alkalinities less than about 80 p.p.m. *It is for this reason that aeration alone will not prevent the corrosion of metal piping when alkalinity of the water is less than about 80 p.p.m.* as $CaCO_3$. Fig. 13 illustrates this factor. Concentrations of carbon dioxide in excess of the lower solid curve represent aggressive carbon dioxide for the respective alkalinities shown on the horizontal scale of the curve. For instance, the alkaline minerals of waters having an alkalinity of 100 p.p.m. would be in equilibrium with about 4.5 p.p.m. carbon dioxide, and any concentration in excess of that value would be aggressive.

While aerators are the most economical means of eliminating aggressive carbon dioxide in excess of about 5 p.p.m., they are otherwise detrimental because oxygen is introduced, which will increase the corrosiveness of the water to an extent that more than counteracts the benefits of the lower carbon dioxide content. Well water devoid of dissolved oxygen but having a high carbon dioxide content is best treated with an alkali in the absence of aeration for carbon dioxide removal and pH correction. The benefits secured through the use of deaerators in removing dissolved oxygen can be secured with well water devoid of oxygen by adding sufficient alkali to produce a pH in excess of about 7.5 p.p.m., provided that the water so treated is not again exposed to the atmosphere in tanks or reservoirs.

FIG. 13
RELATIONSHIP BETWEEN ALKALINITY, CARBON DIOXIDE CONTENT AND THE CORROSIVENESS OF WATER



Modified from Cox (1952) p. 186 by permission of the New York State Department of Health.

Limited exposure in elevated tanks, however, will result in only moderate aeration of the water, and the dissolved oxygen so introduced may be less than 1.0 p.p.m., which can be economically removed through the addition of sodium sulfite.

Protective coatings

Protective coatings may be subdivided into coatings deposited by special treatment of the water and coatings applied at the time the pipe is manufactured. With the exception of the recent development of deaerators, as outlined above, practically all the successful results secured in protecting distribution systems have been through the treatment of the water with alkali to precipitate a protecting mineral film on the inner surface of the pipes. Such water treatment may involve the deposition of a metaphosphate film, a sodium silicate film, or a calcium carbonate film.

Metaphosphates

Calgon, or sodium hexametaphosphate, and Nalco, or sodium pyrophosphate, have been used to a considerable extent recently as corrosion-control agents, with conflicting results. It is desirable to experiment with the treatment under adequate laboratory control before it is adopted for a given supply. It appears that the phosphate compounds render the metal surfaces passive or coat them with a very thin film that protects the metal against the corrosive action of the water. It may require 4-8 weeks for this protective action to be completed; during this introductory period, doses of 5-10 p.p.m. are advocated. When satisfactory results have been secured, the dose may be reduced to 1-2 p.p.m., which should be maintained thereafter. During the transition period the phosphate compounds may dissolve previously precipitated iron, leading to the release of iron films over pipe surfaces and giving a misleading impression of unfavourable results. Thorough and frequent flushing of the distribution system is advocated during this period.

Metaphosphate crystals are noncorrosive, so typical dry-feed units may be used. Concentrated solutions of metaphosphates, however, are moderately corrosive to most metals, so solution feeders made of earthenware, glass, rubber, or stainless steel should be used.

Sodium silicate

Water-glass, or sodium silicate, has been used to a considerable extent in the protection of hot-water systems of single buildings and to a limited extent in the treatment of public water-supply systems, especially zeolite-softened waters, because the use of a sodium compound does not increase the hardness of the treated water. Doses of 28 p.p.m., or 28 kg/1000 m^3 (280 lb/million UK gal; 233 lb/million US gal) of commercial sodium silicate should be effective. A hard, dense calcium silicate film is formed with waters of adequate calcium content or when a limited dose of lime is added with the sodium silicate. The advice of the manufacturers should be secured in selecting the sodium silicate best suited to a specific water, to avoid the deposition of an excessive amount of material near the point of treatment. This usually is controlled by modifying the concentration of excess alkalinity in the sodium silicate solution. No special control procedure need be followed, except to ensure uniform application of the material. Sodium silicate solutions are noncorrosive to metals, therefore various types of chemical feeders may be used.

Calcium carbonate

In accordance with the principles outlined above, calcium carbonate film is deposited when the alkalinity and pH equal or exceed those obtaining at the calcium carbonate equilibrium point. This occurs when an alkali is added to produce the alkalinities and pH values shown by the solid curve of Fig. 14, that is, pH values of 7.5-9.6 and alkalinities of 150-15 p.p.m. These conditions are favourable to the precipitation of calcium carbonate from the water and its deposition on the pipe.[1]

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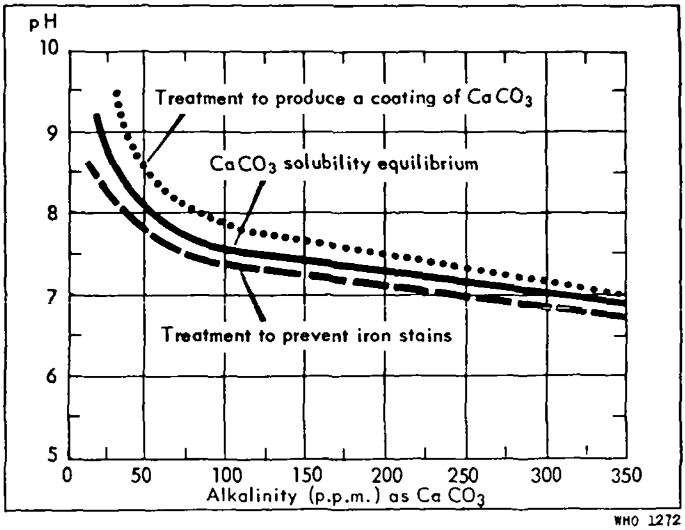
[1] Technically speaking, the solubility of calcium carbonate is dependent on temperature, total-solids content, and calcium ion concentration as well as pH and alkalinity, but the latter are used in simplified plant-control practice.

FIG. 14

RELATIONSHIP BETWEEN ALKALINITY, HYDROGEN-ION CONCENTRATION, CALCIUM CARBONATE STABILITY AND IRON STAINING



Modified from Cox (1952) p. 185 by permission of the New York State Department of Health.

If the alkalinity due to calcium carbonate is less than about 30 p.p.m., lime must be used to supply the deficiency in calcium. Soda ash can also be used when the alkalinity due to calcium carbonate exceeds 30 p.p.m. In most cases lime is added because of its cheapness and greater effectiveness. For instance, 1 p.p.m. 90% hydrated lime or 98% soda ash will remove, respectively, 0.56 p.p.m. or 0.41 p.p.m. carbon dioxide, and will increase the alkalinity of the treated water by 1.28 p.p.m. or 0.92 p.p.m. respectively.

The essential aspects of these reactions are shown by the following equations:

$$\underset{\text{carbonic acid}}{H_2CO_3} + \underset{\text{lime}}{Ca(OH)_2} = \underset{\text{calcium carbonate}}{CaCO_3} + \underset{\text{water}}{2H_2O}$$

$$\underset{\textit{calcium carbonate}}{Ca(HCO_3)_2} + \underset{\text{lime}}{Ca(OH)_2} = \underset{\text{calcium carbonate}}{2CaCO_3} + \underset{\text{water}}{2H_2O}$$

In other words, the lime reacts with carbonic acid (carbon dioxide) to form calcium carbonate, and also with the alkalinity of water, usually calcium bicarbonate, to form more calcium carbonate. If too high a dose of lime is added, an undue amount of scale is formed. If too small a dose of lime is added, the reactions will not go to completion and no calcium carbonate will be precipitated as the desired protective coating.

This treatment procedure is controlled by the somewhat inconvenient and time-consuming " marble test " (see page 311), which is simplified by use of the Enslow stability indicator (page 310), which functions automatically to give significant results whenever they are needed.

Artificial coatings

All metal piping used in constructing distribution systems is coated inside and outside with tar compounds, enamel, or the like, to protect the pipe from the action of water and the surrounding soil. The older tar coatings were effective only for a few years; it is too early to tell how long the effectiveness of the new enamels will last. Tar coatings cause chlorophenol tastes when they are in contact with chlorinated waters. In general, it is wise to treat the water for corrosion prevention even though internal protective coatings are used.

Cement-lined cast-iron pipe and asbestos-cement pipe. The cement lining of iron pipe, especially the spun lining now being produced, is effective in preventing the corrosion of the metal by the water. Because of the material used in its manufacture, asbestos-cement pipe likewise is resistant to the corrosiveness of water. Corrosive water piped through cement-lined cast-iron pipe or asbestos-cement pipe will corrode plumbing systems, however, so treatment of such water is desirable irrespective of the resistance of the material of the distribution system.

Chlorination

The decomposition of the organic content of water in distribution systems, especially in dead-end mains, results in the production of organic acids, which in turn leads to localized corrosion of the piping. Organic decomposition is associated with the absence of dissolved oxygen and thus could be prevented by aerating the water; but as indicated above, aeration increases the corrosiveness of water. Because of this situation, chlorination has been used to destroy the organisms responsible for the decomposition of organic matter and thus control the difficulty before corrosion develops.

Difficulty from this source is likely to be experienced when well waters devoid of dissolved oxygen are pumped directly into distribution systems. The situation is intensified when sulfates in the water are reduced to sulfides, which in turn leads to the presence of iron sulfides in the water, that is, the black deposits associated with water secured from dead-end mains, where objectionable odours are produced.

Because of the necessity of maintaining residual chlorine throughout distribution systems, free-residual chlorination is utilized for this purpose. Effective results have been secured when doses are such as to result in residuals of 0.2-0.4 p.p.m. throughout the system.

Corrosion of plumbing systems

The treatment of water supplies to protect distribution systems will greatly minimize corrosion of plumbing systems in buildings. Complete protection, however, is impossible when a portion of the water is heated, because calcium carbonate is *less* soluble in hot water than in cold. Thus the protection of the distribution system can be made only partly effective during winter months, as the cold water treated to equilibrium would be oversaturated when heated and water heaters would become unduly coated with calcium carbonate. This problem can be solved only by using restricted alkali doses in the winter to secure values less than those of equilibrium, thus providing partial protection to the distribution system without harmful effects on hot-water systems. The doses should be increased in the spring to produce values slightly *above* equilibrium so that in summer a film is produced of sufficient thickness to provide protection the following winter.

Furthermore, water treated with an alkali to make it noncorrosive to iron may be corrosive to zinc, galvanized iron, and *yellow* brass, because zinc seriously corrodes at pH values *less* than 7.5 and *more* than 8.5. The use of galvanized iron and yellow-brass piping should be restricted to supplies having pH values in the zone of 7.5-8.5. Waters having alkalinity above 30 p.p.m. and pH values above 7.8 are not seriously corrosive to copper and *red* brass. Some yellow brasses (duplex brasses) are subject to dezincification by corrosive waters. Copper will produce a metallic taste and staining in concentrations above 1-3 p.p.m.; a value of 1 p.p.m. may serve as an approximate guide. The zinc content should be less than 5 p.p.m.

Lead is corroded by waters of low pH value, and toxic effects are noted when the lead content exceeds 0.05 p.p.m. Lead piping should not be used with waters having pH values less than about 7.8. Waters rendered caustic by " excess lime " treatment also are corrosive to lead, so lead piping should not be used with lime-softened waters unless they are recarbonated to give resulting pH values less than about 9.6. Lead piping may therefore be used with waters having pH values between 7.8 and 9.6, provided there is assurance that this pH range will be maintained. The *European Standards for Drinking-Water,* page 28, states that the content of lead in the water reaching the tap of the consumer should not exceed 0.1 mg/l after 16 h contact with lead piping.

Cement-lined iron or steel piping is more resistant to internal corrosion than galvanized iron or steel piping. While the cement is acted on to some extent by waters containing appreciable concentrations of carbon dioxide, in general such piping is well adapted for use when corrosion-prevention is not practised and when the corrosive water delivered to consumers would attack the metal piping discussed above. Cement-lined iron piping also may be used instead of lead, brass, or copper when these metals are not readily available. The alkalinity of the water contacting the lining is increased.

Plastic pipe has been developed in recent years by a number of manufacturers. The indications are that it will be used to an increasing extent because of its noncorrosive characteristics and its ease of handling.

Many soils are corrosive to metal piping, especially some clays and soils of high organic content. Resistance to external corrosion of the various metals is as outlined above.

Electrolysis

Isolated cases of corrosion of plumbing systems should be carefully investigated to see if they are due to electrolysis, that is, the effects of stray electric currents in water pipes, resulting from the grounding of electric fixtures, radios, etc., to plumbing fixtures. Serious corrosion of distributing systems in the vicinity of power-houses and the like may also be due to electrolysis, and in such instances the problem is the responsibility of the power authorities. Corrosion due to electrolysis also may occur in plumbing systems when dissimilar metals are used, such as iron and copper, especially when the water has a low pH value.

Application of lime and soda ash

The common use of lime to deposit calcium carbonate film or coating on the inside of water piping is of great significance in the protection of distribution systems of public water supplies. The reactions occurring in water treated with lime are complex in nature, but the simple marble test which determines alkalinity after calcium carbonate equilibrium has been established gives adequate information as to the lime dose initially needed.

For instance, assume the alkalinity of the raw water to be 50 p.p.m. and the alkalinity after equilibrium to be 70 p.p.m. Then the alkalinity of the corrosive supply would have to be increased by 20 p.p.m. to control its corrosiveness. Hydrated lime of 95% purity normally is available. Each 1.0 p.p.m. of such lime will increase the alkalinity of the water by 1.28 p.p.m. Therefore the required dose would be $20.0/1.28 = 15.6$ p.p.m. When corrosion-control treatment is started, the best procedure is to overdose for about a month. In the above example the dose could be 20.0 p.p.m. instead of 15.6 p.p.m.

The marble test also will disclose the increase in pH value as a portion of the calcium carbonate is dissolved by the carbonic acid to form calcium bicarbonate. For example, assume that the marble test discloses an increase in pH from 7.0 to 7.8. This increase in pH value cannot be used to compute the required lime dose, but the simple pH test may be used in plant control for the adjustment of the lime dose from day to day. Thus if the marble test discloses a drop in the pH value at equilibrium from 7.8 to 7.6, the dose is decreased and the pH of the treated water determined again. Should the pH then be, say, 7.5, the dose would be increased slightly. Experience will soon supply guidance in this procedure. In essence, the procedure is to determine each day the pH of the effluent of the Enslow stability indicator (or the pH at equilibrium by the regular marble test) and also the pH of the treated supply, then to adjust the lime dose so as to raise or lower the pH of the treated water until it equals the pH at equilibrium.

Soft water containing low concentrations of bicarbonates or carbon dioxide will not react with lime to produce the required concentration of calcium carbonate; in such a case the lime dose must be supplemented with soda ash. In practice, the Enslow stability indicator or the regular marble test is used to disclose both pH and alkalinity at equilibrium. The needed increase in alkalinity indicated is computed by assuming that 80% of the increase will be with lime, to provide the calcium, and 20% with soda ash to supply the deficiency in carbonate ion. The supply is treated with these two chemicals and the alkalinity and pH of the treated water determined. The alkalinity should be that desired. If it is not, *both* doses are adjusted proportionally and the pH value is again determined. Three types of results are possible:

1. When the values for both alkalinity and pH are those desired, the 8 to 2 ration of lime to soda ash doses is continued until future tests disclose that a change in water quality requires a different ratio.
2. When the alkalinity is at the desired value but the pH is *higher* than that produced by the marble test, the lime dose is decreased and the soda ash dose increased proportionally.
3. When the alkalinity is that desired but the pH value is *less* than desired, the lime dose is increased and the soda ash decreased proportionally.

For example, suppose the corrosive water has an alkalinity of 20.0 p.p.m. and a pH value of 6.8 and the marble test shows the alkalinity at equilibrium to be 35.0 p.p.m. and the pH 8.3. This increase in alkalinity of 15.0 p.p.m. would be made by a trial dose of 80% lime and 20% soda ash—that is, 12.0 p.p.m. with lime and 3.0 p.p.m. with soda ash. The dose required would be $12 \times 0.78 = 9.4$ p.p.m. 95% hydrated lime, and $3.0 \times 1.06 = 3.2$ p.p.m. 100% soda ash, using the factors given in Table 22 (page 229). To continue this example, assume that the dose of 9.4 p.p.m. hydrated lime and 3.2 p.p.m. soda ash produced the desired increase in alkalinity of 15.0 p.p.m. but that the resulting pH was *higher* than 8.3. Then the lime dose would be *decreased* to provide, say, 75% of the total increase in alkalinity of 15.0 p.p.m., or 11.25 p.p.m., leaving 3.25 p.p.m. to be supplied by soda ash. The doses then would become $11.25 \times 0.78 = 8.8$ p.p.m. lime and $3.25 \times 1.06 = 3.4$ p.p.m. soda ash. After applying the adjusted dose, the pH of the treated water would be determined to see whether it is near the desired value of pH 8.3. Minor adjustments then can be made in the relative doses of lime and soda ash and in the sum of the doses, to meet changing characteristics of the water.

Coagulation will increase the corrosiveness of waters unless iron coagulants are used with lime in the high pH range. Inasmuch as the coagulation process is sensitive to the pH of the water being treated, corrosion control must be accomplished by adding the alkali to water after filtration. Final chlorine doses will be most effective in corrosive waters of lower pH before alkali treatment, so the best procedure is to add the chlorine to water as it enters clear wells or reservoirs and to add the alkali to it near the outlet piping or the suction piping of high-lift pumps. Clear wells or reservoirs should be baffled to prevent chlorinated water from short-circuiting directly from the filter-effluent pipes towards the outlet piping or the suction pipes.

Laboratory control

Corrosion-prevention treatment is controlled by laboratory tests for pH, alkalinity, and the related content of carbon dioxide determined by use of Fig. 19 (page 307), all related to the marble test or to the use of the Enslow stability indicator. The latter is much simpler than the marble test and provides at all times water in equilibrium with calcium carbonate, so that the pH and alkalinity of stabilized water may be determined at will.

Samples from distribution systems should be examined periodically to determine their pH and alkalinity as compared with plant effluents, to ascertain whether equilibrium values are being maintained. Tap samples also should be inspected for signs of rust particles, or " red water ". The test for iron will disclose any significant increase in iron content due to corrosion.

The water supplies serving toilets, showers and laboratories of treatment plants should be piped from points beyond where corrosion-control treatment is applied so that the effectiveness of treatment may be observed in the plants. Observation is facilitated by the use of a section of piping arranged as a bypass around a valve, the bypass being equipped with two valves separated by 0.6 m (2 ft) of pipe with flanged unions. The valve on the main pipeline serving the plant is normally closed and the water flows through the bypass. When it is desired to determine whether the water is corrosive, noncorrosive or supersaturated with calcium carbonate and hence depositing scale on the inside of the piping, the valve on the main piping is opened and the two on the bypass are closed. The length of test piping can then be disconnected at the flanged unions and the interior inspected. The test section should be of the same material as the rest of the piping. Inasmuch as hot water is more corrosive than cool or cold water, another test portion should be installed on hot-water piping of plants.

The effectiveness of corrosion-control treatment should also be checked by observing the conditions inside water mains opened for repairs or replacement. The experience of consumers with plumbing systems, and especially with hot-water tanks, also provides a guide. Any serious deposition of scale through overtreatment may be eliminated by undertreating the supply until the scale is dissolved and then returning to the normal procedure.

CHAPTER 12

IRON AND MANGANESE REMOVAL

Iron and manganese, either separately or together, are objectionable in waters because they stain clothing and plumbing fixtures and detract from the appearance of the water. The *International Standards for Drinking-Water* specify a concentration of 0.3 p.p.m. iron as the maximum acceptable and 1.0 p.p.m. as the maximum allowable. Manganese is more troublesome, so the limits are 0.1 p.p.m. and 0.5 p.p.m., respectively. Water to be used in the textile, dyeing, beverage and white-paper industries should contain less than 0.05 p.p.m. iron or manganese.

High concentrations of these minerals are associated with deep well waters from shale, sandstone and other rocks. The minerals are dissolved by ground waters containing carbon dioxide but no oxygen, the insoluble oxides being reduced and transformed to soluble bicarbonates. These same reactions occur in the lower portions of deep reservoirs that flood iron- or manganese-bearing soils and rocks. Well waters containing soluble iron or manganese are clear as pumped from the ground-water strata, but exposure of the water to the air leads to the oxidation and precipitation of the minerals.

Shallow wells or reservoirs yielding iron-bearing waters are likely to be contaminated with *Crenothrix* and similar iron bacteria, which may clog well screens or develop in distribution systems, thus creating serious nuisances. This is aggravated when sulfate-bearing waters are involved, because the reduction of iron and sulfate compounds in dead-end mains leads to the formation of disagreeable odours and black deposits of iron sulfide.

Iron- and manganese-removal practice is intended to ensure that the water pumped into distribution systems will be of satisfactory quality in so far as these minerals are concerned.

When funds are not available for iron- or manganese-removal plants, it is feasible, with well-water supplies devoid of oxygen, to use sodium hexametaphosphate to prevent the precipitation of iron and manganese while the water passes through the distribution system. The dose should be twice the content of iron and/or manganese. This process must be used only with moderate concentrations of these minerals, however, otherwise clothing will be stained in laundering.

The control of *Crenothrix* is discussed in Chapter 4 (page 32).

Principles

The literature on iron and manganese removal is confusing, because the characteristics of various waters and the varying chemical composition of the iron and manganese compounds make it necessary to select one of ten processes to meet specific local conditions. These ten processes are summarized in Table 17.

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 that oxidation is prevented by excluding air. This eliminates the need for aeration (see process 10 of Table 17). 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, chlorination or use of potassium permanganate results in oxidation and precipitation at lower pH values.

The removal of manganese is more complicated than iron removal. Oxidation by catalytic action may require a pH of at least 7.5-8.5, and a pH of at least 10.0 is needed with treatment by aeration, lime application, sedimentation and filtration. 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 is then oxidized to insoluble manganic hydroxide by aeration through a ventilated contact bed or aerator, and the water is 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. The water under study, therefore, should be tested to determine whether iron or manganese is present alone or whether both minerals are involved. Washing white cotton cloth in the water with laundry soap will indicate the significance of these minerals. Tests will show whether aeration without lime treatment results in precipitation or whether lime treatment or contact beds are needed for oxidation and precipitation; in the latter case, usually a small pilot plant is needed to establish the best and most economical design for the plant under study.

Engineering study will establish whether the iron and/or manganese is readily oxidized and precipitated by aeration alone without pH correction, or whether the pH, carbon dioxide and organic content are such as to form stable soluble compounds that cannot be removed 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: will oxidize upon aeration without need of further pH adjustment or of catalytic action.

2. Waters containing iron and also a limited amount of manganese: oxidation requires catalytic action in contact beds or chlorination or pH adjustment prior to filtration. More recently oxidation with potassium permanganate, used as in process 6 of Table 17 or applied continuously prior to contact-bed treatment as in process 4, has been proven superior to chlorine (Willey & Jennings, 1963; Welch, 1963).

3. Waters containing iron bound with organic matter: pH adjustment to values of 8.5-9.0 is required to secure oxidation and precipitation; or waters containing manganese may: not oxidize at pH values below 7.5-10.0.

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

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

Practice

The various units of iron- and manganese-removal plants are interrelated and hence must be co-ordinated in the design. The 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 odour-producing substances and to introduce oxygen, as discussed in Chapter 3. 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 a primary contact bed for catalytic action. The gravel is placed on four to six trays

TABLE 17

PROCESSES OF IRON AND MANGANESE REMOVAL

Treatment process	Oxidation required	Character of water	Equipment required	pH range required	Chemicals required	Remarks
1. Aeration, sedimentation, and filtration	Yes	Iron alone in absence of appreciable concentrations of organic matter	Aeration, settling basin, sand filter	Over 7	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-acid content	Contact aerator of coke, gravel or crushed pyrolusite; settling basin; sand filter	Over 7 for iron removal, 7.5-10 for manganese	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 for iron removal, 7.5-10 for manganese	Lime for manganese removal	Double pumping required unless air compressor or "sniffler" valve 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 for iron removal, over 8.5 for manganese	Filter bed reactivated or oxidized with chlorine at intervals or with potassium permanganate applied continuously	Single pumping; aeration not required
5. Catalytic action, aeration, sedimentation, filtration	Yes	Manganese in combination with organic matter	Closed pyrolusite bed, aerator, second open-contact bed, sand filter	Over 7	None	Manganese changed to manganous hydroxide by catalytic action in absence of air, then oxidized

6. Aeration, chlorination, sedimentation, sand filtration	Yes	Iron and manganese loosely bound to organic matter	Aerator and chlorinator or chlorinator alone, settling basin, sand filter	7-8	Chlorine or potassium permanganate	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; organic acids	Effective aerator, lime-feeder mixing basin, settling basin, sand filter	8.5-10	Lime	pH control required
8. Aeration, coagulation and lime treatment, sedimentation, sand filtration	Yes	Coloured turbid surface water containing iron and manganese combined with organic matter	Conventional rapid sand filtration plant	8.5-9.6	Lime and ferric chloride or ferric sulfate, or chlorinated copperas, or lime and copperas	Complete laboratory control required
9. Zeolite softening	No	Well water devoid of oxygen, 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 bypassed water	Over 6.5	None added continuously but bed is regenerated at intervals with salt solution	Only soluble ferrous and manganese bicarbonate can be removed by base exchange, so aeration or double pumping 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, pressure filter	8.1-8.5	Lime	Iron precipitated as ferrous carbonate in absence of oxygen; minimizes or prevents corrosion; double pumping not required

in layers about 30 cm (12 in) deep, the trays being separated by air spaces and perforated to distribute the falling water. Single beds 1.2-1.8 m (4-6 ft) deep are satisfactory, provided ventilation is ensured by perforation in the retaining walls. Louverlike 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 24 m/h (80 ft/h; 10 US gal/min/ft^2) are satisfactory. Aeration need not be excessive, as only 0.14 p.p.m. oxygen is needed to oxidize 1.0 p.p.m. iron. Aeration should be reduced in degree with water having a high content of organic matter.

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 upward flow at rates up to 9.6 m/h (32 ft/h; 4 US gal/min/ft^2) is preferred, but a lower rate may be used. Bed depth should be 1.8 m (6 ft) or any greater depth found necessary by pilot-plant studies. Provision must be made for the *rapid* draining of the beds, so as to wash excess oxides from the gravel or ore, and for the use of a hose stream for periodic cleansing of the gravel or ore. The beds are regenerated by backwashing with potassium permanganate solution when permanganate is not applied continuously to the raw water.

Contact beds of pyrolusite ore, for manganese removal without lime or potassium permanganate treatment (see process 5), 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 4.8 m/h (16 ft/h; 2 US gal/min/ft^2) with a bed depth of 1.8 m (6 ft) is suggested, giving a contact period of 9 min, with a usual void volume of 40%. The effluent from such beds should be aerated in a downward-flow contact-bed aerator constructed to facilitate passage of air. Final filtration is needed, as discussed below.

Manganese zeolite, formed by treating sodium zeolite with a solution of potassium permanganate, is an effective contact material that will remove by oxidation about 1.63 kg manganese per cubic metre of zeolite (0.1 lb/ft^3) per cycle. Re-oxidation or regeneration of the material in each cycle is secured by backwashing with a solution of potassium permanganate containing about 3.26 kg of this chemical for each cubic metre of zeolite (0.2 lb/ft^3). Incomplete re-oxidation will result in the passage of manganese through the contact bed. The need to regenerate may be anticipated by computing the volume of the raw water which contains 1.63 kg manganese per cubic metre of zeolite (0.1 lb/ft^3). For example, a water with a content of 1.0 p.p.m. manganese will contain 1 kg/m^3. Then a contact bed with a volume of, say, 4 m^3 would treat $4 \times 1.63 \times 1.0 = 6.52$ m^3 of this water before regeneration is necessary. A solution of potassium permanganate, such as 10 kg/m^3, would be used to re-oxidize the bed. Then 1.3 m^3 of this solution would be needed per cycle: $(3.26/10) \times 4 = 1.3$.

Lime treatment

Equipment for feeding lime for pH correction should be selected with due consideration of the factors discussed in Chapter 13. The capacity of the lime feeder should be at least 50% greater than the 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

See Chapter 8 for factors to be considered in this connexion. Before adopting this process, the operator should determine whether chlorine, in the absence of dissolved oxygen, will be effective in oxidizing iron or manganese in the specific water supply under design, and whether the cost of chlorination equipment and chlorine will be less than the cost of aeration and double pumping that would otherwise be required. This process is especially adaptable to those instances in which disinfection of a well supply is also needed or in which chlorination will oxidize the iron without lime treatment for pH adjustment. Manganese is oxidized by free-residual chlorine, but the reaction rate is slow unless the pH exceeds 8.5-10.0. The dose of chlorine should be selected to provide about 1.25 p.p.m. free chlorine for each p.p.m. manganese to be oxidized. Iron is more readily oxidized by chlorine with the same dose, but over a wide pH range. Oxidation is aided by the use of 0.2 p.p.m. copper sulfate, the copper acting as a catalytic agent.

Oxidation by potassium permanganate treatment

Potassium permanganate provides much more effective oxidation of iron and manganese than does chlorine in process 6. The reaction is independent of the pH of the water in the range above 7.0, so manganese may be oxidized without lime treatment, except with soft waters. The reaction is rapid, except when reduced substances such as organic matter or hydrogen sulfide are present in high concentrations, in which case reaction periods of 5-20 min should be provided prior to filtration. If permanganate treatment precedes process 4, the oxidizing power of the contact beds will not be depleted and hence they will not need regeneration.

The dose of potassium permanganate may be determined as that needed to produce a faint pink colour in the treated water prior to filtration. Usually the dose required for iron removal is about two-thirds the content of iron in the raw water, and for manganese removal about twice the content of manganese. The dose is not critical; any excess will be removed in the filters or contact beds.

A commercial grade of potassium permanganate is available in the USA at a cost of about US $0.30 lb. Potassium permanganate may be applied as crystals or as a solution. This chemical is harmful to rubber hose and to the diaphragms of certain chemical feeders but not to metals used in waterworks practice.

Sedimentation basins

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 p.p.m. Refer to Chapter 6 for design features of sedimentation basins. Sedimentation periods should be established for a specific water but usually will be 1-2 h for iron removal.

A plant designed to remove iron *in the absence of oxygen*, in accordance with process 10 in Table 17, requires flocculation, sedimentation and filtration *under pressure* 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 five 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 one hour.

Filters

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 7. Rates of filtration of 9.6 m/h (32 ft/h; 4.0 US gal/min/ft^2) or less are satisfactory. The filter sand should have an effective size of 0.7-1.0 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 so rigid as with conventional filters used for bacterial removal.

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

Ion-exchange softeners for iron and manganese removal

Conventional ion-exchange softeners, discussed in Chapter 13, should be considered for hard waters containing iron and/or manganese, provided that the raw well water is devoid of oxygen, inasmuch as the ion-exchange process removes only *soluble* iron and manganese together with the calcium and magnesium. This process should not be used when the iron and manganese content exceeds 0.5 p.p.m. for each 17 p.p.m. of hardness, up to a maximum of 10 p.p.m. This establishes the lower limits of hardness as 340 p.p.m. for an iron content of 10 p.p.m. As aeration or pH adjustment is not needed, the cost of double pumping is avoided.

Operation

The supervision and operation of iron- and manganese-removal processes should be considered in two categories: those followed at conventional filtration plants with coagulation, sedimentation and sand filtration; and those followed at special iron- and manganese-removal plants.

Conventional filtration plants

Raw surface waters containing iron and manganese in limited quantities may be coagulated with alum, aided by prechlorination, especially when colour is present as complex compounds of iron, manganese and organic matter. If, however, the content of iron and manganese is high and turbidity and colour removal are also needed, oxidation must be by aeration and pH correction. This increase in pH value—that is, alkalinity of the water—by lime treatment creates conditions under which alum is not effective as a coagulant. Therefore either ferric sulfate or ferric chloride when available or else copperas and lime or chlorinated copperas should be used. Chlorinated copperas has the advantage of combining prechlorination with coagulation. Refer to Chapter 5, pages 63-65, and Annex 2 for a description of these coagulants.

Iron floc is usually heavier than alum floc, but there is a tendency for feathery, fragile floc to form unless flocculation is effective. Coagulation must also be effective, because any uncoagulated iron would be as objectionable as for the iron content of the raw water to remain unmodified. In other words, soluble iron in the filter effluent should be under 0.3 p.p.m., a lower value than the 0.5-1.0 p.p.m. soluble alumina that may be present when alum coagulation is not entirely effective without creating an objectionable effluent. (Soluble alumina should not be confused with alum floc that may pass through a filter. The first is due to incomplete coagulation, whereas the latter is due to faulty filtration). Iron floc is also likely to coat filter sand and be more difficult to remove by the washing process than alum floc. Some coating is desirable in that it acts as a catalytic agent in the further oxidation of any iron remaining in the applied water.

Manganese is more difficult to remove than iron. The lime dose, with ferric coagulants, should be higher, as coagulation occurs at a pH value of

8.5-9.0. Prechlorination, however, may so alter oxidation that manganese may be coagulated at lower pH values. Only trial in laboratory flocculating equipment will disclose the best process to follow. Manganese removal is especially aided by contact with previously oxidized manganese on filter sand, so the washing process must be controlled so as to remove excess floc, without actually cleaning the sand. Observation of the length of filter runs and the rate of increase in loss of head will serve as a guide to the degree of washing needed at a given plant.

Iron- and manganese-removal plants

The operation of these plants differs with the process used. Processes 1, 2 and 5 of Table 17 require no addition of chemicals. Lime is needed for pH correction in processes 3, 7 and 10. Processes 9 and 10 require that no oxygen be present in the water, because unoxidized ferrous and manganous bicarbonates are removed by base-exchange softening in process 9, and ferrous and manganous carbonate by precipitation and filtration in process 10. The more complex process 5 also requires absence of oxygen at the first step. Therefore care must be exercised with these processes to exclude air from suction-piping of well pumps or from the first contact bed. In the first instance this is accomplished by having the well water rise upward through the primary bed of the manganese ore, pyrolusite, for catalytic action leading to the production of manganese hydroxide. The water is then aerated as it flows down an open bed of the same ore, which is constructed as a conventional contact aerator.

Contact beds depend for their action either on the catalytic action of oxidized iron and manganese compounds deposited on stone or coke or on the same effect from pyrolusite. Therefore these beds should never be washed clean, but the surplus deposits should be removed at intervals. The special closed beds should be backwashed at high rates of at least 60 m/h (200 ft/h; US gal/min/ft^2). Open beds of aerators may be cleaned with strong streams of water from a hose. Occasionally the contact material may have to be removed so that the inner portions of the beds may be cleaned.

Laboratory control

Tests for iron, manganese, residual chlorine, pH and alkalinity are needed at special treatment plants. The carbon dioxide content may be determined from the results of the tests for pH and alkalinity, as discussed in the Supplement, pages 300 and 306. These same tests, together with turbidity, colour and the flocculation jar test, are needed at filtration plants where iron and/or manganese complicate the treatment processes. The control of pH is especially important with many of the processes.

CHAPTER 13

SOFTENING AND DEMINERALIZATION OF WATER

Purpose

Softening and demineralization of water have two basic purposes: first, hardness-producing compounds of calcium and magnesium are reduced or removed by softening processes to prevent the scaling of boilers, water heaters and industrial equipment and to reduce the amount of soap needed to produce a lather in homes, laundries, textile mills, etc.; second, minerals are removed from brackish waters, brines and sea water in areas where fresh-water resources are limited and where the cost of piping fresh water from distant points would be so great that demineralization is more economical.

The ever-expanding population and the growth of industry are creating a serious drain on fresh-water resources, so extensive research is in progress to achieve simplified and more economical demineralization processes, and the situation should be more favourable in the future than in this period of development.

Minerals in natural waters

The dissolved minerals in fresh waters are those associated with hardness and alkalinity, whereas saline waters are characterized by their neutral salts. The common minerals encountered in natural waters are listed in Table 18 as alkaline, neutral or acid. Alkaline minerals are subdivided into two groups, one causing alkalinity only and the other causing alkalinity and hardness. The neutral salts have also been subdivided into those causing salinity and non-carbonate hardness and those causing salinity only. Except for rare mineral waters, most waters have no content of mineral acids except when polluted by acid industrial wastes. (The heating of water containing calcium or magnesium bicarbonate will drive off the carbon dioxide and precipitate calcium or magnesium carbonate, thus

TABLE 18

PRINCIPAL MINERALS PRESENT IN NATURAL WATERS

Alkaline, causing:		Saline (neutral), causing:		Acid
sodium and potassium alkalinity	carbonate hardness	non-carbonate hardness	salinity only	
Potassium bicarbonate ($KHCO_3$)	Calcium bicarbonate [$Ca(HCO_3)_2$]	Calcium sulfate (gypsum) ($CaSO_4$)	Potassium sulfate (K_2SO_4)	Mineral acids and acid salts are restricted to acid mine wastes and rare mineral waters
Potassium carbonate (K_2CO_3)	Calcium carbonate ($CaCO_3$)	Calcium chloride ($CaCl_2$)	Potassium chloride (KCl)	
Sodium bicarbonate (soda) ($NaHCO_3$)	Magnesium bicarbonate [$Mg(HCO_3)_2$]	Magnesium sulfate(Epsom salts) ($MgSO_4$)	Potassium nitrate (KNO_3)	
Sodium carbonate (Na_2CO_3)	Magnesium carbonate ($MgCO_3$)	Magnesium chloride ($MgCl_2$)	Sodium sulfate (Na_2SO_4)	Ferrous sulfate ($FeSO_4$)
			Sodium chloride (common salt) (NaCl)	
			Sodium nitrate ($NaNO_3$)	

From Cox (1952) p. 204 by permission of the New York State Department of Health.

removing the hardness-producing minerals; hence such carbonate hardness is sometimes called " temporary hardness ". Non-carbonate hardness, on the other hand, is due to calcium or magnesium sulfate and chloride, which are not precipitated upon heating, so this has been called " permanent hardness ".)

The alkaline minerals in natural waters comprise two groups, the carbonates and the bicarbonates. A third group is involved in water treatment, namely, caustic or hydroxide alkalinity introduced with lime treatment but otherwise not present in natural waters. The relationship of the three types of alkalinity to the pH scale is discussed in the Supplement and is represented in Fig. 18, page 302. The determination of alkalinity, using two indicators, phenolphthalein and methyl orange, provides information as to the type of alkaline minerals present (see page 300).

Minerals in natural waters not referred to above are not of significance in water treatment, though some, such as arsenic and selenium, may be significant in terms of water quality.

The desirable degree of hardness of public water supplies depends on the experience of the consumers. Those used to very soft waters will consider

100 p.p.m. hardness as high, whereas those used to hard waters will consider this value acceptable. Waters softer than 30-50 p.p.m. are corrosive. The cost of softening is usually not warranted when the hardness is less than 150 p.p.m. Accordingly, softening practice is usually limited to supplies having hardnesses greater than 150 p.p.m., and the waters are softened to hardnesses of about 80 p.p.m.

The terms " soft " and " hard " are used in the United States as follows:

Soft : Hardness less than 50 p.p.m.
Moderately hard : Hardness of 50-150 p.p.m.
Hard : Hardness of 150-300 p.p.m.
Very hard : Hardness over 300 p.p.m.

Benefits of soft waters

The availability of soft water in the home leads to appreciable savings in the consumption of soap and to improved cleansing action. The increasing use and distribution of the newer detergents is minimizing the saving from soft water, however, as some of them will function in hard water. Table 19 shows the savings resulting from water-softening when soap is used, rather than one of the newer detergents.

TABLE 19
AMOUNT AND COST OF COMMERCIAL SOAP SAVED BY SOFTENING

Raw-water hardness, p.p.m.	Softened-water hardness, p.p.m.	Soap saved per family of 5 per year, lb	Value of soap saved per family of 5 per year, US$
200	85	50	8
300	85	94	15
400	85	138	22
500	85	182	29

Assumptions: Soap cost = $0.16/lb; water used with soap = 1.2 gal per capita per day; soap consumption in excess of that needed with soft water = 0.2 lb/1000 gal for each part per million of hardness.

Adapted from American Water Works Association (1950) p. 312 by permission of the Association.

The savings in fuel and maintenance in the operation of boilers, heaters, etc. through the use of soft water are less easily computed, but they are great enough for the softening of boiler feed water and industrial water supply to be generally practised on individual properties.

Demineralization procedures

Waterworks officials are usually concerned only with water softening, but for the reasons noted above, a broadening in perspective is needed and attention must be given to the old as well as the new processes. These include:

(1) *lime-soda softening*, in which most of the calcium and magnesium are precipitated and the alkalinity is reduced;

(2) *ion-exchange softening and demineralization :*

(*a*) base-exchange softening, in which calcium and magnesium ions are replaced by sodium ions and the equivalent sodium compounds remain in the water;

(*b*) ion-exchange demineralization, in which both cations (bases) and anions are removed (i.e., calcium and magnesium, as well as carbonates, chlorides and sulfates, producing mineral-free water or an effluent having a moderate mineral content to make it noncorrosive;

(3) *electrodialysis*, in which a direct current causes the ions to migrate through suitable membranes from the water being treated, leaving demineralized water in suitable compartments;

(4) *distillation*, in which evaporation of brines or sea water produces mineral-free distilled water; this includes solar distillation, in which solar heat is used for evaporation;

(5) *freezing*, in which the fresh-water ice crystals are separated from freezing brines.

Details are given here only for the first two processes; electrodialysis, distillation and freezing processes are described briefly.

Lime-soda softening

Principles

This process of softening depends on the use of lime and soda ash to change soluble calcium and magnesium compounds into nearly insoluble compounds, which are flocculated, settled and filtered as with conventional coagulation.

The reactions are as shown below, but in practice are influenced by water temperature, pH and complex physicochemical relationships. The compounds underlined are those which are precipitated.

$$CO_2 + Ca(OH)_2 = \underline{CaCO_3} + H_2O \tag{1}$$

$$Ca(HCO_3)_2 + Ca(OH)_2 = 2\underline{CaCO_3} + 2H_2O \tag{2}$$

$$Mg(HCO_3)_2 + Ca(OH)_2 = \underline{CaCO_3} + MgCO_32H_2O \tag{3}$$

$$MgCO_3 + Ca(OH)_2 = \underline{Mg(OH)_2} + \underline{CaCO_3} \tag{4}$$

$$MgSO_2 + Ca(OH)_2 = \underline{Mg(OH)_2} + CaSO_4 \tag{5}$$

$$CaSO_4 + Na_2CO_3 = \underline{CaCO_3} + Na_2SO_4 \tag{6}$$

Note that: (*a*) carbon dioxide is not hardness-forming but it is removed by the lime and hence the lime dose is influenced accordingly; (*b*) the magnesium carbonate produced in reaction (3) is not sufficiently insoluble for effective removal and so must be changed by additional lime, as in reaction (4), to magnesium hydroxide; (*c*) the calcium sulfate produced in reaction (5) is soluble and so must be changed to calcium carbonate by reaction (6); (*d*) calcium sulfate in the raw water is also removed by reaction (6), indicating the need for using soda ash; (*e*) soluble calcium and magnesium chlorides are also removed by soda ash, with reactions similar to (6).

Calcium carbonate theoretically is soluble only to the extent of 17 p.p.m. at pH 9.4. Treatment with lime and soda ash, however, will not result in the precipitation of all but 17 p.p.m. calcium carbonate; but a carbonate hardness of 35 p.p.m. can be secured in practice. Treatment is by one of the following four methods:

1. Excess-lime treatment

An excess of lime (10-50 p.p.m.) is added to produce a pH of 10.6 for the precipitation of magnesium and calcium. Soda ash is then added to convert the excess of lime to sodium hydroxide and calcium carbonate. Sodium hydroxide (caustic alkalinity) is undesirable, so split treatment is preferred unless the water is recarbonated to remove caustic alkalinity and produce a pH of 8.

2. Split treatment

A major portion of the raw water is given the excess-lime treatment, and the remaining portion of the raw water is added after the softening reactions have occurred. The carbon dioxide and bicarbonates in this portion of the raw water react with the excess of lime and precipitate the calcium at pH 9.4 in a secondary basin.

3. Excess-lime treatment followed by recarbonation

A more elaborate process consists of primary flocculation and sedimentation, with lime treatment to produce a pH of 10.6 to precipitate the magnesium. The water is then recarbonated with carbon dioxide to lower the pH to 9.4 and precipitate calcium carbonate in the secondary basin, and soda ash is added as needed to precipitate non-carbonate hardness. Finally carbon dioxide is added to the settled water to produce a pH of about 8.7, so as to convert the residual calcium carbonate into soluble bicarbonate and thus prevent after-precipitation on the filter sand (which may also be prevented by adding sodium hexametaphosphate to the settled water).

4. Substitution of base-exchange softening for soda ash

In areas where salt may be purchased at low cost, it is more economical to remove non-carbonate hardness by base exchange than by soda ash. The procedure is to use sufficient lime to reduce the carbonate hardness to as low a value as possible, then recarbonate and filter, and then pass a portion of the filtered water through the base-exchange units to produce the desired quality in the mixture of lime-softened filtered water and base-exchange-softened water.

Coagulation

Lime-soda softening is aided by the use of alum, ferric sulfate, ferrous sulfate, sodium aluminate or activated silica to coagulate the fine crystals formed by the softening reactions. The use of alum at such high pH may be thought to be inconsistent, but the purpose is to form not aluminium hydroxide but magnesium aluminate, so that the magnesium will be more effectively precipitated. Coagulation may be needed when turbid waters are being softened or to aid in precipitation.

Variations of lime-soda softening

The above comments refer to lime-soda softening when both calcium and magnesium, as well as carbonate and non-carbonate hardness, have to be removed. However, if calcium bicarbonate alone is present as hardness, then only lime need be used and the treatment is aimed to produce the optimum pH of 9.4. Soda ash can be dispensed with also when only calcium and magnesium bicarbonates are present, but in this case excess-lime treatment is needed to produce a pH of 10.6 to remove magnesium. In other words, soda ash is needed only to remove non-carbonate hardness, as in reactions (5) and (6) above, or, more technically, when the content in equivalents of calcium and magnesium cations exceeds the alkalinity anions.

Lime-soda softening practice

The softening plants designed to achieve the above reactions are similar in principle to the usual filtration plants, but chemical handling, storage and feeding equipment must be ample to apply the very large chemical doses involved. Flocculation must be more prolonged. Mechanical sludge-removal equipment is also justified. The softening processes are assisted by the solid contact provided in upward-flow basins.

Chemicals

Either hydrated lime or quicklime may be used, the choice being influenced by availability and relative costs and to some extent by the simplicity of applying hydrated lime. Quicklime ordinarily is cheaper, but it must be slaked. In the past it was difficult to gauge the precise amount of water to be added to secure the high temperatures needed without drying or caking the batch of lime being slaked. More recently pebble lime, which slakes readily, and continuous-flow slaking equipment have been developed, which regulate the water to provide temperatures of 66°-77°C. If quicklime is ground, it may be applied with typical dry-feed equipment, using controlled amounts of water and rapid agitation in the solution pot. The slurry or suspension of lime must be forced through solution hose at sufficient velocity to prevent deposition of lime. A hose too small will clog, and one too large will favour sedimentation. All hose should be provided with a connexion to water under pressure for flushing. The best procedure is to discharge the slurry into open troughs discharging above the flow line of the water being treated.

Soda ash may be applied with the lime or separately. Soda ash is quite soluble, so solution feed is feasible; in fact, a 30% solution can be prepared with warm water at 30°-40°C, or an 8% solution with cold water at temperatures of 10°-15°C. Usually dry-feed equipment is more convenient.

Flocculation

The cost of chemicals and the critical reactions occurring in softening justify the use of ample flocculation facilities with detention periods of 30-60 min. Mechanical flocculators are preferred. Flocculation is favoured by the return of a portion of the settled sludge to the raw water when continuous sludge removal is practised. The amount of sludge returned to the raw water is determined by experience.

Coagulation

Alum will function at the high pH values prevailing in lime-soda softening because aluminates are formed. Ferric sulfate, ferrous sulfate, sodium aluminate or activated silica also are used to improve the formation of floc. Sodium aluminate is favoured at small plants, but activated silica seems to be especially suited as a coagulant with softening, inasmuch as doses as low as 2 p.p.m. are successful. This is of special significance with solid-contact basins in which cold waters are softened.

Control is similar to that with conventional coagulation. The lime dose must be increased when alum, or ferric sulfate, or ferrous sulfate are used to counteract the acidity of the coagulants. The calcium sulfate resulting is hardness-producing, so soda ash may be added instead of lime with the coagulants to secure coagulation without increasing the hardness.

Sedimentation

Sedimentation basins should have detention periods of 4-6 h to favour precipitation and settling. Large basins are frequently provided with mechanical rakes that continuously remove the sludge.

Although solid-contact basins are well adapted to lime-soda softening, the velocity of upward flow is frequently such that heavy floc formation is essential and auxiliary coagulation is needed. This complicates the process and requires very careful supervision. Therefore when full-time technical control is not available, relatively larger basins of this type should be used to provide flow rates not greater than 24 m/h (8 ft/h; 1 US gal/min per square foot of basin area).

Recarbonation

Recarbonation is the process of adding carbon dioxide gas to water to lower pH values and to dissolve the unsettled crystals of calcium carbonate. The gas is produced by burning coke, coal, gas or oil in suitable equipment, the products of combustion being scrubbed and dried when necessary to remove tars, sulfur compounds, etc. About 12% of the gas from the furnaces is carbon dioxide. The weight of carbon dioxide produced is about 3 kg per kilogram of coke; 1.3 kg per cubic metre of manufactured gas (82 lb per 1000 ft^3); 1.8 kg per cubic metre of natural gas (115 lb per 1000 ft^3); and 2.4 kg per litre of kerosene (24 lb per UK gal; 20 lb per US gal). Improved types of pressure burners, through which compressed air is forced to completely burn coke or coal and produce non-corrosive gas not needing to be scrubbed, have eliminated the corrosion associated with moist carbon dioxide.

The gas is compressed and forced through pipes to diffusers or perforated pipe submerged in the water being treated, as in diffused-air aeration practice. Perforated pipe with 2.5-3 mm (3/32- to 1/8-in) diameter, opening on 15 cm (6-in) centres, and with 3-m (10-ft) submergence of piping, provides easily maintained facilities. Recarbonation basins should have detention periods of 15 to 30 min. These basins are located between sedimentation basins and filters or between two sedimentation basins operated in series before filtration.

As noted previously, the excess-lime treatment process includes the use of soda ash, which reacts with the excess calcium hydroxide to form sodium hydroxide and precipitate calcium carbonate. The caustic alkalinity so produced may be changed to sodium bicarbonate by recarbonation of the filtered water; or the split-treatment process may be used to produce a mixture of raw water and water subject to excess-lime treatment, so as to neutralize the caustic alkalinity by the action of the carbon dioxide and bicarbonates naturally present in the raw water. Recarbonation in two steps is, however, the best procedure for producing a softened water that will not encrust filter sand, because the settled water is in equilibrium with calcium carbonate and hence neither deposits nor dissolves this compound. This third softening process is discussed more fully below.

There is a trend towards replacing recarbonation by stabilization with sulfuric acid, whereby the excess of carbonates is replaced by soluble sulfates.

Sodium hexametaphosphate treatment

If equipment is not available for recarbonation, the residual calcium carbonate and magnesium hydroxide crystals in the settled water may be prevented from precipitating onto the filter sand or in water mains by the addition of sodium hexametaphosphate. This chemical is the opposite of a coagulating agent, as it disperses colloids so that they remain in suspension.

Doses of 0.25-1.00 p.p.m. are adequate and need not be varied once the best dose has been found for a given water.

Filtration

Conventional rapid sand filters are used. The sand should have an effective size of about 0.5 mm.

Chlorination

In chlorinating softened water, allowance should be made for the adverse effect of the high pH; in other respects chlorination is conventional.

Sludge disposal

The large amount of sludge produced by lime-soda softening—2 p.p.m. for each part per million of hardness removed—makes its disposal difficult. If space is available, the sludge may be discharged into lagoons, where it settles and only the excess of water escapes to the stream. When continuous sludge removal is practised, part of the sludge is returned to the raw water to secure solid-contact benefits, and the rest is discharged continuously into the receiving stream. If periodic removal is necessary, the sludge should be discharged when the stream flow is at a maximum, and the period of discharge should be extended as long as possible to dilute the sludge to the maximum extent, otherwise fish will be killed by the high alkalinity produced. At large plants lime may be recovered from sludge by the use of burners and associated equipment.

The operation of lime-soda softening plants

The complex processes and equipment require a trained staff with laboratory facilities to secure effective softening. Conditions, therefore, are not favourable for the lime-soda softening of small supplies; local conditions will determine when this process is practicable.

The following factors represent aspects of operation not encountered at the usual filtration plant and are intended to supplement the normal procedures used in the application of chemicals, flocculation, sedimentation and filtration.

Chemical doses

The doses of hydrated lime and quicklime required for the removal of free carbon dioxide, bicarbonate alkalinity and magnesium are computed by using the factors shown in Table 20. If the hydrated lime or quicklime is known to have a degree of purity differing from the 95% and 88% respective values given in the table, the various factors should be altered in accordance with the following formula:

$$\frac{\text{Factor in table}}{\%\text{ of purity}} = \text{new factor for specific lime in use}$$

For example, if 90% hydrated lime is used, the factor for the dose for magnesium removal becomes 3.20/0.90 = 3.55. In other words, a greater quantity of a less-pure hydrated lime is needed.

The use of Table 20 is shown by the following example: If the raw water contains 40.0 p.p.m. magnesium and this content is to be reduced to 3.0 p.p.m., then the removal of 37.0 p.p.m. will require 37.0 × 3.20 = 118.4 p.p.m. 95% hydrated lime.

TABLE 20

FACTORS FOR COMPUTING DOSES OF HYDRATED LIME AND QUICKLIME NEEDED TO REMOVE 1 p.p.m. CARBON DIOXIDE, BICARBONATE ALKALINITY OR MAGNESIUM

Constituent	95% hydrated lime	88% quicklime
Free carbon dioxide, as CO_2	1.77	1.44
Bicarbonate alkalinity, as $CaCO_3$	0.78	0.64
Magnesium, as Mg	3.20	2.61

Soda ash is available commercially as 100% sodium carbonate. The factor of 1.06 is used in computing the dose of soda ash required to remove non-carbonate hardness. Then the formula for reduction in non-carbonate hardness is: Reduction, p.p.m. × 1.06 = p.p.m. of soda ash required. For example, if the non-carbonate hardness is to be reduced from 50.0 to 35.0 p.p.m. or 15.0 p.p.m., then 15.0 × 1.06, or 15.9 p.p.m. soda ash will be required.

The dose of carbon dioxide required to change 1 p.p.m. calcium carbonate into calcium bicarbonate and to change 1 p.p.m. magnesium hydroxide into magnesium bicarbonate is computed by one factor, inasmuch as alkalinity is always expressed in terms of calcium carbonate, including hydroxide alkalinity. This factor has to be increased by 25% because this excess of carbon dioxide must be used. This adjusted factor is 0.55, which means that the carbon dioxide dose is 0.55 times the reduction desired in carbonate and hydroxide alkalinity.

The use of all the factors is shown in Table 21. The assumptions are that lime-soda softening is practised and that the carbon dioxide content, pH, bicarbonate alkalinity, non-carbonate hardness and magnesium of the raw water have been determined by laboratory tests, with the results shown in the first column. The constituents in the water to be reduced by the softening reactions, and thus influencing the chemical doses, are those italicized in the table: the reduction of bicarbonate alkalinity, carbon dioxide content and magnesium content determines the lime dose, and the reduction in non-carbonate hardness determines the soda ash dose. The

TABLE 21

ASSUMED RESULTS OF ANALYSIS OF WATER IN PROCESS OF TREATMENT

(in parts per million)

Constituent[a]	Raw water	After lime and soda ash treatment but before recarbonation		After recarbonation and filtration	
		Desired quality of settled water	Change in content of constituent	Desired quality of effluent	Change in content of constituent
Carbon dioxide	25.0	0	− 25.0	0	0
pH value	7.5	10.6	+ 3.1	8.8	− 1.8
Alkalinity to phenolphthalein	0	57.5	+ 57.5	15.0	− 42.5
Alkalinity to methyl orange	250.0	75.0	− 175.0	35.0	− 40.0
Bicarbonate alkalinity[b] . .	250.0	0	− 250.0	5.0	+ 5.0
Carbonate alkalinity[b] . . .	0	35.0	+ 35.0	30.0	− 5.0
Hydroxide alkalinity[b] . . .	0	40.0	+ 40.0	0	− 40.0
Non-carbonate hardness[c] .	50.0	35.0	− 15.0	35.0	0
Total hardness	300.0	110.0	− 190.0	70.0	− 40.0
Magnesium as Mg.	40.0	3.0	− 37.0	3.0	0

[a] Italics indicate constituents to be reduced.

[b] See Supplement for determination of alkalinity (page 300).

[c] Non-carbonate hardness = total hardness − alkalinity to methyl orange.

previously mentioned factors then are used as shown in Table 22. The dose of lime would be 357 p.p.m. plus 30 p.p.m. *excess* needed to secure effective results (10-50 p.p.m.), making the total dose 387 p.p.m.

TABLE 22

COMPUTATION OF HYDRATED LIME AND SODA ASH DOSES USING ASSUMED RESULTS IN TABLE 21

Constituent	Reduction in content of constituent, p.p.m.	Factor	Chemical dose, p.p.m.
Carbon dioxide	25	1.77	hydrated lime, 44
Bicarbonate *alkalinity* . .	250	0.78	*hydrated lime*, 195
Magnesium as Mg	37	3.20	hydrated lime, 118
Non-carbonate hardness .	15	1.06	soda ash, 16

Total computed hydrated lime dose = 375 p.p.m.

Soda ash dose = 16 p.p.m.

Recarbonation

The dose of carbon dioxide needed for recarbonation is determined by the amount needed to eliminate hydroxide alkalinity of 40 p.p.m. in Table 21 and change the carbonate alkalinity from 35 to 30 p.p.m., thus producing 5 p.p.m. bicarbonate alkalinity. This end result, together with a final pH of 8.8, is based on the assumed results of the marble test for calcium carbonate equilibrium, thus avoiding over-saturation with carbonate and resulting deposits on filter sand. The total change, then, is 40+5 p.p.m., or 45 p.p.m. The dose of carbon dioxide then is $45 \times 0.55 = 25$ p.p.m.

Laboratory control

The control of the complex processes involved in lime-soda softening requires much more laboratory work than in ordinary filtration, because the doses are computed from the results of analysis. The tests routinely used are those for methyl orange and phenolphthalein alkalinities; pH; total hardness, magnesium; and flocculation. From the two tests for alkalinity, the bicarbonate, carbonate and hydroxide alkalinities may be computed, as shown in the Supplement, page 302. The alkalinity and pH may be used to determine the content of carbon dioxide from Fig. 19, page 307.

A modification of the jar test provides the best means of testing softening reactions in the laboratory before they are used in the plant, as described on page 316.

Use of results of the softening-reactions test

The test described on page 316 does not provide for recarbonation, but it duplicates what would happen if settled water were not recarbonated but filtered.

Tables 31, 32 and 33 in the Supplement (pages 303, 304 and 305) are used to determine from the three tests—total hardness, methyl orange alkalinity and phenolphthalein alkalinity—the nature of the mineral compounds in the water tested and hence the effect of the computed doses of lime and soda ash as used in the laboratory. The computations are shown in Table 23, using a typical raw water and assuming that the treated water is to have the quality noted. Using the conversion factors in Table 22 for computing the lime and soda ash doses from the reduction in mineral content shown in Table 23, the total dose of 95% hydrated lime is 239 p.p.m. plus an excess of, say, 30 p.p.m., or 269 p.p.m. The dose of soda ash is 26 p.p.m.

The intended quality of the treated water, as shown in Tables 21 and 23, represents normal values, which should be the goal of softening. The results shown in Table 24 represent the anticipated quality and the actual quality of the test sample after lime and soda ash treatment and the softening-

TABLE 23

COMPUTATION OF DOSES FOR A TYPICAL RAW WATER

Constituent	Raw water, p.p.m.	Treated water, proposed p.p.m.	Reduction, p.p.m.	Factor	Chemical dose, p.p.m.
Total alkalinity	175	35	140	0.78	hydrated lime, 109
Phenolphthalein alkalinity	0	20	0	0	
Magnesium	30	3	27	3.20	hydrated lime, 86
Carbon dioxide	25	0	25	1.77	hydrated lime, 44
Total hardness	250	85	165	0	
Non-carbonate hardness	75	50	25	1.06	soda ash, 26

reactions test. The values shown are for softened waters prior to recarbonation. The intended pH of 10.6 was selected to precipitate magnesium in the presence of an excess of caustic alkalinity. Assume that the results of analysis of the softened sample are as shown in Table 24. The discrepancy between the intended and actual results is evident. These errors in the test are greater than would be expected, but they have been selected to illustrate the procedure for making corrections.

At first glance it is evident that the water has been under-softened by the use of an inadequate lime dose. The detailed meaning of the results of the softening-reactions test is disclosed by using Tables 31, 32 and 33 of the Supplement. Start with the analytical results and use Table 31, then use these new detailed values in Table 32 and also in turn in Table 33. Thus

TABLE 24

COMPARISON OF RESULTS OF SOFTENING TEST WITH INTENDED RESULTS

Constituent	Intended results	Test results	Error
Total alkalinity	35	60	25 high
Phenolphthalein alkalinity	20	30	10 high
Magnesium	3	25	22 high
Carbon dioxide	0	0	0
Non-carbonate hardness	50	40	10 low
Total hardness	85	100	15 high
pH	10.6	9.0	1.6 low

Inasmuch as the phenolphthalein alkalinity is equal to one-half the total, or methyl orange, alkalinity, Table 31 discloses that:

Hydroxide alkalinity = 0
Carbonate alkalinity = 60 p.p.m.
Bicarbonate alkalinity = 0

Inasmuch as the total hardness is greater than the total alkalinity, Table 32 discloses that:

Non-carbonate hardness = 100−60 = 40 p.p.m.
Carbonate hardness = 60 p.p.m.
Sodium alkalinity = 0

Inasmuch as hydroxide and sodium alkalinities are equal (both zero), Table 33 discloses that:

Sodium hydroxide = 0
Sodium carbonate = 0
Calcium hydroxide = 0
Calcium and magnesium carbonate = 60 p.p.m.

The detailed results so computed with the guidance of the tables confirm the inadequacy of the lime dose actually used to remove the calcium and magnesium, and also the fact that soda ash treatment removed more non-carbonate hardness than intended. The errors assumed to illustrate the procedure are not errors of theory but errors in the selection of lime and soda ash doses when making the softening-reactions test. First, *less* lime was added than needed. This could have been due to the use of a lime suspension having a lower concentration than intended. For the purpose of illustration, however, the error is assumed to have been due to the use of the lime dose as computed, i.e., 239 p.p.m., *without the excess of 30 p.p.m. needed*, which would have given a total of 269 p.p.m. Second, the soda ash dose was high and thus reduced the non-carbonate hardness to 40 p.p.m. rather than to 50 p.p.m. as intended. Thus the soda ash dose was high by 10 × 1.06 = 10.6 p.p.m. It is not surprising, therefore, that the pH of the treated water as tested was 9.0 rather than the 10.6 intended and that there was no caustic alkalinity. The increase in the lime dose would provide the caustic alkalinity needed and produce a settled water of the quality intended. The softening-reactions test would be repeated, therefore, with modified doses and the results again checked with the intended character of the treated water.

Experience should prevent errors of this type, but if they occur they may be appraised and corrected as illustrated. In normal operation the settled water would be analysed and minor changes would be made in the chemical doses as needed.

The laboratory control of the softening process must also be directed to determination of the dose of carbon dioxide required for recarbonation or of the additional amount of soda ash dose needed to change the excess of calcium hydroxide to sodium hydroxide, thus precipitating calcium carbonate.

Recarbonation with carbon dioxide is controlled by computing the dose required to change hydroxide alkalinity to carbonate alkalinity, plus about 5.0 p.p.m. bicarbonate alkalinity. The computations would be as indicated above for the typical results shown in Tables 21 and 22, using the factor of 0.55.

If recarbonation facilities are not available and excess-lime treatment is being used, the soda ash dose would be that needed to remove non-carbonate hardness plus that needed to react with the hydroxide alkalinity. Since both hardness and hydroxide alkalinity are expressed in terms of calcium carbonate, the factor of 1.06 shown in Table 22 would be used to compute the soda ash dose. For example, suppose the reduction in non-carbonate hardness is to be 15 p.p.m. and the hydroxide alkalinity is 20 p.p.m.; then 15 + 20 = 35 p.p.m., so a soda ash dose of 35 × 1.06, or 37 p.p.m., is required.

Ion-exchange softening and demineralization

Purpose

This water-treatment procedure derives its name from the fact that certain materials have the power to exchange their ions for those in water coming in contact with them. The best known are the base-exchange zeolites, which will exchange the sodium cation (positive ion) for the calcium and magnesium ions in the water. Although only the ions are exchanged, the net result is that calcium and magnesium bicarbonates are changed to sodium bicarbonate, and calcium and magnesium sulfate or chlorides are changed to sodium sulfate or chloride. This is known as base-exchange softening, or as zeolite softening when the material used is the natural greensand or synthetic zeolite. More recently developed materials, however, are also available. These have characteristics that permit the removal of both cations and anions, thus extending the older softening process to demineralization. When both bases and acid radicals are removed, the process is known as ion-exchange demineralization. The two processes are here discussed separately.

BASE-EXCHANGE SOFTENING

The reactions involved in this process are shown below, the letter R being used to indicate the anionic portion of the material which *does not enter into the reactions.* Thus Na_2R means the base-exchange material containing sodium, whereas CaR and MgR represent the same material after the sodium has been exchanged for calcium and magnesium by the removal of these minerals from the water and the release of an equivalent amount of sodium to the water. The ionic reaction is as follows:

$$Ca^{++} + Na_2R \longrightarrow CaR + 2Na^+$$

In terms of hardness-producing compounds, the following reactions occur:

$$\begin{aligned} Ca(HCO_3)_2 + Na_2R &\longrightarrow CaR + 2NaHCO_3 \\ CaSO_4 + Na_2R &\longrightarrow CaR + Na_2SO_4 \\ Mg(HCO_3)_2 + Na_2R &\longrightarrow MgR + 2NaHCO_3 \\ MgSO_4 + Na_2R &\longrightarrow MgR + Na_2SO_4 \end{aligned}$$

After the sodium has been removed from the material by base exchange, regeneration is practised to restore the sodium. Solutions of common salt are used to reverse the process, as follows:

$$\begin{aligned} CaR + 2NaCl &\longrightarrow CaCl_2 + Na_2R \\ MgR + 2NaCl &\longrightarrow MgCl_2 + Na_2R \end{aligned}$$

The calcium and magnesium chloride are discharged to waste with the spent salt solution.

Since water of zero hardness is corrosive, a portion of the raw water is mixed with the softened effluent to provide the desired hardness.

Advantages

The ion-exchange process has the great advantages of simplicity of operation and control, compactness in size, and ability to demineralize waters to the extent desired, irrespective of changes in the mineral content of the raw water. For these reasons the process is widely used in industry and in small units for the home and is better suited to smaller municipalities than lime-soda softening plants. In general, carbonate hardness is more economically removed by lime treatment; non-carbonate hardness is usually removed more cheaply by the ion-exchange process than with soda ash.

Disadvantages and limitations

Base-exchange units are uneconomical to operate when the total hardness exceeds 850-1000 p.p.m. In general, demineralization is restricted to those waters having a total solids content below 3000-5000 p.p.m. The applied water should preferably be clear or have a turbidity under 5 p.p.m., and preferably be free from iron or have a content of less than 0.5 p.p.m. in the precipitated, oxidized state, otherwise the material will become coated and lose effectiveness. (See page 216 for the ion-exchange procedure as used to remove iron and manganese in the reduced state.) Greensand and synthetic zeolites should not be used to soften waters of high sodium alkalinity or those with a pH below 6.0 or over 8.5. Residual chlorine reacts with some of the organic exchange materials, so chlorine should not be applied to *raw* waters treated by such materials.

Standards of quality of final effluents

The *International Standards for Drinking-Water* specifies as " acceptable " a sulfate- and chloride-ion content of 200 p.p.m. each, 400 p.p.m. sulfate ion and 600 p.p.m. chloride ion being the maximum allowable. When waters of high non-carbonate hardness are softened by the ion-exchange process, the effluent has a high content of sodium sulfate and soduim chloride. If the " acceptable " limits would be exceeded by ion-exchange or lime-soda softening, one of the demineralization processes discussed later must be used to remove the sulfates and chlorides.

Base-exchange softening practice

Units of measurement

The capacity of ion-exchange materials to remove hardness is expressed in terms of grains per cubic foot in the United States and in the United Kingdom. A grain is 1/7000 lb; 1 grain/gal US = 17.1 mg/l, or 17.1 p.p.m. In France this capacity is expressed in degrees per cubic metre of water per litre of the material. A French degree is 10 p.p.m., or 10 mg/l, which is 10 g per cubic metre of water, hence 1 degree/m^3/l = 10 g per cubic metre of water per litre of material. This quantity can also be expressed, using only metric units, as grammes of hardness removed per litre of material, which would be one-tenth the value of the French unit. The unit of measurement used in this text is grains per cubic foot; 1 grain/ft^3 = 64.8 mg/ft^3 or 2.29 g/m^3.

Base-exchange materials

The selection of the base-exchange materials should be based on the consideration of local conditions, including availability, costs, hardness of raw waters, relative quantity required (size of bed), and proposed length of operating cycle between regenerations. Base-exchange materials are patented, so detailed information must be secured from the manufacturer in order to determine the size of the 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 to remove 1000 grains of hardness. The grains of hardness removed per cubic foot of material per cycle range from 2800 to 5500 for natural zeolites and from 9000 to 12 000 for synthetic inorganic zeolites; the figure is about 7000 for phenolic and about 30 000 for non-phenolic organic materials. The amount of salt required per 1000 grains of hardness removed varies from 0.25-0.35 lb for organic materials to 0.50 lb for inorganic zeolites.

Softening units

Either pressure or open gravity units are satisfactory, and either upward or downward flow may be utilized, but downward-flow units 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 gal/min/ft^2. Backwash rates of 6 to 8 gal/min/ft^2 are normal. The minimum depth of material in the unit should be 30 inches. The freeboard 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.

Bypass

A bypass 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 bypass should be fitted with a metering device so that the proportion of hard water that is bypassed can be controlled. Consideration should be given to the removal of iron and/or manganese from the bypassed water if the concentration of these minerals in the *blended effluent* exceeds the desirable limit of 0.30 p.p.m. Refer to Chapter 12 for a discussion of the factors governing the design of equipment suitable for this purpose.

Salt storage and solution facilities

Facilities should be provided for the storage of at least 1½ truck-loads or 1 car-load of salt in a basin to which water is added to form a saturated solution with a strength of about 25%. Concrete basins must be painted with Inertol or other salt-resistant paints. 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 that the saturated brine will be diluted as used. The ejector should have sufficient capacity to produce a rate of flow through the bed of 14-19 m/h (48-64 ft/h; 6-8 US gal/min/ft^2). A wash-water connexion should be provided for flushing salt from the unit after regeneration.

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 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 water-bearing 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 in which technical supervision is lacking.

Operation of base-exchange units

Softening cycle

Continue this cycle as long as water of zero hardness is produced. Stop the cycle upon the appearance of 1-5 p.p.m. hardness in the effluent or when any hardness becomes apparent by the soap test.

Regeneration cycle

Backwash downward-flow units to loosen the bed. Force diluted salt solution through the bed. Rinse with water to remove excess salt solution. Start the following softening cycle.

The volume of salt solution is usually established by the manufacturer, but usually a 5-10% solution is slowly forced through a bed for a period of 30 min. The whole process may be standardized, especially when waters of uniform hardness are being softened, such as many well waters.

Saturated salt solution contains about 26% salt by weight, or 331 g/l. In practice, however, the strength is assumed to be 25%, or 2.97 g/l (2.98 lb/UK gal; 2.48 lb/US gal). The saturated solution is diluted by the water used to operate the solution injector to 5% to 10% strength, in volumes to provide 241-349 g salt per 100 g hardness to be removed (0.35-0.50 lb/1000 grains). Using for example the more efficient value of 0.35 lb/1000 grains and a unit containing 20 ft^3 of an exchange material with a unit capacity of 20 000 grains/ft^3, the following values may be computed:

$$20 \times 20\,000 = 400\,000 \text{ grains of hardness removed per cycle}$$

$$\frac{400\,000}{1\,000} \times 0.35 = 140 \text{ lb salt required}$$

$$\frac{140}{2.48} = 56.4 \text{ US gal saturated salt solution required}$$

This volume of saturated solution would be diluted as injected, or would be diluted in a separate tank to a strength of, say, 5%, that is, to $^1/_5$ strength, with 4 parts water and 1 part saturated solution, giving a final volume of 282.0 gal dilute solution in the separate dilution tank.

The volume of water softened per cycle may be computed as follows: Assume that the raw water has a hardness of 250 p.p.m. and that the unit described above will remove 400 000 grains (25.9 kg) per cycle. Inasmuch as water containing 250 p.p.m. hardness contains 250/17.1 = 14.6 grains/US gal, the unit will soften in one cycle 25.9 × 1 000 000/250 = 103 500 l or 103.5 m^3 (400 000/14.6 = 27 000 US gal).

Rinsing cycle

The regeneration of ion-exchange softeners is followed by a thorough rinsing of the bed with raw water, the effluent containing the unused salt being piped to waste until it has a hardness of about 1 p.p.m. and contains no chlorides by test (see the Supplement).. The effluent is then diverted to the supply as the start of the next softening cycle.

Usually rinsing is at the rate of about 4.8 m/h (16 ft/h; 2 US gal/min/ft^2), and the rinsing period is about 20 min for zeolites and about 30 min with carbonaceous and resin materials. The directions of the manufacturer should be followed for each specific installation. Rinsing may be stopped when the test for chlorides indicates that the rinse water no longer has a high content of salt.

Use of bypass

As indicated, the procedure is to bypass a portion of the raw water for mixing with the softened water of zero hardness to produce a final effluent that is not corrosive. For example, assume that the raw water has a hardness of 250 p.p.m. and that the mixed effluent is to have a hardness of 85 p.p.m. or 85/250 = 34% of the original hardness; then 34% of the total flow would be bypassed and 66% would be softened to produce a mixed effluent having a hardness of 85 p.p.m.

Corrosion prevention

The mixed effluent may not be entirely non-corrosive, so the marble test should be made to determine the best proportion of raw water to use and whether chemical treatment also is needed. Usually soda ash, caustic soda or sodium silicate is used when necessary for corrosion prevention, as these chemicals, unlike lime, do not increase the hardness.

Ion exchange with carbonaceous materials and organic resins

Carbonaceous exchange materials may be used in the sodium cycle described above to provide exchange capacities of 10 000-40 000 grains/ft^3. This material is not influenced by acid waters or by waters of a pH below 9.5, so long as calcium carbonate does not precipitate on the grains. Such a precipitate is prevented by using acid rather than salt solution to regenerate the material. This is the hydrogen cycle of operation, discussed below.

Organic resins have been developed with exchange capacities of 10 000-40 000 grains/ft^3. They may be operated on the sodium cycle, the hydrogen cycle, or the hydroxide cycle (thus providing the complete demineralization process described below, with cations removed by the hydrogen cycle and anions by the hydroxide cycle).

Hydrogen cycle for base exchange

The reactions with the hydrogen cycle are as follows:

$$Ca(HCO_3)_2 + H_2R \longrightarrow CaR + H_2O + 2CO_2 \qquad (1)$$

$$CaSO_2 + H_2R \longrightarrow CaR + H_2SO_4 \qquad (2)$$

The same reactions would take place for magnesium compounds, or when sodium and potassium cations are removed for demineralization. Regeneration with sulfuric acid is practised to restore the hydrogen:

$$CaR + H_2SO_4 \longrightarrow H_2R + CaSO_4 \quad (3)$$

$$MgR + H_2SO_4 \longrightarrow H_2R + MgSO_4 \quad (4)$$

$$Na_2R + H_2SO_4 \longrightarrow H_2R + Na_2SO_4 \quad (5)$$

The amount of sulfuric acid required for regeneration is 136-272 g (0.3-0.6 lb) per 1000 grains of hardness removed.

It will be noted that carbon dioxide is produced when *only* carbonates are being removed, as in reaction (1). Under these conditions calcium, magnesium, sodium and/or potassium bicarbonates may be completely removed by the hydrogen cycle, followed by aeration to remove the carbon dioxide, producing demineralization.

The sulfuric acid produced when sulfates are removed, or the hydrochloric acid produced when chlorides are removed, is neutralized by split treatment or by the use of an anion-exchange resin as discussed under Ion-Exchange Demineralization, below.

Split treatment

This procedure is the use of two units in parallel, one operated on the sodium cycle and the other on the hydrogen cycle. The first provides an alkaline softened water, the second an acid softened water, the two being mixed for mutual neutralization. The mixed natural effluent, however, would contain sodium sulfate and sodium chloride. Therefore split treatment cannot be used when non-carbonate hardness is sufficiently high to produce more than 200 p.p.m. sulfate or chloride ion. This is equivalent to a calcium sulfate content of 283 p.p.m. or a calcium chloride content of 314 p.p.m.

ION-EXCHANGE DEMINERALIZATION

Hydroxide cycle for anion exchange

If resins are regenerated with sodium hydroxide or caustic soda, they may be operated to exchange or absorb anions, or acid radicals, such as carbonate, sulfate and chloride ions. This is just the reverse of the base-exchange cycle. Thus if these units are used in series with units using the hydrogen cycle, complete demineralization is secured. For instance, the first hydrogen-cycle unit would produce, through base exchange, an effluent containing carbon dioxide, sulfuric acid and hydrochloric acid. The carbon dioxide would have to be removed by aeration before treatment by weakly basic resins, but could be removed as carbonic acid with the other acids if strongly basic resins were used. Thus the first unit would remove the cations as in the hydrogen cycle (reactions 1 and 2 above), and the acids produced would be removed in the second unit operated in series as follows:

$$2R(OH) + H_2SO_4 = R_2SO_4 + 2H_2O \quad \text{(weakly basic resins)} \qquad (6)$$

or

$$2R(OH) + H_2SO_4 = R_2H_2SO_4 \quad \text{(strongly basic resins)} \qquad (7)$$

In other words, the effluent of the second unit would be demineralized. Regeneration of these units with caustic soda is as follows:

$$R_2SO_4 + 2NaOH = 2R(OH) + Na_2SO_4 \qquad (8)$$

The capacity of these resins to exchange or absorb acids varies from 12 000 to 25 000 grains/ft^3. The amount of caustic soda used in regeneration is 272-454 g (0.6-1.0 lb) per 1000 grains of hardness removed. The cost of this process at present exceeds that of distillation when the total-solids content is greater than about 3000 p.p.m.

Operation of demineralization units

Units containing carbonaceous exchange materials and organic resins are operated on the sodium cycle as with conventional zeolites. When one unit is used on the sodium cycle in parallel with another used on the hydrogen cycle, the rates of flow through the two units are controlled so that the alkalinity of the effluent of the first unit neutralizes the acidity of the effluent of the second unit, producing a neutral combined effluent. The mixed effluents, however, may have to contain a surplus of the alkaline sodium-cycle effluent to provide a non-corrosive water. There is no simple test for the effluents of two units operated in series on the hydrogen cycle (reactions 1 and 2) and then on the acid-absorbing cycle (reactions 6 and 7). In practice the demineralized effluent is checked with equipment for pH determination and for disclosing the conductivity of the water—the latter because the capacity of water to conduct an electric current is proportional to the content of ions in the water, hence the conductivity is very low when demineralization is complete. The pH test indicates when the two units need to be regenerated.

Detailed directions are provided by manufacturers for the operation and maintenance of these units. Because they differ, it is not possible to present general guide-lines.

Other methods of demineralization

Electrodialysis

Special membranes are available that have the property of permitting the passage of ions without allowing water to pass through. One type of membrane permits the passage only of positive ions, or cations; a second type permits negative ions, or anions, to pass through. The energy needed for this action is supplied by a direct electric potential, which causes the cations to flow in one direction and the anions in the other. The net result is that the ions pass from the saline water through the two membranes into adjacent chambers, where strong brine solutions develop and are discharged to waste. In practice a number of membranes are arranged in alternate layers with spaces between, so that saline water being demineralized will flow in the spaces between all membranes and the ions of the mineral salts in the water will pass from alternate spaces into adjacent spaces on each side. The water in one group of alternate spaces is thus demineralized, while the brine in the other group of spaces becomes more highly mineralized. The saline water is thus separated into demineralized water and more concentrated brine, the latter being wasted. Disposal of the waste brine presents a serious problem and may result in appreciable increase of unit costs.

Large plants of this type are under construction in the United States, whose aim is to produce water at a cost of US$ 0.25-0.40 per 1000 gal. A small plant in California produces an effluent having a hardness of 38 p.p.m., a chloride content of 15 p.p.m., and a sulfate content of 195 p.p.m., at the rate of 20 gal/min, or 28 000 gal/day, from raw water having a hardness of 574 p.p.m., chlorides 257 p.p.m., and sulfates 1158 p.p.m. About 0.5 gal of brine is wasted per gallon of fresh water produced. The current consumption is 12.3 kW per 1000 gal, costing US$ 0.21 per 1000 gal. The building cost is US$ 105 000, and the equipment US$ 80 000. The unit costs of producing water are as follows: operation, US$ 0.89; capital charges, US$ 0.54; total, US$ 1.43. Costs elsewhere have exceeded the US$ 2.00 value, so a great deal of work is necessary to reach the goals noted above.

This process is best suited to the partial demineralization of saline waters rather than sea water, as the output of water per unit of power is inversely proportional to the saline content.

Distillation

The cost of power for the evaporation of water is not influenced by its saline content, so sea water may be distilled as cheaply as brackish waters. The cost of distillation with currently available equipment, using low-priced fuel oil, is about US$ 2.00 per 1000 gal. New equipment is under development, which is expected to reduce the cost to US$ 1.00 per 1000 gal.

Solar-heat distillation

Solar heat is free, so it has been assumed that its use in the evaporation of saline waters and sea water would be economical. Evaporation of saline waters is readily accomplished in large shallow basins, but the equipment needed to condense the vapours presents problems. Solar heat, at the relatively high rate of 5000-6500 kcal/m^2 (2000-2600 Btu/ft^2) per day prevailing in areas with a sunny climate, will evaporate 4-6 mm (0.013-0.02 ft; 0.1-0.15 US gal/ft^2) per day, which represents about 50% efficiency in using solar heat in areas where this intensity prevails. An area of 700-1000 m^2 (about 7500-11 000 ft^2) is therefore needed to produce 4 m^3 (1000 gal) per day. The evaporating area must be covered with glass or transparent plastic to retain the water vapour after it is evaporated from the water and also to serve as the condensing surface. Current research is directed towards the transfer of the water vapour by an air current to separate condensers. Present cost is about US$ 1.75/1000 gal, with the prospect of reducing it to about US$ 1.00/1000 gal.

Freezing

The freezing of saline waters produces fresh-water ice crystals, which may be separated and then melted. The energy needed is independent of mineral content and is less than that for evaporation, so active research is now being directed to the development of equipment for handling sea water, with the anticipation of reducing costs with full-scale plants to the range of US$ 0.50-1.00 per 1000 gal.

Economics

The economic advantage of demineralizing otherwise unsuitable waters must be appraised in the light of broad social and industrial factors underlying the well-being and advancement of communities in areas where fresh-water resources are very limited. At times the very existence of an industry has depended on the distillation of sea water—e.g., the oil industry in parts of the Middle East and on the island of Aruba—notwithstanding the cost. In other instances the comparison of costs between the demineralization of a local brackish water supply and the piping of fresh water from distant points is quite complex and would require detailed engineering analysis. In general, the cost of demineralization will have to be reduced much below US$ 1.00/1000 gal to be competitive with fresh-water supplies. For instance, the average total cost of fresh-water supplies in the United States is US$ 0.12/1000 gal, treatment alone accounting for $ 0.05 of this total. The figure of US$ 1.00/1000 gal of salt water converted does not include depreciation of equipment, distribution of water and administration. So far none of the processes of demineralization of brines or sea water produces water of acceptable mineral content at this low cost. The need is so great, however, that over ten million dollars is being spent in the United States alone each year in research. The hope is that intensive research and industrial development will enable saline and sea waters to be demineralized at costs comparable to the value of fresh water in areas where fresh-water resources are limited.

Summary

Equipment under development is complex, and maintenance and operation are more involved than conventional water-treatment processes. Research is being directed to economy and greater simplicity. There is hope for concrete advance in the distillation and freezing processes for the demineralization of sea water and in electrodialysis for the demineralization of saline waters.

CHAPTER 14

RECORDS

The maintenance of files of information has little purpose unless they are available for ready reference and record experience in a form which can be used as a guide to the solving of current problems. Records represent condensed information; they should be systematically kept and regularly filed, to be referred to as needed.

Forms

The type and complexity of record forms required depend on the size and character of a treatment plant. Thus simple forms may be adequate for a chlorination plant treating a well supply: they will provide space for recording the time of starting and stopping pumps, readings of the kilowatt-hour meter, volume of water treated, indicated chlorine dose, loss in weight of chlorine cylinder—i.e., the amount of chlorine compound used (actual dose)—concentration of residual chlorine as determined at specified intervals, and a remarks column for recording pertinent events or observations. These data may be recorded on two forms, one dealing with pump operation and the other with chlorination, especially when pumps are engine-driven and records of consumption of fuel, lubricating oil, etc. are needed.

By contrast very extensive records must be kept at large lime-soda-softening plants where fluoridation is carried out and where laboratory control is available. Under these circumstances shift operators should keep individual records of the various chemical doses, volume of water treated, etc. at hourly intervals, on forms suitable for an 8-hour-shift period or a 24-hour period. The information recorded on these working forms should be transcribed daily on to daily summarizing forms. The chief operator or chemist would also use special forms for records of laboratory tests, which would include volume of each specific sample examined, volume of reagents used and the computations involved, so that the work may be rechecked later if the results are found to be inconsistent. An

FIG. 15

SAMPLE FORM FOR REPORTING ON OPERATION AND LABORATORY CONTROL OF FILTRATION PLANTS

OPERATING REPORT

PLANT

MONTH AND YEAR

DATE	SOURCE, 1000 gal				RAINFALL, inches	WATER TREATMENT RECORD													LABORATORY RECORD																				COLIFORM INDEX, MPN	37° COLONY COUNT, FINISHED WATER
						CHEMICALS USED, lb						CHLORINE, lb		FILTERS					CHEMICAL ANALYSES, p.p.m.							BACTERIOLOGICAL EXAMINATION*														
																										PRESUMPTIVE									CONFIRMED					
																										RAW			SETTLED		FILTERED		TAP		FILTERED		TAP			
	PURCHASED WATER	WELL WATER PRODUCED	SURFACE WATER PRODUCED	TOTAL WATER								PRE	POST	NO.OF FILTERS USED	TOTAL FILTER HOURS	AVER. LENGTH OF FILTER RUNS, h	NO.OF WASHES	WASH WATER, 1000 gal								ml	ml	1.0 ml	1.0 ml	10 ml	1.0 ml	10 ml	1.0 ml	10 ml	1.0 ml	10 ml	1.0 ml	10 ml		
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	BB	CC	DD	EE	FF	GG	HH	II	JJ	KK	LL	MM	NN
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AVG																																								
MAX																																								
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*Note volumes of raw water examined in columns Z, AA and BB. Record results as + for positive and — for negative in appropriate columns.

From US Air Force (1959) p. 17.

example of the laboratory work sheet is that for microscopic examinations shown in Fig. 22 (page 356). The daily summary of all the pertinent data should be on forms having 31 horizontal lines, so that one form will cover a period of one month. Space should be provided for a monthly summary of totals, maximum and minimum values and computed monthly averages.

Many treatment plants are more complex than chlorination installations but yet less comprehensive in processes involved than large softening plants. Fig. 15 shows a typical form used at filtration plants where daily records on forms designed for a period of one month are adequate. This form includes space for laboratory results representing plant performance, i.e., the results achieved by the use of the chemical doses and operating data recorded. A plant log should also be kept so that casual daily events and observations may be noted for future guidance. For instance, water temperature, as influenced by meteorological conditions, may be observed to affect coagulation or the washing of filters, and study of such observations may disclose trends which can be useful guides in future operation.

An often-neglected aspect of records is the systematic notation of information on maintenance and repair of equipment. Protective maintenance should be organized according to instruction manuals for specific equipment, and routine work should be recorded. For example, a chemical feeder should be checked for lubrication and functioning characteristics each day, shut down for cleaning and adjusting on a weekly basis, and given an overhaul for replacement of worn parts on a semi-annual or annual basis. Pertinent records of such protective maintenance will serve as a reminder of the need for such periodic checking to forestall the postponement of repairs with the risk of failure of equipment.

Files

One of the surprisingly frequent deficiencies encountered at treatment plants is the absence of a set of construction plans and specifications applying to the plant *as built*, together with catalogues and instruction manuals for the equipment. These are essential to the economical operation and maintenance of any plant, especially the more elaborate equipment at large filtration and softening plants. The design and construction engineers should be required to furnish at least two complete sets of plans and specifications, one to be filed in the central office and the other at the treatment plant. Operators should study these so that they know the location, characteristics and functions of all equipment. Colour coding should be used to indicate the nature of piping, conduits, electric wiring, etc. in treatment plants, for guidance in repairs and maintenance.

CHAPTER 15

PERSONNEL

Although there is no substitute for ability and experience, the realities of life often necessitate compromise. The need for well-trained staffs may be greatest just where funds and facilities are scarcest, as where water resources are so limited that a heavily polluted and mineralized water must be used in spite of the cost and complexity of treatment. No hard-and-fast rules can be laid down, therefore, as to operating personnel, but certain procedures can assist in ensuring the adequacy of personnel for the operation and control of treatment plants.

Qualifications of operators

Many epidemics of water-borne diseases in the past were traceable to faulty operation (due in large part to poor training of operators or to individual neglect of duty) rather than to inadequate equipment. One factor in the situation was often the appointment of operators for political motives rather than for ability. In the United States this led to the organized training of operators and the establishment of minimum qualifications for operators of specific types and sizes of plant, so that trained and experienced operators could not be replaced by unqualified persons, and all qualified operators would have the incentive of advancement. These programmes, however, cannot be applied on a world-wide basis. Minimum qualifications must be considered in the light of budgetary realities and the supply of trained personnel.

The operator in charge of a plant should be technically skilled and experienced. The desirable qualifications may be summarized as those of a good manager, familiar with hydraulics as applied to waterworks and with the principles of physics, chemistry and biology underlying water-treatment processes as well as the public health objectives of delivering attractive water of safe, sanitary quality to the consumer. Ideally a sanitary engineer should be employed for this position, but in many parts of the world this ideal cannot be reached because of the shortage of sanitary engineers or because salary scales are too low to attract engineers to such positions. Therefore the ultimate responsibility for technical supervision frequently has to be assigned to the waterworks superintendent or chief engineer or to a consultant. The employment of a consultant has the advantage of making available the services of a specialist to a number of plants when they are located close enough together to make the travelling practical.

In practice the qualifications of chief operators cover a wide range—from technically trained superintendents of filtration to skilled mechanics. When the latter work under the general supervision of someone with technical training, the experience, innate ability and integrity of the operator are more important than formal training. The qualifications of shift operators at large plants should be the same as those of chief operators but with less experience required, so that experience will eventually qualify them for advancement.

Considering the need to grasp the principles of technical subjects and their application, the minimum education qualification for chief operators of small plants or for shift operators of large plants would appear to be a secondary school education. The realistic approach to this question is based not so much on consideration of the subjects taught in secondary schools but rather on the fact that completion of secondary school testifies to a certain ability to study and learn, as well as to the character and status associated with continuing in school for the time required.

Because opportunities to gain specific experience in a subordinate post at a water-treatment plant are rare, it is impossible to standardize the previous experience required for operators. Their experience qualifications must often be judged in terms of previous occupation and aptitude for skills similar to those called for in water-treatment plants.

The task of training operators becomes all the more important when the desirable technical qualifications cannot be met. Training may be subdivided into (*a*) in-service training of operators at specific plants in the course of their routine duties and (*b*) short courses given at some centrally located large treatment plant, where operators from a region may gather for more formal instruction, supplemented by demonstrations and field work.

In-service training of operators

The success of any in-service training programme depends on many factors. Especially at large plants, a chain of command must first be established, so that each member of the staff will be supervised by a superior who is recognized as such and hence is respected. Until this respect is gained, instruction and suggestions will not be received in the right spirit, and resentment may destroy the open-minded attitude necessary for learning. Once respect is gained, however, each daily order or suggestion can be given by a superior in a manner which explains its objective or purpose, the principles underlying the equipment or process involved, etc. until the subordinate has mastered the details.

Second, a more formal, organized programme is necessary to provide basic explanation of complex equipment or processes. For instance, the whole subject of chlorination could be covered in a series of discussions of the points dealt with in Chapter 8 as exemplified in the chlorination practice actually followed in the specific plant: the discussion of chlorine cylinders and the properties of chlorine gas, for instance, would use the diagrams of cylinders, valves, etc. which should have accompanied the equipment from the manufacturers. This type of instruction would be followed at intervals until the whole treatment plant had been covered.

Third, the operators and subordinate personnel should be helped to improve their powers of observation by having their attention drawn to examples in the plant; for instance, the character of floc during periods of relatively poor flocculation would be compared with good or standard floc. Even relatively unimportant details such as hard-opening valves may be used to demonstrate the effect of poor maintenance. Such in-service training should aim at increasing awareness of the appearance, " feel ", sound, odour, etc. of the plant when functioning normally, so that abnormal events will be noted at once and corrective action taken.

Fourth, the orthotolidine test made by shift operators should be used as the introduction to other plant-control tests in the laboratory. The approach here should be to expand the range of " tools " available to each operator, so that he will recognize the measurement of, for example, residual-chlorine content or of pH value as similar in nature and purpose to the measurement of water pressure by a gauge or of volume by a meter. This training should include the actual use of laboratory results in the control of the plant.

The fifth aspect of in-service training should embrace organized reading of catalogues, manuals and any other available literature. This requires the acknowledgement by the supervisor that reading by a subordinate while on duty is not neglect of duty but rather a good use of spare time, which should be available at well-functioning plants during periods when major repairs are not needed and normal chemical doses are satisfactory. Regularity in the performance of duties will create time for minor activities and also periods when an operator may study. Such study must be rewarded, however, otherwise spare time will be wasted or else will cease to be available because of a more indolent performance of duty.

Number of operators

The number of operators needed at treatment plants depends on many factors. Chlorinated supplies from wells may be operated by one person, responsible for operation and maintenance, who will visit the plants several times each day to check the pumps and chlorinators. Under these circumstances the motors should be equipped with an overload switch and the chlorinators should be semi-automatic, that is, they should start and stop automatically when the motors start and stop. Since the chlorine doses required for most well waters are rather constant, continuous supervision is not needed, so the operator can be assigned other duties. In fact, many small supplies of this type are supervised and operated by one operator, who reads meters and otherwise acts as superintendent of water. Part-time labour may be employed for distribution-system repairs under proper supervision, to relieve the scarce experienced personnel of this work.

Another type of plant that is simple to operate is that for zeolite softening of hard well supplies, with or without chlorination. Usually the hardness of well waters varies only on a seasonal basis, if at all, and the zeolite units may be of the automatic type, where the regeneration cycle is controlled by a water meter after a specified volume of water has been softened. Manually controlled zeolite units require the operator to be in attendance at least one hour per regeneration per unit; usually two units are used, so about three hours attendance per day is needed for effective operation of such units.

A note of caution is pertinent here. While automatic units save in labour, they require more intelligent maintenance of the equipment than the simpler, manually controlled units.

Small filtration plants treating water from impounding reservoirs are often relatively easy to supervise, because the turbidity and colour of stored waters do not fluctuate rapidly. Coagulant doses may be adjusted and filters washed every 24 hours, and a well-established routine will permit operators to devote time to other duties. All plants operated on a part-time basis, however, should be equipped with automatic shut-off switches on motors of pumps, chemical feeders and chlorinators. One common procedure, where both low-lift and high-lift pumps are used, is to control the low-lift pumps by float switches in the clear wells or filtered-water reservoirs and to control the high-lift pumps by water-pressure switches on the discharge mains; a minimum-level float shut-off in the clear well is also provided to stop the high-lift pumps if necessary. In this way the high-lift pumps are shut down automatically when the distribution-system pressure reaches a selected point, but the plants continue to operate until the clear wells are filled, at which point the pumps, chemical feeders and chlorinators are shut down automatically. One important precaution, however, is that with such control the plant must be started manually so that the operator can select the chemical doses and check performance at the start of operation each day.

Semi-automatic control of supplies flowing by gravity requires that a very reliable source of electricity be available, because water will continue to flow during periods of power failure while motor-driven chemical feeders will stop operating. This is especially important when chlorination is provided by gas chlorinators, with motor-driven pumps to furnish water under pressure for the ejectors, or when motor-driven hypochlorinators are used. Water-driven hypochlorinators are available for use where electric current is not available or is not reliable. Signal lights or bells are desirable to disclose failure of electric power, if an operator is not in constant attendance.

At more complicated treatment plants in continuous operation the minimum number of operators is considered to be four: the chief operator and three shift operators. They should be assisted as needed by maintenance mechanics and labourers.

At very large plants with associated pumping plants, where separate groups are employed in the pumping station, chemical building, filter-operating floor, laboratory, etc., the ideal is to have four operators for each group, supervised by a plant superintendent; this allows a relief for the three shift operators in each group. Several maintenance mechanics and labourers will also be needed.

Specific duties and instructions

Operating manuals are frequently complex, so simplified directions and orders should be typed and displayed at appropriate points for the guidance of shift operators. In the same way, protective maintenance programmes should be established in the form of orders, so that specified procedures will be placed on a routine orderly basis. One method is to have the various procedures pertaining to each type of equipment tabulated on check-lists, specifying the frequency of the performance of each listed activity, on which the operator inserts the time and date when the procedure was carried out. For instance, the delivery capacity of a chemical feeder might be checked every week, but the lubrication and proper functioning of the unit would be checked at each shift.

General

The basic consideration in regard to personnel is that the health of the consumers and the proper maintenance and operation of costly treatment plants rest upon the ability and adequacy of the operating staff. The functioning of treatment plants depends largely on the character of supervision and control, so investments in structures are not warranted unless the quality of the maintenance and operation provided enables them to function properly and economically.

The quality of potable water served to the public is the concern of both water and health officials. There should therefore be close co-operation between the two groups; small plants especially may need to draw on the technical resources of the health department. In this connection it may be noted here that operating personnel should be given health checks, especially to determine whether they are carriers of typhoid fever. When the raw water is heavily polluted they should be vaccinated against typhoid (also cholera when appropriate).

CHAPTER 16

OPERATION OF DISTRIBUTION SYSTEMS

Because the treatment processes discussed in this monograph have a direct bearing on the supervision and operation of distribution systems, a few related points are reviewed here.

Continuous service

The very unfortunate practice of supplying water for only a portion of each day, because of shortage of water, creates a serious menace to the health of consumers, as well as inconvenience. The zero or negative pressure produced by this practice leads to the entrance of pollution by seepage from nearby sewers, cesspools, etc., because no system is watertight, especially one that has been in use for many years. In areas of limited economic resources a choice must often be made between continuous service to a restricted number of people and intermittent service to a larger number of people. The trend is to favour the first alternative.

Cross-connexions

No direct connexion should be made between the piping of distribution systems and non-potable water supplies, sewers or industrial-plant tanks used to store polluted water, acids or the like. Blow-off drains, intended to dewater distribution systems, should terminate above the level of any nearby surface waters; they should never be connected to sewers.

Modern plumbing fixtures, with provisions against back-siphonage of sewage, are not available in many areas, but every effort should be made to encourage the enactment and enforcement of modern plumbing codes.

Dead-end mains

Many distribution systems have extensions around the periphery, usually not connected to other extensions. These dead ends prevent the free circulation of the water, which in turn imposes hydraulic restrictions to the flow of water. Without free circulation of water, residual chlorine cannot be maintained throughout a system and conditions are favourable for the reduction of sulfates to taste-producing sulfides and for corrosion of the mains. The policy should therefore be to avoid dead-end mains and to make a practice of flushing all such existing mains through hydrants at routine intervals.

Open reservoirs and tanks

It is not economically feasible to cover many equalizing reservoirs, because of their size, but tanks should be covered to prevent the entrance of dust, birds, and also sunlight, the latter to prevent the growth of algae. Many covered tanks have ventilating openings. These are not needed, because there is no significant aeration of water in quiescent storage, and in any case no covered tank is airtight.

The only exception is when ground water is aerated by spraying it into the air above the maximum water level of a tank, when air circulation is necessary. This method of aeration is attractive because it avoids double-pumping. Openings for aeration purposes should be screened to exclude birds and shaded to minimize the intensity of the light reaching the water.

Maintaining quality of water in distribution systems

The quality of water reaching the homes of consumers depends on the supervision, maintenance, operation and laboratory control of the whole supply, including the distribution system. Samples of water must be collected at appropriate points on a system to disclose the physical, chemical and bacteriological quality and hence the effectiveness of treatment. Samples collected at well-selected sampling points will disclose any localized deterioration in quality, due, for instance, to secondary pollution through cross-connexions, or to backflow from tanks and reservoirs, or to biochemical changes in the water standing in dead-end mains.

New or repaired mains should be chlorinated, including the packing material used at joints. The inner surface of piping exposed during repairs should be inspected to check the effectiveness of corrosion-prevention measures and the presence of slime organisms, or iron and manganese deposits, or after-precipitation of alum floc in the mains following the passage of improperly flocculated alum through the filters.

In summary, the distribution system is the testing ground for the effectiveness of all the supervision, operation and control being exercised from the source of supply to the consumer's tap.

ANNEXES

Annex 1

COMPUTATION OF SOLUTIONS AND DOSES

Strength of solutions

Recommended strength of solution for application to water supplies is given in grams of chemical per litre of solution (or lb/gal), i.e., the weight of chemical in a specified volume of solution. If a solution of 50 g/l (or say 0.5 lb/gal) is used, it is obvious almost without computation that 5 kg would be added to sufficient water to make up 100 l (or, say, 50 lb made up to 100 gal) of solution. Then if the chemical dose is to be 15 kg/h, 15/5 × 100, or 300 l of the solution would be added per hour; for 15 lb/h, the amount would be 15/50 × 100, or 30 gal.

Contrast this with the complexity introduced when strength of solution is given as a percentage. Suppose a 5% solution is to be used. This is not a solution of 5 parts chemical to 100 parts water, by weight, but rather 5 parts of chemical to 95 parts of water to give 100 parts (by weight) of solution. Were the procedure followed of adding 100 parts of water, the solution would have a strength of 5/(100 + 5), or 0.0476 or 4.76%. Even a correctly made 5% solution would contain an awkward amount (about 52.7 g/l) of chemical in solution.

For these reasons the strength of solution usually is not given as a percentage in this monograph. If, however, it should be necessary to prepare a solution of a certain strength given in percentage, the simplest way is to subtract the weight of chemical from 100 units of weight and to add that weight of water. For example, a 7% solution requires 7 kg (or lb) chemical, and 100 − 7, or 93 kg (or lb) water. To make up this solution 93 litres (93 kg) of water would be added for each 7 kg of chemical (93/10 = 9.3 UK gal/7 lb; 93/8.34 = 11.1 US gal/7 lb).

The formula for computing per cent strength of solution is as follows:

$$\text{Per cent strength of solution} = 100 \times \frac{\text{wt. of chemical}}{\text{wt. of chemical} + \text{wt. of water}}$$

Table 25 shows the weight of chemical per unit volume of solution for three commonly used waterworks chemicals.

TABLE 25

WEIGHT OF CHEMICAL PER UNIT VOLUME FOR VARIOUS PER CENT SOLUTIONS

% solution	Alum ($Al_2(SO_4)_3$)			Soda ash (Na_2CO_3)			Salt (NaCl)			Calcium chloride ($CaCl_2$)		
	g/l	lb/gal UK	lb/gal US	g/l	lb/gal UK	lb/gal US	g/l	lb/gal UK	lb/gal US	g/l	lb/gal UK	lb/gal US
1	10.09	0.1009	0.0842	10.09	0.1009	0.0842	10.05	0.1005	0.0839	10.06	0.1006	0.0840
2	20.39	0.2039	0.1702	20.38	0.2038	0.1701	20.25	0.2025	0.1690	20.30	0.2030	0.1694
4	41.62	0.4162	0.3473	41.59	0.4159	0.3471	41.07	0.4107	0.3428	41.26	0.4126	0.3444
6	63.71	0.6371	0.5317	63.64	0.6364	0.5311	62.48	0.6248	0.5214	62.92	0.6292	0.5251
8	86.70	0.8670	0.7235	86.53	0.8653	0.7221	84.47	0.8447	0.7049	85.27	0.8527	0.7116
10	110.6	1.106	0.9232	110.3	1.103	0.9204	107.1	1.071	0.8935	108.4	1.084	0.9042
12	135.5	1.355	1.131	134.9	1.349	1.126	130.3	1.303	1.087	132.2	1.322	1.103
14	161.4	1.614	1.347	160.5	1.605	1.339	154.1	1.541	1.286	156.8	1.568	1.308
16	188.3	1.883	1.572				178.6	1.786	1.490	182.2	1.822	1.520

Mixing of solutions of the same chemical but of different strengths is simple when strength is expressed as unit weight per unit volume. For instance, suppose that a 50-l tank contains 20 l solution with a strength of 100 g/l and it is desired to prepare 50 l solution containing 200 g/l. Then:

50 × 200 = 10 000 g = 10 kg needed
20 × 100 = 2 000 g = 2 kg available
8 kg to be added

Then 8 kg chemical would be added to the tank and enough water to fill it.

When solutions are expressed as per cent strength, the same method of total weights is used, but the weight of the chemical must first be computed from the per cent strength. For example, a 100-l tank contains 15 litres 3% solution of Na_2CO_3 and is to be filled with a 4% solution. Then, using the values shown in Table 25, the computation would be as follows:

4% solution = 41.59 g/l
3% solution (by interpolation) = 30.98 g/l
41.6 × 100 = 4160 g needed
31.0 × 15 = 465 g available
3695 g to be added

Therefore 3695 g, or 3.7 kg, sodium carbonate would be added to the 15 l in the tank, and sufficient water to make 100 l when the chemical is dissolved.

Chemical doses

The following are convenient formulae for computing chemical doses:

1. $$\text{Pounds per million gallons} = \frac{\text{lb of chemical}}{\text{millions of gallons of water}}$$

2. $$\begin{aligned}\text{Parts per million} &= \frac{\text{grams of chemical}}{\text{cubic meters of water}}\\ &= \frac{\text{lb of chemical}}{\text{millions of pounds of water}}\\ &= \frac{\text{lb of chemical}}{10 \times \text{millions of UK gallons of water}}\\ &= \frac{\text{lb of chemical}}{8.34 \times \text{millions of US gallons of water}}\end{aligned}$$

3. $$\begin{aligned}\text{Wt. of chemical per day} &= \frac{\text{p.p.m.} \times \text{wt. of water treated in 1 day}}{\text{1 million}}\\ \text{grams} &= \text{p.p.m.} \times \text{m}^3 \text{ treated in 1 day}\\ \text{pounds} &= \frac{\text{p.p.m.} \times \text{UK gal treated in 1 day} \times 10}{\text{1 million}}\\ &= \frac{\text{p.p.m.} \times \text{US gal treated in 1 day} \times 8.34}{\text{1 million}}\end{aligned}$$

Examples :

When 340 lb chemical is added to 1.5 million US gal water, the dose is (by formula 2) 340/(1.5 × 8.34) = 27.2 p.p.m.

A dose of 4.2 p.p.m. for 0.78 m.g.d. US (780 000 US gal/24 h) requires (by formula 3) the application of 4.2 × 780 000 × 8.34/1 000 000 = 27.3 lb chemical per 24 h.

Annex 2

PROPERTIES OF CHEMICALS USED IN WATER TREATMENT

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft³	Solubility, lb/gal US	Commercial strength	Characteristics
Activated carbon C	Aqua Nuchar, Hydrodarco, Norite	bags, bulk	dry: iron, steel; wet: rubber, silicon iron, stainless steel	black granular powder	15	insoluble (suspension used)		
Alum (see Aluminium sulfate)								
Aluminium sulfate $Al_2(SO_4)_3 14H_2O$	alum, filter alum, sulfate of alumina	100–200-lb bags, 300–400-lb barrels, bulk (carloads)	dry: iron, steel; solution: lead, rubber, silicon iron-asphalt	ivory-coloured powder granule lump	 38-45 60-63 62-67	4.2 (60°F)	15-22 % Al_2O_3	pH of 1 % solution: 3.4
Ammonium aluminium sulfate $Al_2(SO_4)_3(NH_4)_2 \cdot SO_4 \cdot 24H_2O$	ammonia alum, crystal alum	bags, barrels, bulk	Duriron, lead, rubber, silicon iron, stoneware	lump nut pea powdered	64-68 62 65 60	0.3 (32°F); 8.3 (212°F)	11 % Al_2O_3	pH of 1 % solution: 3.5
Anhydrous ammonia NH_3	ammonia	50-, 100-, 150-lb cylinders	glass, iron, Monel metal, nickel, steel	colourless gas		3.9 (32°F); 3.1 (60°F); 1.8 (125°F)	99 %-100 % NH_3	
Aqua ammonia NH_4OH	*ammonia water*, ammonium hydrate, ammonium hydroxide	carboys, 750-lb drums, 8000-gal tank cars	glass, *iron*, Monel metal, nickel, steel	colourless liquid		completely soluble	29.4 % NH_4 (26°Baumé)	
Bentonite	colloidal clay	100-lb bags	iron, steel	powder, pellet, mixed sizes	60	insoluble (colloidal sol. used)		
Calcite (see Calcium carbonate)								
Calcium carbonate $CaCO_3$	marble	bags or bulk	iron, steel	powder or mixed sizes				
Calcium hydroxide $Ca(OH)_2$	hydrated lime, slaked lime	50-lb bags, 100-lb barrels, bulk (carloads)	asphalt, cement, iron, rubber, steel	white powder: light dense	 26-48 40-70	0.014 (63°F); 0.012 (90°F)	85 %-99 % $Ca(OH)_2$, 63 %-73 % CaO	Hopper agitation required for dry-feeding of light form

CHEMICALS USED IN WATER TREATMENT *(continued)*

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft³	Solubility, lb/gal US	Commercial strength	Characteristics
Calcium hypochlorite $Ca(OCl)_2.4H_2O$	HTH, Perchloron, Pittchlor	5-lb cans; 100-, 300-, 800-lb drums	glass, rubber, stoneware, wood	white granule, powder	52.5		70 % available Cl_2	1 %-3 % available Cl_2 solution used
Calcium oxide CaO	burnt lime, chemical lime, quicklime, unslaked lime	50-lb bags, 100-lb barrels, bulk (carloads)	asphalt, cement, iron, rubber, steel	lump pebble granule	50-65 60-65 50-70	Slakes to form hydrated lime	75 %-99 % CaO	pH of saturated solution: 12.4; detention time, temperature and amount of water critical for efficient slaking
Chlorinated lime $CaO.2CaOCl_2.3H_2O$	bleaching powder, chloride of lime	100-, 300-, 800-lb drums	glass, rubber, stoneware, wood	white powder	48	Clorine portion soluble	25 %-37 % available Cl_2	Deteriorates
Chlorine Cl_2	chlorine gas, liquid chlorine	100-, 150-lb cylinders; 1-ton tanks; 16-, 30-ton tank cars	dry: black iron, copper, steel; wet, gas: glass, hard rubber, silver	liquefied gas under pressure	91.7	0.07 (60°F); 0.04 (100°F)	99.8 %Cl_2	
Chlorine dioxide ClO_2	chlorine dioxide	Generated as used	plastic, soft rubber (avoid hard rubber)	yellow-red gas		0.02	26.3 % available Cl_2	
Copper sulfate $CuSO_4.5H_2O$	blue vitriol, bluestone	100-lb bags, 450-lb barrels, drums	asphalt, silicon iron, stainless steel	crystal lump powder	75-90 73-80 60-64	1.6 (32°F); 2.2 (68°F); 2.6 (86°F)	99 % $CuSO_4$	
Disodium phosphate $Na_2HPO_4.12H_2O$	basic sodium phosphate, DSP, secondary sodium phosphate	125-lb kegs, 200-lb bags, 325-lb barrels	cast iron, steel	crystal	60-64	0.4 (32°F); 6.4 (86°F)	19 %-19.5 % P_2O_5	Precipitates Ca, Mg; pH of 1 % solution: 9.1
Ferric chloride			glass, rubber, stoneware, synthetic resines					
(*a*) $FeCl_3$ solution	Ferrichlor, chloride of iron	5-, 13-gal carboys, trucks,tank,cars		dark-brown syrupy liquid		completely soluble	35 %-45 % $FeCl_3$, 12 %-17 % Fe	
(*b*) $FeCl_3.6H_2O$	crystal ferric chloride	300-lb barrels		yellow-brown lump	60-64	5.0 (98°F) 4.0 (86°F)	59 %-61 % $FeCl_3$, 20 %-21 % Fe	Hygroscopic (store lumps and powder in tight containers); no dry feed; optimum pH: 4-11
(c) $FeCl_3$	anhydrous ferric chloride	500-lb casks; 100-, 300-, 400-lb kegs		green-black powder	85-90		98 % $FeCl_3$, 34 % Fe	

Ferric sulfate $Fe_2(SO_4)_3 \cdot 9H_2O$	Ferrifloc, Ferrisul	100–175-lb bags, 400–425-lb drums	ceramics, lead, plastic, rubber, 18-8 stainless steel	red-brown powder or granule	70-72	Soluble in 2-4 parts cold water	90 %-94 % $Fe_2(SO_4)_3$, 25%-26 % Fe	Mildly hygroscopic; coagulant at pH 3.5-11.0
Ferrous sulfate $FeSO_4 . 7H_2O$	copperas, green vitriol	bags, barrels, bulk	asphalt, concrete, lead, tin, wood	green crystal, granule, lump	63-66	4.0 (68°F)	55 % $FeSO_4$, 20 % Fe	Hygroscopic; cakes in storage; optimum pH: 8.5-11.0
Fluosilicic acid H_2SiF_6	fluosilicic acid	rubber-lined drums	rubber-lined steel	liquid		approx. 1.2 (68°F)	approx. 35 %	
Lime (see Calcium hydroxide and Calcium oxide)								
Potassium aluminium sulfate $K_2SO_4 . Al_2(SO_4)_3 . 24H_2O$	potash alum	bags, barrels, bulk (carloads)	lead, lead-lined rubber, stoneware	lump granule powder	62-67 60-65 60	0.5 (32°F); 1.0 (68°F); 1.4 (86°F)	10 %-11 % Al_2O_3	Low, even solubility; pH of 1 % solution: 3.5
Soda ash (see Sodium carbonate)								
Sodium aluminate $Na_2O . Al_2O_3$	soda alum	100–150-lb bags, 250–440-lb drums, solution	iron, plastic, rubber, steel	white or greenish-yellow crystal liquid (27° Baumé)	50-60	3.0 (68°F); 3.3 (86°F)	55 % Al_2O_3, 35 % Na_2O, 5 % excess NaOH 32 % $Al_2O_3Na_2O$, 10 % NaOH	Hopper agitation required for dry feed
Sodium carbonate Na_2CO_3	soda ash	bags, barrels, bulk (carloads)	iron, rubber, steel	white powder: extra light light dense	23 35 65	1.5 (68°F); 2.3 (86°F)	99.4 % Na_2CO_3, 58 % Na_2O	Hopper agitation required for dry feed of light and extra-light forms; pH of 1 % solution: 11.2
Sodium chloride NaCl	common salt	bags, barrels, carloads	bronze, cement, rubber	rock fine grains	50-60 58-70	2.9 (32°F); 3.0 (68°-86°F)	98 % NaCl	
Sodium chlorite $NaClO_2$	technical sodium chlorite	100-lb drums	metals (avoid cellulose materials)	light-orange powder, flake	70	2.0 (68° F)	82 % $NaClO_2$, 30 % available Cl_2	Generates ClO_2 at pH 3.0

CHEMICALS USED IN WATER TREATMENT *(concluded)*

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft³	Solubility, lb/gal US	Commercial strength	Characteristics
Sodium fluoride NaF	fluoride	bags, barrels, fibre drums, kegs	iron, lead, steel	Nile blue or white powder: light dense	50 75	0.35 (most temperatures)	90 %-95 % NaF	pH of 4 % solution: 6.6
Sodium hexametaphosphate $(NaPO_3)_6$	Calgon, glassy phosphate, vitreous phosphate	100-lb bags	hard rubber, plastic, stainless steel	crystal, flake, powder	47	1-4.2	66 % P_2O_5 (unadjusted)	pH of 0.25 % solution: 6.0-8.3
Sodium hydroxide $NaOH$	caustic soda, soda lye	100–700-lb drums; bulk (trucks, tank cars)	cast iron, rubber, steel	flake, lump, liquid		2.4 (32°F); 4.4 (68°F); 4.8 (104°F)	98.9 % $NaOH$, 74 %-76 % NaO_2	Solid hygroscopic; pH of 1 % solution: 12.9
Sodium hypochlorite $NaOCl$	sodium hypochlorite	5-, 13-gal carboys; 1300–2000-gal tank trucks	ceramic, glass, plastic, rubber	light-yellow liquid			12 %-15 % available Cl_2	
Sodium silicofluoride Na_2SiF_6	sodium silicofluoride	bags, barrels, fibre drums	iron, lead, steel	Nile blue or yellowish-white powder	72	0.03 (32°F); 0.06 (72°); 0.12 (140°)	99 % Na_2SiF_6	pH of 1 % solution: 3.5
Sodium thiosulfate $Na_2S_2O_3.5H_2O$	hypo	bags, barrels, drums, kegs	cast iron, low-carbon steel, stoneware	standard crystal rice fine granule	64-65 65 70 61	6.2 (32°F)	98 %-99 % $Na_2S_2O_3$	
Sulfur dioxide SO_2	sulfur dioxide	100-, 150,-200-lb cylinders	dry: steel	colourless gas			100 % SO_2	
Trisodium phosphate $Na_3PO_4.12H_2O$	normal sodium phosphate, tertiary sodium phosphate, TSP	125-lb kegs, 200-lb bags, 325-lb barrels	cast iron, steel	crystal: coarse medium standard	56 58 61	0.1 (32°F); 13.0 (158°)F	19 % P_2O_5	pH of 1 % solution: 11.9

Annex 3

CONVERSION TABLES

Chemical doses

(Equivalents valid for dilute solutions whose density is very close to 1)

French degree . . . . = 1 part per 100 000 as $CaCO_3$
= 10 mg/l as $CaCO_3$
= $^{1}/_{5}$ milli-equivalent (or 1/5000 equivalent)
Milli-equivalent . . . . . . = 5 French degrees
Milligram per litre (mg/l) . . = 1 part per million (p.p.m.)
Gram per cubic metre (g/m³) = 1 p.p.m.

Conversion of fractions to decimal values

$^{1}/_{32} = 0.03125$
$^{1}/_{16} = 0.06250$
$^{1}/_{12} = 0.08333$
$^{3}/_{32} = 0.09375$
$^{1}/_{9} = 0.11111$
$^{1}/_{8} = 0.12500$
$^{5}/_{32} = 0.15625$
$^{1}/_{6} = 0.16667$
$^{3}/_{16} = 0.18750$
$^{7}/_{32} = 0.21875$
$^{2}/_{9} = 0.22222$
$^{1}/_{4} = 0.25000$
$^{9}/_{32} = 0.28125$
$^{5}/_{16} = 0.31250$
$^{1}/_{3} = 0.33333$
$^{11}/_{32} = 0.34375$
$^{3}/_{8} = 0.37500$
$^{13}/_{32} = 0.40625$
$^{5}/_{12} = 0.41667$
$^{7}/_{16} = 0.43750$
$^{4}/_{9} = 0.44444$
$^{15}/_{32} = 0.46875$
$^{1}/_{2} = 0.50000$
$^{17}/_{32} = 0.53125$
$^{5}/_{9} = 0.55556$
$^{9}/_{16} = 0.56250$
$^{7}/_{12} = 0.58333$
$^{19}/_{32} = 0.59375$
$^{5}/_{8} = 0.62500$
$^{21}/_{32} = 0.65625$
$^{2}/_{3} = 0.66667$
$^{11}/_{16} = 0.68750$
$^{23}/_{32} = 0.71875$
$^{3}/_{4} = 0.75000$
$^{7}/_{9} = 0.77778$
$^{25}/_{32} = 0.78125$
$^{13}/_{16} = 0.81250$
$^{5}/_{6} = 0.83333$
$^{27}/_{32} = 0.84375$
$^{7}/_{8} = 0.87500$
$^{29}/_{32} = 0.90625$
$^{11}/_{12} = 0.91667$
$^{15}/_{16} = 0.93750$
$^{31}/_{32} = 0.96875$

Units of length

Symbol			
in	inch	=	0.0254 m
ft	foot (12 in)	=	0.3048 m
yd	yard (3 ft or 36 in)	=	0.9144 m

Symbol		
—	statute mile (1760 yd)	= 1609.344 m = 1.6 km
μ or μm	micron or micrometre = 0.000001 m (10^{-6} m)	= 0.000039 in
mm	millimetre = 0.001 m	= 0.039 in
cm	centimetre = 0.01 m	= 0.39 in
dm	decimetre = 0.1 m	= 3.9 in
m	metre	= 1.093 yd 39.37 in 3.281 ft 3 ft 3 $^3/_8$ in
km	kilometre = 1000 m	= 0.621 statute mile = 0.539 nautical mile

Units of area

in^2	square inch	= 6.4516 cm^2
ft^2	square foot (144 in^2)	= 0.0929 m^2
yd^2	square yard (9 ft^2)	= 0.83613 m^2
sq mile	square mile (640 acres)	= 2.5899 km^2
—	acre (4840 yd^2)	= 4046.8 m^2
cm^2	square centimetre = 0.0001 m^2	= 0.155 in^2
dm^2	square decimetre = 0.01 m^2	= 0.1076 ft^2
m^2	square metre or centare = 1 m^2	= 1.196 yd^2
hm^2, ha	square hectometre or hectare = 10 000 m^2	= 2.471 acres
km^2	square kilometre = 1 000 000 m^2	= 0.3861 sq miles

Units of mass and weight

—	grain	= 0.065 g
oz	ounce	= 28.349 g
lb	pound (16 oz)	= 453.592 g
—	stone (14 lb)	= 6.350 kg
—	quarter (2 stone)	= 12.70 kg
cwt	hundredweight (UK) (112 lb)	= 50.802 kg
sh cwt	short hundredweight (US) (100 lb)	= 45.359 kg

Symbol			
ton	long ton (UK) (2240 lb)	=	1.016 t
sh tn	short ton (US) (2000 lb)	=	0.907 t
mg	milligram = 0.001 g	=	0.0154 grain
g	gram	=	15.432 grain
kg	kilogram = 1 000 g	=	35.2736 oz
		=	2.2046 lb
t	metric tonne = 1 000 000 g	=	19.6841 cwt (UK) 22.0462 sh cwt 1.1023 sh ton 0.9842 ton (UK)

Units of volume and capacity

in^3	cubic inch	=	16.387 cm^3
pt	pint (UK) (4 gills)	=	0.5682 l
liq. pt.	liquid pint (US) (4 gills)	=	0.4732 l
qt	quart (UK) (2 pt)	=	1.1364 l
liq. qt.	liquid quart (US) (2 liq. pt.)	=	0.9464 l
gal (UK)	UK (Imperial) gallon (8 UK pt)	=	4.54596 l
		=	1.2 gal (US)
gal (US)	US gallon (8 US liq. pt.)	=	3.78533 l
		=	0.833 gal (UK)
bu. (US)	US bushel (4 pecks)	=	35.2383 l
ft^3	cubic foot	=	28.3161 l
yd^3	cubic yard	=	0.7645 m^3
cm^3 ml	cubic centimetre[a] = 0.000001 m^3 or millilitre[a] = 0.001 litre	=	0.06102 in^3
l	litre = 0.001 m^3	=	0.220 gal (UK) 0.264 gal (US) 0.03531 ft^3 1.7597 pt (UK) 2.113 liq. pt. (US)
m^3	cubic metre	=	1.3079 yd^3

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[a] Metric units of length, and hence cubic centimetres for volume, are based on a temperature of 0°C, whereas units of capacity such as the litre and millilitre are based on a temperature of 4°C, the point at which water has its maximum density. Therefore 1 ml actually equals 1.000027 cm^3. For practical purposes this distinction may be disregarded.

Units of rate of flow and of filtration

Symbol		
ft^3/s	cubic foot per second	= 28.315 l/s
		= 101.934 m^3/h
gal/min (US)	US gallon per minute	= 227 l/h
		= 0.227 m^3/h
		= 0.0625 l/s
gal/min (UK)	UK (Imperial) gallon per minute	= 272.758 l/h
		= 0.2727 m^3/h
		= 0.07576 l/s
ft^3/min	cubic foot per minute	= 0.4719 l/s
		= 1.6989 m^3/h
l/s	litre per second	= 0.03531 ft^3/s
		= 2.119 ft^3/min
l/h	litre per hour	= 0.03531 ft^3/h
m^3/h	cubic metre per hour	= 0.589 ft^3/min
		= 3.667 UK gal/min
m.g.d (UK)	million UK gallons per day . .	= 189.4 m^3/h
		= 52.6 l/s
m.g.d (US)	million US gallons per day . .	= 157.7 m^3/h
		= 43.8 l/s
gal/ft^2/min (US)	gallon per square foot per minute (US)	= 2.421 m^3/m^2/h
gal/ft^2/min (UK)	gallon per square foot per minute (UK)	= 2.934 m^3/m^2/h
ft^3/ft^2/min	cubic foot per square foot per minute	= 18.29 m^3/m^2/h
		= 0.340 UK gal/ft^2/min
m^3/m^2/h m/h	cubic metre per square metre per hour or linear speed of 1 metre per hour	= 0.413 US gal/ft^2/min 0.621 mile/h 0.0546 ft^3/ft^2/min

Units of pressure

lbf/in^2	pound-force per square inch . . .	= 0.0703 kgf/cm^2
		= 0.068 atm
lbf/ft^2	pound-force per square foot . . .	= 4.882 kgf/m^2
tonf/in^2	ton-force per square inch (UK) . .	= 1.575 kgf/mm^2
		= 157.5 kgf/cm^2
kgf/cm^2	kilogram-force per square centimetre	= 14.223 lbf/in^2
		= 0.9678 atm

Symbol			
kgf/m^2	kilogram-force per square metre . .	=	0.2049 lbf/ft^2
atm	atmosphere	=	1.033 kgf/cm^2
		=	14.696 lbf/in^2

Units of specific weight and concentration

lb/yd	pound per yard	=	0.496055 kg/m
lb/ft	pound per foot	=	1.48816 kg/m
lb/in^3	pound per cubic inch	=	27.6799 g/cm^3
lb/ft^3	pound per cubic foot	=	0.016019 kg/l
		=	16.085 kg/cm^3
grain/gal (UK)	grain per UK gallon	=	14.2542 mg/l
grain/gal (US)	grain per US gallon	=	17.1184 mg/l
lb/gal (UK)	pound per UK gallon	=	99.7 g/l
lb/gal (US)	pound per US gallon	=	119.0 g/l
grain/in^3	grain per cubic inch	=	3.96 g/l
grain/ft^3	grain per cubic foot	=	2.296 mg/l
kg/m	kilogram per metre	=	0.672 lb/ft
		=	2.016 lb/yd
g/cm^3	gram per cubic centimetre . .	=	0.036127 lb/in^3
mg/l	milligram per litre	=	0.0702 grain/gal(UK)
		=	0.0584 grain/gal(US)
		=	0.4356 grain/ft^3
g/l	gram per litre	=	0.253 grain/in^3 435.6 grains/ft 70.2 grains/gal (UK) 58.4 grains/gal (US)

Concentrations

1 per cent (%) × density of the solution . . . = 10 g/l
1 per thousand ($1^0/_{00}$) × density of the solution = 1 g/l
1 part per million (p.p.m.) × density of the solution = 1 mg/l

Units of linear speed

in/s	inch per second	=	91.444 m/h
ft/s	foot per second	=	1.0972 km/h
yd/s	yard per second	=	0.9144 m/s
mile/h	statute mile per hour	=	1.6 km/h
m/s	metre per second	=	3.281 ft/s
km/h	kilometre per hour	=	0.621 statute mile/h

Units of energy and heat

Symbol		
Btu	British thermal unit	= 0.252 kcal = 1.0548 kJ
Btu/ft^3	British thermal unit per cubic foot	= 8.899 kcal/m^3
kcal or kcal IT	kilocalorie or large calorie or I.T. kilocalorie (1000 small calories)	= 4.1868 kJ
kJ	kilojoule (1000 joules)	= 0.9478 Btu = 0.2388 kcal
th	thermie	= 3967 Btu
kcal/m^3 mth/m^3	kilocalorie per cubic metre, or millithermie per cubic metre	= 0.112 Btu/ft^3
kWh	kilowatt-hour	= 3412.14 Btu = 3600 kJ

Units of power

hp	horsepower (British)	= 0.7457 kW = 1.0139 ch
ft · lbf/s	1000 foot pound-force per second	= 1.35582 kW
Btu/s	British thermal unit per second	= 1.05418 kW
ch	cheval vapeur or metric horsepower	= 0.73575 kW = 0.9863 hp
kgf · m/s	kilogramme-force metre per second	= 0.0098 kW = 0.0132 hp
kW	kilowatt	= 1.341 hp = 1.36 ch

Equivalent values of metric, UK and US rates of flow

Unit	*m^3/day*	*gal/min (US)*	*gal/min (UK)*	*m^3/h*	*l/s*	*m.g.d. (US)*	*m.g.d. (UK)*
gal/min (US)	5.45	1	0.833	0.227	0.063		
gal/min (UK)	6.55	1.20	1	0.273	0.076		
m^3/h	24	4.40	3.667	1	0.278		
l/s	86.40	15.85	13.198	3.600	1		
1000 m^3/day	1 000	183.46	152.765	41.667	11.574		
m.g.d. (US)	3 785	694.44	579.72	157.726	43.811	1	
m.g.d. (UK)	4 546	834.06	694.445	189.415	52.615	1.200	1
m^3/s	86 400	15 851	13 198	3 600	1 000	22.825	19.005

Temperature

(Centigrade and Fahrenheit)

C	F	C	F
0	32	40	104
5	41	50	122
10	50	60	140
20	68	70	158
30	86	80	176
37	98	90	194
(normal body temperature)		100	212
		(boiling point)	

To compute Fahrenheit: Multiply centigrade by 1.8 (or 9/5) and add 32.

To compute centigrade: Subtract 32 from Fahrenheit and divide by 1.8 (or multiply by 5/9).

Multiples and submultiples of metric units

Abbreviation	Prefix	Multiple or submultiple
M	mega-	× million
k	kilo-	× thousand
h	hecto-	× hundred
d	deci-	one-tenth
c	centi-	one-hundredth
m	milli-	one-thousandth
μ	micro-	one-millionth

Annex 4

CHEMICAL EQUATIONS

The chemical equations given in this monograph may be used to compute weight of chemical involved through the use of atomic and combining weights. This is because the proportion of atoms of any given element or compound which enters into a reaction is constant. Furthermore, the same proportions prevail whether expressed as atomic weights or in grams, pounds, milligrams per litre, or parts per million. In fact, atomic weights are not real weights, but relative weights based on the value of 16 for oxygen. For example, water, H_2O, consists of 2 atoms of hydrogen and one atom of oxygen. The atomic weight of hydrogen is 1, that of oxygen is 16. Then $H_2 = 2 \times 1$ and $O = 16$, so $H_2O = 18$. This means that in 18 g water there are 2 g hydrogen and 16 g oxygen.

As stated in Chapter 5, compounds which separate into positive and negative portions when dissolved in water are called *electrolytes*, and the process is called *ionization*, each electrically charged portion being an *ion*. The positive ions are called *cations*, and the negative ions, *anions*. Cations will not react with one another, nor will anions react with other anions, because they have like electrical charges. Furthermore, when a cation reacts with an anion, their charges must be opposite and equal. Hence ferric chloride ionizes thus: $Fe^{+++} + 3Cl^-$; three chloride ions with a single negative charge each are required to balance the three positive charges of the ferric ion. Contrast this with sodium chloride, Na^+Cl^-, where only two single opposite charges are involved.

Many compounds have negative ions that consist of more than one element. For instance, sulfuric acid, $H_2^{++}SO_4^{--}$, consists of two positive hydrogen ions and one negative group, SO_2^{--}, having two negative charges Such groups of atoms, when ionized as a group, are called *radicals*.

Ions, then, combine with ions of opposite charge in definite proportions, depending on their electric charge. This charge, in controlling the reacting ratios of ions, is called *valence*. Thus chlorine, Cl^-, has a valence of 1, Ca^{++} and SO_4^{--} have valences of 2, and aluminium, Al^{+++}, has a valence of 3. Since an atom or radical having a valence of, say, 2 will combine with two atoms or radicals having a valence of 1, the *combining weight* of any given atom is its atomic weight divided by its valence. For example, chlorine has an atomic weight of 35.46 and a valence of 1, so its combining weight is 35.46; but oxygen, having an atomic weight of 16.0 and a valence of 2, has a combining weight of 8.0.

Oxidation and reduction

Certain elements have more than one valence, so the magnitude of reaction with such elements is measured by the increase or decrease in valence that occurs in a specific reaction. For instance, carbon, C, has valences of 2 and 4. When carbon is oxidized to carbon dioxide, CO_2, 4 valence units are involved; when it is oxidized to carbon monoxide, CO, only 2 valence units are involved. If, however, carbon monoxide is oxidized to carbon dioxide, 2 valence units are involved. The amount of oxygen entering into these reactions is dependent on the change in valence of the carbon, but the amount of carbon involved does not change.

Because reactions involving an increase in the valence of those elements with more than one valence frequently involve oxygen, the term *oxidation* is used broadly to denote those reactions in which the valence is increased. For instance, ferrous chloride is said to be oxidized by chlorine to ferric chloride ($FeCl_2 + Cl = FeCl_2$) even though no oxygen is involved, because the valence of the iron is increased from 2 to 3. Conversely, the term *reduction* is used when oxygen is removed or when a valence is reduced. Thus when insoluble ferric oxide is reduced to ferrous oxide ($2Fe_2O_3 = 4FeO + O_2$), the valence of iron is reduced from 3 to 2. Incidentally, the ferrous oxide in turn is dissolved by waters containing carbon dioxide (carbonic acid) to form the soluble ferrous bicarbonate: $FeO + 2H_2CO_3 = FeH_2(CO_3)_2 + H_2O$. These are the reactions which occur when ground waters devoid of oxygen but containing carbon dioxide come in contact with iron-bearing rocks.

Combining weights

Table 26 gives the basic data as to these properties of the common atoms and radicals encountered in water treatment.

The use of Table 26 may be illustrated by the neutralization of hydrochloric acid by sodium hydroxide.

$$\begin{array}{ccccccc}
HCl & + & NaOH & \longrightarrow & NaCl & + & H_2O \\
(1 + 35.46) & + & (23 + 16 + 1) & \longrightarrow & (23 + 35.46) & + & (2 + 16) \\
(36.46) & + & (40) & \longrightarrow & (58.46) & + & (18)
\end{array}$$

It will be noted that the weight of compounds formed equals the weight of the reacting substances, so no loss occurs. Therefore 36.46 parts hydrochloric acid by weight reacts with 40 parts sodium hydroxide to form 58.46 parts neutral sodium chloride, or salt, and 18 parts water; so 36.46 g of this acid would combine with 40 g sodium hydroxide to form 58.46 g sodium chloride and 18 g water.

TABLE 26

ATOMIC WEIGHTS, VALENCES AND COMBINING WEIGHTS OF ELEMENTS AND RADICALS INVOLVED IN WATER TREATMENT

(Only the most common valences shown)

Element or radical	Atomic symbol	Atomic weight	Valence	Combining weight
Elements :				
Aluminium	Al	26.98	+ 3	8.99
Calcium	Ca	40.08	+ 2	20.04
Carbon	C	12.01	+ 4	3.00
Chlorine	Cl	35.46	− 1	35.46
Copper	Cu	63.54	+ 2	31.77
Hydrogen	H	1.008	+ 1	1.008
Iodine	I	126.91	− 1	126.91
Iron	Fe	55.85	+ 3	18.62
Magnesium	Mg	24.32	+ 2	12.16
Manganese	Mn	54.93	Several	
Nitrogen	N	14.01	Several	
Oxygen	O	16.00	− 2	8.00
Potassium	K	39.10	+ 1	39.10
Sodium	Na	23.00	+ 1	23.00
Sulfur	S	32.06	Several	
Radicals :				
Bicarbonate	(HCO_3)		− 1	61.01
Carbonate	(CO_3)		− 2	30.00
Hydroxyl	(OH)		− 1	17.00
Nitrate	(NO_3)		− 1	62.01
Nitrite	(NO_2)		− 1	46.01
Phosphate	(PO_4)		− 3	31.66
Silicate	(SiO_3)		− 2	38.05
Sulfate	(SO_4)		− 2	48.03

If, however, sulfuric acid, for example, is involved, the equation is:

$$H_2SO_4 + 2NaOH \longrightarrow Na_2SO_4 + 2\ H_2O$$

It will be noted here that the sulfate radical or ion, having a valence of 2, combines with two sodium atoms having a valence of 1. This means that twice as much sodium hydroxide must be used to neutralize each molecule of sulfuric acid, as compared with hydrochloric acid. As it is desirable to use a valence of 1 as the unit for comparison, the above formula is computed as follows:

$$^1/_2\,H_2SO_4 + NaOH \longrightarrow {}^1/_2\,Na_2SO_4 + H_2O$$

In other words the combining weight of sulfuric acid is its molecular weight divided by the valence of 2 of the sulfate radical, SO_4, or

$$\frac{2 + 32.06 + 64}{2} = \frac{98.06}{2} = 49.03$$

That is, 49.03 parts sulfuric acid will neutralize 40 parts sodium hydroxide.

In the same way, the weights of chemicals used in water treatment may be computed. For example, the precipitation of calcium bicarbonate, or carbonate hardness, by hydrated lime is as follows:

$$\begin{array}{ccccccc} Ca(HCO_3)2 & + & Ca(OH)_2 & = & 2CaCO_3 & + & 2H_2O \\ 162 & + & 74 & = & 200 & + & 36 \end{array}$$

Therefore 74 parts pure calcium hydroxide are required to precipitate 162 parts calcium bicarbonate. The same ratio may be expressed in terms of combining weights: 74/2 = 37, and 162/2 = 81, or 37 to 81.

The general procedure is to use the ratio of combining weights of chemicals in computing the quantities of reacting substances. As an example, the combining weight of calcium bicarbonate, 81, is equivalent to the value of 50 for calcium carbonate. Since hardness and alkalinities are in terms of calcium carbonate, the above ratio becomes: 37 parts pure calcium hydroxide to 50 parts alkalinity.

The combining weights may be expressed in terms of grams, pounds, milligrams per litre, or parts per million. For instance, 37 p.p.m. pure calcium hydroxide will precipitate 81 p.p.m. calcium bicarbonate, expressed as 50 p.p.m. hardness or alkalinity computed as calcium carbonate.

The ratio of combining weight to weight of reacting chemicals may be expressed as follows:

$$\begin{array}{l} \text{Combining weight} \\ \text{of substance } a \end{array} : \begin{array}{l} \text{combining weight} \\ \text{of substance } b \end{array} :: x : 100$$

Then

$$x = \frac{a}{b}$$

The factor x may be recorded and used thereafter for the specific reacting substance in the *pure state*. For example, the reaction between calcium hydroxide and calcium hardness would be computed as follows:

$$x = \frac{37}{50} = 0.74$$

Therefore the dose of pure calcium hydroxide needed to precipitate, say,

TABLE 27

MOLECULAR WEIGHTS AND COMBINING WEIGHTS OF PRINCIPAL COMPOUNDS INVOLVED IN WATER TREATMENT

Substance	Formula	Molecular weight	Combining weight
Aluminium hydroxide	$Al(OH)_3$	78	26
Aluminium sulfate	$Al_2(SO_4)_3$	342	57
Calcium bicarbonate	$Ca(HCO_3)_2$	162	81
Calcium carbonate	$CaCO_3$	100	50
Calcium hydroxide	$Ca(OH)_2$	74	37
Calcium oxide	CaO	56	28
Calcium sulfate	$CaSO_4$	136	68
Ferric chloride	$FeCl_3$	162.5	54.2
Ferric hydroxide	$Fe(OH)_3$	106.84	35.61
Ferric oxide	Fe_2O_3	159.70	26.61
Ferric sulfate	$Fe_2(SO_4)_3$	400	66.6
Ferrous oxide	FeO	71.84	35.92
Ferrous sulfate	$FeSO_4$	152	76
Hydrochloric acid	HCl	36.5	36.5
Magnesium bicarbonate	$Mg(HCO_3)_2$	146	73
Magnesium carbonate	$MgCO_3$	84	42
Magnesium hydroxide	$Mg(OH)_2$	58	29
Magnesium sulfate	$MgSO_4$	120	60
Sodium bicarbonate	$NaHCO_3$	84	84
Sodium carbonate	Na_2CO_3	106	53
Sodium chloride	$NaCl$	58.5	58.5
Sodium hydroxide	$NaOH$	40	40
Sodium silicate	Na_2SiO_3	122	61
Sodium sulfate	Na_2SO_4	142	71
Sulfuric acid	H_2SO_4	98	49

185 p.p.m. calcium hardness would be 185 × 0.74, or 137 p.p.m. The dose of 95% hydrated lime required would be 137/0.95, or 144 p.p.m.

Combining weights are shown in Table 27 for the principal compounds involved in water treatment.

Standard solutions

The use of the combining weights of atoms or radicals in the laboratory is made convenient by the preparation of standard, or normal, solutions of the various reagents. Such a solution is prepared by adding to 1 litre of distilled water the weight in grams of the substance equal to its combining weight. This is called the *gram-equivalent weight*, because it is equivalent in chemical activity to 1 g hydrogen in 1 litre, the latter having the unit value of 1.

$$\text{Gram-equivalent weight} = \frac{\text{molecular weight}}{\text{valence}}$$

A standard solution of sulfuric acid, for example, would be prepared by adding its gram-equivalent weight (98/2 = 49 g) to 1 litre distilled water. This is normal, or N, H_2SO_4 solution. The standard solutions used in the analysis of water are usually diluted to $^1/_{50}$ their normal strength, so as to secure greater accuracy. These diluted solutions are expressed as N/50 or 0.02N.

Since all gram-equivalent weights of reacting substances have the equivalent values, it is possible to measure the unknown concentration of a reacting substance by the known amount of another substance with which it reacts. Thus the unknown alkalinity of a water sample is measured by adding 0.002N H_2SO_4 to the point where the alkalinity is neutralized by the acid, as shown by the methyl orange indicator. As the 0.02N acid solution has a definite weight of acid per unit of volume, measuring the volume with a burette means that a weight is being measured: each millilitre of 0.02N sulfuric acid will neutralize 1 mg calcium carbonate. Since 1 mg per litre is the same as 1 p.p.m., and 1 mg per 100 ml is equal to 10 p.p.m., 1 ml of 0.02N sulfuric acid will neutralize 10 p.p.m. alkalinity in a 100-ml portion of a sample. Thus the alkalinity of a sample is computed by multiplying by 10 the amount of 0.02N acid used when 100 ml of the sample is titrated. This is the origin of the factor of 10 mentioned in that portion of the Supplement dealing with the test for alkalinity.

SUPPLEMENT

LABORATORY PROCEDURES FOR WATER TREATMENT PLANT OPERATORS

LABORATORY PROCEDURES FOR WATER TREATMENT PLANT OPERATORS[1]

Introduction

Laboratory examinations of water are probably most frequently undertaken to aid in forming an opinion of the suitability of a water supply for public use. This involves consideration of several factors: whether the water is safe for human consumption as shown by the presence or absence of pollution, whether it is corrosive to metal pipes or will form scale on cold- or hot-water systems, whether it is of attractive appearance and taste, whether it is satisfactory for use in home laundry and dishwashing, and whether it can be used for industrial purposes. Regular laboratory analyses are essential for the control of water treatment processes and to ensure a satisfactory effluent at all times. The various tests are tools that supplement and extend the human senses. Thus the gradual deterioration of coagulation and filtration processes can be detected by laboratory measurements before it becomes evident to visual observation: microscopic organisms can be magnified and counted, variation in dissolved constituents of water can be detected and the reasons for changes in plant operation discovered so that corrective measures can be adopted. Unless the significance of the tests is well understood, however, they are of little value to operators. The results of these tests should be interpreted in the light of sanitary surveys of the sources of water supply, and conversely, used for additional information as to the character of the water being treated. Many of the tests are small-scale replicas of plant operation by means of which chemical doses or methods of treatment can be tried before application in the treatment plant. Only those tests concerned with the control of water-treatment processes are here considered.

Laboratory examinations may be divided into physical, chemical, bacteriologic, and microscopic. The physical tests measure and record the properties detectable by the senses. Chemical analyses determine the amounts of mineral and organic matters present that affect the quality of the water; provide data as to past sewage pollution; and show variations due to treatment—a necessary requirement in the control of water treatment processes. Bacteriologic examinations indicate the presence of bac-

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[1] This Supplement is adapted and expanded from Cox (1958).

teria characteristic of pollution and hence the safety of the water for consumption. Microscopic examinations provide information concerning growths in water that are often the cause of disagreeable tastes and odours or clogging of filters.

International Standards for Drinking-Water (World Health Organization) and *Standard Methods for the Examination of Water and Waste-Water* (American Public Health Association) are generally accepted guides when laboratory facilities are available to chemists. It is urged that these standard procedures be followed when facilities and trained personnel are available, although portable kits may be used by shift operators for routine tests such as residual chlorine and pH value. Many kits are available in various countries for these two tests, with detailed directions for their use, as well as other kits that are more comprehensive in the number of tests possible.

For the sake of brevity and unity in this monograph, and in order to assist those not trained to use the standard, more technical, procedures, the equipment associated with the Taylor Water Analyzer has been selected. This Supplement gives the procedure for using this unit for the following determinations: colour, pH, residual chlorine, chlorine demand, iron, manganese and fluorides. More conventional equipment is described for the tests for odour, taste, hardness, magnesium, alkalinity, chlorides, and calcium carbonate stability (marble test). Equipment for testing for turbidity and the coagulating properties of water and that for the chlorine-demand test are not portable, but these tests may be made at filter plants with the equipment described.

The equipment may be carried in a specially designed case that also contains the reagents, glassware and an alcohol lamp to furnish heat when testing for iron and manganese in the presence of organic matter; or it may be placed on a work bench with sink and running water in treatment plants.

The Taylor Water Analyzer consists of a wooden support for three special glass tubes held at an angle of about 45°, down which light passes to a small mirror, in which natural colour or colours produced by specific reagents for each test may be compared with images produced by the graded standard colour solutions for each specific test. A slide containing nine colour standards is available for each test. It is moved back and forth until a match between the colour of the sample and that of one of the nine standard coloured solutions is secured. The desired result is the value of the matching standard solution as marked on the slide. Details for each test are given later.

The glass tubes, the small plastic tubes containing the series of colour standards in each slide, and the mirror should be cleaned so as not to interfere with the chemical reactions of the tests and so as to ensure the free passage of light through the large tubes, the colour standards and similar small tubes containing clear water, and their reflection as images from the mirror to the eye. Glassware should be cleaned in hot water with soap, using the brush furnished for the purpose, then rinsed with water (a portion of a clear sample if the test is made in the field) and finally with distilled water.

Many of the reagents must be imported from large laboratory-supply firms or prepared by a qualified laboratory equipped for such work. The colours produced by the various reagents reacting with samples of some waters may not have the precise tint and depth of colour of the permanent standard providing the nearest match. This is unavoidable when simple equipment must be used. It is for this reason, however, that standard methods specify, for precise laboratory procedures, standards that are made as needed with the same reagents used in testing a sample and that use known quantities of the elements being tested for.

The section on bacteriologic examinations includes the membrane-filter technique, which requires no central laboratory but only a portable incubator and associated items transported therein for the test for coliform organisms in the field.

Sampling

The value of any laboratory test depends on the method of sampling. The samples must be representative of the water to be examined or the results will have no significance. The frequency of collection of samples for chemical analysis depends on the uniformity in the quality of raw water, the types of treatment processes under control and other local factors. Sampling is therefore discussed separately in this monograph for each chemical analysis or bacteriologic examination.

The containers for the collection of samples for the chemical analysis should be clean but need not be sterile. For complete sanitary analysis about 1 gal of sample is required, but for plant operation this is seldom necessary and a 1-litre sample is sufficient. The point at which the sample is collected depends on what the results are to be used for, but in any case, representative samples free from extraneous matter should be taken and care exercised that the neck or stopper of the bottle does not become soiled.

PHYSICAL TESTS OF WATER

Turbidity

Purpose of the test

Turbidity is the optical effect caused by dispersion of and interference with light rays passing through water containing small particles in suspension. It may be caused by silt extracted from soil, surface wash containing suspended organic and mineral matter, precipitated calcium carbonate in all waters, aluminium hydrate in treated waters, precipitated iron oxide in corrosive water, microscopic organisms and similar material. Measurement of turbidity is important because it is one of the visual factors affecting consumer acceptance of water. In well water it may indicate the entrance of surface wash and thus potential contamination. The turbidity of raw waters influences the doses of coagulants required for their clarification and discloses the effectiveness of coagulation. If alum floc is passing the filters, or if floc is being formed in the clear-water well instead of in the coagulation basin, this will be indicated by turbidity of the filtered water.

The degree of turbidity depends upon the concentration, fineness or dispersion of the suspended matter and the light-absorption properties of the suspension. Turbidity is therefore not measured in parts per million but in *units*, because, for example, 1.0 mg/l fuller's earth does not necessarily produce the same degree of turbidity as the same concentration of finer or coarser material and hence does not necessarily produce a turbidity of one unit as determined by the use of the Jackson turbidimeter.

Sampling

Samples of raw water should be taken to determine the fluctuation in raw-water quality and the amount of turbidity to be removed by treatment. Samples of coagulated and filtered effluent should be taken to demonstrate the effectiveness of coagulation and filtration. Samples from the distributing system will allow comparison with filter effluent to determine whether turbidity is being increased in the distribution system. Samples from the distribution system should be collected at predetermined points selected to be representative of the entire volume of water, not affected by dead ends or sudden variations in flow. Only clean glass bottles of at least 500-ml capacity should be used for collecting samples.

Types of equipment

Turbidity of water may be determined by various types of turbidimeter, as follows:

1. Turbidity rod
2. Jackson candle turbidimeter
3. Turbidity standards
4. Baylis turbidimeter (low turbidities)
5. St. Louis turbidimeter (low turbidities)
6. Photoelectric-cell turbidimeter

The turbidity rod is convenient for field use but lacks precision. The Jackson candle turbidimeter is the standard instrument for measuring turbidity; it is most accurate in the range of turbidity between 100 and 1000 units, but accurate standards prepared locally with its use permit the determination of turbidities in the range of 5-100 units.

The Baylis and St. Louis turbidimeters are precision instruments and are capable of measuring turbidities as low as 0.2 unit. Both are based on the principle of comparing the turbidity of filtered waters under 2 units with carefully prepared standards. They are cumbersome to use. Recently a turbidimeter has become available[1] that uses photoelectric cells to measure the intensity of light scattered by suspended solids causing low turbidities. Such measurements can be recorded to give continuous information as to the turbidity of filter effluents and hence of the effectiveness of filtration. This turbidimeter, therefore, is an important tool for the control of larger filtration plants where funds are available for its purchase. For our purpose, however, the tests for turbidity with the Jackson turbidimeter and locally prepared standards and with a home-made St. Louis turbidimeter will be described.

JACKSON TURBIDIMETER. The Jackson turbidimeter consists of a long glass tube calibrated from 100 to 5000 turbidity units; a standard candle; and a support to hold the tube a distance of 76 mm or 3 inches above the top of the candle. The candle is specially made to produce a uniform flame, so that comparable results are secured from day to day.

Procedure :

1. Place the unit in a dark room.
2. Place the tube in the support.
3. Stir the sample of water and pour a portion into the tube.
4. Light the candle.
5. Look down the length of the tube, through the water, and observe the flame of the candle.
6. Continue to add water slowly until the image of the candle *just* disappears from view.
7. Note the turbidity-unit calibration at the surface of the water in the tube; this is the turbidity of the water.

Precautions :

1. Trim wick of candle.
2. Adjust holder so that the top of the candle is always 76 mm (3 in) below the bottom of the tube.
3. Do not burn the candle for more than a few minutes, otherwise the wax will become hot and the size of the flame will increase.

---

[1] C. R. Low-Range Turbidimeter, Hack Chemical Co., Ames, Iowa, USA.

4. Avoid draughts.

5. Keep the tube clean and, especially, free of soot or drops of water on the outer bottom surface.

6. Do not place an *empty tube* in the support over a lighted candle.

TURBIDITY STANDARDS :

Preparation of standards :

1. Add 4 to 6 g fuller's earth (clay) to about 2 l distilled water. Shake the mixture several times during a period of 1 h. Let stand for 24 h for the coarser particles of the earth to settle, leaving the very fine material in suspension.

2. Withdraw the *upper portion* of the turbid solution carefully so as not to disturb the sediment.

3. Determine the turbidity of the stock suspension so prepared with the Jackson turbidimeter. If the turbidity is found to be less than about 200 units, add more fuller's earth, shake and allow to settle for another 24 h. Test the turbidity of the upper portion of the suspension with the Jackson turbidimeter. This procedure should be followed until the suspension has a turbidity of about 200 units. The suspension should be tested *very carefully*, as the accuracy of this test determines the accuracy of standards prepared from it by dilution.

4. Dilute measured amounts of the stock suspension with distilled water to secure standards in a range of 5-100 units. Examples of dilutions are shown in Table 28.

A general formula for preparing 1000 ml of standard is as follows:

$$\frac{\text{Turbidity of desired standard}}{\text{turbidity of stock suspension}} = \frac{\text{ml of stock suspension needed}}{1000}$$

For instance, using a stock suspension with a turbidity of 225 units and needing a standard with a turbidity of 70, the amount of stock suspension required would be computed as follows:

$$\frac{70}{225} = \frac{x}{1000} \qquad x = 311 \text{ ml}$$

That is, 311 ml stock suspension would be diluted to 1000 ml with water.

Procedure for measuring turbidity with standards :

1. Collect sample of water in 1-litre bottle of identical size and shape to those containing the standards.

2. Agitate the standards and arrange them in a row in a position in which they are all well lighted but where there is a black background.

TABLE 28

EXAMPLE OF DILUTIONS FOR TURBIDITY STANDARDS

Desired units	Ratio of volume of stock suspension (200 units turbidity) to volume of standard [a]	Volume of stock suspension required, ml	Volume of distilled water required (ml) to make up to 1000 ml [b]
5	5 : 200 or 25 : 1000	25	975
10	10 : 200 or 50 : 1000	50	950
15	15 : 200 or 75 : 1000	75	925
20	20 : 200 or 100 : 1000	100	900
25	25 : 200 or 125 : 1000	125	875
30	30 : 200 or 150 : 1000	150	850
40	40 : 200 or 200 : 1000	200	800
50	50 : 200 or 250 : 1000	250	750
60	60 : 200 or 300 : 1000	300	700
70	70 : 200 or 350 : 1000	350	650
80	80 : 200 or 400 : 1000	400	600
90	90 : 200 or 450 : 1000	450	550
100	100 : 200 or 500 : 1000	500	500

[a] Based on the ratio of turbidity of desired standard to turbidity of stock suspension.

[b] Note that a dilution of, for example, 200 to 5 is secured by adding 975 ml distilled water to 25 ml suspension, *not* 1000 ml water to 25 ml suspension.

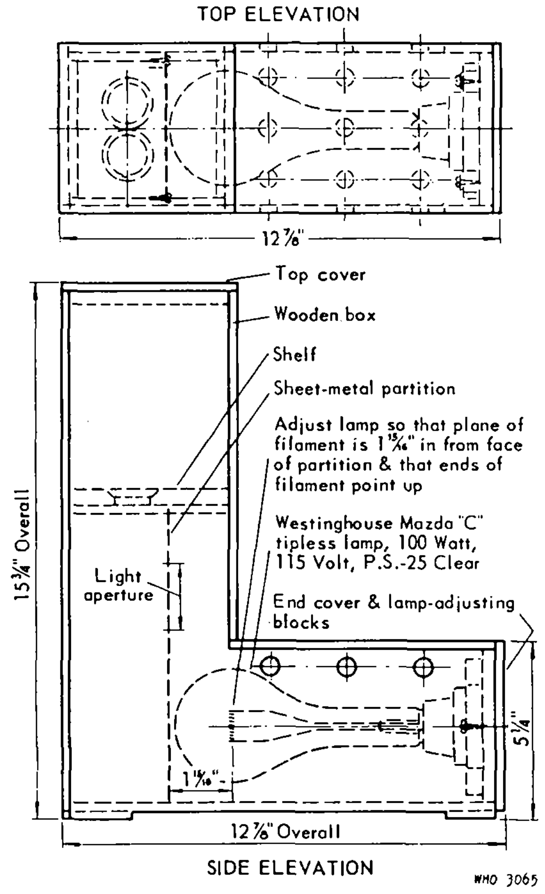
3. Compare the cloudiness or turbidity of the sample by placing it between standards until it is determined which standard best matches the turbidity of the sample.

4. Use judgement in estimating turbidity of the sample. For instance, a turbidity greater than the 30-unit standard but not approaching that of the 40-unit standard might be estimated at 33 units.

ST. LOUIS TURBIDIMETER. This turbidimeter was developed in St. Louis, Mo., for the determination of turbidities of filtered waters in the range of 0.1-5.0 units; Fig. 16 shows the design of the unit, which may be locally made. Briefly, it consists of a wooden box of the dimensions shown, coated on the inside with dull black paint. The light aperture in the vertical metal partition extends across the width of the box and is 2 inches in height. The bottom edge of the aperture is located 3 inches above the centre line of the electric bulb. The two holes cut in the shelf must have a diameter slightly less than that of 100-ml Nessler tubes, and the upper part of the holes should be bevelled to hold the bottom of the two tubes. The two 100-ml Nessler tubes should have uniform flat bottoms and have 100-ml marks equidistant from the bottoms.

FIG. 16

ST. LOUIS TURBIDIMETER



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The light from the bulb passes diagonally upward through the aperture and then through the lower part of the two Nessler tubes. No light will reach the eye, however, except that reflected by the suspended solids in the sample in one tube and in the selected standard suspension in the other tube. Hence the intensity of this reflected light permits comparison between the sample and the standard.

Preparation of low-turbidity standards for the St. Louis turbidimeter:

1. Select a stock turbidity suspension, the turbidity of which has been determined carefully with the Jackson turbidimeter.

2. Prepare by dilution 300 ml each of standard suspensions having turbidities of 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0 and 5.0 units. The quantity needed is computed by the formula:

$$x = \frac{vt}{T}$$

where x = required volume of stock turbidity standard
v = desired volume of low-turbidity standard
t = desired turbidity of low-turbidity standard
T = turbidity of stock standard

Thus 300 ml of 0.2-unit standard would be prepared from a 100-unit stock standard as follows:

$$x = \frac{300 \times 0.2}{100} = 0.6 \text{ ml}$$

Therefore mix 0.6 ml stock standard having a turbidity of 100 units with 299.4 ml clear distilled water to give 300 ml of standard having a turbidity of 0.2 units.

The other standards in the series would be prepared by diluting each to 300 ml with the following volumes of the 100-unit standard: 1.2, 1.8, 2.4, 3.0, 6.0, 9.0, 12.0, and 15.0 ml respectively.

These low-turbidity standards should be stored in clean, glass-stoppered bottles. Fresh standards should be prepared frequently from a suspension whose turbidity has been determined carefully with the Jackson turbidimeter.

Procedure :

1. Fill one tube to the mark with the sample and place in turbidimeter.
2. Estimate the turbidity of the sample from previous experience.
3. Fill the other tube to the mark with a portion of the turbidity standard corresponding to the estimated turbidity and compare the appearance of the turbidity in the two tubes.
4. Change the standard until a match is reached.

For example, assume that the estimated turbidity of the sample is 0.4 units and that the sample is found in step 3 to have a turbidity somewhat higher than the 0.4-unit standard. Then the 0.4-unit standard solution would be returned to the storage bottle and the tube rinsed with distilled water and refilled to the 100-ml mark with a portion of 0.6-unit standard. If the turbidity of the sample is then considered to be somewhat less than that of the 0.6-unit standard, it would be estimated as 0.5 units.

Precautions :

1. Glassware must be clean at all times. The standards should be vigorously shaken each time before they are used.

2. Bubbles of air in the sample and standards should be allowed to rise before comparisons of turbidity are made.

3. The accuracy of this procedure depends on the accuracy with which the turbidity of the stock turbidity standard is determined by use of the Jackson turbidimeter.

Interpretation

Turbidities greater than 5 units are readily noted by consumers and indicate unsatisfactory conditions. Coagulation and filtration should always reduce the turbidity to less than 1 unit, and in well-operated plants to less than 0.5 units.

Colour

Purpose of the test

The colour of water is commonly caused by the extraction of colouring material from the humus of forests or the deposit of vegetable matter in swamps and low-lying areas. This colouring matter is composed of humus and tannic acid compounds which cause the yellowish-brown tea colour of surface waters. The colour of water is of two types: " true colour " is that present in the water after the suspended matter has been removed, and " apparent colour " is true colour plus any other colour produced by substances in suspension. In certain cases colour may be imparted to water by dissolved iron or by the discharge of industrial wastes, but generally colour has little relation to pollution except as indicating surface water reaching ground-water supplies. The attractiveness of water is markedly dependent on the colour to which the public of any locality has become accustomed. The removal of colour is a function of water treatment, therefore decrease of colour is a measure of plant efficiency.

Colour is measured in units and not in parts per million, because the weight of colouring matter is not a measure of its colour-producing properties. The standard unit is the degree of colour produced by 1 mg/l of platinum in a solution of potassium chloroplatinate and cobaltous chloride.

Sampling

The samples collected for turbidity determination may be used for the colour test.

Equipment

Taylor Water Analyzer, three 150-mm tubes and Taylor colour slide 1300A, containing nine standardized coloured solutions with colours from 0 to 70 units.

Procedure

1. Place distilled water (colourless) to the 150-ml mark in the two outer tubes.
2. Fill the middle tube to the 150-ml mark with a portion of the sample being examined.
3. Place the colour slide in the Analyzer, which should be located where natural light (not direct sunlight) or strong artificial light passes down the tubes to the underlying mirror.
4. Move the slide until the colour of one of the two outer images equals that of the central image of the sample. The colour of the sample then is equal to the matching standard, as marked on the slide. The colour of the sample may be midway between that of two adjacent standards, e.g., 30 and 40 units, in which case the colour would be estimated as 35 units.

Note : If the colour of the sample exceeds 70 units, dilute the sample with distilled water to reduce the colour of the mixture to a value less than 70 units. For instance, a highly coloured water may be diluted 1 to 3 by adding 2 parts of distilled water to 1 part of sample. Suppose such a diluted sample has a colour of 60 units. Then the original sample has a colour of 60 × 3, or 180 units.

Interpretation

In general, consumer complaints will be received if the colour is greater than 10 to 15 units, and colour less than 5 units is desirable. Coagulation and filtration should reduce the colour to less than 5 units. Slow sand filters generally remove 50% of the colour of the raw water. Gradual increase in colour of filtered water is an indication of impaired efficiency of either coagulation or filtration. Increase in the colour of a supply as it flows through the distribution system may be caused by corrosion or growths in the pipes.

Odour

Purpose of the test

Odours in water are caused by extremely small concentrations of volatile compounds. Some of those compounds are produced when organic matter decomposes and are, therefore, likely to be present in surface waters because of the presence of organic matter from surface wash. Their intensity and offensiveness vary with the type, some being earthy and musty while others are putrefactive. Some odours of surface waters are produced by pollution from industrial wastes typical of specific industries, such as phenol or oil waste. For the most part, objectionable odours in surface waters are caused by plankton, i.e., free-floating microscopic organisms. These organisms liberate minute traces of volatile essential oils, which impart various objectionable odours according to the type and concentration of the oils liberated. Some are aromatic, others are sweetish and flowery, and still others are fishy or suggestive of pigpens. While the organisms are increasing in concentration the odours are not so strong as when they are decreasing and the dead organisms are decomposing. Other and various types of odours are produced by the addition of chlorine to water. In some cases odours are accentuated by marginal chlorination. In other instances chlorinous odours are formed which were not noticeable before the chlorine was added, and in still others the type of odour is changed from an unobjectionable one to one which may cause complaints. On the other hand, free-residual chlorine frequently destroys odour-producing substances.

Because odour-producing substances are usually present in extremely small concentrations, analytical procedures are unsatisfactory for their measurement and reliance is placed on the sense of smell. This varies with individuals, and therefore the results will vary somewhat. The sense of smell is easily fatigued; the ability to detect slight odours is rapidly lost when used for any length of time or if strong odours are encountered. It is important that the test be performed *in an odour-free room and with odour-free equipment.*

Sampling

Odour is generally determined only on the raw and finished water, although samples from the distribution system should be examined at frequent intervals. The samples should be taken with as little aeration as possible or odour will be lost. When hydrogen sulfide is present the odour determination should be made immediately after sampling, as hydrogen sulfide is liberated by aeration or modified by dissolved oxygen or the oxygen of the air. Fill the bottle two-thirds full.

Equipment

Clean, odour-free glass bottles. The use of a good detergent and rinsing in distilled water will remove odour from glassware. Avoid the use of scented soaps.

Procedure

1. Warm sample and bottle if necessary to room temperature.
2. Shake sample.
3. Remove stopper and sniff odour at mouth of bottle.

Results

The odour should be indicated or described according to the following code:

A — Aromatic or spicy (e.g., odour of camphor, cloves, lavender, lemon)
  Ac — Cucumber (e.g., *Synura*)

B — Balsamic (flowery) (e.g., geranium, violets, vanilla)
  Bg — Geranium (e.g., *Asterionella*)
  Bn — Nasturtium (e.g., *Aphanizomaenon*)
  Bs — Sweetish (e.g., *Coelosphaerium*)
  Bv — Violets (e.g., *Mallomonas*)
C — Chemical (odour due to industrial wastes or chemical treatment)
  Cc — Chlorinous (odour of free chlorine)
  Ch — Hydrocarbon (odour of oil refinery wastes)
  Cm — Medicinal (e.g., phenol or iodoform)
  Cs — Sulfuretted (odour of hydrogen sulfide)
D — Disagreeable (pronounced unpleasant odour)
  Df — Fishy (e.g., *Uroglenopsis* and *Dinobryon*)
  Dp — Pigpen (e.g., *Anabaena*)
  Ds — Septic (odour of stale sewage)
E — Earthy (odour of damp earth)
  Ep — Peaty (odour of peat)
G — Grassy (odour of crushed grass)
M — Musty (odour of decomposing straw)
  Mm — Mouldy (odour of damp cellar)
V — Vegetable (odour of root vegetables)

A number is prefixed to indicate the intensity of the odour, as follows:

0 — None: no odour perceptible
1 — Very faint: an odour that would not be detected ordinarily by the average consumer but could be detected in the laboratory by an experienced observer
2 — Faint: an odour that the consumer might detect if his attention were called to it but that would not attract attention otherwise
3 — Distinct: an odour that would be detected readily and that might cause the water to be regarded with disfavour
4 — Decided: an odour that would force itself upon the attention and that might make the water unpalatable
5 — Very strong: an odour of such intensity that the water would be absolutely unfit to drink (a rating to be used only in extreme cases)

Threshold-odour test

The above procedure is not precise enough when control of taste and odour present problems and costly treatment procedures are involved. In such cases the threshold-odour test is recommended.

Principles

This test is based on the dilution of portions of a sample of water with odour-free water until the mixture has only a barely discernible odour. The

" threshold-odour number " is the number of times the sample must be diluted with odour-free water for the odour to be just detectable. For example, if 50 ml of sample must be diluted to 200 ml before the odour of the mixture is just noticeable, then the threshold-odour number is 4. The degrees of dilution corresponding to a range of threshold-odour numbers from 1 to 200 are as follows:

Volume (ml) of sample that must be diluted to 200 ml	*Threshold-odour number*	*Volume (ml) of sample that must be diluted to 200 ml*	*Threshold-odour number*
200	1	12	17
140	1.4	8.3	24
100	2	5.7	35
70	3	5	40
50	4	4	50
35	6	2.8	70
25	8	2	100
17	12	1.4	140
14	14.3	1.0	200

Equipment

Six 500-ml glass-stoppered Erlenmeyer flasks
Thermometer
Graduated cylinders of 10-, 50- and 200-ml capacity
Pipettes, 10 ml graduated in 0.1 ml; 1 ml graduated in 0.1 ml
500-ml glass-stoppered sample bottles
2-litre bottles to hold odour-free water
Odour-free water producer
Hot plate

Home-made odour-free water producer. Odour-free water is produced by the slow flow of 1 litre per minute or less of tap water through granular activated carbon placed in a 1-gal bottle (Fig. 17).

Procedure

1. Shake 200 ml of the sample in a 500-ml flask, the sample first being warmed to a temperature of 40°C.

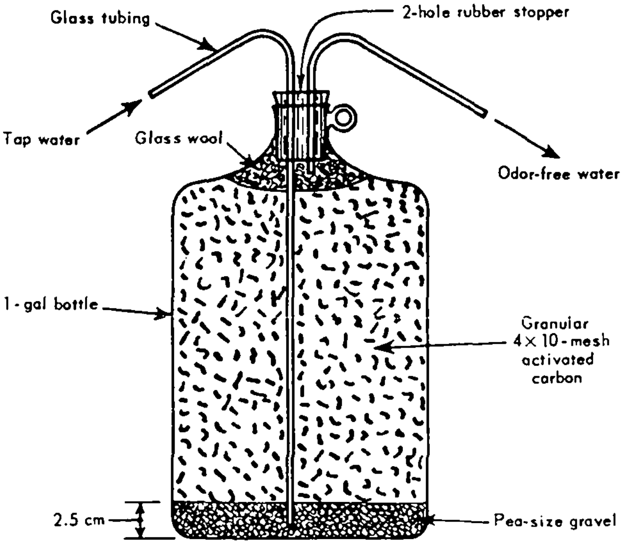
2. Sniff the odour by removing the stopper and holding the mouth of the flask to the nose.

3. Record the characteristics of the odour and its apparent intensity in terms of the number and letter codes listed on pages 288-289. If the odour of this undiluted sample is only just discernible, the threshold-odour number is 1. If its intensity is greater, proceed to step 4.

4. Place 200, 50, 14 and 5 ml of the sample respectively in four flasks and dilute the last three to 200 ml with odour-free water to determine the approximate range of the threshold-odour number. Add 200 ml of odour-free water to another flask.

FIG. 17

ODOUR-FREE WATER GENERATOR



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5. Heat the diluted sample and the odour-free water to 40°C before testing.

6. Shake the flask containing the odour-free water, remove the stopper and sniff the odour. (Always start comparing odours with the flask containing odour-free water, followed by the flask containing the *least* amount of sample.)

7. Compare this with the odour of the mixture containing 5 ml of sample.

8. If this mixture has a perceptible odour, further dilution must be made, such as 1 ml sample and 199 ml odour-free water. If the mixture has no odour, compare the mixture of 14 ml sample and 186 ml odour-free water, and so on, until a definite odour is noticed from the mixture in one of the flasks.

9. Having thus established the approximate range of the threshold-odour number, prepare a series of five dilutions, starting with the dilution that had a definite but faint odour, as shown in Table 29.

10. Repeat the test with these five selected dilutions to find the highest dilution having a just-detectable odour.

If, for example, the original tests disclosed that the mixture of 14 ml sample and 186 ml odour-free water had a discernible odour, the subsequent

TABLE 29

DILUTIONS FOR VARIOUS ODOUR INTENSITIES

Sample volume in which odour is first noted			
200 ml	50 ml	14 ml	5 ml
Volume (ml) of sample to be diluted to 200 ml:			
200	50	14	5.0
130	40	11	3.3
100	29	9.1	2.2
67	20	6.7	1.3
50	14	5.0	1.0

dilutions would be as shown in the third column of Table 29. Assuming, then, that the dilution of 9.1 ml sample and 190.9 ml odour-free water has a barely discernible odour and the higher dilutions have no odour, then the threshold-odour number would be 22:

$$\frac{9.1 + 190.9}{9.1} = \frac{200}{9.1} = 22$$

Anomalous responses of the sense of smell sometimes occur, as when a higher dilution may have an odour while a lower dilution is odourless. For example, in the following series of dilutions of a sample to 200 ml there might be a positive odour for those with a + sign and no odour for those with a – sign:

Volume of sample, ml:	5.0,	6.7,	9.1,	11,	14
	–	+	–	+	+

In this case the dilution of 11 ml of sample to 200 ml by odour-free water would be considered the significant degree of dilution of the sample, giving a threshold-odour number of 18.

Experience will disclose what dilution series should be used with any given water, so that the routine tests can be started with step 10 above. For example, threshold-odour numbers may be in the range of 14 to 40, so the series of dilutions shown in the third column of Table 29 would be used as a routine, or shifted later if the water changed in character so that its odour was outside this range.

Interpretation

The character and intensity of odour often discloses the nature of pollution or the prevalence of micro-organisms and hence the type of treatment or preventive measures needed. Consumers will judge a supply by its taste and odour; while many consumers will become accustomed to a prevailing odour of low intensity, the objective should be the delivery of water having an intensity rating of 1 or a threshold-odour number of 2 or less.

Determination of optimum dosage of activated carbon for odour control

This procedure is based on the use of the threshold-odour test and the laboratory flocculating equipment described elsewhere in this Supplement (p. 313).

Equipment

Mixing equipment with five beakers
Five narrow-mouthed Erlenmeyer flasks
Five Buchner-type funnels
Supply of very fine glass wool

A stock slurry of 1.0 g activated carbon in 1 litre odour-free water prepared as described under Threshold Odour Test (each millilitre of this slurry would provide a dose of 1 p.p.m. when added to 1 litre of water; slurry must be agitated each time it is used).

Procedure

1. Determine the threshold-odour number of the sample.

2. Add 1 litre of the sample to each of the five beakers and place them in the mixing equipment.

3. Add to four of the beakers the graded doses of the slurry of activated carbon according to the range in which the threshold odour falls in Table 30. For instance, if the threshold-odour number is found by step 1 to be 30, then the second row of graded values would be used, as follows:

beaker No. 1: no slurry
beaker No. 2: 4 ml
beaker No. 3: 8 ml
beaker No. 4: 16 ml
beaker No. 5: 32 ml

giving corresponding dosages in parts per million.

4. Agitate the samples by operating the stirring mechanism at about 100 r.p.m. and continue for the period equivalent to the time the carbon will remain in contact with the water in the plant—4 h for many sedimentation basins.

5. Place glass wool in each of the five funnels and rinse with odour-free water.

TABLE 30

ACTIVATED-CARBON DOSES FOR DIFFERENT CONCENTRATIONS OF ODOURS[a]

Threshold-odour number of water to be treated	Activated carbon dose, p.p.m.				
	Beaker No. 1	Beaker No. 2	Beaker No. 3	Beaker No. 4	Beaker No. 5
10 to 20	None	2	4	8	16
20 to 40	None	4	8	16	32
40 to 80	None	8	16	32	64

[a] These are suggested trial values only. Experience will indicate the range in doses for any given range in threshold-odour values. In practice it may be found that the range of odours from 20 to 40, for example, will require doses between 5 and 40 p.p.m. rather than between 4 and 32 p.p.m.

6. Stop the agitation and filter *each of the five portions* of the sample through the glass wool, discarding the first 200 ml of each filtrate.
7. Determine the threshold-odour number of the remaining portion of each filtrate.

Interpretation

Assume that the threshold-odour numbers of the five portions were found to be 30, 22, 14, 7 and 2 respectively. These assumed results would indicate that the activated-carbon dose used in beaker 5, viz., 32 p.p.m., was adequate, because it reduced the odour to a threshold value of 2. A more accurate interpretation can be arrived at by plotting the results and then finding from the resulting curve the dose corresponding to the final threshold-odour value desired.

If the above procedure indicates that the highest dose of carbon used in a specific test does not produce the desired threshold odour, repeat the test with a higher series of graded values.

Precautions

All equipment must be clean and should be rinsed with odour-free water just before use.

The test should be made when the laboratory or room is free of odours and before the operator has eaten or smoked.

When an operator is uncertain of the results of the odour test, a rest period of at least 15 min should be taken to free the nose of the odours under study.

The bias of an operator who has conducted the tests may be compensated for by asking an associate to check the odour of selected flasks that are in doubt, the identity of which is not known to the associate.

The laborious procedure discussed need be followed only when there is no previous experience with activated-carbon treatment of a specific supply, or when a new type of odour is encountered or a new brand of activated carbon is used, or when other treatment procedures are altered. Doses may usually be raised or lowered on a plant scale as dictated by the results of the threshold-odour test of samples of the plant effluent.

Taste

Purpose of the test

Taste in water is generally closely related to odour and is caused by the same conditions; however, dissolved mineral matter may impart taste but no odour to water. Metallic ions such as copper, zinc, or iron may cause metallic tastes. Chlorinated water containing phenol-like compounds may have a distinct taste in concentrations below that detectable as odour.

Sampling

Tests for taste are made if the water is safe to drink; the same samples are generally used as were collected for the odour determination.

Equipment

Clean, odour-free sample bottles
One 50-ml beaker

Procedure

1. Warm sample to room temperature.
2. Pour a small quantity of sample into beaker.
3. Taste on both tip and back of tongue.

Results

Indicate the characteristics and intensity of the taste according to the same code as used for odour (pages 288-289).

Interpretation

Taste is a measure of consumer acceptance of water, although, as with odour, a change in character may cause complaints if consumers have become accustomed to one type of taste. Generally, finished water should have a taste intensity of 2 or less. A metallic salty taste may indicate pollution in the water supply.

CHEMICAL TESTS OF WATER

Hardness

Purpose of the test

Water is a universal solvent and dissolves varying amounts of different mineral substances. Those producing hardness do not affect the sanitary quality but are of importance in the domestic use of water, particularly for laundry and boiler purposes. Calcium and magnesium salts, the principal mineral constituents, consume soap and precipitate as insoluble compounds or soap curds. Until all the calcium and magnesium is precipitated, no lather or washing action is obtained from the soap; the soap-consuming power of water is therefore a measure of its hardness. Calcium and magnesium generally are dissolved as soluble bicarbonate but may change, owing to heating, to the less soluble carbonate which precipitates and is one source of scale in distributing systems and hot-water heaters. Hard waters are usually less corrosive than soft waters.

The most accurate test for hardness due to calcium is the EDTA procedure described in *International Standards for Drinking-Water* (page 122). While this procedure is simple, the four solutions used in the test are not stable and must be prepared in a laboratory at rather frequent intervals. It is therefore not suitable for use in areas where laboratories are not available. For plant-control purposes, the soap test for total hardness is here described.

Sampling

Hardness is generally determined only in finished water, hence samples of filtered water or of that from the distributing system are used, except when softening is practised, in which case more detailed sampling is involved.

Equipment

Burette, 50 ml
Burette stand
Glass-stoppered bottles, 250-ml capacity
Standard soap solution
Volumetric flask, 50-ml capacity

Procedure

1. Fill burette to mark with standard soap solution.

2. Measure 50 ml of sample, using volumetric flask, and transfer it to glass-stoppered bottle.

3. Add standard soap solution from the burette in small increments (each not over 0.5 ml) to test portion in bottle. Shake vigorously 25 times after each addition of standard soap solution. Reduce the volume added as the end point of the titration is approached.

4. Lay bottle on side and observe lather.

5. Continue addition of standard soap solution until a *continuous lather* on surface of test portion persists for at least 5 min.

6. Read burette and determine amount of standard soap solution used. To ensure that the end point is a true one, add an additional 0.5 ml of soap

solution, shake and observe lather. A lather which then does not persist for 5 min indicates that the tentative end point was a false or " ghost " one. In such circumstances, continue the titration until the true end point is reached.

7. Subtract from amount of standard soap solution used the lather factor, usually 0.3 ml (lather factor is the amount of standard soap solution required to produce lather when added as in above procedure to distilled water of zero hardness. This factor is noted on the label of standardized soap solution).

8. Multiply corrected volume of standard soap solution by 20 to give hardness in parts per million.

9. If volume of soap solution required to produce a continuous lather is greater than 7 ml, repeat the test using a smaller volume of sample diluted to 50 ml with boiled and cooled distilled water.

Results

Hardness is reported as parts per million calcium carbonate regardless of whether it is calcium or magnesium or both:

$$\text{Hardness, p.p.m., as CaCO}_3 = (\text{ml soap solution} - \text{lather factor}) \times 20 \times \frac{50}{\text{volume of sample}}$$

The total hardness as determined may be subdivided into that due to calcium and that due to magnesium, as noted under Computations, page 299.

Interpretation

Water for domestic and laundry use preferably should contain less than 100 p.p.m. hardness. Water with hardness of 300 p.p.m. or greater is not suitable for ordinary use, although there are many areas where harder water must be used and where softening is economically not feasible. Very soft waters having hardness less than 30 p.p.m. are likely to be corrosive; such waters are generally treated with lime, which increases the hardness.

Magnesium

The accurate, rapid and not too complex determination of the concentration of magnesium in water that is presented in *International Standards for Drinking-Water* cannot be used at many water-softening plants because the special reagents are not widely available and they deteriorate in storage. The two procedures given in *Standard Methods for the Examination of Water and Waste-Water* are for chemists having complete laboratory facilities at their disposal. For the control of softening plants with limited laboratory facilities the following procedure is presented.

Purpose of the test

Reference to Chapter 13, Demineralization (page 228), will indicate that the lime dose needed for softening is influenced by the magnesium content of the water to be softened. Therefore the test under discussion is used for routine control of softening plants when the magnesium content is significant, as when deposits of dolomite contribute both magnesium and calcium to ground waters flowing as springs into surface waters.

The chief reason that the magnesium content is important to the softening process is that magnesium hydroxide is best precipitated at pH 10.6, as contrasted to 9.4 for calcium carbonate. The lime dose used to produce this higher pH must be adequate to allow for magnesium removal, hence the necessity of determining the magnesium content. If this content is found to be low, it can be ignored and the lime dose selected to remove the hardness due to calcium compounds.

Principles

The basis of this test is the use of limewater to precipitate magnesium as the insoluble magnesium hydroxide. The quantity of limewater required for this purpose is a measure of the content of magnesium.

For example, assume that magnesium is present, as a part of non-carbonate hardness, as magnesium sulfate. Then limewater would react as follows:

$$MgSO_4 + Ca(OH)_2 = Mg(OH)_2 + CaSO_4$$

If, in addition, magnesium bicarbonate is present, it would be precipitated as follows:

$$Mg(HCO_3)_2 + 2Ca(OH)_2 = Mg(OH)_2 + 2CaCO_3 + 2H_2O$$

The insoluble magnesium hydroxide is precipitated and removed by filtration through laboratory filter paper. The resulting drop in alkalinity then is determined.

Equipment

Two 250-ml Erlenmeyer flasks
Two 200-ml volumetric flasks
Two funnels
Filter paper
Burette
Steam bath[1]
0.02N sulfuric acid reagent
Phenolphthalein indicator
One 100-ml pipette
One 25-ml pipette
Limewater

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[1] If a laboratory steam bath is not available, use a large beaker to hold the boiling water; cover with a copper sheet, in which two holes have been cut somewhat smaller than the diameter of the flasks, so that their lower portion will be exposed to steam.

Preparation of limewater. Lime is only partly soluble, but the soluble portion may be prepared as a true solution as follows:

1. Add 3 g hydrated lime to 1 litre distilled water in a 2-l bottle.
2. Shake thoroughly for several minutes.
3. Allow to settle so that undissolved lime collects in the lower portion of the bottle and the solution above is *clear.*
4. Remove the clear supernatant limewater as needed very carefully with the 25-ml pipette, being sure not to disturb the settled lime. (This procedure maintains a saturated solution in the bottle.)

Procedure

1. Determine the alkalinity of a 100-ml portion of the sample to the phenolphthalein indicator, if this has not been done previously as part of the tests for the several types of alkalinity (page 301), OR

1a. Add to the 100-ml sample precisely the same amount of 0.02N sulfuric acid as was needed to measure the alkalinity to phenolphthalein.

2. Boil the sample in the flask until its volume is reduced to one-half, or about 50 ml.
3. Transfer the boiled sample to one of the 200-ml volumetric flasks.
4. Boil a portion of distilled water to remove carbon dioxide and transfer 100 ml to the second 200-ml volumetric flask.
5. Add 25 ml of the *clear* limewater to *each* flask.
6. Fill each flask to the mark with hot, boiled distilled water. Place a rubber stopper *loosely* in each flask and mix.
7. Place both flasks in the holes of the cover of the steam bath for 1 h to complete the reaction.
8. Remove both flasks from the steam bath and immediately filter a portion of the content of each through filter paper, collecting the filtrates.
9. Immediately transfer 50 ml of each filtrate to the two Erlenmeyer flasks.
10. Add 4 drops of phenolphthalein indicator to each flask and determine by titration the amount of 0.02N sulfuric acid reagent required to neutralize the alkalinity of both filtrates.

Computations

Subtract the amount of 0.02N acid used to neutralize the alkalinity of the sample from that required for the distilled water treated as above, then multiply this difference by the factor 9.72. The results are in *parts per million magnesium ion,* not in terms of calcium carbonate.

Example : Assume that 15.7 ml and 11.3 ml acid reagent were used to neutralize the alkalinity of the treated distilled water and of the treated sample respectively, giving a difference of 4.4 ml. Then the magnesium content of the sample is 4.4 × 9.72 = 42.7 p.p.m.

The content of magnesium multiplied by the factor 4.11 gives the magnesium hardness in terms of calcium carbonate; the total hardness minus the magnesium hardness equals the hardness due to calcium.

Example : A water with a total hardness of 150 p.p.m. and a magnesium content of 10 p.p.m. would have a magnesium hardness of 10 × 4.11, or 41.1 p.p.m. The calcium harndess would be 150—41.1, or 109.9 p.p.m.

Alkalinity

Purpose of the test

Alkalinity is a measure of the basic constituents of water. In natural waters it is usually present as the carbonate and bicarbonate salts of calcium, magnesium, sodium, and potassium. Within reasonable limits alkalinity has no sanitary significance, but it is very important in connexion with coagulation, softening, and corrosion prevention. Alum is an acid salt which, when added in small quantities to natural water, reacts with the alkalinity present to form floc. If insufficient alkalinity is present to react with all the alum, coagulation will be incomplete and soluble alum will be left in the water. It may therefore be necessary to add alkalinity in the form of soda ash or lime to complete the coagulation or to maintain sufficient alkalinity to prevent the coagulated water from being corrosive. Alkalinity can exist as hydroxide, carbonate, or bicarbonate. Ordinarily the total alkalinity, as determined with methyl orange indicator, gives sufficient information for the control of coagulation and corrosion prevention when pH also is determined. Control of lime-soda softening of hard waters, however, requires information as to the characteristics of the alkaline compounds originally present or formed by the softening reactions. For this purpose the alkalinity to methyl orange and also to phenolphthalein indicator must be determined.

Sampling

Samples of both raw water and treated water may be required, depending on the treatment processes.

Methyl orange alkalinity

Equipment :

Burette, 50 ml
Burette stand
Two 250-ml Erlenmeyer flasks
Methyl orange indicator solution, 0.5 g per litre of distilled water
0.02N sulfuric acid
100-ml volumetric flask

Procedure :

1. Using volumetric flask, measure two 100-ml portions of sample and transfer each to a 250-ml Erlenmeyer flask.
2. Fill burette to mark with 0.02N sulfuric acid.
3. To each sample in flasks add 2 drops of methyl orange solution.
4. Add the sulfuric acid slowly from burette to one of the test portions in flask, mixing thoroughly by rotating flask.
5. Continuously compare the colours in the two flasks as the acid is added, and *at the first appearance of permanent faint pink colour in the test flask, stop adding the acid.*
6. Read burette. The amount of 0.02N acid used (in millilitres) multiplied by 10 will give the alkalinity in parts per million.[1]

Results. Results are reported as parts per million calcium carbonate:

$CaCO_3$ alkalinity to methyl orange, p.p.m. =

$$\text{ml } 0.02\text{N acid} \times 10 \times \frac{100}{\text{volume of sample}}$$

Phenolphthalein alkalinity

Phenolphthalein indicator discloses the point at which carbon dioxide ceases to prevail and carbonate alkalinity begins to be present, this change occurring at pH 8.3. In other words, the " acid " range with this indicator is below pH 8.3. and the " alkaline " range is above this value. Alkalinity to both methyl orange and phenolphthalein indicators is determined in order to disclose the chemical nature of the alkaline compounds and their relation to hardness-producing compounds, as discussed below.

Equipment. Same as in procedure for methyl orange alkalinity, except for substitution of phenolphthalein indicator solution for methyl orange indicator solution. (*Phenolphthalein indicator solution :* Dissolve 5 g phenolphthalein in 500 ml 95% ethyl or isopropyl alcohol and add 500 ml distilled water. Then add 0.02N sodium hydroxide drop by drop until a *faint* pink colour appears.)

Procedure :

1. Place 100 ml of sample in Erlenmeyer flask.
2. Add 2 drops of phenolphthalein indicator solution. If the indicator in the sample is colourless, the water is " acid " to phenolphthalein, so the phenolphthalein alkalinity is zero.

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[1] For complex physicochemical reasons, alkalinity values secured when methyl orange is used as the indicator are about 5 p.p.m. higher than the true values when the sample being tested has an alkalinity of 30 p.p.m. or *less*. Therefore, unless the more complex *Standard Methods* procedure is followed, alkalinity values of 30.0 p.p.m. or less with methyl orange indicator should be adjusted by *subtracting* 5 p.p.m. from the observed values. For example, if the alkalinity of a sample appears to be 22 p.p.m., the actual alkalinity is 22.0 − 5.0, or 17.0 p.p.m.

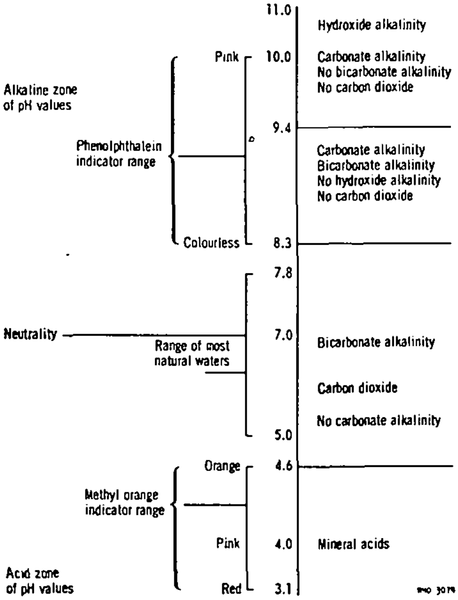
3. If the indicator in the sample has a pink colour, titrate by adding the 0.02N sulfuric acid reagent until the colour changes from pink to colourless.

4. Read burette. The amount of 0.02N acid used (in millimetres) multiplied by 10 gives the phenolphthalein alkalinity in parts per million.

Computing bicarbonate, carbonate and hydroxide alkalinities

The relationship of alkalinity as determined with methyl orange and phenolphthalein indicators to bicarbonate, carbonate and hydroxide alkalinities of a sample may be clarified by reference to Fig. 18. If a sample is acid to methyl orange, its pH is less than 4.6 and mineral acids are present; this is due to pollution by acid mine wastes or other industrial wastes, since natural waters contain no mineral acids.

FIG. 18
ZONES OF pH VALUES WHERE FREE CARBON DIOXIDE, BICARBONATES, CARBONATES AND HYDROXIDE ALKALINITIES PREVAIL



Note: The full pH range is from 1.0 to 14.0. From Cox (1958).

If a sample is alkaline to methyl orange but acid to phenolphthalein, its pH is in the range of 4.6 to 8.3 and the alkalinity is in the bicarbonate form, with no carbonate alkalinity, since carbonate alkalinity would react with free carbon dioxide to form more bicarbonates.

If the phenolphthalein alkalinity is *less than one-half* the methyl orange alkalinity, bicarbonates and carbonates are present, and the pH is greater than 8.3.

If the phenolphthalein alkalinity is *more than one-half* the methyl orange alkalinity, carbonate and caustic or hydroxide alkalinities are present and the pH is greater than 9.4.

Finally, if the alkalinities to both indicators are the same, then *all* of it is hydroxide alkalinity.

The alkalinity to phenolphthalein is never greater than that to methyl orange. The latter is the *total alkalinity*.

TABLE 31

PROPORTION OF ALKALINE MATERIAL IN WATERS IN TERMS OF THREE TYPES OF ALKALINITY[a]

Result of titration	Caustic or hydroxide alkalinity	Carbonate alkalinity	Bicarbonate alkalinity
Acid (colourless) to phenolphthalein Alkaline to methyl orange	0	0	Total alkalinity to methyl orange
Phenolphthalein alkalinity less than one-half methyl orange alkalinity	0	Twice phenolphthalein alkalinity	Total alkalinity to methyl orange minus twice phenolphthalein alkalinity
Phenolphthalein alkalinity equal to one-half methyl orange alkalinity	0	Methyl orange alkalinity	0
Phenolphthalein alkalinity greater than one-half methyl orange alkalinity	Twice phenolphthalein alkalinity minus methyl orange alkalinity	Twice the difference between the methyl orange alkalinity and the phenolphthalein alkalinity	0
Phenolphthalein alkalinity equal to methyl orange alkalinity	Total alkalinity	0	0

[a] Alkalinity expressed as calcium carbonate.

The above relationships are detailed in Table 31, which is used to compute the content of the three types of alkalinity. Strictly speaking, this tabulation deals with the basic ions, namely, OH, HCO_3 and CO_3, so that it is not necessary to consider the cations of calcium, magnesium or sodium. Subsequent procedures, however, distinguish between these cations.

The use of Table 31 may be illustrated by an example. Assume that titration of a sample shows a methyl orange, or total, alkalinity of 25 p.p.m. and a phenolphthalein alkalinity of 15 p.p.m. The latter is greater than one-half the methyl orange alkalinity, so the conditions agree with the next-to-last row of Table 31. Then the hydroxide alkalinity is " twice the phenolphthalein alkalinity minus the methyl orange alkalinity " or $(2 \times 15) - 25 = 5$ p.p.m. The carbonate alkalinity is " twice the difference between the methyl orange alkalinity and phenolphthalein alkalinity", or $2(25-15) = 20$ p.p.m. Note that the hydroxide alkalinity of 5 p.p.m. plus the carbonate alkalinity of 20 p.p.m. equals the total alkalinity of 25 p.p.m., as shown by the titration with methyl orange indicator.

Computing non-carbonate and carbonate hardness

The tests for *total hardness* and *total alkalinity* give results which can be used in the control of lime-soda softening, where the non-carbonate hardness determines the soda ash dose and the carbonate hardness in part determines the lime dose needed to secure the desired degree of softening (non-carbonate hardness does not contribute alkalinity, as it is due to neutral salts of calcium and magnesium, usually sulfates). Table 32 shows the basis of the computations.

For example, assume that a sample has a total hardness of 150 p.p.m. and a total alkalinity of 100 p.p.m. Then the hardness is the greater and the conditions of the last row prevail. That is, there is zero alkalinity due to sodium or potassium bicarbonates, a non-carbonate hardness of 150—100, or 50 p.p.m., and a carbonate hardness of 100 p.p.m. Or assume that a sample has an alkalinity of 80 p.p.m. and a total hardness of 50 p.p.m. Then the conditions of the first row prevail, and the alkalinity due to sodium and/or potassium bicarbonates, which contribute no hardness, is 80—50, or 30 p.p.m. There is zero non-carbonate hardness, and the carbonate hardness is 50 p.p.m.

Computing content of sodium hydroxide, sodium carbonate, calcium and magnesium carbonates and calcium hydroxide

The hydroxide alkalinity and sodium alkalinity, that is, the alkalinity in excess of the hardness, as determined in Table 32, may be used to compute the other values shown by Table 33 when total hardness and carbonate alkalinity have once been determined.

For example, assume that a sample has a total hardness of 105 p.p.m., a total alkalinity of 70 p.p.m. and a phenolphthalein alkalinity of 40 p.p.m., and that Tables 31, 32 and 33 are used in succession. The phenolphthalein alkalinity is greater than one-half the total alkalinity, so line 4 would be used in Table 31. This shows that the bicarbonate alkalinity is zero; that the carbonate alkalinity is 2 (70—40) = 60 p.p.m.; and that the hydroxide

TABLE 32

COMPARISON OF TOTAL HARDNESS AND TOTAL ALKALINITY

Results of laboratory tests	Alkalinity due to sodium or potassium bicarbonates	Non-carbonate hardness	Carbonate hardness
$H < A$	$A - H$	0	H
$H = A$	0	0	H
$H > A$	0	$H - A$	A

H = total hardness; A = total, or methyl orange, alkalinity.

TABLE 33

COMPARISON OF HYDROXIDE AND SODIUM ALKALINITIES

Results of laboratory tests	Sodium hydroxide	Sodium carbonate	Calcium and/or magnesium carbonate	Calcium hydroxide
HA < SA	HA	SA — HA	Total hardness	0
HA = SA	HA	0	Carbonate alkalinity	0
HA > SA	SA	0	Carbonate alkalinity	HA — SA

HA = hydroxide alkalinity; SA = sodium alkalinity.

alkalinity is (2 × 40) — 70 = 10 p.p.m. Table 32 is used next. Inasmuch as the assumed results used as an example show a total hardness greater than the total alkalinity, line 3 is followed. This shows that alkalinity to sodium bicarbonate is zero; that non-carbonate hardness is 105—70 = 35 p.p.m.; and that bicarbonate hardness is 70 p.p.m. Finally Table 33 is used. The total hardness by test is 105 p.p.m. The hydroxide alkalinity was found by Table 31 to be 10 p.p.m. The sodium alkalinity was found by Table 32 to be zero. Then the conditions of the bottom line of Table 33 prevail, according to which the content of sodium hydroxide is zero; the content of sodium carbonate is zero; the content of calcium and magnesium carbonate equals the carbonate alkalinity of 60 p.p.m.; and the content of calcium hydroxide equals the caustic alkalinity minus the sodium alkalinity, or 10—0 = 10 p.p.m.

The values determined from these three tables are used in the control of lime-soda softening, as discussed in Chapter 13.

Interpretation

The determination of alkalinity provides an estimate of the basic constituents. If these are present as salts of calcium and magnesium, the alkalinity will equal the hardness. If the alkalinity is greater than the hardness, it means there are basic salts present other than those of calcium and magnesium—generally sodium or potassium salts. If the alkalinity is less than the hardness, neutral salts of calcium or magnesium must be present that are not carbonates; usually these are sulfates. Coagulation generally requires a concentration of alkalinity equal to about half the amount of alum added to produce a good floc. Thus 20 p.p.m. alum requires 10 p.p.m. alkalinity to form floc. As coagulation lowers the alkalinity, the water becomes more corrosive and, unless an excess of alkalinity was present before coagulation, soda ash or lime must be added to the filtered water to prevent corrosion. (See discussion of the Marble or Calcium Carbonate Stability Test for control of corrosiveness of water, page 310.)

Carbon dioxide

Purpose of the test

The content of carbon dioxide in water is directly related to the type and degree of alkalinity and the pH value and therefore influences coagulation, corrosion prevention, softening and iron removal.

The content of carbon dioxide may be titrated with standardized sodium hydroxide reagent, but this reagent deteriorates upon standing. The easiest procedure is to compute the content of carbon dioxide from the total or methyl orange alkalinity and pH value of a sample. The accuracy is influenced by any aeration of the sample before making the test for pH; the best procedure is therefore to collect the sample with as little agitation as possible and to make the pH test immediately at the point of sampling. The test for alkalinity of the same sample may be postponed until convenient.

Computing content of carbon dioxide

The most accurate method of computing the content of free carbon dioxide is the use of a nomogram using the temperature of the water, the total-solids content and the bicarbonate alkalinity, as given in *Standard Methods for the Examination of Water and Waste-Water.* The diagram shown in Fig. 19, however, is adequate for plant control purposes.

Use of Fig. 19:

1. Locate the *diagonal* line for the known pH value.
2. Follow along this line to the point where it crosses the *vertical* line for the known alkalinity.
3. The *horizontal* line passing through this point of intersection gives the content of carbon dioxide in parts per million.

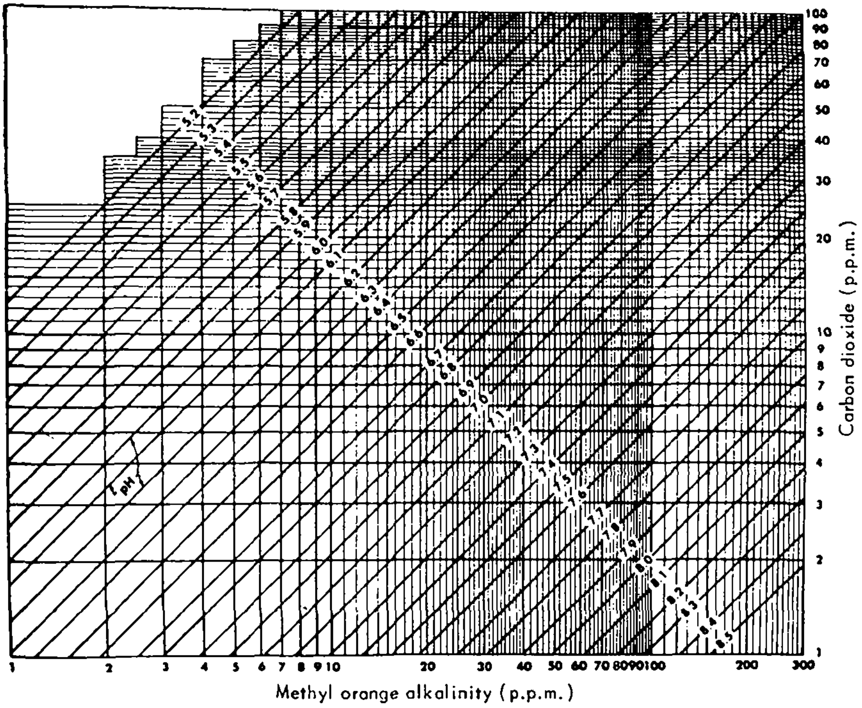
Example: Assume the pH to be 6.6 and the alkalinity to be 19 p.p.m. The horizontal line passing through the point of intersection of the lines for these two values shows, on the scale at the right, a carbon dioxide content of 10 p.p.m.

Hydrogen-ion concentration—pH value

Purpose of the test

The hydrogen-ion concentration, or pH value, measures the *intensity* of the acid or alkaline reaction of water. It does not measure the total *concentration* or weight of acid or alkali present. Water containing no acid or alkali has a pH value of 7, which is termed the neutral pH value. Addition of strong acid such as sulfuric or hydrochloric acid markedly reduces the pH value, while an equal amount of weak acid such as carbonic acid only slightly lowers the pH value. Similarly, alkali increases the pH value above 7, and the degree of change depends on the intensity and amount of the alkali

FIG. 19
RELATIONSHIP BETWEEN METHYL ORANGE ALKALINITY, CARBON DIOXIDE CONTENT AND pH



From US Air Force (1959) p. 207.

added. Thus pH values from 1 to 7 indicate acidity, a pH value of 7 indicates neutrality, and pH values from 7 to 14 indicate alkalinity. Most natural waters have pH values between 5.0 and 8.6. Alum coagulation of water occurs at an optimum pH value which varies for different waters and is generally determined by means of the jar or coagulation test. The corrosiveness of water is a function of pH and can often be corrected by decreasing the acid intensity by addition of alkali; this is usually controlled by determination of the pH value. Similarly, the deposition or dissolving of scale on pipes can be controlled by changing the reaction or alkalinity and pH value by addition of lime or soda ash, also controlled by pH determinations. Finally, lime-soda softening is controlled in part by the test for pH.

A part of the pH scale is shown in Fig. 18 (page 302), illustrating the relationship between pH and types of alkalinity and acidity.

The range in pH values between pH 5.2 and pH 9.6 is covered by the four indicators noted below.

Sampling

Samples for pH determination must be collected carefully, because small quantities of contaminants such as particles of lime can affect the pH of a small sample to a marked extent. Loss of carbon dioxide by aeration will change the pH value, so samples should be collected without agitation and preferably examined at once. Completely fill the bottle when a sample is to be stored before examination.

Note. The following procedure for using indicators to determine the pH values of samples of water is *not* satisfactory for waters with alkalinities *below* 20 p.p.m. An electrometric method using a "flowing electrode" must be used with such waters of low alkalinity. It is beyond the scope of this monograph to outline procedures for using such electrodes.

Equipment

Taylor Water Analyzer and three 150-ml tubes
Taylor pH slide 1301 F (chlorphenol red, pH 5.2 to 6.8)
Taylor pH slide 1301 H (bromthymol blue, pH 6.0 to 7.6)
Taylor pH slide 1301 I (phenol red, pH 6.8 to 8.4)
Taylor pH slide 1301 M (thymol blue, pH 8.0 to 9.6)
Taylor special 27-ml pH pipette
Taylor special 0.18-ml pH pipette

Reagents

Chlorphenol red indicator
Bromthymol blue indicator
Phenol red indicator
Thymol blue indicator

Care and adjustment of indicators. The indicator solutions must be protected from contamination and should be stored in Pyrex glass or polyethylene plastic bottles. Do not use cork or rubber stoppers. The indicators may change under long storage, especially if exposed to acid or alkaline fumes when the bottles are opened. The procedure for adjusting them, requiring the use of a pair of Taylor adjustment tubes, is as follows:

1. Place 0.5 ml of indicator in the larger tube.

2. Fill the smaller tube with distilled water.

3. Place the smaller tube inside the larger tube, thus causing the indicator to form a thin film between the two tubes.

4. Compare the colour of this thin film with the colour of the midmost colour standard on the slide for the indicator being tested. For illustration, suppose bromthymol blue indicator is being tested. Then its colour would be compared with the colour of the midmost standard, that is, the one having a pH of 6.8. If the colours are practically the same, the indicator is satisfactory for continued use without adjustment. If, however, the colour

of the indicator is noticeably different from that of the midvalue standard, then the indicator needs adjustment.

5. If the error is on the acid side (lower pH value than midvalue), add 0.1N NaOH *a drop at a time* to the stock solution, the solution being mixed and tested after each drop is added. If the error is on the basic side (higher pH value than the midvalue), add 0.1N HCl a drop at a time to the stock indicator solution, mix and retest.

It is important to ensure perfect adjustment of the indicators, otherwise they will be useless. Fortunately, adjustment is not needed if the indicators are protected and handled carefully. The best procedure is to transfer a small portion of an indicator solution from the large bottle to a small bottle for use in the pH test, so that the stock solution need not be handled so often.

Procedure for determining pH

1. Fill two tubes to the mark with the water to be tested and place in the outside compartments.
2. Wash the third tube with a portion of the sample and drain.
3. With the special pipette measure *exactly* 0.18 ml of the indicator solution selected as having the pH range anticipated to embrace the pH of the sample and place it in the third tube. If in doubt, use the bromthymol blue indicator, since many natural waters have pH values in its range.
4. With the second special pipette measure exactly 27 ml of the sample and add it to the third tube by placing the tip of the pipette on the bottom of the tube; mix it with the indicator by stirring with the pipette. *Never mix by inverting the tube with its opening covered with a finger, as this will contaminate the sample.*
5. Remove the excess of the sample from the tube with the large pipette until the water surface or meniscus is level with the line marked on the tube.
6. Place the third tube with sample in the middle compartment of the Analyzer.
7. Place in the Analyzer the Taylor slide corresponding to the indicator solution used.
8. Locate the Analyzer so that light passes down the tubes to the mirror below.
9. Move the slide until the colour of the image of one of the standards corresponds with that of the central image of the sample. Thus if a midrange of pH values was anticipated, so that bromthymol blue indicator was selected, the colour of the sample might, for instance, equal that of the 6.6 colour standard in the slide, indicating a pH of 6.6 for the sample.

A pH value above or below the range of a slide will produce colours equal to its highest or lowest standards. For this reason the indicator solutions have been selected to have overlapping ranges. If bromthymol blue indicator and slide were used for a sample with a pH of 5.8, for instance, the pH would appear to be 6.0, an indeterminate result with this indicator. Repeating the test with chlorphenol red indicator and slide would give the accurate pH of 5.8, well within the midrange of that indicator.

Interpretation

The use of the pH value and alkalinity of a sample in the control of corrosion is discussed in the next section, under Marble or Calcium Carbonate Stability Test; their use in the control of coagulation, under Coagulation and Flocculation or Jar Test, immediately following that.

Marble or calcium carbonate stability test

Purpose of the test

The corrosive action of water is reduced by the application of an alkali to increase the alkalinity and pH of the water. Lime is generally used for this purpose, to deposit a calcium carbonate protective film on the surface of the piping. If too much lime is added, an undue amount of deposit will occur; if too little is added, no film will be produced. It is impossible to compute the lime dose required for this purpose because there are so many influencing factors. Fortunately it is possible by the marble test to determine the alkalinity and pH which prevail at calcium carbonate equilibrium, where chemical forces are balanced and the tendency to deposit a film is counteracted by a tendency to dissolve a film of calcium carbonate. The lime dose then is selected to produce this alkalinity and pH.

The most convenient manner of making this test is with the Enslow stability indicator; the conventional marble test is inconvenient and time-consuming.

The Enslow stability indicator

The Enslow stability indicator functions automatically to give significant results whenever they are needed. It may be made as follows:

Select two glass or plastic tubes about 2 inches in diameter and 24 inches long, and secure rubber stoppers which fit into the ends of the tubes. One tube is nearly filled with *powdered* calcium carbonate, and glass wool is inserted in each end, after which the rubber stoppers are inserted in place. The second tube is filled with *granular* calcite (calcium carbonate), and glass wool and the rubber stoppers inserted in each end as before.

The rubber stoppers at the ends of each tube are perforated to permit the insertion of small glass or plastic tubing so that water being tested may be forced first through the tube containing powdered material and then through the second tube containing granular material. The small tubing is fitted with a stopcock to regulate the rate of flow of water through the larger tubes. In this way the water to be treated with alkali for corrosion prevention (corrosive water) is brought into contact with calcium carbonate, which dissolves until equilibrium is reached. The water flowing from the tubes is discharged into a sampling bottle or flask so that it may be tested for pH and alkalinity when desired.

Water should be allowed to *flow very slowly and continuously* through the tubes so as to ensure equilibrium being reached. In general, the rate of flow through the tubing should be adjusted so that the computed period of contact is 2 h or more.

Conventional marble test

Sampling. Sample should be representative of the supply to be treated and should be collected in glass-stoppered 250-ml bottle of Pyrex or similar resistant glass. Sample bottle should be completely filled so as to avoid aeration and loss of carbon dioxide, which is an important constituent of corrosive waters.

Equipment :

Mechanical shaking device when available
250-ml glass-stoppered bottle of Pyrex or other resistant glass
Precipitated calcium carbonate, chemically pure
Equipment for determination of alkalinity and pH value
Filter paper, Whatman No. 50
Filter funnel and flask

Procedure :

1. Fill a glass-stoppered bottle with the sample, without agitation.
2. Add approximately 0.25 g chemically pure precipitated calcium carbonate per 250 ml of sample and replace stopper without entrapment of air.
3. Mix by shaking at frequent intervals, or continuously on a mechanical shaker, for at least three hours to aid in the solution of the calcium carbonate and attainment of equilibrium.
4. Allow the sample to settle overnight and carefully remove a portion of the *clarified supernatant* for determination of its pH value and alkalinity.
5. Filter a portion of the supernatant sample through a " hard " filter paper (Whatman No. 50 or equivalent).
6. After discarding the first portion passing through the filter paper, determine total alkalinity to methyl orange of the remainder.
7. Determine pH value of the *unfiltered* supernatant, as passage of the sample through filter paper will alter its pH.

Results. The results are reported as the pH value and alkalinity prevailing at calcium carbonate saturation or equilibrium for the specific sample.

Interpretation

The increase observed in the total alkalinity and in the pH value indicates the corresponding values which should be obtained by addition of lime in the treatment of water to eliminate corrosive action and to provide for deposition of a protective coating. In the treatment of the supply the dosage of lime should be controlled to maintain the total alkalinity and the pH value at the concentration determined by these tests; 1 p.p.m. of hydrated lime will increase the alkalinity 1.35 p.p.m. The resulting change in pH value cannot be computed but will be indicated by the trial use of graduated doses of lime.

The relationship between the alkalinity, carbon dioxide and pH of water, when calcium carbonate equilibrium prevails, is shown by Figs. 13 and 14 (pages 201 and 203). For instance, equilibrium prevails with an alkalinity of 50 p.p.m. when the pH is 8.1 or when the carbon dioxide content is only about 0.75 p.p.m. (very near the end point for phenolphthalein indicator when carbon dioxide content is zero and pH is 8.3). If, for instance, the alkalinity of the untreated water is 50 p.p.m. and the pH is, say, 7, the water would be corrosive. The marble test would show the increase in alkalinity and pH resulting from the solution of some of the calcium carbonate, which, as already indicated, would be a measure of the amount of lime needed for corrosion prevention. If the alkalinity increased from 50 to 70 p.p.m., then 20/1.35 or 14.8 p.p.m. lime would be needed to raise the alkalinity from 50 to 70 p.p.m.

Coagulation and flocculation or jar test

Purpose of the test

The coagulation and flocculation of water is influenced by a number of interrelated variable factors such as temperature, turbidity, colour, pH, alkalinity, the period and degree of agitation during flocculation and the characteristics of the coagulant in use. Therefore the optimum dose of coagulant cannot be determined from the results of water analysis but must be based on experience with any given water; for an unknown water this is supplied by the coagulation and flocculation or jar tests under discussion. Since alum and lime are commonly used, the problem is to determine the most effective doses of these two chemicals for a specific water in the control of coagulation and flocculation at a filter plant, especially when water quality fluctuates rapidly.

Sampling

A large sample of the raw water under study should be collected so that all portions tested will be from the same sample. At least 25 litres of water should be available, to permit at least four series of tests of six portions each.

Equipment

Stirring or mixing device to provide controlled agitation equivalent in degree to plant-scale flocculators (usually provided by 30- to 100-r.p.m. stirring paddles)

Six glass jars or beakers holding 1 ½-2 litres

Glass funnels

Filter paper

100-ml pipette

Equipment for colour, turbidity, pH and alkalinity tests

Stock alum solution (1 g of the filter alum to be used at the plant dissolved in 1 litre distilled water)[1]

Stock lime suspension (1 g powdered hydrated lime mixed in 1 litre distilled water; should be mixed by shaking every time it is used)[1]

Laboratory stirring equipment for coagulation and flocculation or jar test. Laboratory stirring equipment may be purchased from laboratory-supply houses or may be locally made. Fig. 20 shows one such device, which includes a water motor as the source of power, but a small electric motor with speed-reducing gears may be used. The driven pulleys have their own bearings, and these pulleys are perforated at their centres for the vertical brass rods to which the paddles are attached. The brass rods have a brass disk attached to the upper end so that they may be raised vertically when the beaker is removed, even when the pulleys are moving. The weight of the rods, however, creates sufficient friction to cause the rods to revolve with the pulleys.

The area of the paddles should be not more than one-fourth the vertical cross-section of the litre of water in each beaker. Beakers of 1 ½-litre capacity should be used. The paddles should revolve at speeds in the range of 30-100 r.p.m., depending on the degree of agitation desired.

Procedure

1. Determine the colour, turbidity, pH and alkalinity of the raw water.
2. Measure 1-litre portions of the sample into each of the six jars or beakers and place them in the stirring equipment.
3. Start the motor of the stirrer.
4. Add as quickly as possible graded doses of alum and lime to the six portions being agitated (see Interpretation below for selection of doses).
5. Agitate for 20 min or for the period of time provided for flocculation at the plant. If flash-mixing is practised on a plant scale, operate the agitator at full speed for 1 min, followed by the selected degree of agitation.

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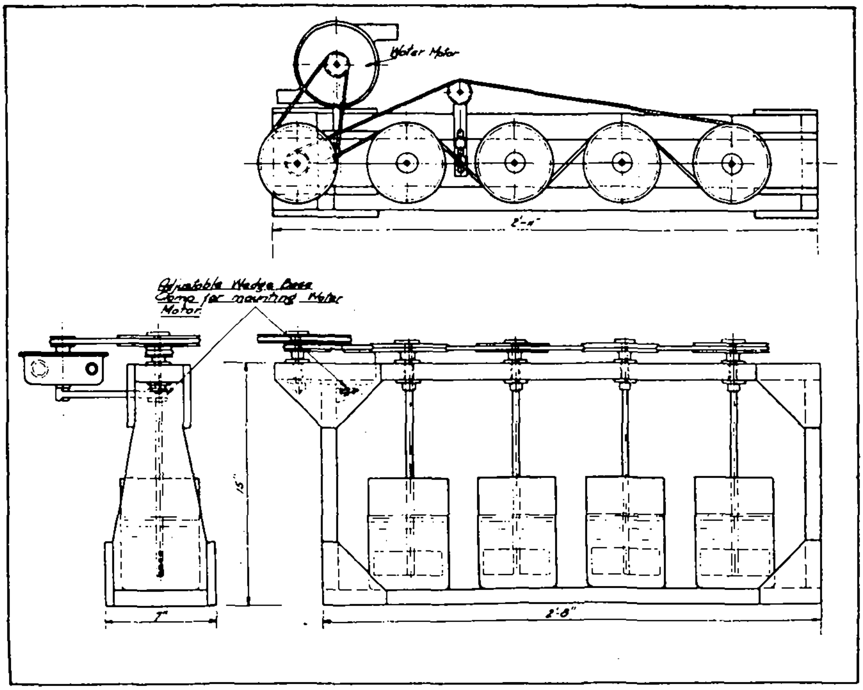
[1] 1 ml of this added to 1 litre water will give a dose of 1 mg/l, or 1 p.p.m.

FIG. 20
LABORATORY STIRRING EQUIPMENT FOR COAGULATION AND FLOCCULATION OR JAR TEST



From Cox (1952) p. 40 by permission of the New York State Department of Health.

6. Observe the time of the *first appearance* of visible floc in each of the six portions of the sample, and also the appearance, size and quantity of floc at the end of the agitation or flocculation period.

7. Let all portions settle for 1 h in the jars or beakers.

8. Observe the extent of sedimentation and the character of the unsettled floc remaining in the six portions.[1]

9. Determine the colour, turbidity, pH and alkalinity of the *supernatant* settled water above the settled floc in each portion, using a large pipette to withdraw the water without agitating the floc.

10. If none of the test portions after 1 h of sedimentation in step 7 has a colour below 10 to 20 units or a turbidity below 5 to 10 units, discard the six portions tested and start another series of six portions of the sample

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[1] A supplement to step 8 is the determination of the volume of floc produced by the optimum chemical doses by repeating steps 2 to 6 with portions of the sample to which the optimum doses have been added, then letting 1 litre of each of the flocculated waters settle in Imhoff cones. Read the volume of sediment at the end of 1 h and 4 h.

with another range of chemical doses. Repeat, if necessary, until several portions are shown to be effectively coagulated.

11. When effective coagulation is secured in one or two portions, filter the supernatant settled water from each effectively coagulated portion through the filter paper placed in the funnels.

12. Determine the colour, turbidity and alkalinity of the filtered water. The filter paper influences the pH of the filtered water, so determine the pH of the settled water.

Interpretation

The test portion giving the best flocculation and reduction of colour and turbidity by sedimentation and filtration will indicate the optimum doses of alum and lime for the specific sample under test.

Only preliminary trials will disclose whether alum alone or both alum and lime are needed. The tests can then be planned to show the influence of graded doses of the chemicals. Other tests, with a constant selected dose but different degrees of agitation, will disclose the optimum degree of agitation.

Table 34 shows typical results of the coagulation and flocculation of a *turbid* water where lime or an equivalent alkali is not needed, since the natural alkalinity of the raw water is sufficient to react with the alum to

TABLE 34

TYPICAL RESULTS OF COAGULATION AND FLOCCULATION TEST OF A TURBID ALKALINE WATER (NO LIME NEEDED)

Jar no.	Alum dose, p.p.m.	Lime dose, p.p.m.	Result of flocculation	Analysis of settled water			
				Colour	Turbidity	pH	Alkalinity, p.p.m.
—	0	0	Raw water	10	100	7.6	125
1	10	0	None	10	90	7.4	120
2	20	0	Poor	8	50	7.1	115
3	30	0	Good[a]	5	8	6.8	110
4	40	0	Very good[a]	5	5	6.6	105
5	50	0	Good	5	10	6.4	100
6	60	0	Poor	7	30	6.2	95

[a] Satisfactory coagulation and flocculation of portions 3 and 4 would justify filtering the coagulated, flocculated and settled water through filter paper, the following typical results supporting the conclusion that the 30-p.p.m. alum dose was the lowest effective dose:

	Visible colour	*Turbidity*	*pH*	*Alkalinity, p.p.m.*
Jar No. 3:	0	0	6.9	110
Jar No. 4:	0	0	6.7	105

TABLE 35

TYPICAL RESULTS OF COAGULATION AND FLOCCULATION TEST OF A COLOURED WATER OF LOW ALKALINITY (LIME NEEDED FOR pH CORRECTION)

Jar no.	Alum dose, p.p.m.	Lime dose, p.p.m.	Result of flocculation	Analysis of settled water			
				Colour	Turbidity	pH	Alkalinity, p.p.m.
—	0	0	Raw water	80	10	5.6	15
1	20	0	Poor	50	10	4.6	6
2	20	3	Fair	30	8	5.0	10
3	20	6	Good	10[a]	5	5.4	13
4	30	8	Good	10[a]	5	5.4	13
5	30	10	Fair	25	8	5.8	15
6	30	12	Poor	70	10	6.2	18

[a] The colour of 10 units of portions 3 and 4 is due in part to the colour floc remaining in the settled water. Filtration through filter paper would reduce this to 5 units or less, and this would confirm the effectiveness of 20 p.p.m. alum plus 6 p.p.m. lime in coagulating the colour at pH 5.4.

form aluminium hydroxide floc. Table 35 shows typical results of the coagulation and flocculation of a *coloured* water of low alkalinity, where the " colour floc " is formed in the range of low pH values, requiring little added lime for pH correction. Any excess of lime will prevent floc formation, as shown by the hypothetical test portions 5 and 6 in Table 35. Alum alone proved to be ineffective, as shown by test portion 1. Coagulation is shown by these typical results to occur at the pH value of 5.4, characteristic of coloured waters of low alkalinity and mineral content. The dose of lime is needed to establish the optimum pH value of 5.4 used as an example and to prevent the pH from being lowered by the alum to a point where coagulation would be hindered. In fact, the typical results for coloured waters of low alkalinity show that optimum coagulation occurs at a pH of 5.4—less than that of the raw water, pH 5.6.

Water coagulated at a pH of 5.4 will be corrosive. The additional lime needed for corrosion prevention should be added to the *filtered* water, otherwise the lime will prevent the formation of the colour floc, as shown by test portion 6 in Table 35. In other words, the marble test would be made on the filtered water to indicate the pH and alkalinity prevailing at calcium carbonate equilibrium. The separate lime dose needed for this purpose can then be computed and can be checked by the resulting pH and alkalinity values.

Softening-reactions test

The laboratory control of lime-soda softening reactions consists of the separate tests for methyl orange and phenolphthalein alkalinities, pH, hardness, magnesium, and the computation of carbon dioxide content, using the flocculation test as modified to observe the formation of calcium carbonate floc from the softening reactions.

Equipment

Laboratory flocculator
1-litre beaker
Two 10-ml pipettes
Glass funnel
Filter paper
Stock lime suspension (10 g hydrated lime added to 1 litre water previously boiled and cooled to drive off carbon dioxide)[1]
Stock soda ash solution (10 g soda ash added to 1 litre water previously boiled and cooled)[1]

Procedure

1. Place 1 litre raw water to be softened in beaker of flocculator.
2. Add the doses of lime and soda ash calculated to remove carbon dioxide, reduce bicarbonate alkalinity and precipitate magnesium (see example in Tables 21, 22 and 23, pages 229 and 231).
3. Mix for 30 min, using intermediate degree of agitation.
4. Settle for 30 min in beaker.
5. Siphon off about one-half the clear water in the upper part of the beaker, being careful not be disturb the settled floc, and warm to 50°C.
6. Filter this portion through laboratory filter paper.
7. Analyse the filtrate for methyl orange and phenolphthalein alkalinities, hardness and magnesium and compute the content of carbon dioxide.
8. Test an unfiltered portion for pH.
9. Proceed as noted on page 230. Refer to *Coagulation*, page 225, in case a coagulant also is needed to secure adequate flocculation.

Residual chlorine

Orthotolidine-arsenite test

Purpose of the test. Chlorine when added to water reacts with organic matter and other substances which destroy its disinfecting power. Therefore it is necessary to add sufficient chlorine to react with all the various substances present and still leave an excess or residual, if bacteria are to be destroyed. This residual chlorine may be present in the *free* state, which has a very rapid disinfecting power; it may be *combined* with ammonia to form less active chloramine; or it may be absorbed by organic matter

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[1] 1 ml of this, added to 1 litre water, will give a dose of 10 p.p.m.

to form relatively inactive chloro-organic compounds with little or no disinfecting power. A residual of free available chlorine may be secured by the use of a sufficient amount of chlorine to oxidize the organic matter or by the addition of relatively small amounts of chlorine to water free from appreciable quantities of ammonia or organic matter. In the control of chlorination it is important to known whether the residual chlorine is present as free chlorine or as the less active combined chlorine.

Two tests may be used for this purpose: the Palin procedure, using diethyl-*p*-phenylene diamine reagent with soluble tablets, manufactured by Tintometer Ltd., Salisbury, England; and the orthotolidine-arsenite test (OTA). The former is described in *Approved Methods for Physical and Chemical Examination of Water*, published by the Institution of Water Engineers of England. The latter is used in this monograph to unify the control procedure discussed in Chapter 8.

The orthotolidine-arsenite test measures the free-residual chlorine employed for more effective disinfection or for the destruction of tastes and odours; it measures the combined-residual chlorine when chloramination is practised or when ammonia is naturally present in the water; and in addition it measures free and combined residuals when both are present as the result of partial destruction of organic matter by chlorine. It also measures the error due to the presence of interfering substances, such as manganese and nitrites.

When orthotolidine reagent is added to water containing chorine, a greenish-yellow colour develops, the density of which is proportional to the amount of residual chlorine present. Free-residual chlorine reacts with orthotolidine practically instantaneously, requiring less than 15 s for development of the colour. Combined-residual chlorine reacts relatively slowly with orthotolidine, requiring 5 min at 70°F for full colour development. Thus the presence or absence of an immediate flash colour indicates the presence or absence of free-residual chlorine. This flash colour can be estimated quantitatively when a weak reducing agent such as sodium arsenite is added to the sample, in a tube marked A, immediately after the addition of the orthotolidine. The sodium arsenite neutralizes combined chlorine in that portion of the sample in tube A before it can react with the orthotolidine reagent. The flash colour is not affected by the sodium arsenite reagent and can be read by comparison with permanent standards at any time within 5 min, unlike the old flash test, which had to be read within a matter of seconds.

As stated, the orthotolidine reagent also reacts with nitrites and manganese to form a false colour simulating that due to residual chlorine. Furthermore, the permanent colour standards furnished with the several kits used in this test, including the Taylor Water Analyzer, have been prepared to match or simulate the colour resulting from the reaction between residual chlorine and orthotolidine in clear water free from natural colour and turbidity. The errors due to interfering substances, natural colour and turbidity can be compensated for by placing another portion of the sample of water in a tube marked B, to which sodium arsenite is added *first* to neutralize the residual chlorine, after which the orthotolidine is added to react with any interfering substances which may be present and may form a false colour. The portion in tube A also will be influenced by the same false colour, but in addition it also has colour due to free-residual chlorine. Therefore the difference in the degree of colour in tubes A and B (A—B) will indicate the concentration of free-residual chlorine. Both tubes also will be influenced equally by natural colour and turbidity. Therefore the procedure to be described consists in part of placing the tubes in the kits in such a manner that the *difference* in the degree of colour is apparent, i.e., the colour due to free residual chlorine *alone*, which is compared with the colour standards to determine its value in parts per million.

The orthotolidine reagent is also added to a third portion of the sample of water placed in a tube marked OT and allowed to react for 5 min. The resulting colour is due to reactions between the orthotolidine reagent and (1) free-residual chlorine, (2) combined-residual chlorine, and (3) interfering substances. As the test in tube A described above gives a colour due to items (1) and (3), the difference in colour between tubes OT and A (OT—A) will indicate the concentration of combined residual chlorine.

Sampling. Samples must be collected in clean bottles, and the tubes used in the test kit must be clean. Otherwise any dirt on the glassware will react with the chlorine, giving low results. If possible, samples should be collected at points where the chlorine has been in contact with the water for at least 10 min. If this cannot be done, the sample should be allowed to stand sufficient time to make the total contact time 10 min.

Equipment :

Orthotolidine reagent
0.5% arsenite reagent (0.5 g sodium arsenite in 100 ml distilled water)
Taylor Water Analyzer
Taylor residual-chlorine standard slide 1300 Fl
Four 150-mm tubes

Procedure :

1. Let samples stand for required 10-min contact period, unless a longer time has elapsed between application of chlorine to the flowing water and the time required for the water to reach the sampling point.
2. Label the four tubes OT, A, B_1 and B_2.
3. To tube marked OT add 1 ml orthotolidine solution, fill to the mark with the sample being tested and mix. Set aside to be compared with standards in 5 min.

4. To tube marked A add 1 ml orthotolidine, fill to the mark with the sample, and then *quickly* add 1 ml sodium arsenite solution and mix. Set aside to be compared with standards after step 5.

5. To tubes B_1 and B_2 add 1 ml of sodium arsenite each, fill to the mark with the sample, then add 1 ml orthotolidine and mix. Place these two tubes in the outside compartments of the Analyzer. Any colour developing in these tubes will be due to interfering substances, as the chlorine was removed by the arsenite, which was added *before* the orthotolidine. Therefore tubes B_1 and B_2 provide an optical compensation for an equal degree of false colour in the central tube A of step 6, or tube OT of step 7. This also compensates for errors due to turbidity and natural colour of the water.

6. Place tube A in the middle compartment and compare the three images produced in the mirror by light passing down through the three tubes. Move the residual-chlorine slide until the central image matches one of the outside images. Record the value of the matching standard as the A value for *free* or *active* residual chlorine.

7. In exactly 5 min place the tube marked OT (step 3) in the central compartment and compared the colour of the three images, as in step 6. Record the value of the matching standard as the OT value for *total residual chlorine*.

8. Subtract value A from value OT to secure concentration of *combined-residual* chlorine.

9. If the numerical value of the false residual is desired, place tube B_1 in the central compartment and add a portion of the sample, *without any reagents*, to the two outer tubes, compare the images and move the slide until a colour match is obtained. Usually, however, any false residual is moderate and has a value which does not precisely match any of the standards. The optical compensating procedure, which is more accurate, should be followed.

Precautions. Some of the combined-residual chlorine will react with the orthotolidine reagent within the period of 15 s or less before the sodium arsenite is added in step 4, so that this portion would be incorrectly recorded as free-residual chlorine. This difficulty may be minimized by cooling the sample to near freezing point by placing the sample bottle in ice water *before* the test is made; the low temperature greatly reduces the speed of reaction of the combined-residual chlorine without affecting the activity of the free-residual chlorine.

Furthermore, some of the manganese, nitrites and other interfering substances may react more slowly than shown by the colour produced in 15 s, so tube B should be examined again at the end of 5 min. This is done as an incidental part of step 7.

Results. Assume that the A value is 0.15 p.p.m. and the OT value 0.4 p.p.m. Then the results are as follows:

OT = 0.4 p.p.m. total residual chlorine

A = 0.15 p.p.m. free-residual chlorine

OT—A = 0.4—0.15 = 0.25 p.p.m. combined-residual chlorine

If free-residual chlorination, or breakpoint chlorination, is being practised and these results are secured, the dose of chlorine should be increased to oxidize the ammonia and any other substances forming the combined-residual chlorine, so that the resulting residual will all be free-residual chlorine. For instance, an increase in the dose might give an OT value of 0.7 p.p.m. and an A value of 0.7 p.p.m. Then:

OT = 0.7 p.p.m. total residual chlorine

A = 0.7 p.p.m. free-residual chlorine

OT—A = 0 combined-residual chlorine

Conventional orthotolidine test

The frequent use of the orthotolidine-arsenite test in the control of chlorination will provide data as to whether free- or combined-residual chlorine is usually produced and whether a false colour is produced by interfering substances. The conventional orthotolidine test, exactly like the OT portion of the more elaborate test, may then be used for routine plant control.

The shorter test is as follows:

1. Let sample stand 10 min.
2. Fill outer two tubes with sample (no reagent).
3. Add 1 ml orthotolidine solution to a tube and fill to the mark with the sample.
4. Place tube in the centre compartment of the Analyzer.
5. Immediately compare any flash colour with the standards.
6. Let stand 5 min. and again compare with standards.

This procedure compensates for errors due to natural colour and turbidity of the sample and gives an approximate idea of the concentration of free-residual chlorine (flash test) but does not distinguish false colour due to interfering substances such as manganese.

Interpretation. The disinfecting power of chlorine depends on the form of residual chlorine present, the contact time, the temperature, and the pH value of the water. If the pH value is less than 8, 0.2 p.p.m. of free-residual chlorine will destroy bacteria in a 10-min. contact period at all temperatures. To accomplish the same results with combined chlorine, a residual of 1.0 p.p.m. at pH 6, 1.5 p.p.m. at pH 7, and 1.8 p.p.m. at pH 8 must be maintained with a contact period of 60 min, and this residual must be varied as the temperature of the treated water varies.

Drop-dilution method for high concentrations of residual chlorine

Purpose of the test. Concentrations of residual chlorine well above the range of the conventional orthotolidine test or the orthotolidine-arsenite test should prevail when water mains, reservoirs or tanks are disinfected after construction or repairs. The drop-dilution method is reasonably accurate and uses the same equipment as the regular test.

Equipment :

Chlorine comparator or Taylor Analyzer
Orthotolidine reagent
Medicine dropper (20 drops per millilitre)
Distilled water

Procedure :

1. Determine the capacity of the tube of the comparator in which the sample is to be tested (usually about 5, 10 or 15 ml, but about 30 ml for the larger tubes of the Taylor Analyzer).
2. Add 1.5 ml orthotolidine reagent to the centre tube of the Taylor Analyzer (10-15 drops for smaller tubes of other comparators).
3. Fill to the mark with distilled water.
4. Fill the two compensating tubes with distilled water (no orthotolidine).
5. Add to the centre tube with the dropper *one drop* of the sample being examined and mix.
6. Compare the colour with that of the standards. If no colour is apparent, discard the contents of the centre tube, rinse, and start again at step 2, but use *two drops* of the sample in step 5. If no colour is apparent still, discard the contents of the tube and repeat procedure using *three drops* of the sample, then *four drops*, and so on until colour is produced.
7. Record the value of the matching colour standard (which is the concentration of residual chlorine in the mixture) and the number of drops of the sample used to produce the colour, also the measured capacity of the tube utilized.

Computations. The concentration of residual chlorine in the original sample is computed as follows:

$$\text{Residual chlorine, p.p.m.} = \frac{\text{capacity of tube (ml)} \times \text{residual chlorine of mixture} \times 20}{\text{number of drops}^{1} \text{ of sample}}$$

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[1] Using medicine dropper with tip giving 20 drops/ml.

Example: Assume that the tube used has an actual capacity of 28 ml, that the residual chlorine as measured in the mixture of sample and distilled water was 0.4 p.p.m., and that 5 drops of the sample gave this result. Then:

$$\frac{28 \times 0.4 \times 20}{5} = 45 \text{ p.p.m. residual chlorine in sample}$$

Chlorine demand

Purpose of the test

The chlorine demand of a water is the amount of chlorine which is needed to react with organic matter and other substances in a water; it is therefore the difference between the amount of chlorine added and the amount of residual remaining after a given contact time. The demand of each water varies with the amount of chlorine added, the time of contact, and the temperature of the water. As larger amounts of chlorine are added the chlorine demand of the water increases, because the chemical reactions become correspondingly more active and intense, thus increasing the amount of chlorine that reacts.

Sampling

Collect in clean bottle about ½ gal water to be tested.

Equipment

Taylor Water Analyzer and three tubes
Taylor chlorine standard slide 1300 Fl
Six clean 250-ml bottles

Reagents

Ortholidine reagent
Arsenite solution
Chlorine solution, standardized, 100 p.p.m.

PREPARATION OF CHLORINE SOLUTIONS. Chlorine solutions may be prepared from sodium hypochlorite or calcium hypochlorite or by the use of a gas chlorinator. The alkalinity of hypochlorite solution influences the results, so more comparable results are secured when chlorine solution from a gas chlorinator is used to determine the chlorine demand of a water chlorinated with such a unit. Naturally, hypochlorite solutions give satisfactory results also when such solutions are used to disinfect the water under study.

A 100-p.p.m. chlorine solution may be prepared in one of the following three ways:

1. Dilute 2 ml fresh laundry bleach (sodium hypochlorite) in 1 litre distilled water.

2. *Dilute 10 ml chlorine solution from a chlorine-gas chlorinator with* 990 ml distilled water to give 1 litre of diluted solution.

3. Dissolve 0.06 g high-test calcium hypochlorite (70% strength) in 1 litre distilled water.

STANDARDIZATION OF CHLORINE SOLUTIONS. These solutions must be standardized to secure their correct concentration and they should be re-standardized *each day* they are used, since they lose strength rapidly. Two procedures for standardization are as follows:

Procedure 1. To 150 ml distilled water add 1-2 g potassium iodide crystals and dissolve, then add 50 ml of the chlorine solution being tested and 1 ml glacial acetic acid and allow 5 min for liberation of iodine, then titrate with 0.025N sodium thiosulfate, using starch solution as indicator in the usual way, until the blue colour has disappeared. Since 1 ml 0.025N sodium thiosultate is equivalent to 0.886 mg chlorine, then

$$\text{mg chlorine per ml solution} = \frac{\text{ml thiosulfate} \times 0.886}{50}$$

For example, suppose 6 ml thiosulfate solution was needed to react with the chlorine in 50 ml of solution. Then the chlorine concentration would be 6 × 0.886/50 = 0.106 mg/ml, or 106 mg/l, or 106 p.p.m. This concentration at 106 p.p.m. may be modified to the desired value of 100 p.p.m. by adding 6 ml distilled water to each 100 ml of the solution. Then each millilitre of the adjusted solution when added to 100 ml of sample will give a dose of 1 p.p.m.

Procedure 2. The above procedure requires the use of 4 chemicals. A less precise but simple procedure for use in the field is as follows:

1. Add 2 drops of laundry bleach (5% strength) to 100 ml distilled water to provide a dilute chlorine solution having a concentration of about 100 p.p.m. chlorine.

2. Add 1 ml of the dilute chlorine solution to 200 ml distilled water and let stand for 1 h for the chlorine to react with any impurities in the water and thus satisfy its chlorine demand.

3. Boil the chlorinated distilled water for about 5 min to expel all the remaining chlorine and cool.

4. Check the water for residual chlorine and boil longer if necessary to expel all chlorine. This provides distilled water of zero chlorine demand.

5. Add 0.5 ml of the dilute chlorine solution to 100 ml of this cooled distilled water of zero chlorine demand.

6. Test immediately for residual chlorine in the usual manner. This will disclose the amount of chlorine in each 0.5 ml of the dilute chlorine solution to be used in the procedure outlined below in the test for chlorine demand.

Calculations for Procedure 2. Steps 1 to 3 should satisfy any chlorine demand of the distilled water and also expel by heat the remaining residual chlorine. The chlorination of this water of zero chlorine demand should produce a *residual equal to the dose.* For instance, suppose step 6 discloses that 0.5 ml of the chlorine solution in 100 ml water gives a concentration of 0.7 p.p.m. residual chlorine instead of 0.5 p.p.m. Then the solution prepared in step 1 contains 0.7 × 100/0.5 or 140 p.p.m. chlorine rather than 100 p.p.m.

In this example, the series of doses suggested in the procedure for the chlorine-demand test would be reduced to 5/7 of the noted values to give 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 p.p.m. *actual* doses of chlorine to the series of six 100-ml portions of the sample being tested. The best procedure would be to dilute the overstrong solution with distilled water of zero chlorine demand to provide a final strength of solution of 100 p.p.m.; that is, in the example cited, 40 ml water would be added to 100 ml solution of 140-p.p.m. strength to give 140 ml of weaker solution of 100-p.p.m. strength.

Procedure for chlorine-demand test

1. Measure 100-ml portions of the sample into each of the six bottles.
2. Add 0.5 ml chlorine solution to the first bottle, 1.0 ml to the second and 1.5, 2.0, 2.5 and 3.0 ml, respectively, to the others, with thorough mixing, thus providing doses of 0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 p.p.m.
3. Let stand 10 min or a time equal to the contact time at the point of control.
4. At the end of this time determine the amount and kind of residual chlorine in each portion by the orthotolidine-arsenite test (see Residual Chlorine, page 317). The lowest dose should show no residual and the highest should be well above the amount required in plant operation.
5. If the amounts of chlorine solution added do not produce these results, different increments should be chosen and added to fresh portions until the desired results are obtained.

Results

The chlorine demand equals the parts per million of chlorine added minus the parts per million of residual chlorine at the end of a specific contact time as read by the orthotolidine-arsenite test. This will be different for various doses of chlorine, so both the dose and the resulting concentration of residual chlorine should be recorded.

Interpretation

The chlorine-demand test indicates the amount of chlorine that will be necessary to produce given residuals in a particular water after a definite period of contact. Since organic matter in water absorbs chlorine, water having colour or turbidity will have a high chlorine demand.

Iron

Purpose of the test

The presence of more than 0.3 p.p.m. iron in a water will result in the staining of plumbing fixtures and laundry, and even smaller amounts may be troublesome. The determination of the iron content of a water, therefore, is essential in determining the suitability of a proposed supply and in the control of iron-removal treatment. The test is especially valuable in the appraisal of ground waters, which are more likely to contain iron than surface waters, except when the latter are stored in deep reservoirs flooding iron-bearing soils.

The test under discussion discloses the total concentration of iron in a sample, that is, the sum of oxidized, or ferric, iron and the reduced, or ferrous, iron. Ferrous iron is soluble, hence its concentration can be determined by testing a sample of water after filtration through laboratory filter paper. The difference between the total iron content of the unfiltered sample and that of the filtered sample is the content of iron in the ferric state.

Equipment

Taylor Water Analyzer
Three 150-mm tubes
Taylor slide of iron standards 1300G
200-ml beaker (or Erlenmeyer flask, or evaporating dish)
50-ml volumetric flask with stopper
100-ml volumetric flask
10-ml pipette
50-ml graduate

Reagents

Dilute hydrochloric acid (approximately 6N) (one volume of chemically pure concentrated hydrochloric acid diluted with an equal volume of distilled water)

Thiocyanate solution (2 g potassium thiocyanate dissolved in about 80 ml distilled water and diluted to 100 ml) (*Caution: extremely poisonous!*)

Permanganate solution (0.6 g potassium permanganate dissolved in about 80 ml distilled water and diluted to 100 ml)

Procedures

The determination of the iron content of water is based on the use of potassium permanganate to oxidize to the ferric state any ferrous iron that may be present. Organic matter present in many surface waters, especially highly coloured waters from swamps, interferes with the test unless precautions are taken. Therefore four procedures are given below: the first for use in the field where heat is not available; the second, where heat is available to make the test more precise for water of low organic content; the third, for accurate results with waters of high organic content; and the fourth, for waters containing silt, organic matter and high concentrations of iron.

The *normal* amount of sample to be used with the Taylor Analyzer is 50 ml. If, however, the sample contains silt and organic matter, the content of iron may be greater than 3 p.p.m., the upper limit of the standards, so a smaller amount would be used, such as 10 ml, diluted to 50 ml.

Procedure 1 (for use in field where heat is not available; results approximate):

1. Measure 50 ml of sample into beaker.
2. Add 5 ml dilute hydrochloric acid.
3. Add potassium permanganate *drop by drop* until faint pink colour persists for at least 5 min.
4. Add 5 ml potassium thiocyanate solution and mix well.
5. Fill one of the tubes of the Analyzer to the mark with the sample so treated and place it in the central compartment of the Analyzer.
6. Fill two other tubes to the mark with a portion of the *unmodified* sample so as to compensate for the natural colour and turbidity of the sample, which remain unchanged during this prodecure, and place the tubes in the outer compartments of the Analyzer.
7. Place the slide 1300G in the Analyzer and move it so as to secure a match between the colour of the central image (sample) and the image of one of the standards.
8. Read the iron content in parts per million from the value stamped on the matching standard.

Procedure 2 (when heat is available):

1. Measure 50 ml of sample into a beaker.
2. Add 5 ml dilute hydrochloric acid.
3. Boil until about 10 ml of sample has evaporated.
4. Cool.
5. Add potassium permanganate *drop by drop* until a pink colour persists for at least 5 min.
6. Pour into the 50-ml volumetric flask and add distilled water to the 50-ml mark to compensate for the loss by evaporation.
7. Add 5 ml potassium thiocyanate solution and mix.
8. Fill one of the tubes of the Analyzer to the 150-mm mark with the sample so treated.

9. Fill two other tubes with portions of the sample to provide compensation for the natural colour and turbidity of the sample.

10. Place slide 1300G in the Analyzer and secure a match between the colour of the image of the sample and one of the standards.

11. Read the iron content of the sample from the value of the matching standard.

Procedure 3 (for more precise determination of iron content; suitable for use with water containing appreciable concentrations of organic matter):

1. Measure 50 ml of sample into beaker.
2. Evaporate to dryness.
3. Cool the dry residue in the beaker and add 5 ml dilute hydrochloric acid.
4. Warm several minutes and wash the inner surface of the beaker with the hot acid.
5. Add 10 ml distilled water and warm several minutes.
6. Cool, then add a drop or two of potassium permanganate solution and mix. If the pink colour does not persist for at least 5 min, add more, *drop by drop*, until a faint colour persists.
7. Transfer to the 50-ml flask, washing all iron from the beaker with some distilled water.
8. Dilute to 50-ml mark with distilled water.
9. Add 5 ml potassium thiocyanate and mix well.
10. Fill one tube of the Analyzer to the mark with the sample so treated.
11. Fill two other tubes with *distilled water* and place them in the outer compartments of the Analyzer. (Distilled water is used in the outer tubes, because all natural colour and turbidity of the sample have been precipitated by evaporation and distilled water was used with clear acid to redissolve the residue.)
12. Compare the colour of the images in the usual manner to secure a match with that of one of the standards, and note its value as the concentration of iron in the sample.

Procedure 4 (with waters containing organic matter, silt and/or a high concentration of iron):

1. Estimate the expected range in the concentration of iron in the sample. If this estimate is over 3 p.p.m., select a volume of the sample less than 50 ml, say, 20 ml.
2. Measure 20 ml of sample into an evaporating dish.
3. Add 5 to 10 ml of *concentrated hydrochloric acid* (iron-free).
4. Evaporate to dryness.

5. Ignite by heating until the black, charred residue becomes greyish-white ash, *but do not heat to a red glow*, otherwise the residue will be difficult to dissolve.

6. Follow steps 3 to 12 of the third procedure, above.

7. Multiply the iron content of the portion of the sample examined by the dilution factor (50 ml/ml of sample examined = 50/20 = 2.5 in this example) to secure the iron content of the original sample. For instance, suppose the 20-ml portion of the sample contained 2.2 mg/l of iron. Then 2.2 × 2.5 = 5.5 mg/l or 5.5 p.p.m. is the iron content of the sample.

Manganese

Purpose of the test

Manganese is less frequently present than iron in natural water, but it may occur either alone or associated with iron in ground waters or waters from deep reservoirs flooding manganese-bearing soils. It will produce a dark-purple or black stain on laundry or plumbing fixtures. The critical concentrations depend on the chemical state of the mineral; manganous or reduced manganese is soluble and not troublesome until it is precipitated by oxidization. The content of manganese should not exceed 0.3 p.p.m. Manganese-removal treatment is controlled by this test.

The test discloses the total manganese content of samples of water. Manganese in solution (manganous) may be determined by testing a portion of the sample which has been filtered through filter paper to remove the insoluble manganese (manganic) in suspension.

Equipment

Taylor Water Analyzer
Taylor slide 1300H containing manganese standards
200-ml beaker
100-ml volumetric flask
10-ml pipette
Whatman No. 40 filter paper

Reagents

Dilute nitric acid (100 ml concentrated nitric acid diluted with 100 ml distilled water)

Silver nitrate solution (2 g silver nitrate dissolved in distilled water and diluted to 100 ml)

Ammonium persulfate or potassium periodate, 50 g of crystals, free from chlorides (0.5-g portions of these crystals may be weighed in advance and placed in small paper envelopes for convenient use)

Procedure

1. Measure 100 ml of the sample with the volumetric flask and pour into the Erlenmeyer flask.
2. Add 2 ml dilute nitric acid.
3. Evaporate by boiling to a volume of about 50 ml.
4. Add 5 ml silver nitrate solution to precipitate the chloride content of the water, which otherwise would interfere with the test. (An excess of 1 ml silver nitrate is needed.)
5. Mix and heat to coagulate the silver chloride, then filter through filter paper, washing the precipitate with a small amount of distilled water and collecting the filtrate in a beaker.
6. Add about 0.5 g ammonium persulfate crystals to the filtrate and warm to 80°-90°C until the maximum colour develops (about 10 min).
7. Cool and pour the coloured filtrate into a 100-ml volumetric flask.
8. Dilute to the 100-ml mark with distilled water. Mix well.
9. Fill the central tube of the Analyzer to the mark with the sample so treated.
10. Fill the two outside tubes with distilled water.
11. Place the manganese standards slide 1300H in the Analyzer and move it to secure a match in the colour of the images in the mirror, noting the content of manganese in parts per million from the value of the matching standard.
12. If the colour is deeper than that of any of the standards (over 3 p.p.m. manganese), dilute a portion of the original sample with distilled water to bring the manganese content of the diluted sample to a value less than 3 p.p.m. For instance, if 50 ml of the sample is diluted to 100 ml with 50 ml distilled water and the test as outlined discloses the manganese content of the mixture to be, say, 1.8 p.p.m., then the manganese content of the sample is $2 \times 1.8 = 3.6$ p.p.m.

Interpretation

The significance of the concentration of manganese found in any given sample is determined by the character of the water and the physical state of the manganese (in suspension or in solution). Ordinarily, manganous bicarbonate found in ground water is readily oxidized at pH values over 7, so the permissible concentration is 0.3 p.p.m. or less, to prevent staining. Manganese may be chemically combined with organic matter, as with waters from deep reservoirs, and not readily oxidized at pH values of natural water (that is, without lime treatment), so the concentration of such soluble manganese may be as high as 0.75 p.p.m. without being visible; the increase in pH when alkaline soaps are used in laundering clothes, however, will precipitate the manganese and cause staining.

Fluorides

Purpose of the test

The test for fluorides indicates the content of the element in natural waters and whether the amount is below or above the critical value of 1.5 p.p.m. Above this amount it will cause mottled tooth enamel, that is, unsightly staining of the teeth, among children drinking the water for an appreciable period of time.

The test will also show whether a natural water has a fluoride content between 0.6 and 1.2 p.p.m., which is the range in which the fluoride serves as a nutrient mineral which leads to the development of tooth enamel most resistant to decay when the water is consumed by children during the period when their teeth are developing.

Finally, the test serves to control the fluoridation of public water supplies to which a fluoride compound is added to compensate for a deficiency of natural fluorides and to secure a concentration in the optimum range of 0.6-1.2 p.p.m. The selected value is determined by estimating the per capita consumption of drinking water and the dose required to give 1.5 mg fluoride ion per capita per day. For instance, if the consumption of water is 2 litres per capita per day, the dose would be 0.75 mg/l, or 0.75 p.p.m. fluoride ion, to give the desired 1.5 mg fluoride ion in the 2 litres consumed on an average by each person each day. So far the actual dose used in the tropics has been 0.8 p.p.m. fluoride ion, and that used in temperate climates has been 1.0 p.p.m.

Notes on the test

Unfortunately the test for fluorides (fluoride ion) is based on reactions which produce a blend of two colours, one of which *decreases* in proportion to the content of fluorides in the sample, so the comparison between the blended colours of the sample and the standards is more difficult than with the other colorimetric tests described in this monograph, in which the intensity of the colour development is proportional to the concentration of the element being tested for (e.g., the test for iron).

Certain substances in natural or treated waters interfere with this test by either decreasing or increasing the depth of colour resulting from the reactions and thus decreasing or increasing the *apparent* fluoride content of the sample. Procedures have been developed for eliminating these errors, and these will be discussed later, following the standard test for fluorides.

Equipment (for use in the field)

Taylor Water Analyzer
Taylor slide of fluoride standards, Scott-Sanchis method, 1300SS
3 Taylor tubes for Analyzer, 250-mm depth to mark [1]

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[1] This is the only test with the Taylor Water Analyzer in which tubes marked at 250-mm depth rather than 150-mm depth are used.

10-ml pipette
50-ml measuring cylinder
Erlenmeyer flask

Reagents

The *mixed* Scott-Sanchis reagent cannot be stored for more than two months. The zirconium, alizarin and acid solutions used in preparing the mixed reagent may be stored *separately* for at least a year. These more permanent solutions are prepared as follows:

1. Dissolve 0.30 g zirconium oxychloride in 50 ml distilled water. The solution should be perfectly clear. Store in refrigerator.
2. Dissolve 0.07 g sodium alizarin monosulfonate (alizarin red S) in 50 ml distilled water. Store in refrigerator.
3. 2.7N hydrochloric acid solution. Dilute 112 ml concentrated hydrochloric acid (sp. gr. 1.19) to 500 ml with distilled water.
4. 2.7N sulfuric acid solution. Dilute 37 ml concentrated sulfuric acid (sp. gr. 1.84) to 500 ml with distilled water.
5. After the two acid solutions, 3 and 4, have cooled, mix them.

The three solutions prepared as above should be mixed (see below) *as needed* for use in the field kit. Discard any surplus *mixed* reagent after 2 months even when stored in the refrigerator.

Mixing Scott-Sanchis reagent :

1. Place 50 ml zirconium oxychloride solution in a 1-litre flask.
2. Pour slowly 50 ml alizarin monosulfonate solution into the same flask while swirling the flask to mix. The resulting mixture clears in a few minutes on standing.
3. Fill the flask to the 1-litre mark with the mixed acid solution (5) and mix. This resulting mixed reagent changes colour from red to yellow within 1 h and is then ready for use. *This is the Scott-Sanchis reagent used in the test for fluorides.*

Care of reagents. As noted, the *separate* reagents may be stored for 1 year; but *once they have been mixed* they should be discarded after storage in a refrigerator for 2 months or after one or two weeks at room temperature. The three separate reagents should therefore be stored at a central point and only mixed as needed for a field trip. If only a few samples are to be examined, requiring 5 ml per sample, smaller volumes of the reagents should be mixed. For instance, 5 ml zirconium oxychloride and 5 ml alizarin monosulfate would be mixed in a 100-ml flask, then made up to the 100-ml mark with the mixed acid solution, giving 100 ml of reagent, or enough for the examination of 20 samples.

Procedure for fluoride test

1. Measure with the pipette 5 ml of the mixed Scott-Sanchis fluoride reagent and place in the Erlenmeyer flask.

2. Measure 50 ml of the sample with the graduated cylinder and pour it into the same flask.

3. Mix the reagent and sample well, note the time, and let stand for *exactly* 1 h.

4. In the meantime fill two of the tubes to the mark with portions of the sample *without* reagent and place them in the outer compartments of the Analyzer, so as to secure compensation for any natural colour of the sample.

5. Place the Analyzer where light passes down the tubes to the mirror.

6. *Shortly before the 1-h period has elapsed* fill the third tube to the mark with the mixture of sample and reagent, which have been reacting in the flask.

7. *At the end of exactly 1 h* compare the resulting colour with that of the standards. The fluoride ion content of the sample in parts per million is the value of the matching standard.

Interfering substances

Certain substances in natural or treated waters interfere with this test by either decreasing or increasing the apparent content of fluorides. Following is a list of such interfering substances which *lower* the true value when present in concentrations at or above those noted:

Substance	*Critical concentration, p.p.m.*
Chlorides (salt)	2000.0
Aluminium (alum floc)	0.2
Alkalinity	400.0

The following substances cause an *increase* in the apparent content of fluorides when present in concentrations at or above those noted:

Substance	*Critical concentration, p.p.m.*
Sulfates	300.0
Hexametaphosphate	0.3
Iron	2.0
Manganese	0.05
Residual chlorine	0.2

Critical concentrations of these substances create errors of 0.1 p.p.m.

Errors due to chlorides and sulfates are restricted to waters of high mineral content, which are not encountered very often. Errors due to iron and manganese are of significance with some well waters and with a few surface waters. If iron or manganese removal is practised before fluoridation, such errors are eliminated. Natural waters do not contain appreciable concentrations of aluminium, but waters *improperly* coagulated with alum may retain sufficient soluble aluminium to interfere with the test. Residual chlorine is the most common interfering substance, because most fluoridated supplies are also chlorinated; this interference, of course, does not occur when the fluoride content of unchlorinated waters is being determined.

Directions for eliminating interference by these substances are given below. The preliminary distillation procedure, discussed last, must be followed when interference cannot otherwise be eliminated.

Chloride. Interference by the chloride ion is suppressed by the hydrochloric acid content of the reagent, so that chlorides may be present up to 2000 p.p.m. without causing difficulty. Each additional 2000 p.p.m. will cause the apparent concentration of fluorides to *decrease* by about 0.1 p.p.m. This interference, therefore, is not encountered with acceptable waters.

Aluminium. Residual alum as aluminium in excess of 0.2 p.p.m. will interfere by *decreasing* the apparent concentration of fluorides by 0.1 p.p.m. This interference may be eliminated by adding to a 100-ml portion of the sample one drop (0.05 ml) of 3% hydrogen peroxide solution and mixing. The excess hydrogen peroxide is eliminated by adding one drop of potassium iodide solution, followed by three drops of 0.1N sodium thiosulfate solution. If the sample then is turbid, filter it through filter paper. Aluminium causes interference only when fluorides are also present.

Alkalinity. Alkalinities of 400 p.p.m. or over will cause a *decrease* in the apparent content of fluorides. If interference is by alkalinity alone, it can be eliminated by neutralizing the sample with hydrochloric acid, otherwise the preliminary distillation procedure must be followed with waters of high alkalinity.

Sulfates. Sulfate interference is suppressed by the sulfuric acid in the reagent. Most waters have a sulfate ion content of less than 300 p.p.m.; but if it exceeds this value the apparent content of fluorides will be *increased* by 0.1 p.p.m., and higher concentrations of sulfates will cause even higher errors. Distillation must be practised to eliminate these errors.

Hexametaphosphate. Treatment of water with 1 p.p.m. or more of this compound to minimize corrosion or to prevent the precipitation of calcium carbonate will cause an apparent *increase* in the fluoride content. This interference may be eliminated by bringing a portion of the sample to the boil to change the characteristics of the compound. Cool and then test.

Iron. Iron in the ferric or oxidized state, in a concentration of 2 p.p.m. or more, will cause the apparent content of fluorides to *increase* by 0.1 p.p.m. Fortunately most waters contain less than 2 p.p.m. iron, but if the content exceeds this, the preliminary distillation procedure must be followed.

Manganese. Manganese in the oxidized form, in a concentration of 0.05 p.p.m. or more, will cause an *increase* in the apparent fluoride content of 0.1 p.p.m. (a more reddish colour is produced with the reagent). This interference may be eliminated by first treating a portion of the sample as directed under aluminium, above, and then testing the portion for fluorides.

Residual chlorine. Residual chlorine, when present in excess of 0.2 p.p.m., will cause an *increase* in the apparent content of fluorides. Remove the residual chlorine before testing a sample by adding two drops (0.1 ml) of sodium arsenite solution for each 0.1 mg of chlorine, then add two drops more and mix. When the residual is 1 p.p.m. or 1 mg/l, there will be 0.1 mg residual chlorine in a 100-ml portion of the sample.

Colour and turbidity. Colour and turbidity of untreated waters being tested for natural fluoride content may produce off-shades with the Scott-Sanchis reagent and should be removed from samples. Activated carbon will usually remove the colour, and filtration through paper on a fritted glass funnel or asbestos mat on a Gooch crucible, the turbidity. *Chemical coagulation should not be used to clarify samples.* The Taylor Analyzer compensates for errors due to moderate colour or turbidity. The regular test also may be used with water of low enough colour and turbidity to be satisfactory for delivery to the public. If, however, the colour is high and cannot be removed from a sample by filtration, the preliminary distillation procedure must be carried out.

Preliminary distillation procedure

The distillation procedure cannot be carried out in the field. When interference by the substances noted above cannot be eliminated, especially if a fluoridation programme is under control, then equipment should be installed at headquarters to permit effective elimination of such interference and to serve as a check on results secured in the field.

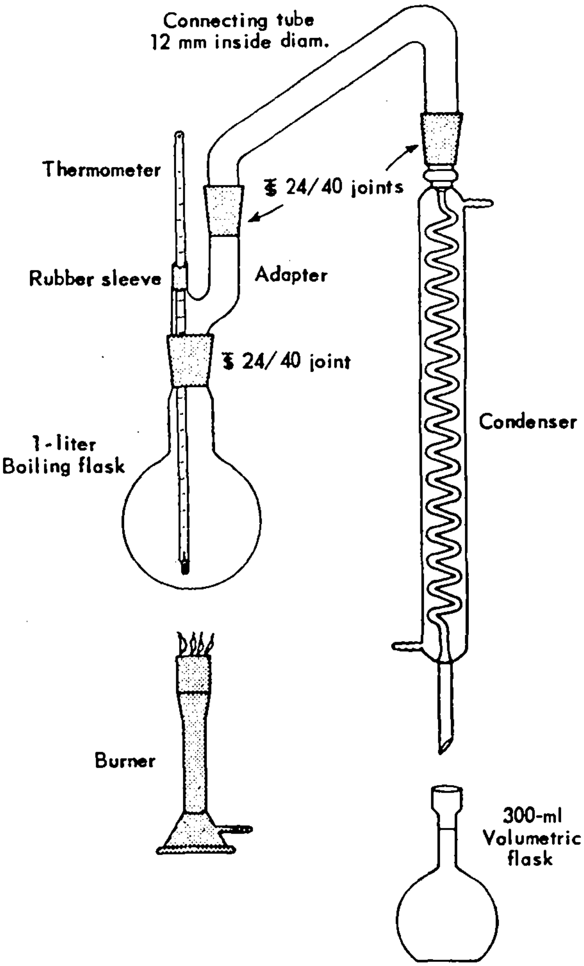
The procedure is based upon the distillation of hydrofluoric acid from an acidified sample, the interfering substances being left in the sample. The distillate then is tested for fluorides.

Equipment :

1-litre, round-bottom, long-neck Pyrex boiling flask
Connecting tube of glass, 12 mm inside diameter
Glass condenser
Thermometer reading to 200°C
300-ml volumetric flask
Supply of glass beads

Assemble the above as shown in Fig. 21.

FIG. 21
FLUORIDE DIRECT-DISTILLATION APPARATUS



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Reagents :

Concentrated sulfuric acid
Solid silver sulfate

Procedure :

1. Place 400 ml distilled water in the distillation flask, and carefully and slowly add 200 ml concentrated sulfuric acid.

2. Swirl to mix.

3. Add about 30 glass beads to the flask and connect it by the tubing to the condenser.

4. Begin heating slowly at first and then as rapidly as permitted by the effectiveness of the condenser to produce a *cool distillate*, until the temperature of the flask contents reaches *exactly* 180°C.

5. Discard the distillate.

(Steps 1 to 5 are intended to remove any fluoride contamination and also to adjust the acid-to-water ratio for the subsequent distillation.)

6. Cool the contents to 120°C or below.

7. Add 300 ml of the sample and mix thoroughly.

8. Connect the flask to the condenser and distil until the temperature reaches 180°C. To prevent sulfate carry-over into the distillate, do not permit the temperature to exceed 180°C.

9. Add distilled water to the distillate in the flask to fill to the mark, since the distillate contains all the fluorides originally present in 300 ml of sample.

10. Test the distillate for fluorides.

Alternative procedure. If the chloride content of the sample exceeds about 100 p.p.m., add 5 mg silver sulfate for each milligram chloride in the sample. For instance, if a sample has a chloride content of 140 p.p.m., or 140 mg/litre, then 300 ml of the sample contains 300/1000 of 140 mg, or 42 mg, so 210 mg silver sulfate ($5 \times 42 = 210$) would be added to the flask at step 7. This alternative procedure is seldom necessary.

Note : The sulfuric acid solution in the flask may be used repeatedly, inasmuch as all fluorides in each sample distilled have been discharged as hydrofluoric acid with the distillates. Contaminants will accumulate, however, in the acid solution. Its suitability for continued use should be checked by the distillation of a sample of known fluoride content. If such checking is impracticable, discard the acid solution after 10 samples have been distilled.

When samples of high fluoride content (over 1 p.p.m.) have been distilled and another sample of suspected low fluoride content is to be distilled, the still should be freed of any fluorides by distilling 300 ml *distilled water.*

Chlorides

Purpose of the test

The test for the content of chloride in water may be used for three separate purposes. First, salt is present in sewage from urine in the range of about 50-200 p.p.m. and higher yet in countries with low water consumption per capita, so the content of chlorides in polluted water is a rough measure of the degree of pollution. This is especially the case with well waters, in which seepage from cesspools into the ground-water tributary of a well may be disclosed. This, however, requires that the normal chloride content of the unpolluted surface and ground waters be known for comparison purposes. For instance, the normal content in ground water in a given neighbourhood may be 0.5 p.p.m., so a content of, say, 2.5 p.p.m. in a well water in the vicinity would disclose significant seepage.

Second, the chloride content of ground waters is used in the study of salt-water intrusion along the sea coast, which destroys the usefulness of wells so affected. Local deposits of rock salt naturally lead to high chloride content in ground waters—that is, salty water not related to pollution or sea-water intrusion and hence without sanitary significance. Water has a salty taste to some people when the content of chlorides exceeds about 150 p.p.m., and 200 p.p.m. is the maximum desirable content.

Finally, the test for chlorides is used in the control of the regeneration of ion-exchange softeners with solutions of salt.

Equipment (for use in field)

Burette
1-ml pipette
Two 250-ml evaporating dishes

Reagents

Potassium chromate solution (dissolve 50 g neutral potassium chromate in small quantity distilled water; add silver nitrate solution to produce *slight* red precipitate; allow to stand overnight and filter; dilute filtrate with distilled water to give volume of 1 litre)

Silver nitrate solution (2.4 g silver nitrate dissolved in 1 litre distilled water)

Sodium chloride solution (16.48 g sodium chloride in 1 litre distilled water: dilute 100 ml of this solution to 1 litre with distilled water)

Standardization of silver nitrate solution. Place 25 ml of the diluted sodium chloride solution in one evaporating dish and add 25 ml distilled water. Place 50 ml distilled water in another dish. Add 1 ml chromate solution to each dish. Place silver nitrate solution in the burette and add it drop by drop to the sodium chloride solution in one of the dishes until the *reddish-brown* colour remains, comparing this colour with the unchanged colour of the distilled water containing only chromate.

Record the amount of silver nitrate, in millilitres, used to secure this colour change. This amount must be corrected for the error due to the increase in volume of the sodium chloride solution resulting from the addition of the silver nitrate. For this correction, subtract the dilution factor, which is (0.003 × volume of liquid at the end of titration) + 0.02.

For instance, assume that 50.2 ml of silver nitrate solution was used. Then 50.2 ml silver nitrate + 25.0 ml sodium chloride + 25.0 ml distilled water + 1.0 ml potassium chromate = 101.2 ml total volume at the end of titration. The dilution factor then is (0.003 × 101.2) + 0.02, or 0.3236 ml; and the corrected value for the silver nitrate solution is 50.2 — 0.3236, or 49.876 ml.

The intent was to have a standardized volume of 50 ml to react with the 25 ml sodium chloride, so that each 2 ml silver nitrate would equal 1 ml chloride ion, or 1 ml silver nitrate would equal 0.5 mg chloride ion. Therefore the silver nitrate solution in this example would be diluted with distilled water so that each 49.876 ml would have a final volume of 50.0 ml, requiring the addition of 0.124 ml distilled water. In other words, the adjustment would be 0.124 × 100/49.876 = 0.24%, or 2.40 ml distilled water added to each 100 ml of the silver nitrate solution being adjusted. This may seem to be so small an adjustment as to be of no practical significance, but the amount of solution used in the test is so limited that any error is greatly magnified, as shown by the computations given below.

Procedure for chlorides test

1. Place 50 ml of sample in one dish.
2. Place 50 ml distilled water in the other dish.
3. Add 1.0 ml chromate solution to each dish.
4. Add standardized silver nitrate solution with the burette drop by drop to the sample until the first reddish coloration appears, compared with the original colour of the distilled water and chromate mixture in the second dish.
5. Record the amount of silver nitrate solution used (ml). If more than 8 ml is required, repeat with smaller volume of sample diluted to 50 ml with distilled water.

Computations

$$\frac{(\text{ml } AgNO_3-0.2) \times 500}{\text{ml of sample}} = \text{p.p.m. chloride ion}$$

For example, if 50 ml of sample was examined and 6.2 ml standardized silver nitrate used, then:

$$\frac{(6.2-0.2) \times 500}{50} = \frac{3000}{50} = 60 \text{ p.p.m. chlorides}$$

BACTERIOLOGIC EXAMINATIONS OF WATER

The purpose of bacteriologic examinations is to indicate the degree of sewage pollution of the water at the time of sampling and thus the possibility that disease may be transmitted by the water so polluted.

Bacteria are minute single-cell organisms belonging to the plant kingdom and consisting of many different types and kinds. There are two main classes, the *saprophytes*, living on dead or decaying organic matter, which are harmless and necessary for the decomposition of such organic materials; and the *parasites*, which have their natural habitat in the living bodies of man and animals. Pathogenic micro-organisms which produce specific diseases in human beings and lower animals are parasites, as are many non-pathogenic bacteria such as *Escherichia coli*, a member of the coliform bacteria inhabiting the animal intestinal tract (these are believed to develop pathogenic properties in rare instances, but this is not of practical significance). The chief diseases caused by bacteria and transmitted through the medium of water are typhoid fever, dysentery, cholera and certain types of gastro-intestinal disturbances. Since these are intestinal diseases, the faecal bacteria are of primary importance in water examinations. Water may also contain many types of organisms whose normal habitat is soil, water or air. Many of these organisms may, in an unfavourable environment, revert to a " seed " or spore stage without being destroyed.

The routine bacteriologic examination of water is based on the approximate determination of the total number of bacteria present and the presence or absence of bacteria of intestinal or sewage origin. Contrary to the usual belief, no examinations are made for specific pathogenic micro-organisms, because this is not practicable. Only non-pathogenic indicator organisms are sought which are characteristic of the intestinal discharges of warm-blooded animals, including man, and thus of pollution by sewage or manure.

Two groups of indicator organisms are used in waterworks practice. First, there is the group of faecal streptococci, typified by *Streptococcus faecalis*. Second, there is the coliform group, deriving its name from the colon or large intestine of man and typified by *Escherichia coli*, formerly known as *Bacillus coli*. *E. coli* is of intestinal origin, and other members of the coliform group may be of intestinal origin or from soils and plants. In any case, these organisms are foreign to potable waters, and hence their presence indicates the presence of pollution.

Inasmuch as *International Standards for Drinking-Water* specifies water quality in terms of the content of coliform organisms, the procedure discussed below does not include tests for faecal streptococci.

All members of the coliform group ferment lactose (milk sugar) with the formation of acid and gas, grow aerobically (i.e., in the presence of oxygen) and are non-spore-forming. These cultural characteristics are the basis of the long-established routine tests for the presence of this group in water. Since coliform bacteria are usually found in the intestines of all warm-blooded animals, their presence cannot be used to differentiate pollution by human beings from that by other animals. Such a differentiation is unnecessary, however, as pollution from either source is undesirable in a water supply.

The test for coliform organisms discloses through statistical computations the most probable number (MPN) of organisms per 100 ml of sample. A great deal of glassware and medium as well as sizable autoclaves and incubators are needed in laboratories to which samples of water are transported for examination. In recent years, however, the new membrane-filter technique has been developed, modifications of which may be used in the field with portable equipment. The modified procedure, as well as the standard test, is described below.

Standard plate count

Purpose of the test

The standard plate count provides an estimate of the total number of bacteria in a sample which will grow at 36°C in 24 h and under the conditions of food supply and moisture provided in the accepted laboratory procedure. This procedure is useful in the control of filtration plants, but the total plate count is not used as a criterion in appraising the sanitary quality of a supply, the coliform index being used for that purpose.

Principles

The agar used in this test contains nutrients for bacteria so that they will multiply during the incubation period of 24 h. The agar is a jelly which hardens upon cooling, holding each organism in the position occupied when the agar hardens. Inasmuch as bacteria multiply very fast, each organism will develop in 24 h into a cluster or colony, which is visible. These colonies are counted to give the number of bacteria in the portion of the sample (usually 1 ml) in the Petri dish or plate, which is termed the plate count.

Sampling

Samples must be collected in sterile, glass-stoppered, clear glass bottles, using great care to avoid contamination of the sample. Select sampling points at locations representative of the water to be examined. Avoid dead ends and leaky taps, and flame taps before use. Allow water to run from tap for 5 min before collecting sample. Sampling quiescent waters requires special care to prevent bacteria from the hand and outer surface of the bottles from contaminating the water entering the bottles. Grasp the lower portion of the bottle and move the bottle *neck first* through the water a few inches below the water surface, keeping the neck somewhat higher than the bottom to permit the air to escape.

Samples should be examined promptly or else refrigerated until examined, Samples of *chlorinated* water should be dechlorinated as sampled, so that disinfection will not continue in the bottle, producing samples unlike the water prevailing at the place and time of sampling. Dechlorinate by placing a crystal of sodium thiosulphate in each bottle before it is sterilized.

Dilution of samples

The number of colonies developing on a plate should be less than 300 to make counting accurate and easy. Therefore, when previous experience with a water discloses that the plate count is likely to be more than 300, less than 1 ml of the sample should be examined. This is accomplished by diluting, say, 1 ml of the sample in 99 ml of diluting water, so that each millilitre of the mixture contains 1/100 the number of bacteria in 1 ml of the sample. Thus a plate count made with 1 ml of the diluted sample might be, say, 42, in which case the count would be multiplied by 100 to give 4200 per millilitre. A 1.0-ml pipette graduated in 0.1-ml divisions may be used to measure 0.1-ml portions of a sample, but the more accurate procedure is to use a dilution of 1 to 10 with dilution water and then place 1 ml on the plate.

Dilution water. Sterile, distilled or tap water should not be used for dilution purposes, because bacteria are harmed when pH values are changed upon dilution, as they would be especially if distilled water were used. Therefore buffered diluting water is used, prepared as shown below (page 350, under Equipment).

Diluting water should be placed in glass bottles with glass or rubber stoppers or screw caps, in quantities needed for specific dilutions, such as 99 ml. The amount of dilution water placed in a bottle, however, should allow for some loss during sterilization in the autoclave, so that 99 ml ± 2 ml will remain. The loss will depend on the functioning of a specific autoclave and hence must be found by experience. The bottles may be marked at the capacity level found from experience to leave about 99 ml after sterilization.

A dilution of 1 to 10 is most conveniently made by removing 9 ml sterile dilution water from a bottle with a sterile 10-ml pipette and adding this to an empty sterile tube into which 1 ml of sample has been placed; 1 ml of the mixture is then placed on the Petri plate with a sterile 1-ml pipette.

Equipment for plate count

Distilled water

Harvard trip balance

Pressure cooker autoclave with solid-flame gas burner

Dehydrated Bacto tryptone/glucose extract agar or its equivalent

Oven hot-air sterilizer to operate at 170°C

Incubator equipped to maintain constant temperature between 34° and 36°C in incubation chamber

Glass Petri plates, 90 mm in diameter and 15 mm deep, with porous covers

Glass straight-walled pipettes to deliver 1 ml

Ten 125-ml Pyrex Erlenmeyer flasks

1000-ml graduate with 10-ml divisions

Illuminated colony counter, if funds are available

200-ml dilution bottles with glass or rubber stoppers or screw caps

Supply of 125-ml glass-stoppered, wide-mouthed bottles for collection of samples

Preparation of agar :

1. Weigh 12 g dehydrated agar.
2. Measure 500 ml distilled water in the graduate.
3. Pour 400 ml of the distilled water into a beaker and heat to boiling.
4. Suspend the 12 g agar in the remaining 100 ml cold water.
5. With constant stirring, add the agar suspension to the 400 ml boiling water. Continue stirring and boiling until the medium is completely dissolved.
6. Pour equal amounts of the medium into each of the ten 125-ml Erlenmeyer flasks.
7. Close the Erlenmeyer flasks with cotton plugs.
8. Sterilize in autoclave for 15 min after the pressure has reached 15 lb.
9. Remove from the autoclave as soon as the pressure returns to zero. The total time in the autoclave, including heating, sterilizing, and cooling, should not exceed 40 min.

Sterilization of glassware :

1. Wash all glassware thoroughly.
2. Wrap Petri plates in groups of four in kraft paper or pack them in metal cans.
3. Wrap pipettes in kraft paper or place in metal pipette cans.
4. Tie kraft paper over the stoppers and necks of the sample bottles to protect them from contamination while in use.
5. Sterilize in oven by heating at 170°C for 1 h.

Procedure for plate count

1. Melt the sterile agar by immersing the 125-ml Erlenmeyer flask in boiling water.
2. Cool the melted agar to 45°C and hold in a water-bath at 43°-45°C.
3. Shake sample violently in an up-and-down motion 25 times.

4. Aseptically, with a sterile pipette, transfer exactly 1 ml well-shaken sample to a sterile Petri dish.

5. Add 10 ml melted agar medium cooled to 43°C.

6. Mix by rotating plate on surface of work-table.

7. Allow medium to harden and place Petri dish in incubator maintained at 34°-36°C.

8. After 24 h, count the number of colonies on or in the agar medium, using an illuminated colony counter if available.

9. If the number of colonies present is too numerous to count, estimate the number by counting a fraction of the plate, such as one-quarter or one-tenth, and multiply this count by the appropriate factor. This would indicate that subsequent testing of samples from the same point should be diluted to a greater extent to secure plates with less than 300 colonies.

Results

Results are reported as 24-hour plate count per millilitre at 36°C.

Interpretation

The standard plate count at 36°C indicates the number of micro-organisms which develop best at this temperature and, since many of them are associated with animal life, is also an indication of pollution. No definite standard can be set for the total count but usually it should be less than 500 per millilitre. Higher counts indicate possible contamination with surface drainage or with sewage. In filtered water, high counts may indicate passage of bacteria through the filter or bacterial growths in filters, filtered-water basin, or pipelines. Filters should remove at least 98% of the bacteria in the applied water.

Standard test for bacteria of the coliform group

Purpose of the test

The aim is to estimate the number of bacteria of the coliform group present in a given volume of water as an index of the degree of pollution.

Sampling

Use the same sample collected for the standard plate count.

Equipment (in addition to that required for standard plate count)

Wire loop at end of small-gauge wire inserted in handle, the loop holding one drop of liquid

Fermentation tubes for 10-ml volumes of sample (culture tubes without lip), 175 × 22 mm

Culture tubes without lip, 75 × 10 mm, for inner tubes

Fermentation tubes for 1-ml volumes of sample (culture tubes without lip), 150 × 18 mm

Volumetric transfer pipettes, 10 ml

Baskets or racks for holding media

Bacto dehydrated lactose broth or its equivalent

Bacto dehydrated brilliant-green/lactose bile broth 2% or its equivalent

Preparation of lactose broth. The medium after inoculation with the sample should contain 0.5% each of lactose and peptone. Thus, in tubes to which 10 ml water is to be added, the medium must be made up to double strength as follows:

1. Weigh 13 g dehydrated lactose broth.
2. Measure 500 ml distilled water in a graduate.
3. Heat 400 ml of the distilled water to boiling.
4. Mix the weighed lactose broth in the remaining 100 ml cold distilled water.
5. With constant stirring, add the mixture to the boiling water and dissolve completely in the total volume of 500 ml.
6. In each of the large tubes (175 × 22 mm) insert one of the small tubes (75 × 12 mm) in an *inverted* position.
7. Add 10 ml of medium to each outer tube.
8. Close each outer tube with a cotton plug.
9. Place tubes in baskets and put in autoclave.
10. Sterilize for 15 min after the pressure has reached 15 lb.
11. Remove from the autoclave as soon as the pressure has returned to zero. The total time for heating, sterilizing and cooling should not exceed 40 min.

In tubes to which 1 ml of sample is to be added, the single-strength medium is used. For this purpose in step 1 weigh 6.5 g and in step 6 use the smaller tubes (150 × 18 mm), otherwise follow the same procedure.

Preparation of brilliant-green/lactose bile broth. The procedure is exactly the same as for the preparation of lactose broth except that 20 g dehydrated brilliant-green/lactose bile broth is used with 500 ml distilled water. Only single-strength medium is prepared, as no portion of a sample is added but merely a drop of previously inoculated broth, so only the small 150 × 18-mm tubes with inner tubes are used.

Sterilization of glassware. Use the same procedure as for the standard plate count (page 343).

Procedure for testing water for presence of coliform organisms

Presumptive test :

1. In an aseptic manner, inoculate with 10 ml of sample each of five large fermentation tubes containing double-strength lactose broth.
2. Inoculate with 1 ml of sample one small fermentation tube containing lactose broth.
3. Inoculate with 0.1 ml of sample[1] one small fermentation tube containing lactose broth.
4. Place all fermentation tubes in incubator maintained at 35°-37°C.
5. At end of 24 h observe whether gas has formed in inner tubes of each of the fermentation tubes.
6. Perform confirmatory test on all tubes in which gas has formed and replace remaining tubes in incubator.
7. At end of 48 h observe whether gas has formed in inner tube of each of the remaining tubes.
8. Perform confirmatory test on all tubes in which gas has formed.

Confirmatory test. The production of gas in the lactose broth does not necessarily indicate the presence of bacteria of the coliform group, because there may be other bacteria present which ferment lactose. If a portion of the media from those lactose-broth tubes which show gas is transferred to brilliant-green/bile broth, the bacteria *other than* coliform organisms are inhibited by the brilliant-green/bile, so any gas produced in these tubes can be assumed to indicate the presence of organisms of the coliform group.

1. Select each fermentation tube showing gas at 24 or 48 h (steps 5 and 7 of presumptive test) and transfer with the wire loop one drop from it to a fermentation tube containing brilliant-green/bile broth, the loop first being sterilized in the flame of a gas or alcohol burner each time it is used.
2. Place in incubator for 24 h.
3. Examine for presence of gas. If gas is present the tube may be recorded as positive and discarded; if no gas is present it should be reincubated for another 24 h and re-examined.
4. If gas is present at the end of the second 24-h period the tube may be considered positive; if no gas is present it is negative.

Interpretation

Any lactose-broth fermentation test showing gas formation after 24 or 48 h of incubation, confirmed by gas formation in the confirmatory medium after 24 or 48 h, indicates the presence of bacteria of the coliform group in

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[1] Smaller volumes of sample would be examined by dilution procedure when polluted raw waters are examined.

the corresponding volume of sample used. By using different volumes of sample, multiples of 1 ml, it is possible to make a partial quantitative estimation of the number of coliform bacteria in the samples. If the number of positive and negative tubes in each dilution is known, it is possible to calculate the most probable number of organisms of this group in a given volume of water—an index of pollution usually expressed as MPN of coliform bacteria per 100 ml of sample. This index represents that number of bacteria of this group which, more frequently than any other number, will give the observed results. Table 36 may be used for determining the MPN when five 10-ml portions, one 1.0-ml and one 0.1-ml portion are used. See *International Standards for Drinking-Water* or *Standard Methods for the Examination of Water and Waste-Water* for complete tables.

Membrane-filter technique for determining number of coliform organisms

Filter pads or discs of cellulose have been developed recently with numerous very small pores. The pores are smaller than bacteria, so bacteria present in samples of water may be removed by filtration through the membrane-filter discs. The bacteria isolated in this manner will develop into visible colonies in less than 24 h in a suitable culture medium at a favourable incubation temperature.

As indicated previously, bacteria of the coliform group are of chief importance in water examinations, so a selective medium may be used to suppress the growth of most other bacteria and produce a distinctive coloration for the colonies of coliform organisms. Such colonies can then

TABLE 36

EXAMPLE OF DETERMINING MPN INDEX

10-ml portions	1.0-ml portion	0.1-ml portion	MPN per 100 ml
-----	–	–	< 2.2 [a]
+----	–	–	2.2
++---	–	–	5.0
+++--	–	–	8.8
++++-	–	–	15.0
+++++	–	–	38.0
+++++	+	–	240
+++++	+	+	2 400 or over

[a] It cannot be assumed that negative results with the 51.1 ml of the sample examined indicate the absence of coliform organisms throughout the whole sample, so the results are expressed as " less than 2.2 per 100 ml ".

be counted separately to give a definite number of coliform organisms which were filtered out of the measured amount of water.

Equipment for the membrane-filter technique is available from several companies. The basic units consist of glass or stainless-steel funnels, membrane-filter holders and suction flasks to facilitate the passage of measured volumes of water through a sterile membrane. The membrane disc is then removed with sterile technique, placed on an absorption pad in a Petri dish previously saturated with liquid M-Endo's medium, and incubated for 18-22 h. This procedure is described in *Standard Methods for the Examination of Water and Waste-Water.* This basic procedure has been simplified for field use by the development of portable incubators, factory-prepared liquid medium or dehydrated medium which may be moistened as used with sterile distilled water, and simple membrane-filter holders and associated equipment.

The unit manufactured by Millipore Filter Corporation, Watertown, Mass., USA, has been selected for this test because it operates without auxiliary electrical units on 6 or 12 V d.c. or 110 or 220 V a.c., no sterilization by heat is required in its use, and sterile technique is facilitated by the simplicity of membrane-filter holders used only once and then discarded. Similar equipment for use in the laboratory is manufactured by the same company and also by Sartorius-Werke of Germany (Bac-T-Flex), and by Oxo, Ltd of England (Oxoid Filters).

Advantages

The advantages of the membrane-filter technique over the standard test for coliform organisms may be summarized as follows:

1. The procedure is more precise than the older procedure for determining the content of coliform organisms by the multiple-tube fermentation test.

2. Much larger volumes of water may be examined, so that low densities of coliform organisms may be detected, e.g., less than 1 per 100 ml.

3. Results are obtained in 18 to 22 h, in contrast to the minimum of 72 h for the confirmatory fermentation-tube procedure.

4. Very much less glassware, medium, etc. is needed than for the standard technique in the laboratory.

5. The field procedure, described below, does not require any glassware or autoclaves, and medium is purchased ready for use. Sterilized sampling bottles are not needed when the procedure is conducted in the field, as intended. Sample bottles, if needed, may be sterilized by use of a small oven placed over a gas or kerosene burner, fitted with a thermometer reading to 200°C, as listed for the standard plate count; 500-ml bottles are preferable, to permit the examination of several hundred millilitres of sample, especially when effluent samples of low bacterial density are involved.

Limitations

1. The membrane will be clogged with suspended solids in turbid waters and with algae in unfiltered waters. This is of significance chiefly when turbid waters *of a low degree of bacterial pollution* are involved. Such waters are not likely to be encountered except when turbid surface waters are disinfected by chlorination but unfiltered. (See Volume of Sample Examined, below.)

2. The cost of membrane-filter discs and prepared medium is high and the equipment must often be imported. Prices in 1960 averaged US$ 0.40 to $0.80 per sample for the two portable kits now available (f.o.b. USA). These prices will be reduced as sales increase and in any case are not excessive when compared with the cost of the laboratory facilities and labour needed for the older procedure.

Those examining many samples of water in their laboratories may find it more economical to use the originally developed equipment consisting of filter holder and suction flask, individually sterilized filter pads and the dehydrated form of M-Endo's medium. The latter may be purchased in the USA, England and Germany. The prepared M-Endo's medium may be preserved as follows: (*a*) Dissolve 12 g pure sodium benzoate in distilled water and make up to 100 ml with distilled water; (*b*) add 3.2 ml of this solution to each 100 ml M-Endo's medium. Discard the unused sodium benzoate solution after 3 months.

Sampling

Samples should be examined in the field as collected or else collected in sterile bottles for examination later. Place samples in refrigerator until examined.

Volume of sample examined

The sampling cup is calibrated from 25 to 250 ml. The volume of sample passed through the filter should be estimated from the anticipated number of coliform organisms in that portion of the sample examined, so that the filter will remove preferably 20-200 organisms. This will produce realistic values and yet the membrane will not be overcrowded with colonies after incubation. Not less than 25 ml of sample should be filtered. For example, if raw river water is being examined, one would anticipate that coliform organisms would be present to the extent of about 5000/100 ml or 50 per millilitre; 25 ml of such water then would contain about 1250 coliform organisms, which would overcrowd the membrane. In this case 25 ml of the sample would be diluted to 250 ml (1 to 10) with sterile buffered dilution water, so that the number of organisms in the

250 ml of diluted sample would be 1250. Then if only 25 ml of the diluted sample is filtered, the number of organisms on the filter membrane would be about 125. This dilution also reduces the influence of the turbidity in the sample to one-tenth. If on the other hand a filter effluent is being examined, one would expect coliform organisms in the range of 0-25/100 ml. Therefore 250-500 ml of such water would be passed through the membrane filter.

The amount of original or diluted sample drawn through the membrane filter is measured by the calibrations in the cup.

Equipment [1]

Stainless steel syringe

Portable incubator, also used as carrying case

Sampling bottles or measuring cup of stainless steel

Supply of " field monitoring kits ", each consisting of two transparent plastic filter-disc holders, two plastic tubes and two glass ampoules of culture medium (modified Endo's medium), all sterilized and ready for single service use

Alcohol

Sterile dilution water (needed only where heavily polluted water or sewage is being tested; buffered dilution water should be used for this purpose)

Buffered dilution water is prepared by dissolving 34.0 g monobasic potassium phosphate (KH_2PO_4) in 500 ml distilled water, which is then adjusted to pH 7.2 with normal sodium hydroxide solution and diluted to 1 litre with distilled water. This is the *stock solution.* Add 1.25 ml stock solution to 1 litre distilled water as needed. Sterilize before use.

The stainless steel sampling cup should be kept clean at all times, as it is the only part of the equipment which is not sterile and which the sample contacts *prior* to its passage through the membrane filter. Therefore, when clean, it should be thoroughly rinsed in the water being tested, by being repeatedly filled with water from a faucet or dipped in a river or reservoir. When used in the field to sample heavily polluted water or sewage, it should be thoroughly cleaned in soap and water before subsequent use. rinsed and then sterilized with alcohol, a small amount of which should be placed in the cup and made to flow over the inner surface by manipulating the cup. The alcohol is then poured off and the cup allowed to dry.

The incubator has an adjustable thermostat, located on the inside bottom, and also a thermometer on the inside of the cover. Before use the incubator should be operated and the thermostat adjusted until the thermometer discloses a constant temperature of 35°C.

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[1] Millipore Filter Corporation.

Procedure

1. Rinse the clean sampling cup in water being sampled.

2. Fill sampling cup with the amount of sample to be examined (25-250 ml) or with diluted samples of polluted waters, as described above.

3. Remove the two small caps from a " field monitor ", or membrane-filter unit, and attach (*a*) syringe to side showing the grated membrane support and (*b*) the small sterile plastic tube to the other opening (inlet side).

4. Insert the end of the tube in the sample and operate the syringe to draw the volume of sample to be tested through the membrane filter, using the calibrations in the cup as a guide. For instance, polluted water, after being diluted 10 to 1 as discussed above, would be drawn through the filter in the amount of 25 ml, that is, the water level in the cup would drop from the 250-ml mark to the 225-ml mark. The most convenient method with undiluted sample is to fill the cup with the precise amount to be withdrawn, such as 100 ml, 250 ml, etc.

5. Invert the syringe (small tube pointing upward) and draw remaining water through the membrane. (In this way *all* of the sample withdrawn from the cup will pass through the membrane and none will remain in the space above the membrane.)

6. Remove tube from field monitor and discard.

7. Break tip from *insertion end* of an ampoule of medium and insert this into the filter side of the field monitor, that is, the opening from which the small tube was disconnected in step 6.

8. Then break the top off the other end of the ampoule to admit air and permit the medium to drain into the monitor while the syringe is slowly pulled to draw the medium through the membrane. Stop drawing with the syringe when all surplus medium disappears from the membrane.

9. Discard ampoule and disconnect the syringe from the monitor. Replace the caps over the two openings in the monitor.

10. Label monitor with a wax pencil and place in the incubator.

11. Incubate for 18 to 22 h at a temperature of 35°C.

12. Remove the monitor from the incubator and remove cover so that the colonies may be viewed for counting.

13. Count the number of colonies with a *metallic sheen* and record the number as the number of coliform organisms in the volume of sample examined.

14. Convert this number to the equivalent number per 100 ml of sample, so as to secure a comparison with MPN values secured by the old technique. For instance, assume that 500 ml of a sample was passed through the filter and the number of colonies counted was 5, then the count would be 5/500 ml or 1/100 ml. If, however, 25 ml of a polluted water was diluted

to 250 ml and if 25 ml of the dilution were tested, then the dilution factor would be 1 to 10. Suppose, for example, the number of coliform colonies counted was 135. Then the number of coliform organisms in the original 25 ml of sample would be 135 × 10 = 1350. The number per 100 ml would be 1350 × 4 = 5400, equivalent to an MPN of 5400/100 ml.

15. Discard the monitor.

Observation of colonies

Experience is needed in the appraisal of colonies in step 13 above. Coliform organisms ordinarily develop a dark metallic sheen or lustre which at times may be restricted to the centre of the colony. Miscellaneous bacteria usually develop smaller colonies having a uniform pink colour. *These should not be counted*, as they are not coliform organisms.

MICROSCOPIC EXAMINATION OF WATER

Taste and odour in water have many causes, one of the most important being the growth of aquatic, free-floating, single-cell plants and animals. The individual cells are so small that they can be seen only under high magnification. Storage reservoirs are particularly subject to such growths. Microscopic plant forms belong to the algae. Because algae contain chlorophyll and require sunlight for their growth and metabolism, most species are found only in open bodies of water and at or near the surface. The animal species of micro-organisms that do not contain chlorophyll and can grow and develop in the dark are found in covered reservoirs and at a depth in open reservoirs to which sunlight will not penetrate. Many types of algae secrete oils, which may be discharged during their life processes and which are always set free after the death and disintegration of the cells. These oils are the cause of the characteristic tastes and odours imparted to the water. In open bodies of water certain species of algae may grow sufficiently to produce a floating scum or " water bloom ". Certain micro-organisms may grow under ice during the winter. Growths of micro-organisms, in addition to producing objectionable tastes and odours in water, will clog filters and reduce the length of filter runs (see Chapter 4 for method of control). Free-floating algae, protozoa and other microscopic organisms are termed plankton.

Grouping of micro-organisms

Algae

Include diatoms—unicellular plants with a silicious cell wall, found usually in colder waters and imparting aromatic, disagreeable tastes and odours; and members of the Cyanophyceae and Chlorophyceae families, chlorophyll-bearing plants with pigment, which are found in summer and produce disagreeable vegetable, fishy or pigpen odours. The Cyanophyceae are known as " blue-green algae " and are responsible for most " water bloom ". Many organisms contain chlorophyll like algae and yet have flagella like many protozoa. Such flagellates are often included among algae, as they are in Tables 37-41.

Fungi

Contain no chlorophyll and therefore do not require sunlight for growth; produce slime in water mains and growth on the beds of polluted streams. These organisms are usually not of importance in water treatment.

Bacteria

Are of many types, including the coliform organisms and pathogens noted previously in the discussion of water quality. Other bacteria present in the soil and natural waters that are of significance in water treatment include: (*a*) bacteria of the *Crenothrix*, or " iron bacteria " group; (*b*) the " sulfur bacteria ", present in sulfur-bearing spring waters; (*c*) bacteria associated with the reduction of iron, manganese and sulfates in waters devoid of oxygen; (*d*) the actinomycetes, soil organisms responsible for the earthy, woody or mouldy tastes and odours of many surface waters; and (*e*) the numerous bacteria associated with the decomposition and alteration of organic matter.

Protozoa

Single-celled animals (or organisms midway between the plant and animal kingdoms) requiring oxygen and organic food; some can live in an anaerobic environment, obtaining oxygen indirectly. A few forms cause disagreeable fishy tastes, sometimes bitter or cucumber.

Higher forms of life

Natural waters support a host of multicellular animals which are microscopic in size or just visible to the unaided eye. These include rotifers, hydras, worms, water-fleas and insect larvae. Usually these organisms are of significance only in the shortening of filter runs or because they may be observed by the consumer in unfiltered surface supplies. They do not cause tastes and odours.

Studies are in progress in the USA as to the sanitary significance of bacteria which are present in the intestinal tract of minute worms from polluted raw waters and may pass through filters. Effective filtration should remove these worms from delivered waters.

Sampling

Samples should be collected at significant points representative of the entire body of water. In open reservoirs and other bodies of water samples should be taken at a depth of 2 ft below the surface, to detect chlorophyll-bearing species. Care should be taken not to contaminate the sample with floating scum growths. For the detection of non-chlorophyll-bearing species of plankton, samples should be collected at a depth of more than 20 ft. For evaluation of the raw water received at the treatment plant, samples should be collected at the intake from the reservoir or other raw-water source. Only bottles of at least 500-ml capacity should be used for the collection of samples of water for microscopic examination.

Equipment

Compound microscope with magnifying power of 100-1000
Whipple eyepiece micrometer
Standard Sedgwick-Rafter counting cell with extra cover slips
Sedgwick-Rafter funnel
Rubber stopper with glass U-tube to fit small end of funnel
Silk bolting-cloth disc to fit over small end of rubber stopper
Standard washed and graded sand for use in Sedgwick-Rafter method of filtration (should be purchased ready for use)
Two 50-ml glass beakers
Pipettes, 1 ml and 10 ml
500-ml graduated cylinder
Reference books to aid in identification of organisms (no one reference book describes micro-organisms on a world-wide basis; available books are restricted to specific regions)

Procedure

For the proper control of plankton growths, enumeration of the species of micro-organisms is essential. Identification of the predominant species will aid in determining the amounts of copper sulfate needed to destroy the growths. Unless the growths in the sample to be examined are extremely heavy, concentration of the sample is required, either by centrifuging (the preferred procedure) or by the use of a Sedgwick-Rafter filter funnel using a small plug of standard graded sand as a filter. The micro-organisms will be retained in the surface of the sand and may be washed off with a small quantity of water. In this way the micro-organisms present in a large volume of sample can be concentrated in a final small volume.

Concentration of sample

1. Place one silk bolting-cloth disk moistened with distilled water over opening in small end of a rubber stopper.

2. Insert rubber stopper with a tube into outlet end of the Sedgwick-Rafter funnel.

3. Pour into funnel sufficient standard sand to provide a layer the top of which will be at the zero graduation at the small end of the funnel.

4. Pour 5 ml distilled water into funnel to wet the sand and leave a 1-ml layer on top of sand filter.

5. Measure 500 ml of sample, using graduated cylinder, and pour gently into funnel, being careful not to disturb sand filter.

6. Allow sample to filter through sand. Discard filtrate.

7. When water level is nearly but not quite down to sand surface, filtration will cease since upper (outlet) end of glass tube in rubber stopper will be at a level just about at top of sand filter.

8. Remove stopper carefully, allowing sand and filter disk to drop directly into 50-ml beaker.

9. Wash funnel with 5 ml distilled water, allowing wash water to run into the beaker.

10. Rotate beaker gently to permit water to wash adhering micro-organisms off the sand grains and filter disk.

11. Allow beaker to stand for a few seconds so that heavy sand grains but not micro-organisms will settle.

12. Decant water to a clean beaker.

13. Repeat rinsing and washing process (steps 9-12) using 5 ml more of distilled water and add to beaker containing first washing.

Examination of sample

1. Shake concentrated sample gently and withdraw 1 ml with pipette.

2. Place cover slip on counting cell, slightly ajar to permit entrance of tip of 1-ml pipette.

3. With pipette introduce the 1-ml portion of concentrated sample into counting cell. Cell holds exactly 1 ml, and when pipette is withdrawn, cover slip will " float " into place.

4. Place counting cell on stage of microscope and adjust to give sharp focus, using combination of lenses to provide magnification of 100.

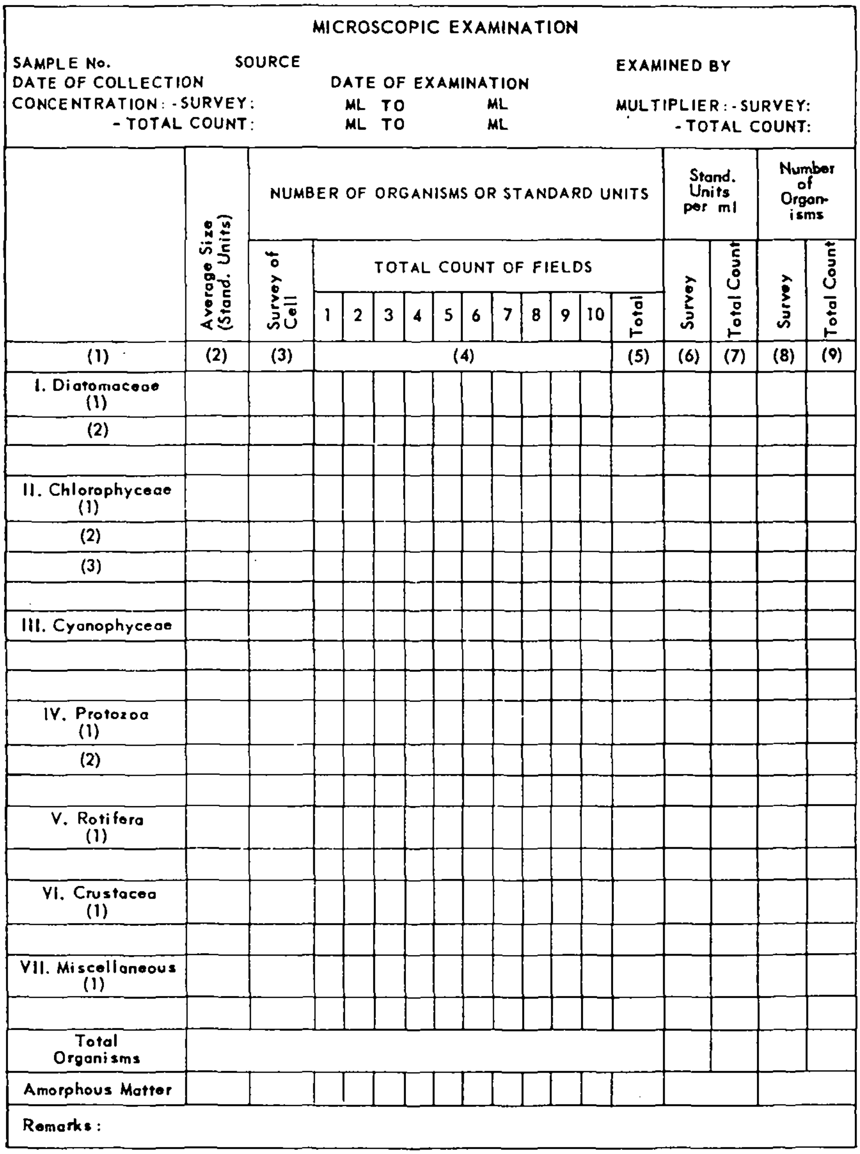
5. Observe micro-organisms with microscope, identify, enumerate and estimate size of each organism.

Enumeration of micro-organisms

The sizes and shapes of micro-organisms differ so greatly that their number alone does not give a representative value for the amount of organic matter involved. Nevertheless present practice in limnology and oceanography is to centrifuge samples and then count the number of plankton per millilitre of the original sample. Waterworks literature refers to area and volume units. Fig. 22 shows a sample bench sheet that provides columns

FIG. 22

SAMPLE FORM FOR REPORTING ENUMERATION OF MICRO-ORGANISMS



MICROSCOPIC EXAMINATION

SAMPLE No. SOURCE EXAMINED BY
DATE OF COLLECTION DATE OF EXAMINATION
CONCENTRATION: - SURVEY: ML TO ML MULTIPLIER: - SURVEY:
- TOTAL COUNT: ML TO ML - TOTAL COUNT:

		NUMBER OF ORGANISMS OR STANDARD UNITS												Stand. Units per ml		Number of Organisms	
	Average Size (Stand. Units)	Survey of Cell	TOTAL COUNT OF FIELDS											Survey	Total Count	Survey	Total Count
			1	2	3	4	5	6	7	8	9	10	Total				
(1)	(2)	(3)	(4)										(5)	(6)	(7)	(8)	(9)
I. Diatomaceae (1)																	
(2)																	
II. Chlorophyceae (1)																	
(2)																	
(3)																	
III. Cyanophyceae																	
IV. Protozoa (1)																	
(2)																	
V. Rotifera (1)																	
VI. Crustacea (1)																	
VII. Miscellaneous (1)																	
Total Organisms																	
Amorphous Matter																	
Remarks:																	

WHO 1273

Sizes in columns 2, 6 and 7 are given in either area or volumetric standard units, depending on which is used locally.

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for recording both the number of organisms and their estimated area or volume, so either procedure for enumeration may be used. The standard unit of area is a square 20 μ on a side, or 400 μ^2. The standard unit of volume is a cube of 20 μ, or 8000 μ^3 (1 μ = 0.001 mm).

The Whipple eyepiece micrometer is ruled so that, when placed in the eyepiece of a microscope giving a magnification of 100, the organisms are observed against the scale of 20 μ or 400 μ^2—that is, area units. This same eyepiece may be used to estimate the volume of familiar organisms when they are viewed in many positions in the counting cell. Thus many organisms are rod-shaped, so their depth equals their width; others are spherical. In any case, both these units are used in the same way: familiar organisms are examined with the eyepiece, and their average area or average volume is determined. Thereafter the number of organisms of the various types is counted and their total area or total volume is computed by the previously determined unit sizes, as noted below, and recorded on a form such as that shown in Fig. 22.

Procedure for enumeration

1. Observe the micro-organisms located in one field covered by the standard unit of area of 20×20 μ, and count the number of each type present. Record the count in the appropriate column of the form.
2. Shift the counting cell so that a new field may be observed and count the number of each type present. Record on the form.
3. Repeat until either five or ten such fields have been counted, recording the results.
4. Compute the total number of each type of organism counted in the five or ten fields examined, and record on the form.
5. Multiply the total count of each organism by its average area or its average volume, as noted from previous experience, to convert the count into the standard unit used. Average areas of recurring organisms may be measured and recorded. The average volumes have to be estimated from the observed shapes of the organisms involved, e.g., spherical or cylindrical. In other instances the thickness of an organism may be equal to its width. In any case the volume is computed by multiplying the area by the estimated average thickness.
6. Add these subtotal counts of individual organisms or their areas or volumes to get the total for all organisms observed.
7. Estimate the area or volume of amorphous matter observed in each field; these individual estimates should be recorded on the form and totalled, inasmuch as amorphous matter has no definite permanent shape or size.

The form shown in Fig. 22 is a convenient one for recording the results of the examination of up to 10 fields and for inserting the names and area or volume units for each type of organism so that totals may be determined readily. The number of organisms identified, measured and listed separately will increase with experience. The area or volume of unknown organisms should be totalled for each field in the " Miscellaneous " row, because no uniform average area or volume can be applied to unknown organisms unless they are determined individually during the examination.

Computation

The formula for computing the average area standard unit for each organism is :

$$\frac{\text{Area of organism in } \mu^2}{400}$$

The average volume standard unit for each organism is given by the formula :

$$\frac{\text{Volume of organism in } \mu^3}{8000}$$

The field counts, as recorded on the form in area or volumetric units, may be computed to these same units per millilitre of the original sample by use of the factor given by the formula :

$$\frac{\text{Number of fields in counting cell}}{\text{number of fields counted}} \times \frac{\text{ml of concentrate}}{\text{ml of original sample}}$$

Example : The counting cells with a volume of 1 ml have a depth of 1 mm and therefore an area of 1000 mm^2, or 1000 fields. The number of fields counted was selected as ten but could have been five or more, depending on the density of organisms encountered. The original volume of the sample was 500 ml, and this was concentrated to 10 ml in the funnel. Then, $(1000/10) \times (10/500) = 2$, which is the factor. Thus, if the total count of 10 fields for a specific organism was 200 volumetric units, there are $200 \times 2 = 400$ volumetric units per millilitre. The factor of 2 is commonly used because it is derived from the frequently used values noted above. The subtotals of the observed units, recorded on the form for each organism, are multiplied by the factor to secure the totals in the selected unit per millilitre of sample and recorded in the last column of the form.

Interpretation

The significance of the results depends on many factors discussed in Chapter 4. Experience in the USA has shown that a total count of 1000 standard area units or 20 000 volumetric units will produce difficulty from tastes and odours ; therefore counts of about 500 area units should serve as an indication for the starting of control measures. Experience will indicate which specific micro-organisms give trouble and when control measures are needed, because the results of the count of micro-organisms may be correlated with taste and odour experience. Thus the presence of the protozoan *Synura*, even in one of ten fields examined, is sufficient reason to make a more detailed examination of samples collected at various depths to determine the number and distribution of this organism. If it is present in numbers greater than about 10 standard area units per millilitre, check for resulting cucumber taste and odour, which would be replaced by a fishy taste and odour at higher concentrations. Usually copper sulfate treatment is warranted even with low concentrations.

Other organisms likely to produce tastes and odours are the blue-green *Anabaena* algae and the diatom, *Asterionella*, which give trouble when present in concentrations of 500 standard area units per millilitre.

Significant numbers of organisms will be determined from experience with each water supply, whether the count is in number, in area units or in volume units.

The identification of micro-organisms requires experience in the use of the microscope and in observing characteristics which may be compared with illustrations in the literature. Unfortunately there is no short-cut to identification of micro-organisms. Many organisms known to experts in various parts of the world have only regional significance. Furthermore, reference books available to waterworks personnel are costly and usually quite technical. The organisms shown in Figs. 23-28 should be considered only as examples, which, however, have been grouped according to their significance in control and operation.

On the other hand, the operator who lacks training in biology should not conclude that he is unable to make microscopic examinations of samples of water in a manner helpful in the control of tastes and odours. The end result sought is palatable water and not some minimum content of micro-organisms, except as this content may become known for a specific supply through experience and may be useful as a guide for prevention of tastes and odours.

Books on the subject include the booklet *Algae in Water Supplies* (Palmer, 1959), which discusses the identification, significance and control of the chief micro-organisms of interest to operators in the USA. This booklet is illustrated in colour, and six of the plates have been reproduced in black and white as Figs. 23-28. Tables 37-42 list the micro-organisms shown, together with other related micro-organisms, the tabulations giving the subdivision of the several groups of blue-green algae, green algae, red algae, diatoms and flagellates, the latter being the pigmented single-celled organisms that get their nutrition as plants but are classified by some as protozoa. It should be noted that these plates and tables apply to organisms prevalent in the USA and are presented to provide examples for general guidance. Experience in other regions must be the basis for similar compilations for those regions when facilities permit.

TABLE 37

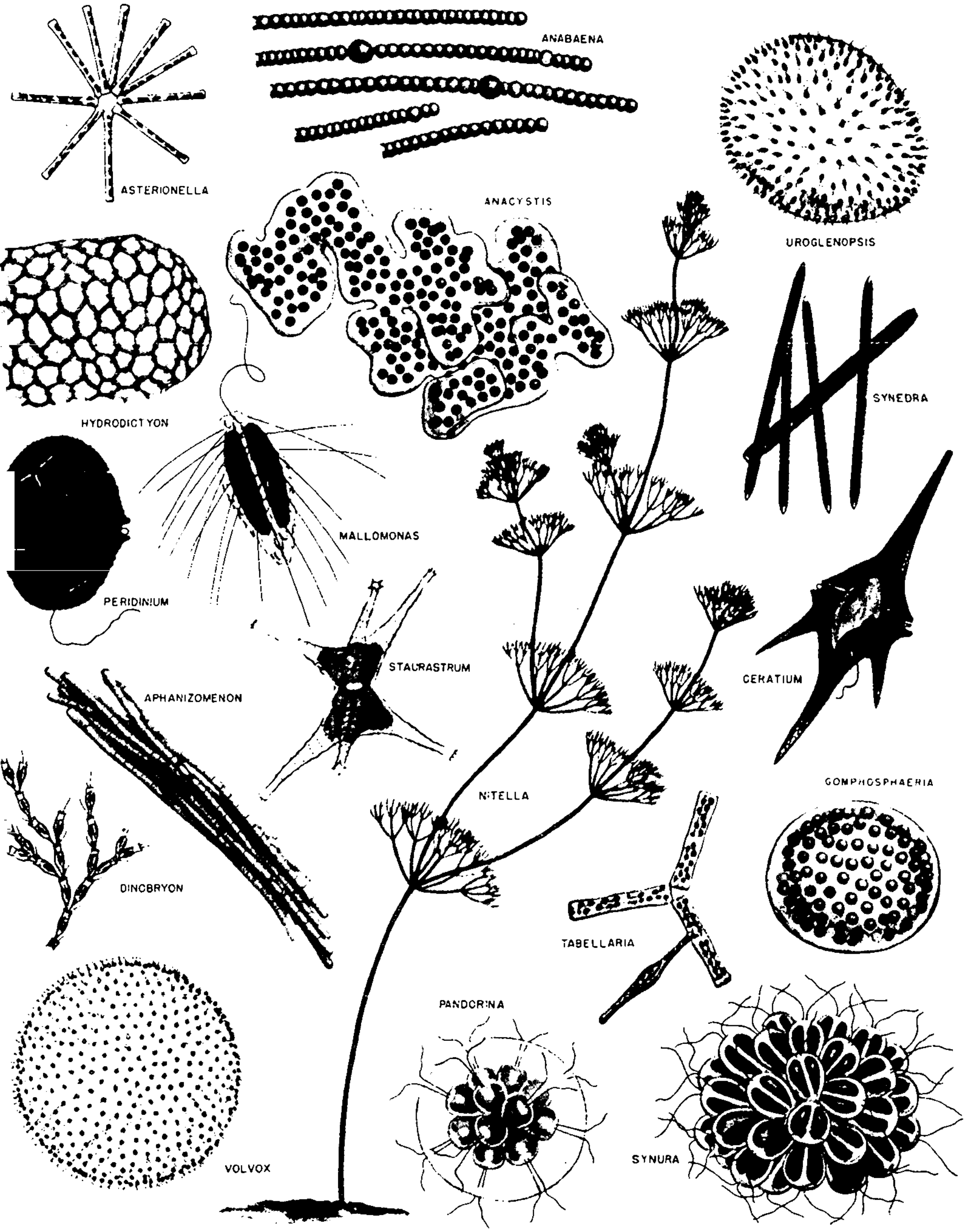
TASTE- AND ODOUR-PRODUCING MICRO-ORGANISMS

Group and algae	Figure and linear enlargement
Blue-green algae (Myxophyceae) :	
Anabaena circinalis	
Anabaena planctonica	23 (×188)
Anacystis cyanea	23 (×188)
Aphanizomenon flos-aquae	23 (×188)
Cylindrospermum musicola	
Gomphosphaeria lacustris	23 (×375)
Oscillatoria curviceps	
Rivularia haematites	
Green algae (nonmotile Chlorophyceae, etc.) :	
Chara vulgaris	
Cladophora insignis	
Cosmarium portianum	
Dictyosphaerium ehrenbergianum	
Gloeocystis planctonica	
Hydrodictyon reticulatum	23 (×8)
Nitella gracilis	23 (¼ reduction)
Pediastrum tetras	
Scenedesmus abundans	
Spirogyra majuscula	
Staurastrum paradoxum	23 (×375)
Diatoms (Bacillariophyceae) :	
Asterionella gracillima	23 (×188)
Cyclotella compta	
Diatoma vulgare	24 (×1125)
Fragilaria construens	
Stephanodiscus niagarae	
Synedra ulna	23 (×188)
Tabellaria fenestrata	23 (×188)
Flagellates (Chrysophyceae, Euglenophyceae, etc.) :	
Ceratium hirundinella	23 (×188)
Chlamydomonas globosa	
Chrysosphaerella longispina	
Cryptomonas erosa	
Dinobryon divergens	23 (×188)
Euglena sanguinea	
Glenodinium palustre	
Mallomonas caudata	23 (×375)
Pandorina morum	23 (×375)
Peridinium cinctum	23 (×375)
Synura uvella	23 (×375)
Uroglenopsis americana	23 (×94)
Volvox aureus	23 (×94)

From Palmer (1959) p. 19.

FIG. 23

TASTE- AND ODOR-PRODUCING MICRO-ORGANISMS



From Palmer (1959) p. 26.

TABLE 38

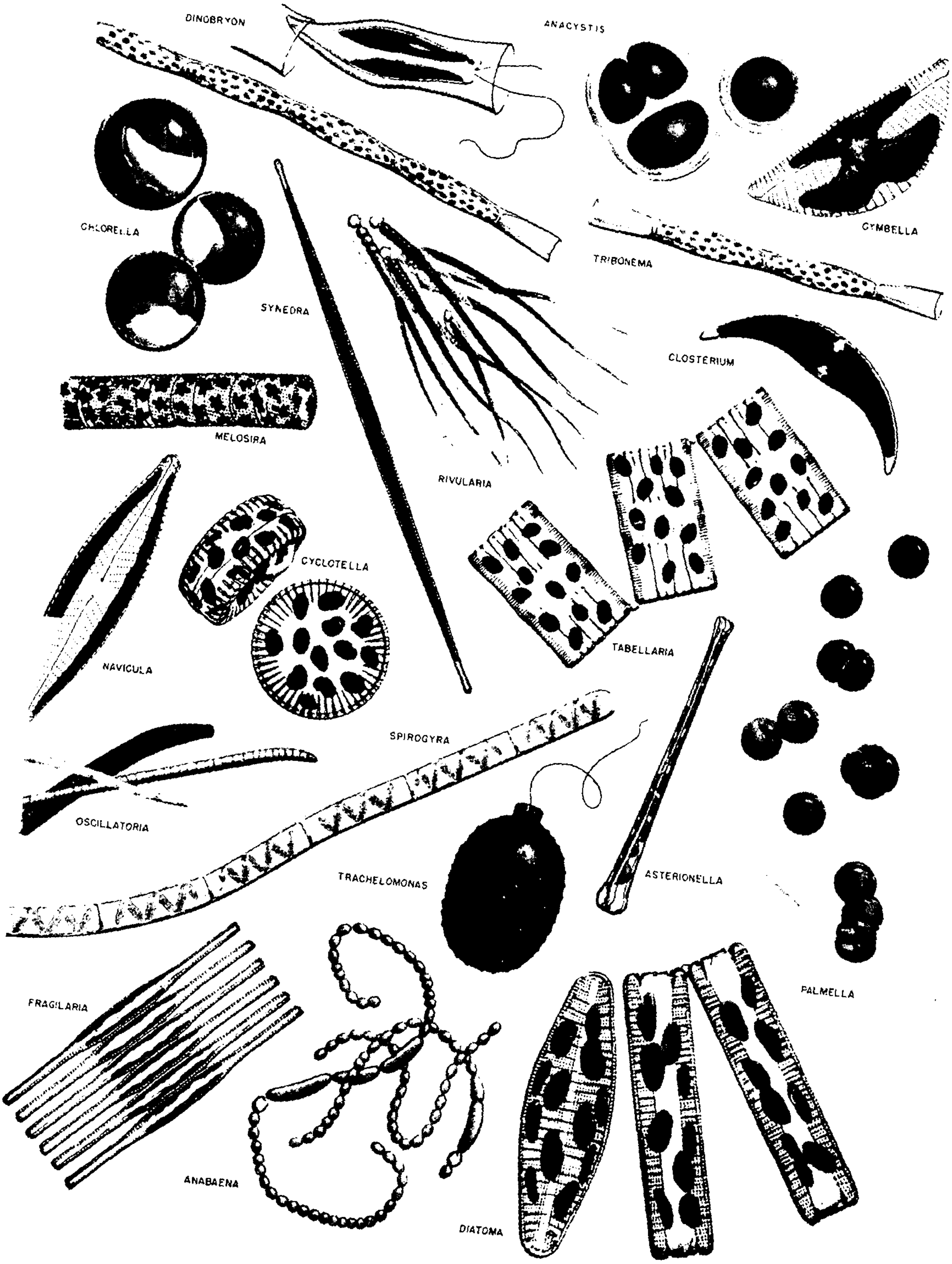
MICRO-ORGANISMS THAT CLOG FILTERS

Group and algae	Figure and linear enlargement
Blue-green algae (Myxophyceae):	
Anabaena flos-aquae	24 (×375)
Anacystis dimidiata (Chroococcus turgidus)	24 (×750)
Gloeotrichia echinulata	
Oscillatoria amphibia	
Oscillatoria chalybea	(middle) 24 (×188)
Oscillatoria ornata	
Oscillatoria princeps	(top) 24 (×188)
Oscillatoria pseudogeminata	
Oscillatoria rubescens	
Oscillatoria splendida	(bottom) 24 (×375)
Rivularia dura	24 (×188)
Green and yellow-green algae (nonmotile Chlorophyceae, etc.):	
Chlorella pyrenoidosa	24 (×3750)
Cladophora aegagropila	
Closterium moniliferum	24 (×188)
Dichotomosiphon tuberosus	
Dictyosphaerium pulchellum	
Hydrodictyon reticulatum	23 (×8)
Mougeotia sphaerocarpa	
Palmella mucosa	24 (×750)
Spirogyra porticalis	24 (×94)
Tribonema bombycinum	24 (×375)
Ulothrix variabilis	
Zygnema insigne	
Diatoms (Bacillariophyceae):	
Asterionella formosa	24 (×750)
Cyclotella meneghiniana	24 (×1125)
Cymbella ventricosa	24 (×1125)
Diatoma vulgare	24 (×1125)
Fragilaria crotonensis	24 (×750)
Melosira granulata	24 (×750)
Melosira varians	
Navicula graciloides	24 (×1125)
Navicula lanceolata	
Nitzschia palea	25 (×1500)
Stephanodiscus binderanus	
Stephanodiscus hantzschii	27 (×750)
Synedra acus	24 (×375)
Synedra acus var. radians (S. delicatissima)	
Synedra pulchella	
Tabellaria fenestrata	23 (×188)
Tabellaria flocculosa	24 (×1125)
Pigmented flagellates (Chrysophyceae, etc.):	
Dynobryon sertularia	24 (×1125)
Peridinium wisconsinense	
Trachelomonas crebea	24 (×1125)

From Palmer (1959) p. 23.

FIG. 24

MICRO-ORGANISMS THAT CLOG FILTERS



From Palmer (1959) p. 28.

TABLE 39

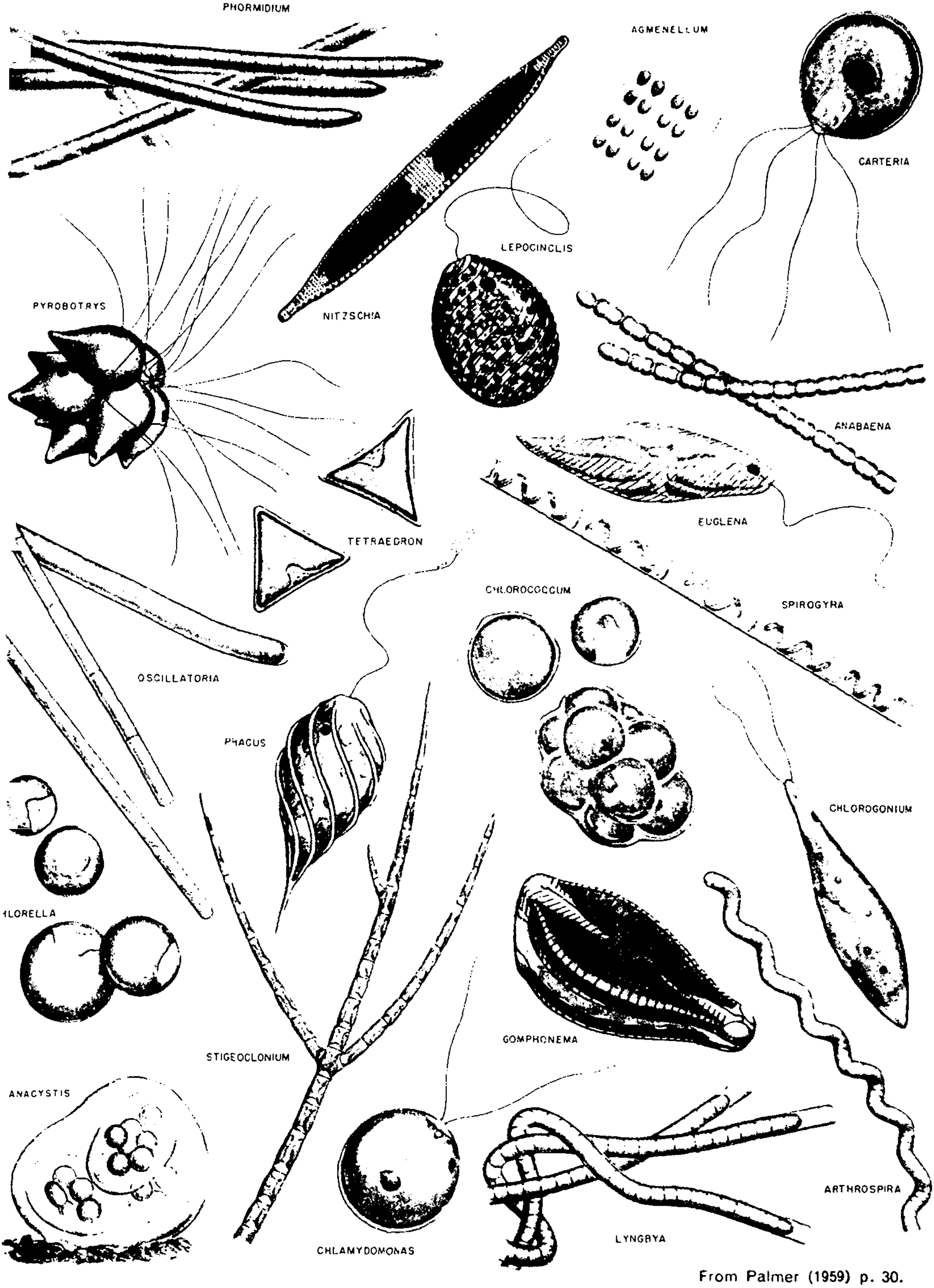
MICRO-ORGANISMS ENCOUNTERED IN POLLUTED WATERS

Group and algae	Figure and linear enlargement
Blue-green algae (Myxophyceae):	
Agmenellum quadriduplicatum	25 (×750)
Anabaena constricta	25 (×375)
Anacystis montana	25 (×750)
Arthrospira jenneri	25 (×750)
Lyngbya diguetii	25 (×750)
Oscillatoria chalybea	(top) 24 (×188)
Oscillatoria chlorina	(middle) 25 (×750)
Oscillatoria formosa	
Oscillatoria lauterbornii	(bottom) 25 (×750)
Oscillatoria limosa	
Oscillatoria princeps	(top) 24 (×188)
Oscillatoria putrida	(middle) 25 (×750)
Oscillatoria tenuis	
Phormidium autumnale	25 (×375)
Phormidium uncinatum	28 (×188)
Green algae (nonmotile Chlorophyceae):	
Chlorella pyrenoidosa	24 (×3750)
Chlorella vulgaris	25 (×1500)
Chlorococcum humicola	25 (×750)
Scenedesmus quadricauda	27 (×750)
Spirogyra communis	25 (×188)
Stichococcus bacillaris	
Stigeoclonium tenue	25 (×375)
Tetraedron muticum	25 (×1125)
Diatoms (Bacillariophyceae):	
Gomphonema parvulum	25 (×2250)
Hantzschia amphioxys	
Melosira varians	
Navicula cryptocephala	
Nitzschia acicularis	
Nitzschia palea	25 (×1500)
Surirella ovata	
Flagellates (Euglenophyceae, Volvocales of Chlorophyceae):	
Carteria multifilis	25 (×1500)
Chlamydomonas reinhardii	25 (×1125)
Chlorogonium euchlorum	25 (×1125)
Cryptoglena pigra	
Euglena agilis	
Euglena deses	
Euglena gracilis	27 (×750)
Euglena oxyuris	
Euglena polymorpha	
Euglena viridis	25 (×750)
Lepocinclis ovum	
Lepocinclis texta	25 (×375)
Pandorina morum	23 (×375)
Phacus pyrum	25 (×1125)
Pyrobotrys gracilis	
Pyrobotrys stellata	25 (×1125)
Spondylomorum quaternarium	

From Palmer (1959) p. 38.

FIG. 25

MICRO-ORGANISMS ENCOUNTERED IN POLLUTED WATERS



From Palmer (1959) p. 30.

TABLE 40

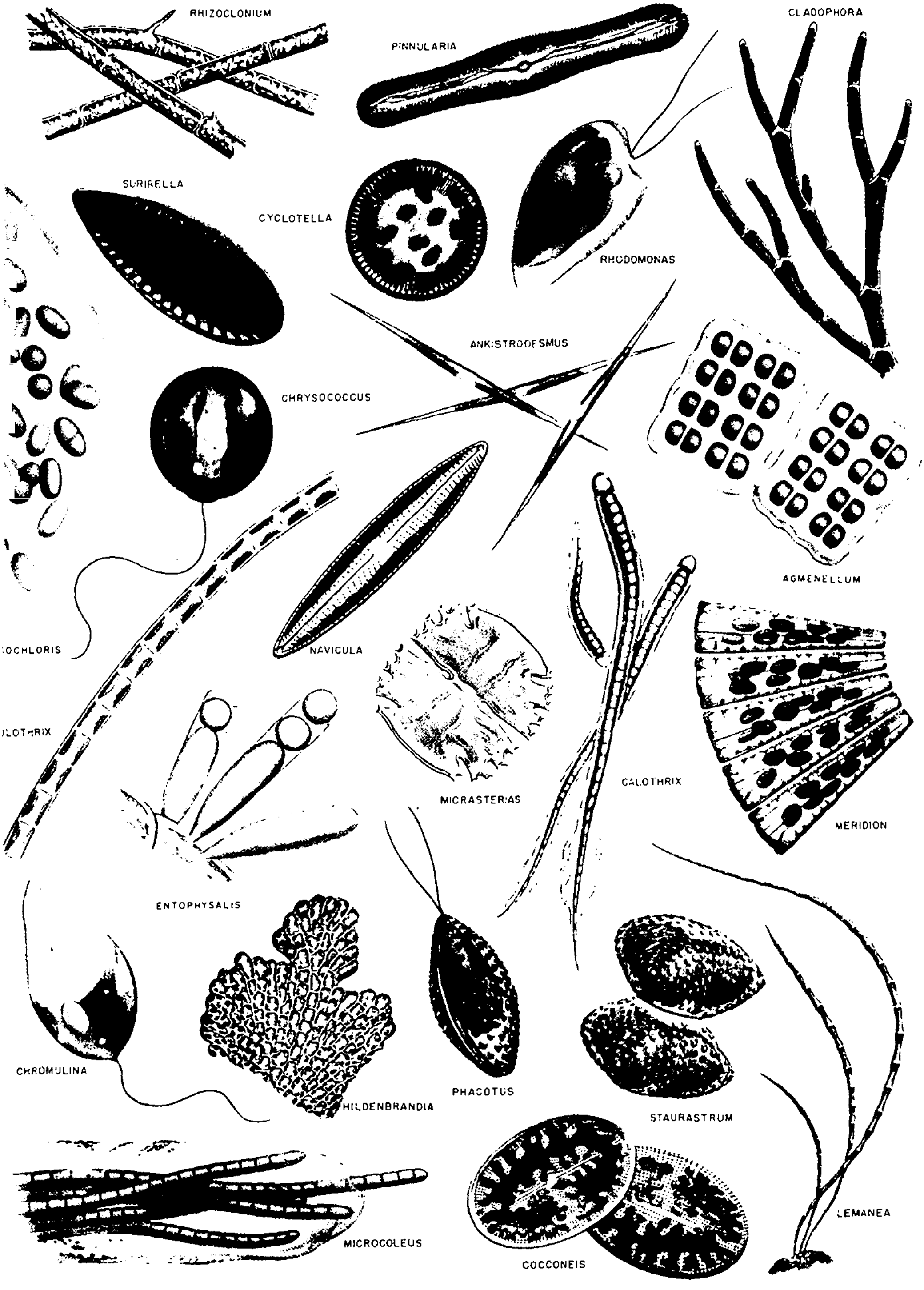
MICRO-ORGANISMS ENCOUNTERED IN CLEAN WATERS

Group and algae	Figure and linear enlargement
Blue-green algae (Myxophyceae):	
Agmenellum quadriduplicatum	26 (×188)
Calothrix parietina	26 (×375)
Coccochloris stagnina	26 (×750)
Entophysalis lemaniae	26 (×1125)
Microcoleus subtorulosus	26 (×375)
Phormidium inundatum	
Green algae (nonmotile Chlorophyceae):	
Ankistrodesmus falcatus var. *acicularis*	26 (×750)
Bulbochaete mirabilis	
Chaetopeltis megalocystis	
Cladophora glomerata	26 (×75)
Draparnaldia plumosa	
Euastrum oblongum	27 (×375)
Gloeococcus schroeteri	
Micrasterias truncata	26 (×188)
Rhizoclonium hieroglyphicum	26 (×188)
Staurastrum punctulatum	26 (×750)
Ulothrix aequalis	26 (×188)
Vaucheria geminata	
Red algae (Rhodophyceae):	
Batrachospermum vagum	
Hildenbrandia rivularis	26 (×375)
Lemanea annulata	26 (¼ reduction)
Diatoms (Bacillariophyceae):	
Amphora ovalis	
Cocconeis placentula	26 (×750)
Cyclotella bodanica	26 (×375)
Cymbella cesati	
Meridion circulare	26 (×750)
Navicula exigua var. *capitata*	
Navicula gracilis	26 (×750)
Nitzschia linearis	
Pinnularia nobilis	26 (×188)
Pinnularia subcapitata	
Surirella splendida	26 (×375)
Synedra acus var. *angustissima*	
Flagellates (Chrysophyceae, Cryptophyceae, Euglenophyceae, Volvocales of Chlorophyceae):	
Chromulina rosanoffii	26 (×3000)
Chroomonas nordstetii	
Chroomonas setoniensis	
Chrysococcus major	
Chrysococcus ovalis	
Chrysococcus rufescens	26 (×3000)
Dinobryon stipitatum	
Euglena ehrenbergii	
Euglena spirogyra	
Mallomonas caudata	23 (×375)
Phacotus lenticularis	26 (×1500)
Phacus longicauda	
Rhodomonas lacustris	26 (×1600)

From Palmer (1959) p. 42.

FIG. 26

MICRO-ORGANISMS ENCOUNTERED IN CLEAN WATERS



From Palmer (1959) p. 32.

TABLE 41

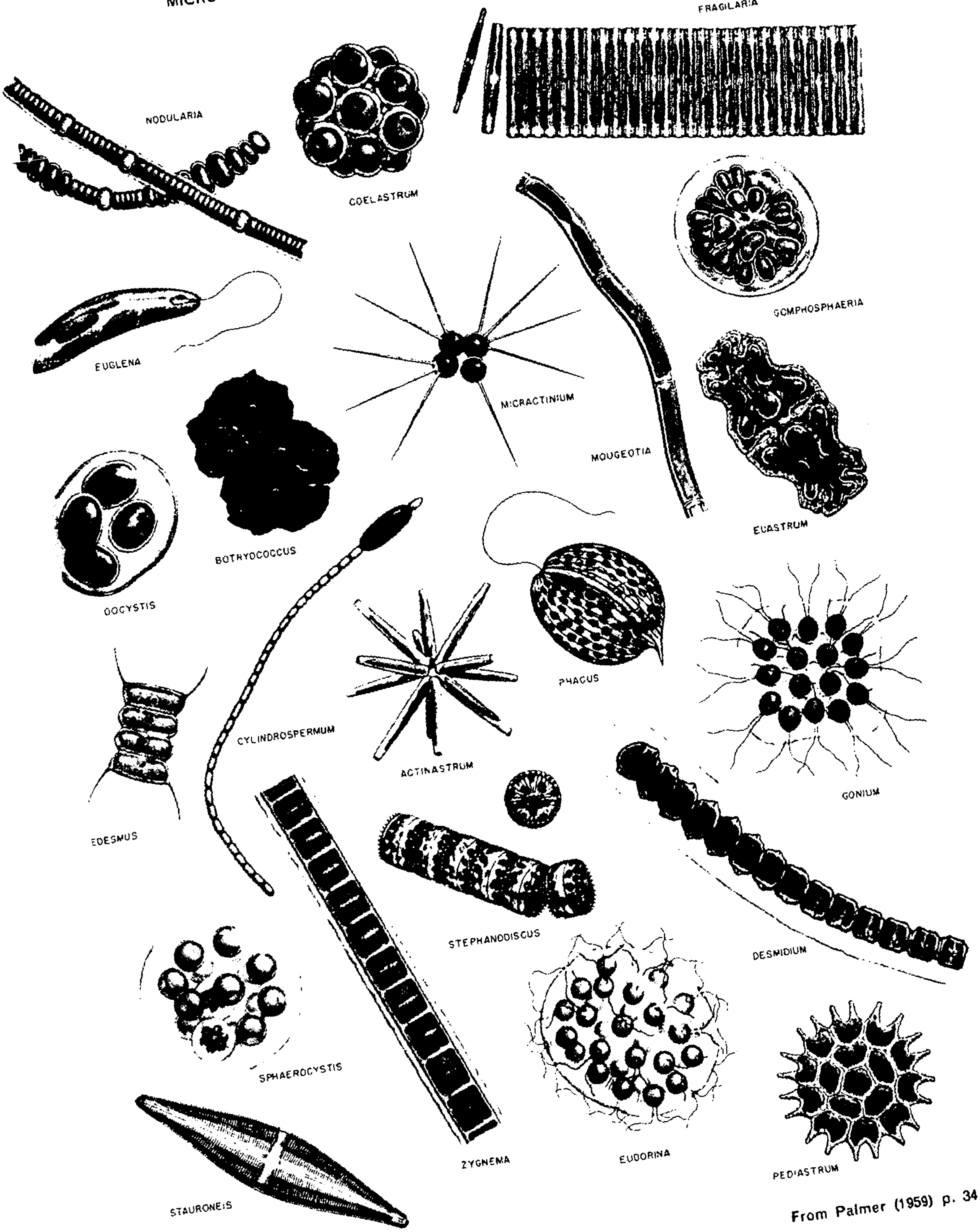
MICRO-ORGANISMS ENCOUNTERED IN SURFACE WATERS

Group and algae	Figure and linear enlargement
Blue-green algae (Myxophyceae) :	
Anacystis thermalis	
Cylindrospermum stagnale	27 (×188)
Gloeotrichia natans	
Gomphosphaeria aponina	27 (×1125)
Gomphosphaeria lacustris	
Gomphosphaeria wichurae	
Lyngbya versicolor	
Nodularia spumigena	27 (×375)
Nostoc carneum	
Oscillatoria agardhii	
Phormidium retzii	
Plectonema tomasiniana	
Scytonema tolypothricoides	
Spirulina nordstetii	
Filamentous green algae (Chlorophyceae, Chrysophyceae) :	
Cladophora fracta	
Desmidium grevillei	27 (×188)
Hyalotheca mucosa	
Mougeotia genuflexa	
Mougeotia scalaris	27 (×188)
Oedrogonium idioandrosporum	
Spirogyra fluviatilis	
Spirogyra varians	
Stigeocolonium stagnatile	
Tribonema minus	
Ulothrix tenerrima	
Vaucheria terrestris	
Zygnema pectinatum	
Zygnema sterile	27 (×188)
Nonfilamentous nonmotile green algae (Chlorophyceae) :	
Actinastrum gracillimum	27 (×750)
Actinastrum hantzschii	
Ankistrodesmus falcatus	
Botryococcus braunii	27 (×750)
Chlorella ellipsoidea	
Closterium aciculare	
Nonfilamentous nonmotile green algae (Chlorophyceae) (*cont.*) :	
Coelastrum microporum	27 (×375)
Cosmarium botrytis	
Crucigenia quadrata	
Dimorphococcus lunatus	
Euastrum oblongum	27 (×375)
Kirchneriella lunaris	
Micractinium pusillum	27 (×750)
Oocystis borgei	27 (×750)
Ophiocytium capitatum	
Pediastrum boryanum	27 (×94)
Pediastrum duplex	
Scenedesmus bijuga	
Scenedesmus dimorphus	
Scenedesmus quadricauda	27 (×750)
Schroederia setigera	
Selanastrum gracile	
Sphaerocystis schroeteri	27 (×375)
Staurastrum polymorphum	
Diatoms (Bacillariophyceae) :	
Cyclotella glomerata	
Cymatopleura solea	
Fragilaria capucina	27 (×750)
Gyrosigma attenuatum	
Melosira crenulata	
Navicula radiosa	
Rhizosolenia gracilis	
Rhopalodia gibba	
Stauroneis phoenicenteron	27 (×375)
Stephanodiscus hantzschii	27 (×750)
Synedra capitata	
Flagellates (Chrysophyceae, Euglenophyceae, Volvocales of Chlorophyceae) :	
Dinobryon sociale	
Eudorina elegans	27 (×188)
Euglena gracilis	27 (×750)
Gonium pectorale	27 (×375)
Phacus pleuronectes	27 (×375)

From Palmer (1959) p. 46.

FIG. 27

MICRO-ORGANISMS ENCOUNTERED IN SURFACE WATERS



From Palmer (1959) p. 34.

TABLE 42

MICRO-ORGANISMS THAT GROW ON WALLS OF RESERVOIRS

Group and algae	Figure and linear enlargement
Blue-green algae (Myxophyceae):	
Calothrix braunii	
Lyngbya lagerheimii	28 (×750)
Lyngbya ocracea	
Nostoc pruniforme	
Oscillatoria tenuis	
Phormidium retzii	
Phormidium uncinatum	28 (×188)
Stigonema minutum	
Tolypothrix tenuis	28 (×375)
Green algae (nonmotile Chlorophyceae, Charophyceae):	
Bulbochaete insignis	28 (×94)
Chaetophora attenuata	
Chaetophora elegans	28 (×188)
Chara globularis	28 (×3)
Cladophora crispata	28 (×94)
Cladophora glomerata	26 (×75)
Draparnaldia glomerata	28 (×94)
Gloeocystis gigas	
Microspora amoena	28 (×188)
Nitella flexilis	
Oedogonium boscii	
Oedogonium grande	
Oedogonium suecicum	28 (×375)
Palmella mucosa	24 (×750)
Phytoconis botryoides	28 (×750)
Pithophora oedogonia	
Rhizoclonium hieroglyphicum	26 (×188)
Schizomeris leibleinii	
Stigeoclonium lubricum	28 (×188)
Tetraspora gelatinosa	28 (×94)
Ulothrix zonata	28 (×188)
Vaucheria geminata	
Vaucheria sessilis	28 (×94)
Red algae (Rhodophyceae):	
Audouinella violacea	28 (×188)
Batrachospermum moniliforme	28 (×2)
Compsopogon coeruleus	28 (×94)
Diatoms (Bacillariophyceae):	
Achnanthes microcephala	28 (×1125)
Cocconeis pediculus	
Cymbella prostrata	28 (×188)
Epithemia turgida	
Gomphonema geminatum	28 (×188)
Gomphonema olivaceum	
Rhoicosphenia curvata	

From Palmer (1959) p. 48.

FIG. 28

MICRO-ORGANISMS THAT GROW ON WALLS OF RESERVOIRS

From Palmer (1959) p. 36.

Fig. 23 shows some of the plankton of greatest interest to operators: those causing tastes and odours. It should be noted that this illustration is not a representation of organisms seen as a group under a microscope but rather a collection of individual drawings of specific organisms, drawn to various scales. In an actual sample, one organism, such as *Anabaena*, may be so numerous that 90 % of all organisms seen when the sample is examined microscopically will be of this one species. This is true of the organisms shown in the following illustrations also.

Fig. 24 shows the appearance of micro-organisms associated with filter clogging; several of these also cause tastes and odours. Rapid clogging of filters has been reported with concentrations of these organisms from 700 to several thousand area standard units.

Fig. 25 illustrates those micro-organisms prevalent in polluted waters. The relative number of algae and protozoa shown in Fig. 25 has no specific reference to the ratio of algae and protozoa present in polluted waters or observed under the microscope when polluted waters are examined.

Fig. 26 shows the micro-organisms prevalent in clean water, where green algae predominate.

Fig. 27 illustrates miscellaneous micro-organisms present in surface waters. Here again algae predominate.

Finally, Fig. 28 shows algae that grow on the walls of reservoirs, causing the characteristic green coating.

It will be noted that many of the organisms listed in Table 2 (pages 37-39) are shown in the six illustrations. Experience will facilitate the co-ordination between: (*a*) observed odours; (*b*) number and types of micro-organisms found by the microscopic examination of appropriate samples; (*c*) doses of copper sulfate found to be effective in suppressing growths of odour-producing micro-organisms; and (*d*) related observations as to conditions surrounding the supply during the period when the specific micro-organisms were increasing in number to a troublesome extent.

Detection of pollution of water by microscopic examination

With experience in the microscopic examination of water, operators can rapidly appraise the degree of pollution by noting the number and type of micro-organisms prevailing at a selected point, such as the site of a proposed water-supply intake. Inasmuch as these organisms are influenced by environmental factors reflecting average, prolonged conditions rather than those which may prevail at the moment when a sample is collected, the biological factors are more representative than those indicated by chemical analysis of a single sample of water. In general, certain organisms are prevalent in unpolluted waters, and others are found in polluted waters. Therefore a stream above a significant source of pollution will support the first group, characterized by green algae. Below the point of pollution, sludge banks may be formed in the bed of a stream, and rat-tailed maggots, bloodworms, the sludge worm *Tubifex*, the sewage fungus *Sphaerotilus*, and the mould *Liptomitus* may be observed even without microscopic examination. Microscopic examination would disclose protozoan ciliates as well. If pollution is heavy, dissolved oxygen will be exhausted and decomposition of organic matter will be accompanied by a black or grey appearance of the water. No green vegetation will be present, and there will be an offensive odour.

Re-aeration of the water as it flows downstream will restore conditions favourable to the growth of green plants, so the protozoa characteristic of polluted waters will be replaced downstream by an increasing proportion of algae, until self-purification has progressed to a point where the stream is restored to normal and algae may predominate again. In other instances higher forms of life using algae as food may predominate. Thus the zone of pollution is self-evident, but the degree of recovery at any given point can be determined only by analysis for the dissolved-oxygen content and the biological oxygen demand (BOD), by microscopic examinations, and by the test for coliform organisms. These will indicate the related environmental factors characteristic of the several zones of a polluted stream.

REFERENCES

American Public Health Association, American Water Works Association, and Water Pollution Control Federation (1960) *Standard methods for the examination of water and waste-water*, 11th ed., New York, American Public Health Association

American Water Works Association (1950) *Water quality and treatment*, 2nd ed., New York

American Water Works Association (1958) *Specifications for deep wells*, New York (Publication A100-58)

Baylis, J. R. (1940) Filter operation and maintenance, *J. New England Water Works Ass.*, 54, No. 4, 393

Cox, C. R. (1952) *Water supply control*, Albany, N.Y., New York State Department of Health [out of print]

Cox, C. R. (1958) *Water analytical procedures for use by water works operators and inspectors*, Kingston, Jamaica, Ministry of Health [out of print; expanded and used as the Supplement to this monograph]

Cox, C. R. (1959) *Laboratory control of water purification*, New York, Reuben H. Donnelley Corporation

Cox, C. R. (1960) *Guides to the design of water treatment plants*, Washington, D.C., US Agency for International Development [not generally available]

Hale, F. (1942) *The use of copper sulfate in control of microscopic organisms*, New York, Phelps Dodge Refining Corporation

Institution of Water Engineers, The Royal Institute of Chemistry, The Society for Analytical Chemistry and the Society for Water Treatment and Examination (1960) *Approved methods for the physical and chemical examination of water*, 3rd ed., London, The Institution of Water Engineers

International Water Supply Association (1961) *Report of Technical Commission on Pollution of Surface Waters*, London

Kellermann, K. F. (1912) The rational use of disinfectants and algicides in municipal water supplies. In: *Proceedings of the Eighth International Congress of Applied Chemistry*, Vol. 26, p. 24

Miller, A. P. (1962) *Water and man's health*, Washington, D.C., US Agency for International Development

New York State Department of Health, *Manual of instructions for water treatment plant operators*, Albany, N.Y., New York State Department of Health

Palmer, C. M. (1959) *Algae in water supplies*, Washington, D.C., Government Printing Office (Public Health Service Publication No. 657)

United States Air Force (1959) *Maintenance and operation of water plants and systems*, Washington, D.C. (Air Force Manual 85-13)

United States Public Health Service (1961) *Effectiveness of water treatment processes*, Washington, D.C. (Public Health Service Publication No. 898)

United States Public Health Service (1962) *Public Health Service drinking-water standards, 1962*, Washington, D.C. (Public Health Service Publication No. 956)

Wagner, E. G. & Lanoix, J. N. (1959) *Water supply for rural areas and small communities*, Geneva (*World Health Organization : Monograph Series*, No. 42)

Welch, W. A. (1963) Potassium permanganate in water treatment, *J. Amer. Water Works Ass.*, 55 (No. 6), 735

Whipple, G. C. (1948) *The microscopy of drinking water*, 4th ed., rev. by G. M. Fair & M. C. Whipple, New York, Wiley

Willey, B. F. & Jennings, H. (1963) Iron and manganese removal with potassium permanganate, *J. Amer. Water Works Ass.*, 55 (No. 6), 729

World Health Organization (1963) *International standards for drinking-water*, 2nd ed., Geneva

World Health Organization (1961) *European standards for drinking-water*, Geneva

SELECT BIBLIOGRAPHY

Akademija nauk URSR, Rada po vyvcermju produktyvnyh syl URSR [Academy of Sciences of the Ukrainian SSR, Council for the Study of the Productive Resources of the Ukrainian SSR] (1959) *Sanitarna ohorona vodojmysc i ocystka promyslovyh sticnyh vod* [Sanitary protection of watercourses and the purification of waste waters], Kyjiv

Agamennone, G. & Zanchi, C. (1956) *Le acque per le caldaie. Loro trattamento e controlli analitici*, Milano, Hoepli

American Chemical Society (1961) *Objectives and status of the federal water conservation program*, Washington, D.C.

American Standards Association (1955) *American standard national plumbing code*, New York, American Society of Mechanical Engineers

Babbitt, H. E. & Doland, J. J. (1955) *Water supply engineering*, New York, McGraw-Hill

Beger, H. (1949) *Leitfaden der Brunnenhygiene*, Freiburg, Cantor

Brix, J., Heyd, H. & Gerlach, H. (1952) *Die Wasserversorgung*, München, Oldenburg Verlag

Buonomini, G. & De Blasi, R. (1950) *L'esame batteriologico delle acque*, Roma, Associazione italiana per l'Igiene

Canaperia, G. A. & Garaci, C. (1951) L'approvvigionamento idrico in Italia, *Ann. Sanità pubbl.*, **12**, 375-532

Centre Belge d'Etude et de Documentation des Eaux (1954) *Livre de l'eau*, Liège

Dahlhaus, C. (1959) *Wasserversorgung*, Stuttgart, Teubner Verlag

Eggenhoffner, G. (19?) *Trattamento dell'acqua per usi industriali e navali*, Torino, Andrea Viglongo

England and Wales, Ministry of Health & Ministry of Housing and Local Government (1956) *The bacteriological examination of water supplies*, 3rd ed., London (Reports on Public Health and Medical Subjects, No. 71)

Fago, S. V. (1941) *Filtrazione rapida per acquedotti*, Milano, Hoepli

Fair, G. M. & Guyer, J. C. (1954) *Water supply and waste-water disposal*, New York, Wiley

France, Ministère de la Santé publique et de la population (1954) Instructions générales relatives aux eaux d'alimentation, *J. off. Rep. franc.*, **86**, 11928 (No. 298)

Gallizio, A. (1952) *Impianti sanitari*, Milano, Hoepli

Gesellschaft Deutscher Chemiker, Fachgruppe Wasserchemie (1960) *Deutsche Einheitsverfahren zur Wasser-, Abwasser- und Schlammuntersuchung*, 3rd ed., Weinheim, Verlag Chemie

Hentze, J. (1951) *Construcciones hidráulicas*, Barcelona, Editorial Labor

Hirose, K. (1961) *Municipal water supply*, Tokyo, Gihodo [in Japanese]

Hubner, H. (1956) *Grundlagen der Siedlungswasserwirtschaft*, Berlin, Verlag Technik

Ibuka, K. (1961) *Execution of waterworks*, Sankaido [in Japanese]

Institution of Water Engineers (1961) *Manual of British water supply practice*, 3rd ed., Cambridge, Heffer

Ippolito, G. (19?) *Appunti delle lezioni di costruzioni idrauliche*, Napoli, Libreria Internazionale Treves

Iwasaki, T. & Tanaka, T. (1957) *Sanitary engineering: Water supply and sewerage*, Tokyo, Morikita [in Japanese]

Jentsch, O. (1956) *Aufbereitung des Wassers*, Berlin, Verlag Technik

Kastal'skij, A. A. & Minc, D. M. (1962) *Podgotovka vody dlja pit'evogo i promyšlennogo vodosnabženija* [Water treatment for drinking and industrial water supply], Moskva, Vysšaja Škola

Lazaro Urra, J. (1955) *Hidrologia urbanista*, Madrid, Dossat

Leclerc, E. (1945) *Eaux potables et eaux résiduaires : les techniques de correction et d'épuration*, Liège, Vaillant-Carmanne

Le Strat, Blasco, J. & Luengo, L. (1962) *Primer coloquio sobre contaminación de aguas superficiales*, Barcelona

Mihajlov, V. A. & Novakovskij, N. S. (1961) *Avromatizacija očistnyh stancij* [Automation of purifying plants], Moskva

Mistrangelo, C. (1952) *Provvista e distribuzione di acqua potabile*, 5th ed., Milano, Hoepli

Mugele, G. F. & Wiseman, A. (1958) *Water treatment*, London, Newnes

Mutschmann, J. & Stimmelmayr, F. (1958) *Taschenbuch der Wasserversorgung*, Stuttgart, Teubner Verlag

Neri, F. (1930) *Acqua potabile, ricerca, raccolta e distribuzione*, Torino, U.T.E.T.

Ohashi, K. (1954) *Water supply*, Tokyo, Riko-Tosho [in Japanese]

Oliver Suñé, B. (1957) *Problemas organolepticos en los procedimientos modernos de depuración de aguas*, Barcelona

Paz Maroto, J. & Paz Casañé, M. (1955) *Urbanismo y servicios urbanos*, Madrid, Dossat

Puig, J. B. (1948) *El agua en la industria textil*, Barcelona

Rivas Mijares, G. (1959) *Abastecimientos de agua y alcantarillados (acueductos y cloacas)*, Madrid, Nuevas Gráficas

Sallovitz, M. & Sallovitz, M. J. (1951) *Tratado de ingeniería sanitaria*, 5th ed., Buenos Aires, El Ateneo

Shaw, J. H. (1954) *Fluoridation as a public health measure*, Washington, American Association for the Advancement of Science

Sociedad Civil de Empresas Summistradoras de Aguas (1958) *La contaminación industrial de los rios*, Barcelona

Société Degrémont-France (1963) *Memento technique de l'eau*, 5th ed., Suresnes [2nd English ed., *Water treatment handbook*, 1960; Spanish ed., *Manual técnico del agua*, 1959]

South African Council for Scientific and Industrial Research, National Chemical Research Laboratory (1960) *Demineralization by electrodialysis*, London, Butterworth

Steel, E. W. (1960) *Water supply and sewerage*, 4th ed., New York, McGraw-Hill [Spanish ed.: *Abastecimiento de agua y alcantarillado*, Barcelona, Gili]

Taylor, E. W. (1958) *The examination of waters and water supplies (Thresh, Beale & Suckling)*, 7th ed., London, Churchill

USSR, Ministry of Health (1963) *Stroitel'nye normy i pravila*, Čast II, Razdel G, Glava 3, Vodosnabženie normy proektirovanija [Building standards and regulations, Part II, Section G, Chap. 3, Water supply: Planning standards] Moskva

Venezuela, Instituto Nacional de Obras Sanitarias (1948) *Normas para el diseño de los abastecimientos de agua*, Caracas, Impr. Nacional

Žukov, A. I., Mongajt, I. L. & Rodziller, I. D. (1963) *Očistka promyšlennyh stočnyh vod* [Purification of industrial effluent], Moskva, Gosstorjizdat [in press]

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INDEX

INDEX

Figures in bold type are page numbers of chapters