Introduction to Potable Water Treatment Processes

Course Content

Introduction

There are various water treatment methods available for making water potable and remove materials that may cause damage to the plant equipment or that cause problems in the major treatment process. The available processes used depend upon the characteristics and source, of the raw, or untreated, water. The raw water that is to be treated comes from either ground-water sources (wells or springs) or surface-water sources (rivers or lakes).

Surface-water sources require more extensive treatment due to the effects of rainfall and runoff. In rivers, runoff from rains causes small particles of solids to mix with the river water. This causes the river to appear "muddy" or turbid. Streams can contain high concentrations of suspended sediment. These small particles must be removed and the water disinfected to make it fit for human consumption and use. Groundwater is often more pure than surface water because it has filtered down through the soil. As a result of this filtration, groundwater contains less bacteria than surface waters do. But it still contains some bacteria and dissolved solids, so it has to be treated before it can be used for potable purposes.

Preliminary treatment processes that are employed for removal of debris and part of the sediment load include:

1) **SCREENS**

   a) *Coarse screens or racks* - Coarse screens, often termed bar screens or racks, are provided to intercept large suspended or floating material. Such screens or racks are made of l/2-inch to 3/4-inch metal bars spaced to provide 1- to 3-inch openings.

   b) *Fine screens* - Surface waters require screens or strainers for removal of material too small to be intercepted by the coarse rack. These may be basket-type, in-line strainers, manually or hydraulically cleaned by backwashing, or of the traveling type, cleaned by water jets. Fine-screen, clear openings should be approximately 3/8 inch. The velocity of the water in the screen openings should be less than 2 feet per second at maximum design flow through the screen and minimum screen submergence.

   c) *Ice clogging* - In northern areas screens may be clogged by frazil or anchor ice. Exposure of racks or screens to cold air favors ice formation on submerged parts and should be avoided to the maximum practicable extent. Steam or electric heating, diffusion aeration and flow reversal may be used to overcome ice problems.

   d) *Disposal of screenings* - Project planning must include provision for the disposal of debris removed by coarse and fine screens.

2) SAND TRAPS

Sand traps are not normally required at surface water treatment plants. Their principal application is for the removal of fine sand from well water. The presence of sand in well water is usually a sign of improper well construction or development. If sand pumping cannot be stopped by reworking the well, the sand must be removed. Otherwise, it will create serious problems in the distribution system by clogging service pipes, meters, and plumbing. Centrifugal sand separators are an effective means of sand removal. These cyclone-separator devices are available assembled from manufacturers and require no power other than that supplied by the flowing water. They operate under system pressure; therefore, re-pumping is not necessary. Water from the well pump enters tangentially into the upper section of the cone and centrifugal force moves the sand particles to the wall of the cone. They then pass down-water into the outlet chamber. Sand is periodically drained to waste from this chamber through a valve that can be manually or automatically operated. The clarified water is discharged from the top of the cone. These units are available in diameters of 6, 12, 18, 24, and 30 inches providing a capacity range from 15 to 4500 gallons per minute (gpm) and are suitable for operation up to 150 pounds per square inch (psi). Pressure drop through the unit ranges from 3 to 25 psi, depending on unit size and flow rate. These separators will remove up to 99 percent of plus 150 mesh sand and about 90 percent of plus 200mesh. The units are rubber lined for protection against sand erosion.

3) PLAIN SEDIMENTATION

Plain sedimentation, also termed "pre-sedimentation" is accomplished without the use of coagulating chemicals. Whether plain sedimentation is essential is a judgment decision influenced by the experience of plants treating water from the same source. Water derived from lakes or impounding reservoirs rarely requires pre-sedimentation treatment. On the other hand, water obtained from notably sediment-laden streams, such as those found in parts of the Middle West, requires pre-sedimentation facilities for removal of gross sediment load prior to additional treatment. Pre-sedimentation treatment should receive serious consideration for water obtained from rivers whose turbidity value frequently exceeds 1,000 NTUs (Nephelometric Turbidity Unit). Groundwater in comparison to surface water is generally lower in turbidity and higher in mineral content.

a) *Plain sedimentation basins* - Plain sedimentation or pre-sedimentation basins may be square, circular, or rectangular and are invariably equipped with sludge removal mechanisms.

b) *Design criteria* - Detention time should be approximately 3 hours. Basin depth should be in the approximate range of 10 to 15 feet, corresponding to upflow rates of 600 to 900 gallons per day (gpd) per square foot for a detention period of 3 hours. Short-circuiting can be minimized by careful attention to design of inlet and outlet arrangements. Weir loading rates should not exceed approximately 20,000gpd per foot. Where pre-sedimentation treatment is continuously required, duplicate basins should be provided. Basin bypasses and overflows should also be included.

AERATION

The term "aeration" refers to the processes in which water is brought into contact with air for the purpose of transferring volatile substances to or from water. These volatile substances include oxygen, carbon dioxide, hydrogen sulfide, methane and volatile organic compounds responsible for tastes and odor. Aeration is frequently employed at plants treating ground water for iron and manganese removal.

1) PURPOSE OF AERATION

The principle objectives of aeration are:

a) Addition of oxygen to ground water for the oxidation of iron and manganese. By mixing the water with air, oxygen in the air comes in contact with these minerals. The minerals are oxidized and precipitate, or settle out of the water. Note that the ground waters are normally devoid of dissolved oxygen.

b) If water has a high concentration of free carbon dioxide, it will have a low pH which can cause corrosion of pipes when the water is distributed. Aeration allows some of the carbon dioxide to leave the water and enter the air, raising the pH to a more normal level.

c) Reduction of the concentration of taste-and-odor producing substances, such as hydrogen sulfides and volatile organic compounds. Hydrogen sulfide and other compounds can cause foul odors in water.

d) Removal of volatile organic compounds which are suspected carcinogens.

2) *SURFACE AREA*

The goal of an aerator is to increase the surface area of water coming in contact with air so that more air can react with the water. As you break air or water up into small drops/bubbles or thin sheets, the same volume of either substance has a larger surface area. Consider three different sets of water drops, drawn as squares to make it easy to measure the volume and surface area.

Volume = 4
Surface area = 8

Volume = 4
Surface area = 10

Volume = 4
Surface area = 16
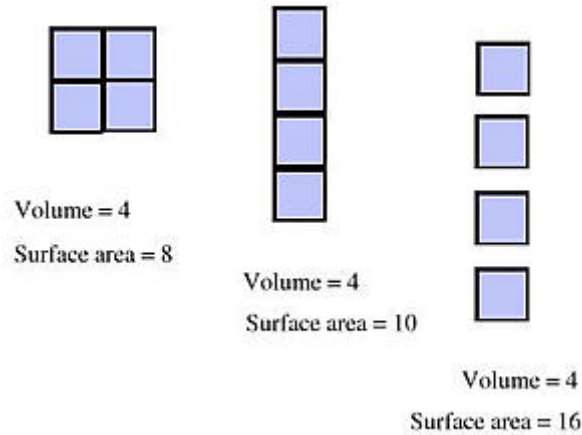
The volume is the amount of water in each drop. The drop on the left has a volume of four, as does the elongated drop in the center. The small drops on the right each have a volume of one, so all four together have the same volume as each of the other two shapes.

The surface area is the length of a line drawn all the way around an object. The large drop has a surface area of 8, but the other two shapes have greater surface areas. As you can see, a thin sheet of water (like the elongated drop) has an intermediate surface area. When a drop of water is broken up into several smaller drops (like the four drops on the right), the surface area is greatly increased.

3) TYPES OF AERATORS

Three types of aerators are commonly employed. These are: a) waterfall aerators exemplified by spray nozzle, cascade, and multipletray units; b) diffusion or bubble aerators which involve passage of bubbles of compressed air through the water; and c) mechanical aerators employing motor-driven impellers alone or in combination with air injection devices. Of the three types, waterfall aerators, employing multiply trays, are the most frequently used in water treatment. The efficiency of multiple-tray aerators can be increased by the use of enclosures and blowers to provide counterflow ventilation.

3) METHODOS OF AERATION

There are several different methods used to aerate water, but all either involve passing water through air or air through water. Water can be exposed to air by spraying or by distributing it in such a way that small particles or thin sheets of water come in contact with the air. Water can also be aerated by pumping large volumes of air through the water.

The method of aeration to be used depends on which materials on the water are to be removed. The chemical characteristics of the water to be treated can also influence the treatment methods. Finally, each method has a different efficiency. In general, pumping water through air is much more energy efficient than pumping air through water. Different types of aeration and other methods of treatment should all be compared to determine the most efficient and practical method of treatment in each case.

4) DESIGN CRITERIA

A. Multiple-tray, tower aerators

a) Multiple-tray aerators are constructed of a series of trays, usually three to nine, with perforated, slot or mesh bottoms. The water first enters a distributor tray and then falls from tray to tray, finally entering a collection basin at the base. The vertical opening between trays usually ranges from 12 inches to 30 inches. Good distribution of the water over the entire area of each tray is essential. Perforated distributors should be designed to provide a small amount of head, approximately 2 inches on all holes, in order to insure uniform flow. In aerators with no provision for forced ventilation, the trays are usually filled with 2- to 6-inch media, such as coke, stone, or ceramic balls to improve water distribution and gas transfer and to take advantage of the catalytic oxidation effect of manganese oxide deposits in the media. The water loading on aerator trays should be in the range of 10 to 20gpm per square foot. Good, natural ventilation is a requirement for high efficiency. For multiple tray aerators designed for natural ventilation, the following empirical equation can be used to estimate carbon dioxide (CO_2) removal:

$C_r = C_o \, (10^{-kn})$

o C_r = mg/L CO_2 remaining after aeration

o C_o = mg/L CO_2 present in water in distribution tray

o   n = number of trays, including distribution tray

o   k = 0.11 to 0.16 depending on temperature, turbulence, ventilation etc

Where icing is a problem and the aerator must be housed, artificial ventilation by fans or blowers is necessary. An enclosed induced- or positive-draft aerator requires approximately 3.5 to 6 standard cubic feet of ventilating air per gallon of water aerated. Thus, for an enclosed aerator operating at a rate of 1.5 million gallons per day (mgd), air requirements will be in the range of 3600 to 6200 standard cubic feet of air per minute. Positive-draft aerators employing the higher air-flow rates exhibit the highest efficiency for the addition and removal of dissolved gases and oxidation of iron, manganese, and sulfide. Power requirements for a natural draft, multiple-tray aerator having an overall height of 10 feet will be approximately 1.7 kilowatts per mgd of aeration capacity. Power demands for forced draft units will be greater.

b) *Counter-current packed column aeration* - A counter-current parked column aerator tower is similar to operation to counter-current multiple tray aerators, but are particularly efficient at the removal of volatile organic compounds (VOCs) through air-stripping. Packed column aerators consist typically of a long thin tower filled with either a random dumped media (Rasching rings, Ber) saddles, Pall rings) or corrugated sheet media, held by a packing support plate. Water is

pumped to the top of the tower over a distribution plate and allowed to fall through the media. Air is blown up through the tower by a fan counter to the falling water. Redistributor plates are used throughout the column to prevent channeling of the water or air stream. Efficiency of the tower is dependent on the extent of contact between the air and water. Detailed design can be found in various chemical engineering literatures and handbooks or AWWA, EPA publications.

B. *Diffusion aerators*

Compressed air is injected into the water as it flows through a rectangular basin. A variety of air injection devices may be employed including perforated pipes, porous plates or tubes and various patented sparger devices. Basin size is determined by desired detention time, which commonly ranges from 10 to 30 minutes. Tank depth is usually from 10 to 15 feet. Air requirements, supplied by a compressor, generally range from 0.1 to 0.2 standard cubic foot per gallon of water aerated. Major advantages of a diffusion aeration system include practically no head loss and freedom from cold-weather operating problems. An additional advantage is that a diffusion aerator may also be used to provide chemical mixing. Power requirements are those associated with air compression and range from 1.0 to 2.0 kilowatts per mgd of aerator capacity. Aeration efficiency in terms of addition of oxygen or removal of carbon dioxide is generally similar to that provided by multiple-tray aerators employing natural ventilation.

C. *Mechanical aerators*

Mechanical aerators typically consist of an open impeller operating on the waters surface. Basin size is determined by detention time required. Basin depth can vary from 5 to 17 feet with the average depth being 10 feet. Major advantages of mechanical aerators are practically *no head loss and the ability to provide mixing.* Mechanical aerators are generally not as efficient as aeration towers or diffused aerators and longer detention times are required.

4) CRITERIA FOR INSTALLATION OF AERATORS

Aeration is a gas transfer process which is not needed at all water treatment plants. A decision as to whether to aerate or not requires assessment of the economic and water quality benefits achieved by its use.

a) *Addition of oxygen* - Aeration processes are commonly used in adding oxygen to ground waters and to oxidize iron, manganese, hydrogen sulfide and to a limited extent, organic matter. Ground waters are usually deficient in oxygen and aeration is an effective means of adding it. Oxygen addition is normally required if iron and manganese removal is a treatment objective. Aeration will also help oxidize hydrogen sulfide and some organic matter.

b) *Partial removal of volatile substances* - Aeration is a useful method of removing volatile substances from water. Ground waters while being deficient in oxygen can contain objectionable levels of carbon dioxide. An efficient aerator will result in near saturation with

oxygen and about 90 percent reduction of the carbon dioxide content of groundwater. At lime-soda water softening plants, any carbon dioxide dissolved in the water at the point of lime application will consume lime without accompanying softening. For high (>50 mg/L) carbon dioxide concentrations, as encountered in some ground waters, aeration for its removal is probably justified. For concentrations on the order of 10 mg/L, or less, aeration is probably not economically valid. Before deciding to aerate for carbon dioxide removal, the cost of purchasing, maintaining and operating the aerator should be compared to the value of the lime saved. At softening plants, each mg/L of carbon dioxide removed will effect a saving of about 1.3 mg/L quicklime (95 percent calcium oxide). It will also reduce the quantity of softening sludge produced proportionately.

c) *Reduction of hydrogen sulfide* - Aeration is also used for removing hydrogen sulfide from well water. It may be sufficient in itself if the hydrogen sulfide concentration is not more than about 1.0 or 2,0 mg/L. Otherwise, it maybe used in conjunction with chlorine to oxidize the hydrogen sulfide not removed by aeration.

d) *Reduction of Volatile Organic Compounds (VOCs)* - Recent studies has shown that aeration can be successfully employed to reduce volatile organic compounds (VOCs) such as total Trihalomethane (THM) concentration in chlorinated water to meet current US EPA regulations limiting TTHM concentrations. Aeration by diffused air or multiple-tray aerators can reduce TTHM concentration at low cost, with cost increasing with higher concentrations of Trihalomethane (THM). Counter-current packed tower aeration is most efficient in achieving mass transfer of VOC.

Aeration Summary

Surface waters usually exhibit low concentrations of carbon dioxide, no hydrogen sulfide and fairly high dissolved oxygen. As a consequence, aeration is not required for the removal or addition of these gases. However, surfaces waters contain higher levels of THM precursors than ground waters and therefore a need for aeration may arise to reduce TTHM following chlorination. Water high in the bromine-containing THMs is difficult to treat by aeration and other methods of removal should be used, such as coagulation and flocculation or contact with granular activated carbon.

The efficiency of the water aeration process depends on the amount of surface contact between the air and water. In general, aeration is worthy of consideration in connection with the treatment of groundwater supplies in conjunction with lime softening and for the removal of some VOCs.

WATER CLARIFICATION

Water clarification is the process of removing suspended solids from water. While a degree of clarification can be accomplished by subsidence (settling), most industrial processes require better quality water than can be obtained from subsidence only. Most of the suspended matter in water would settle, given enough time, but in most cases the amount of time required would not be practical. The time required for settling is dependent on many factors, including:

a) Weight of the particle

b) Shape of the particle

c) Size of the particle

d) Viscosity and/or frictional resistance of the water, which is a function of temperature

The settling rates of various size particles at 50 ⁰F (10 ⁰C) is illustrated in Table below.

Settling Rates by Particle Size

Diameter of Particle (in mm)	Types of Particles(by order of Particle (mm) magnitude)	Time Required to Settle One Foot
10.0	Gravel	0.3 seconds
1.0	Coarse sand	3.0 seconds
0.1	Fine sand	38.0 seconds
0.01	Silt	33.0 minutes
0.001	Bacteria	35.0 hours
0.0001	Clay particles	230.0 days
0.00001	Colloidal particles	63.0 years

Stokes' Law

Settling velocities may be calculated from Stokes' Law.

$$V = \frac{2662(S_1 - S_2)D^2}{z}$$

Where

o   V = Velocity of fall (ft/sec)

o   D = Diameter of particle (in)

o S_1= Density of particle (lb/ft3)

- o S_2= Density of fluid (lb/ft3)

- o   z = Viscosity (centipoises)

In this equation it is assumed that the particles are spherical, failing under viscous resistance, and that they have no electrostatic charges. This is, of course, never true under actual conditions. Most suspended solids smaller than 0.1 mm found in surface waters carry negative electrostatic charges. This charge causes the particles to repel each other, increasing their stability and thus increasing their tendency to remain suspended. Chemicals are often added to water to neutralize particle charge and enhance particle settling. Chemicals used to promote suspended particle subsidence in the clarification process are commonly called coagulants; the particle charge neutralization process is called coagulation.

COAGULATION AND FLOCCULATION

Coagulation means a reduction in the forces which tend to keep suspended particles apart. The joining together of small particles into larger, settle able and filterable particles is "flocculation." Thus, coagulation precedes flocculation and the two processes must be considered conjunctively.

1) PURPOSES OF COAGULATION AND FLOCCULATION

Raw water supplies especially surface water supplies; often contain a wide range of suspended matter, including suspended minerals, clay, silt, organic debris and microscopic organisms ranging in size from about 0.001 to 1.0 micrometer. Small particles in this size range are often referred to as "colloidal" particles. Larger particles, such as sand and silt, readily settle out of water during plain sedimentation, but the settling rate of colloidal particles is so low that removal of colloidal particles by plain sedimentation is not practicable. Chemical coagulation and flocculation processes are required to aggregate these smaller particles to form larger particles which will readily settle in sedimentation basins. The coagulation-flocculation processes are accomplished step-wise by *short-time rapid mixing* to disperse the chemical coagulant followed by a longer period of *slow mixing (flocculation)* to promote particle growth.

2) COAGULATION PROCESS

Coagulation, the first step in complete clarification, is the neutralization of the electrostatic charges on colloidal particles. Because most of the smaller suspended solids in surface waters *carry a negative electrostatic charge*, the natural repulsion of these similar charges causes the particles to remain dispersed almost indefinitely. To allow these small suspended solids to agglomerate, the negative electrostatic charges must be neutralized. This is accomplished by using inorganic coagulants (water soluble inorganic compounds), organic cationic polymers or polyelectrolyte.
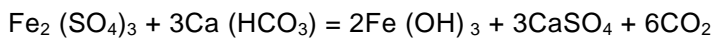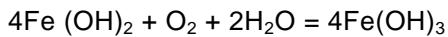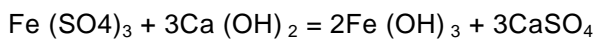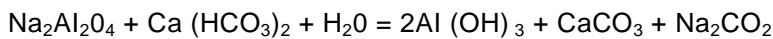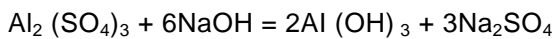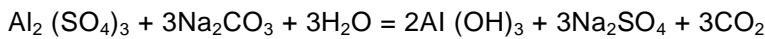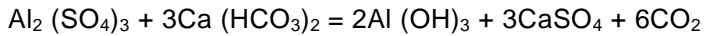
3) INORGANIC COAGULANTS

The most common inorganic coagulants are:

1. Alum-aluminum sulfate- $Al_2(SO_4)_3$

2. Ferric sulfate - $Fe_2(SO_4)_3$

3. Ferric chloride - $FeCl_3$

4. Sodium aluminate - $Na_2Al_2O_4$

Inorganic salts of metals work by two mechanisms in water clarification. The positive charge of the metals serves to neutralize the negative charges on the turbidity particles. The metal salts also form insoluble metal hydroxides which are gelatinous and tend to agglomerate the neutralized particles. The most common coagulation reactions are as follows:

Coagulation Reactions

$Al_2(SO_4)_3 + 3Ca(HCO_3)_2 = 2Al(OH)_3 + 3CaSO_4 + 6CO_2$

$Al_2(SO_4)_3 + 3Na_2CO_3 + 3H_2O = 2Al(OH)_3 + 3Na_2SO_4 + 3CO_2$

$Al_2(SO_4)_3 + 6NaOH = 2Al(OH)_3 + 3Na_2SO_4$

$Al_2(SO_4)_3 (NH4)2SO4 + 3Ca(HCO_3) = 2Al(OH)_3 + (NH_4)_2SO4 + 3CaSO_4 + 6CO_2$

$Al_2(SO_4)_3 K_2SO_4 + 3Ca(HCO_3)_2 = 2Al(OH)_3 + K_2SO_4 + 3CaSO_4 + 6CO_2$

$Na_2Al_2O_4 + Ca(HCO_3)_2 + H_2O = 2Al(OH)_3 + CaCO_3 + Na_2CO_2$

$Fe(SO4)_3 + 3Ca(OH)_2 = 2Fe(OH)_3 + 3CaSO_4$

$4Fe(OH)_2 + O_2 + 2H_2O = 4Fe(OH)_3$

$Fe_2(SO_4)_3 + 3Ca(HCO_3) = 2Fe(OH)_3 + 3CaSO_4 + 6CO_2$

Coagulant Equivalents

The effectiveness of inorganic coagulants is dependent upon water chemistry (pH and alkalinity), and their addition usually alters that chemistry. Table below illustrates the effect of the addition of 1ppm of the various inorganic coagulants.

Coagulant, Acid and Sulfate-1ppm Equivalents

	1 ppm Formula of Chemical	ppm Alkalinity Reduction	ppm SO_4 Increase	ppm Na_2SO_4 Increase	ppm CO_2 Increase	ppm Total Solids Increase
Filter Alum	$Al_2(SO_4)_3 \cdot 18H_2O$	0.45	0.45	0.64	0.40	0.16

	1 ppm Formula of Chemical	ppm Alkalinity Reduction	ppm SO_4 Increase	ppm Na_2SO_4 Increase	ppm CO_2 Increase	ppm Total Solids Increase
Ammonia alum	$Al_2(SO_4)_3 \cdot (NH_4)_2SO_4 \cdot 24H_2O$	0.33	0.44	0.63	0.29	0.27
Potash alum	$Al_2(SO_4)_3 \cdot K_2SO_4 \cdot 24H_2O$	0.32	0.43	0.60	0.28	0.30
Copperas (ferrous sulfate)	$FeSO_4 \cdot 7H_2O$	0.36	0.36	0.61	0.31	0.13
Chlorinated Copperas	$FeSO_4 \cdot 7H_2O + (\frac{1}{2}Cl_2)$	0.54	0.36	0.51	0.48	0.18
Ferric Sulfate (100% $Fe_2(SO_4)_3$)	$Fe_2(SO_4)_3$	0.76	0.76	1.07	0.64	0.27
Sulfuric acid-96%	H_2SO_4	1.00	1.00	1.42	0.88	0.36
Sulfuric acid-93.2% (66° Be)	H_2SO_4	0.96	0.95	1.36	0.84	0.34
Sulfuric acid-77.7% (60° Be)	H_2SO_4	0.79	0.79	1.13	0.70	0.28
Salt cake-95%	Na_2SO_4	-	0.64	0.95	-	1.00
Sodium aluminate-88%	$Na_2Al_2O_4$	Increase 0.54	-	-	Reduction 0.47	0.90

b) **Alkalinity Relationships**

Aluminum salts are most effective as coagulants in a 5.5-8.0 pH range. Because they react with the alkalinity in the water, it may be necessary to add additional alkalinity in the form of lime or soda ash.

Iron salts, on the other hand, are most effective as coagulants at higher 8 -10 pH ranges. Iron salts also depress alkalinity and pH levels; therefore, additional alkalinity must be added.

Sodium Aluminate increases the alkalinity of water, so care must be taken not to exceed pH and alkalinity guidelines. As is evident from the reactions discussed above, a working knowledge of the alkalinity relationships of water is mandatory.

It is important to note, at this point, that the use of metal salts for coagulation may increase the quantity of dissolved solids. One must consider the downstream impact of these dissolved solids. In addition, the impact of carryover of suspended Al^{+++} and Fe^{+++} compounds and their related effect on downstream processes must be considered.

For example, if the alum demand of water is 50ppm, the sulfate increase in the effluent water would be 50ppm x .45, or 22.5ppm. If an equivalent amount of lime had to be added, the dissolved solids content of the water would be further increased by

$$50ppm \times \frac{.45}{1.26} = 17.9ppm$$

It must also be pointed out that using inorganic coagulants produces a voluminous, low-solids sludge that dewaters and dries very slowly.

Alkali and Lime 1ppm Equivalents

Compound - 1 ppm	Formula (1 ppm)	Alkalinity Increase (1 ppm)	Free CO₂ Reduction (1 ppm)	Hardness as CaCO₃ Increases
Sodium bicarbonate	$NaHCO_3$	0.60	-	-
Soda ash (56% Na₂ -99.16% Na₂CO₃)	Na_2CO_3	0.94	.41	-
Caustic soda (76% Na₂O-98.06% Na₂CO₃)	$NaOH$	1.23	1.08	-
Chemical lime (Ouicklime-usually 90% CaO)	CaO	1.61	1.41	1.61
Hydrated lime (usually 93% Ca(OH)₂)	$Ca(OH)_2$	1.26	1.11	1.36

c) **Organic Coagulants - Polymers**

While these coagulants serve the same function as the inorganic metal salts, the process is simpler because the charge neutralization reaction is the only concern. Polymer addition has no effect on pH or alkalinity, so no supplemental chemical feed is required to control either.

Polymers can be envisioned as long chains with molecular weights of 1000 or less to 5,000,000 or more. Along the chain are numerous charged sites. In primary coagulants, these sites are positively charged. The sites are available for adsorption onto the negatively charged particles in the water. To accomplish optimum polymer dispersion and polymer/particle contact, initial mixing intensity is critical. The mixing must be rapid and thorough. Polymers used for charge neutralization *cannot* be over-diluted or over-mixed. The farther upstream in the system these polymers can be added, the better their performance.

Because most polymers are viscous, they must be properly diluted before they are added to the influent water. Special mixers such as static mixers, mixing tees and specially designed chemical dilution and feed systems are all aids in polymer dilution.

4) FLOCCULATION

Flocculation is the second step of the coagulation process. Once the negative charges of the suspended solids are neutralized, flocculation begins. Charge reduction increases the occurrence of particle-particle collisions, promoting particle agglomeration. Portions of the polymer molecules not absorbed protrude for some distance into the solution and are available to react with adjacent particles, promoting flocculation. Bridging of neutralized particles can also occur when two or more turbidity particles with a polymer chain attached come together. It is important to remember that during this step, when particles are colliding and forming larger aggregates, mixing energy should be great enough to cause particle collisions but not so great as to break up these aggregates as they are formed.

In some cases flocculation aids are employed to promote faster and better flocculation. These flocculation aids are normally high molecular weight anionic polymers. Flocculation aids are normally necessary for primary coagulants and water sources that form very small particles upon coagulation. A good example of this is water that is low in turbidity but high in color (colloidal suspension).

5) COAGULATION FOR REMOVAL OF TRIHALOMETHANE PRECURSORS

Recent US EPA regulations limit allowable TTHM concentrations in finished potable water. To help meet the current maximum contaminant level (MCL) of 0.10 mg/L for TTHM, trivalent metal ion coagulant, such as aluminum sulfate or ferrous sulfate, and a variety of organic poly-electrolytes have been used to remove THM precursors before chlorination. Naturally-occurring THM precursors, such as humic and fulvic compounds, are only partially removed by coagulation and filtration. For coagulation with alum, a pH of between 5 and 6 is the optimum for the removal of fulvic and humic acid compounds.

Ferrous sulfate exhibits an optimum pH for removing organic compounds of between 3 and 5. Fulvic acids require twice the dosages of alum needed for humic acids; the addition of anionic polymers at doses from 1 to 10 mg/L can also provide some removal of humic compounds. The efficiency of removal depends upon the type and concentration of organic compounds present in the water supply, pH, coagulant dose, and solids-liquid separation step. Optimum precursor removal can only be estimated using laboratory simulation techniques, such as simple jar testing, followed by settling or removal of precipitated colloids with membrane filters. This procedure can provide the information necessary to determine the optimum conditions for the removal of trihalomethane precursor compounds. Monitoring of the removal of organic precursor compounds by coagulation and filtration can be facilitated by the measurement of total organic carbon.

6) DESIGN CRITERIA FOR MIXING

Criteria for rapid- and slow-mix processes have been developed on the basis of detention time, power input, velocity gradient (G) and the product (Gt) of velocity gradient and detention time. The values of G and Gt are computed from:

$G = (P / uV)^{1/2}$

Where

- o   *G = velocity gradient (fps/foot; of sec^{-1})*

- o   *P = the power dissipated in the water (ft-lb/sec)*

- o   *u = water velocity (lb-sec/ft^2) (u = 2.73 x 10^{-5} @ 50°F)*

- o   *V = volume of mixing basin (cubic feet)*

- o   *t = mixer detention time (seconds)*

a) Rapid Mixing

For rapid-mix units, detention periods usually range from 10 to 30 seconds with installed mixer power approximately 0.25 to 1.0 hp per mgd. Power and detention time should be matched so that values of G will be in the approximate range: 500-1000. A wire-to-water efficiency of 80 percent, a water temperature of 50°F, a power input of 1.0 hp per mgd and a detention time of 10 seconds, yield a G value of about 1000 see-l and a Gt value of 10,000. Similarly, a 30-second detention time gives a G value of about 600 and a Gt value of 18,000. Long detention period for rapid-mix basins should be avoided because of higher power requirements and inferior coagulation results. The rapid-mix basin should be designed to minimize short circuiting.

b) Slow Mix

For slow-mix (flocculating) units, detention periods should range from 30 minutes to 60 minutes with installed mixer power of approximately 0.1 to 3.5 hp per mgd. G values in the range of 20 to 100 are commonly employed. Corresponding Gt values will, therefore, be in the range of 36,000 to 360,000. Tapered, slow mixing with G decreasing from a maximum of about 90 down to 50 and then to 30 can be used and will generally produce some improvement in flocculation. Somewhat higher G values, up to 200 are employed in some water softening plants. For normal flocculation, using alum or iron salts, the maximum peripheral speed of the mixing units should not exceed about 2.0 fps and provision should be made for speed variation. To control short circuiting, two to three compartments are usually provided. Compartmentation can be achieved by the use of baffles. Turbulence following flocculation must be avoided, Conduits carrying flocculated water to sedimentation basins should be designed to provide velocities of not less than 0.5 fps and not more than 1.5 fps. Weirs produce considerable turbulence and should not be used immediately following flocculation.
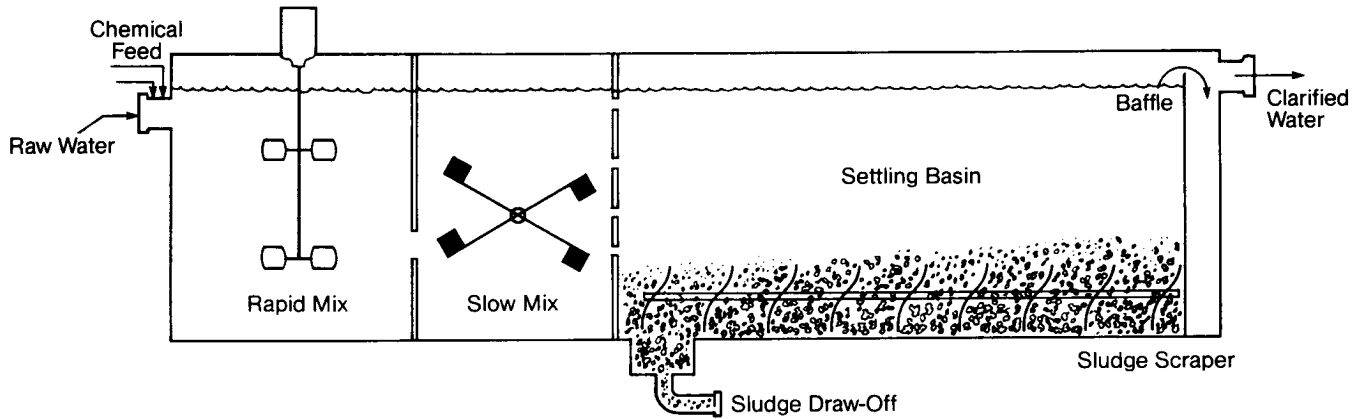
SEDIMENTATION

Sedimentation follows flocculation; the most common types of sedimentation basins use a large tank or horizontal basin for sedimentation of flocculated solids as shown in figures below. The basin may contain separate chambers for rapid mix, slow mix and settling. The first two steps are important for good clarification. An initial period of turbulent mixing is necessary for contact between the coagulant and the suspended matter, followed by a period of gentle stirring to increase collisions between particles and increase floc size. Typical retention times are 3-5 minutes for rapid mix, 15 to 30 minutes for flocculation and 4-6 hours for settling.

The coagulant is added to the waste water in the rapid mix chamber or just upstream, and the water passes through the mix chambers into the settling basin. As the water passes along the length of the basin (or out to the circumference in the case of a circular clarifier), the flocculated particles settle to the bottom and are scraped into a sludge collection basin for removal and disposal. Clear water flows over a weir and is usually held in a tank called a clearwell.
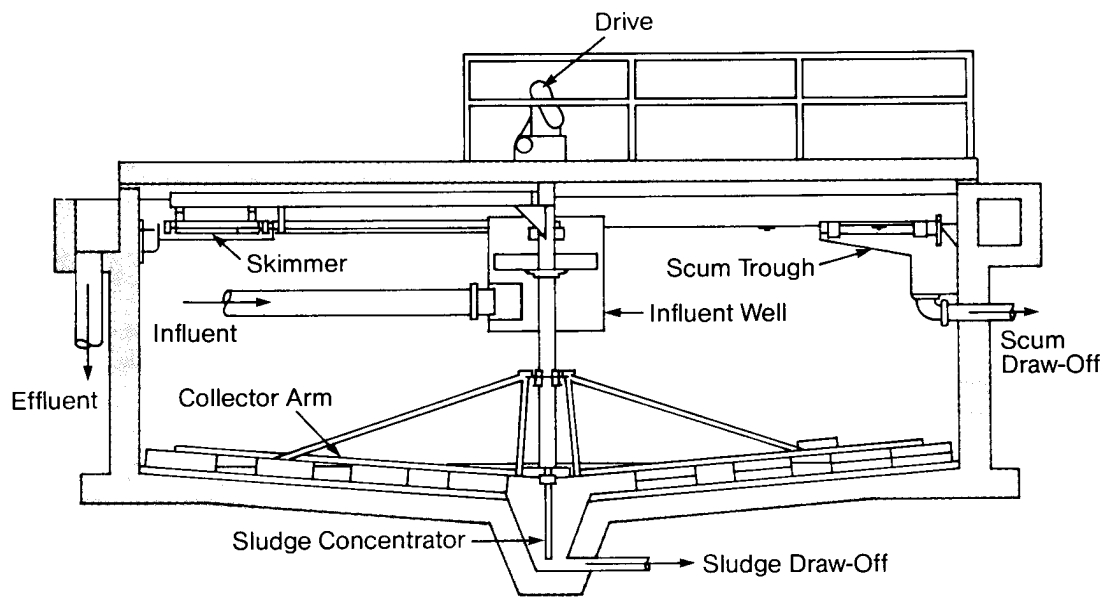
Large Tank Clarifier

Horizontal Basin Clarifier

1) *DESIGN CRITERIA*

The sedimentation basins should have adequate capacity to handle peak flow conditions and to prevent excessive deteriorated effluent water qualities. The design of a sedimentation tank is based on the criterion as listed in table below.

Design Criteria for Sedimentation Tanks

Design Element	Criteria
Tank Depth (ft)	10 – 16
Length/width Ratio (Rectangular)	3:1 to 5:1
Diameter (ft)	10 – 100
Flow-Through Velocity (fpm)	0.5 – 3
Entrance velocity of inlet ports (fps)	0.5 - 2
Detention Time (hour)	2 – 4
Surface Overflow Rate (conventional coagulation plant, gpm/ft^2)	0.35 – 1.5
Weir Overflow Rate (gpm/ft^2)	<15

The above design data represent common conditions; higher overflow rates may be used at lime softening plants and at some plants employing upflow clarification units as indicated in the tables of Water Treatment Plant Design by AWWA. Sedimentation is accomplished by decreasing water velocity and high velocities especially near the bottom of a sedimentation tank can contribute to carry over of settled material.

Detention time in the range of 8 to 12 hours or more provided in several stages maybe necessary for treating highly turbid waters; on the other hand, conical clarifiers are more efficient in softening and/or turbidity removal and require a detention time of one hour or less.

If the space available for sedimentation basins is limited, multiple-story basins, in which the water flows horizontally along one level and then passes upward and then flows horizontally along another level may be utilized. A newer commercial development in sedimentation, the "tube" settler, may be used if the capacity of existing sedimentation basins must be increased or if little space is available for the construction of new sedimentation basins. Detailed information relative to size, capacity and configuration of tube settlers may be obtained from manufacturers. Steel basin conical clarifiers may be used to expand sedimentation plants when space available is limited. The solids sludge which settles to the bottom of the basin must be removed before the depth of the sludge becomes great enough to interfere with effective sedimentation. In general, mechanical sludge removal equipment is requirement. All basins must be provided with drains and overflows. Basin covers, usually in the form of a super structure, will be required in northern localities where ice is a problem.

## 2) FLOCCULATION –SEDIMENTATION BASINS

Units of this type, usually circular, combine the functions of flocculation, sedimentation and sludge removal, Flocculation is accomplished in a circular center well, Sedimentation occurs in the annular space between the flocculation section and the perimeter effluent weir. Design criteria are generally similar to those applicable — to separate units.

3) UPFLOW CLARIFICATION

It is often advantageous to employ a zone of high solids contact to achieve a better quality effluent. This is accomplished in an up-flow clarifier, so called because the water flows upward through the clarifier as the solids settle to the bottom. Most up-flow clarifiers are either solids-contact or sludge-blanket type clarifiers, which differ somewhat in theory of operation.

4) SOLIDS CONTACT BASINS

Basins of this type combine rapid-mixing, flocculation, sedimentation, and sludge removal in a single unit. The raw water is drawn into the primary mixing zone, where initial coagulation and flocculation take place. The secondary mixing zone is used to produce a large number of particle collisions so that smaller particles are entrained in the larger floc. Water passes out of the inverted cone into the settling zone, where solids settle to the bottom and clarified water flows over the weir. Solids are drawn back into the primary mixing zone, causing recirculation of the large floc. The concentration of solids in the mixing zones is controlled by occasional or continuous blowdown of sludge. Cross-section of solid contact unit is shown below:
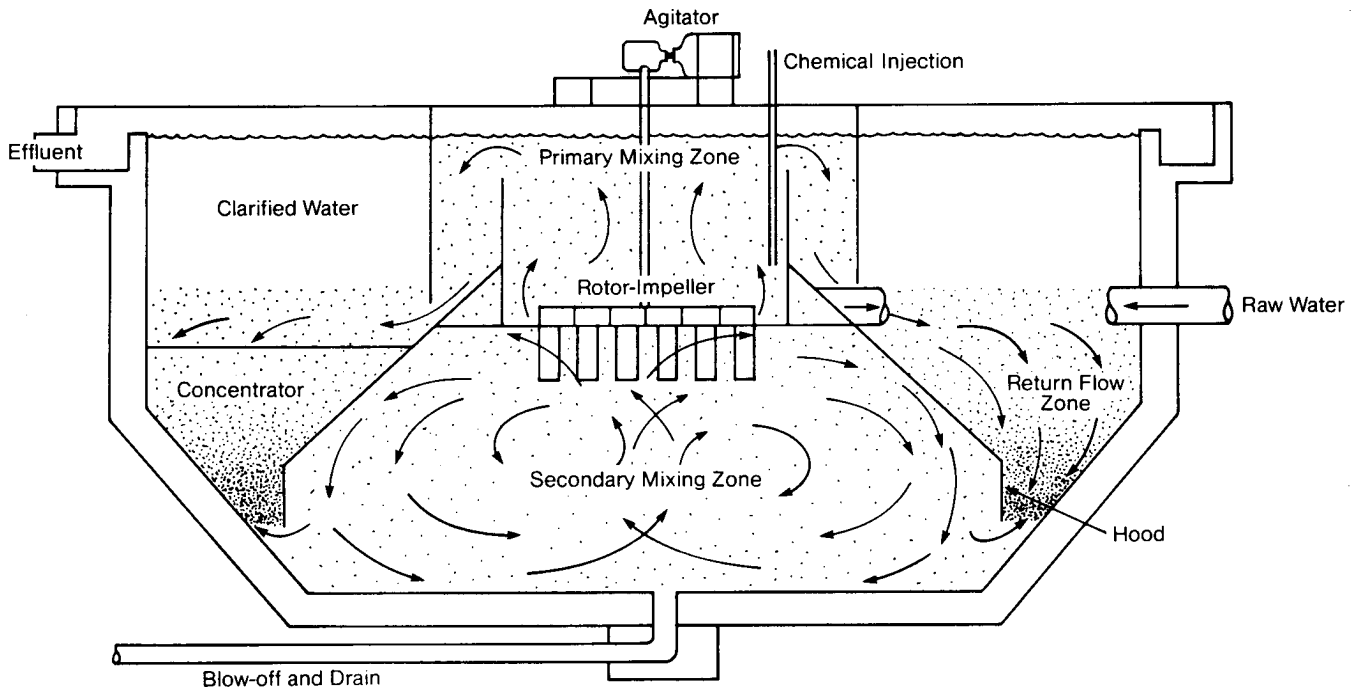
Solids-Contact Clarifier

4) THE SLUDGE-BLANKET CLARIFIER

The sludge-blanket clarifier goes one step further, by passing the water up from the bottom of the clarifier through a blanket of suspended solids that acts as a filter. The inverted cone within the clarifier produces an increasing cross-sectional area from the bottom of the clarifier to the top. Thus, the upward velocity of the water decreases as it approaches the top. At some point, the upward velocity of the water exactly balances the downward velocity of a solid particle and the particle is suspended, with heavier particles suspended closer to the bottom. As the water containing flocculated solids passes up through this blanket, the particles are absorbed onto the larger floc, which increases the floc size and drops it down to a lower level. It eventually falls to the bottom of the clarifier to be recirculated or drawn off. Cross-section of sludge-blanket clarifier is shown below:
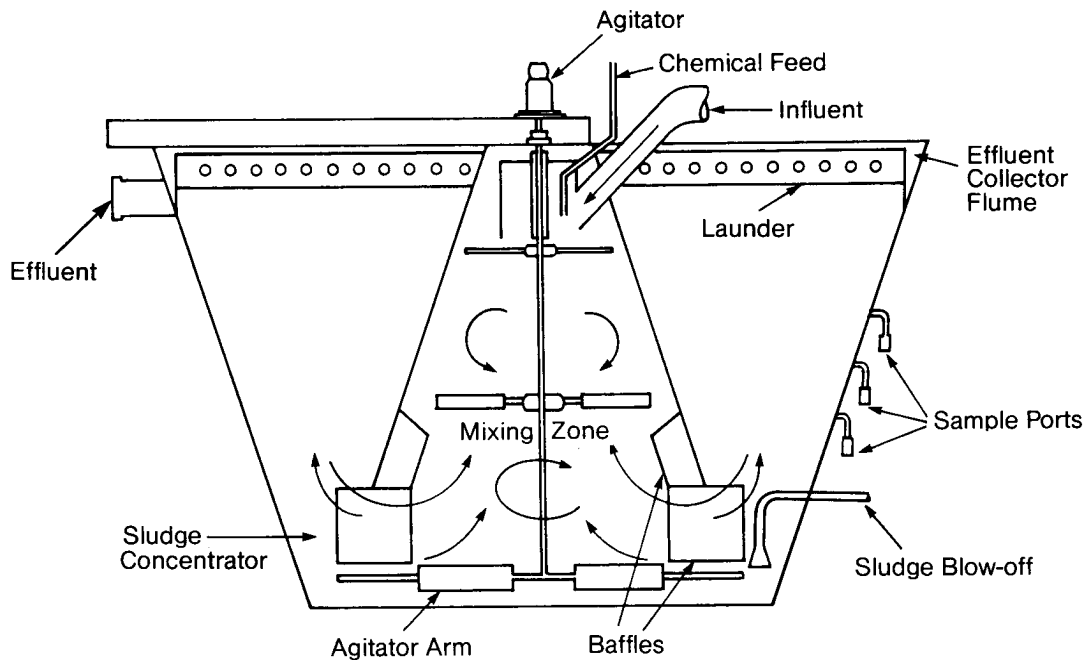
Sludge-Blanket Clarifier

Both solid contact and sludge-blanket clarifier units have an inverted cone within the clarifier. Inside the cone are a zone of rapid mixing and a zone of high solids concentration. The coagulant is added either in the rapid mix zone or somewhere upstream of the clarifier.

FILTRATION

Filtration of water is defined as the separation of colloidal and larger particles from water by passage through a porous medium, usually sand, granular coal or granular activated carbon. The suspended particles removed during filtration range in diameter from about 0.001 to 50 microns and larger. Several different types of medium arrangements and rates of flow through filters can be used. The filtration process most commonly used is gravity filtration, but pressure filtration or diatomite filtration is used at smaller installation. Recently high rate filters have been developed, which require less space and have higher solids loading capacity than conventional filters.

1) **RAPID SAND FILTERS**

   a) Filtration rate – Rapid sand filters are those filters which commonly operate at rates between approximately 2 and 8gpm per square foot. The usual rate is about 3 to 5gpm per square foot. The rate of filtration to be employed at a specific plant can be determined only after careful consideration, by the designer, or raw water quality and the probable efficiency or pretreatment that will be consistently provided. Good quality water is not ensured by low filtration rates.

Adequate pretreatment and filter design will allow application rates of up to 6gpm per sq-ft with little difference in water quality. It is emphasized that if high rate are to be used in design, great care must be taken to insure that all pre-filtration treatment processes including coagulation, flocculation and sedimentation will perform satisfactorily and consistently. High rate filter operation definitely requires excellence in pre-filtration treatment, especially in the case of surface waters. It is recommended that data from laboratory or pilot studies be utilized whenever possible rather than an arbitrary selection of criteria.

b) Filter Medium

➢ Sand – Silica sand is the most commonly used filter medium and its depth should be at least 24 inches and not more than 30 inches. When sand is employed, its "effective size" should fall in the range 0.35 to 0.5 millimeter (mm) with a uniformity coefficient of 1.3 to 1.7. Filter sand usually ranges in size from that passing a 16-mesh sieve to that retained on a 50-mesh (U.S. Series) sieve. Approximately 100 percent by weight, of the sand should pass the 16-mesh sieve and 90 to 100 percent be retained on a 50-mesh sieve. Filter sand should be clean silica sand having a specific gravity of not less than 2.5. The hydrochloric acid volubility of the sand should be less than 5 percent.

➢ *Anthracite* - Anthracite is an alternative medium consisting of hard anthracite coal particles. The effective size commonly ranges from about 0.45 mm to 0.6 mm with a uniformity coefficient not to exceed 1.7. The hardness should not be less than 2.7 on the Moh scale and the specific gravity not below 1.4. Also, the anthracite should be visibly free of clay, shale, and dirt.

➢ *Multimedia* - Multimedia filters employ two or three layers of media of different size and specific gravity. A common arrangement, the dual media filter, is 20 inches of anthracite overlaying a sand layer of approximately 8 to 12 inches. The anthracite layer has size range of about 0.8 to 2.0 mm; the sand layer, about 0.4 to 1.0 mm. Tri-media filters employ an 18-inch anthracite layer, an 8-inch sand layer, and an underlying 4-inch layer of garnet or ilmenite having a size range of 0.2 to 0.4 mm. Garnet has a specific gravity of about 4, and ilmenite about 4.5.

c) *Filter gravel and underdrains* -The filter media is commonly supported by a 10- to 18-inch layer of coarse sand and graded gravel. The gravel depth may range from 6 inches to 24 inches, depending on the filter underdrain system chosen. The gravel should consist of hard, rounded stones having a specific gravity of at least 2.5 and an acid volubility of less than 5 percent. A 3- to 4-inch transition layer of coarse (torpedo) sand, having a size range of about 1.2 to 2.4 mm, is placed on top of the filter gravel. Gravel size usually ranges from about 0.1 inch to about 2.5 inches. Filter underdrains may be constructed of perforated pipe grids or various proprietary underdrain systems. A variety of the latter are available. Design details for pipe underdrains are

given in numerous texts and handbooks. Manufacturers will furnish design and installation criteria for proprietary systems.

d) *Sand, anthracite, gravel specifications* - Detailed specifications for filter sand, anthracite and gravel are contained in AWWA B100.

e) *Number of filters* - Not less than two filters should be installed regardless of plant size. For large plants, rough guidance as to the number of filters to be provided may be obtained from:

$$N = 2.7\sqrt{Q}$$

Where

   o   N = number of filter units

   o   Q = design capacity in mgd

Thus, a 9mgd plant would require eight filters.

e) Size of filter units - The maximum filter size is related to wash water flow rate and distribution. Normally, individual filters sizes do not exceed about 2100 square feet corresponding to a capacity of about 6 mgd at a flow rate of 2.0 gpm per square foot. A unit of this size would require a maximum backwash water rate of about 60 mgd, which is excessive. Consequently, it should be divided into two parts of equal size arranged for separate backwashing. Total filter depth should be at least 9 feet.

f) *Filter backwash* - Backwash facilities should be capable of expanding the filter media 50 percent. This will require wash a normal backwash rates in the range of 10 to 20gpm per square foot of filter surface area. Backwash water can be supplied by a backwash pump or from elevated storage provided specifically for this purpose. Filter down-time during wash periods commonly average 10 to 20 minutes including a 5- to 15-minute wash period. For a 15-minute backwash of a single unit, at maximum rate, the wash water volume will be 300 gallons per square foot of filtration area in that unit. In addition to backwashing, auxiliary scour is commonly provided. This aids in cleaning the filter and is commonly accomplished by rotary or fixed surface-wash equipment located near the top of the bed. It is operated for a time period equal to that of the backwash, Water pressures of 40-100 psi are required for surface-wash operation at a rate of 0.5 gpm per square foot. Air scour may also be employed but is not generally used. If an independent washwater storage tank is used, it must refill between washes. Tank capacity should be at least 1.5 times the volume required for a single wash.

g) *Wash water troughs* - Wash water troughs equalize the flow of wash water and provide a conduit for removal of used water. Two or more troughs are usually provided. The elevation of the trough bottoms should be above that of the expended bed. The clear horizontal distance

between troughs should not exceed 5 to 6 feet, and the top of the troughs not more than 30 inches above the top of the bed.

h) *E*ach filter must have five operating valves: influent, wash water, drain, surface wash, and filter-to-waste. Design velocities commonly employed for major filter conduits are as follows:

Conduit	Design Velocity (ft/sec)
Influent	1 – 4
Effluent	3 – 6
Wash water	5 – 10
Drain	3 – 8
Filter to waste	6 - 12

The effluent conduit must be trapped to present backflow of air and provide a seal for the rate controllers. The filter pipe gallery should have ample room and good drainage, ventilation, and lighting.

It is emphasized that the filter-to-waste piping must not be directly connected to a plant drain or sewer. An effluent sampling tap must be provided for each filter. Valves can be manually, electrically, hydraulically, or pneumatically operated. Butterfly type valves are recommended for filter service.

Filters should be covered by a superstructure except under favorable climatic conditions. Drainage from the operating floor into the filter should be prevented by a curb. Access to the entire bed should be provided by a walkway at operating floor level around the filter. Filters may be manually or automatically controlled from local or remote locations. Facilities permitting local, manual control are recommended irrespective of other control features. Used backwash water should be discharged to a wash water recovery basin or to a waste disposal facility. Regulatory agencies generally view filter wash water as a pollutant and forbid its direct discharge to the natural drainage.

i) *Essential instrumentation* - Minimum essential instrumentation for each filter will be provided as follows: rate=of-flow indicator; loss-of-head indicator; effluent turbidity indicator; wash water rate-of-flow indicating and totalizing meter. If a wash water storage tank is provided, it must be equipped with a water level indicator. While not absolutely required, a turbidity indicator on the main filter influent is desirable.

2) *DIATOMITE FILTERS*

Filtration is accomplished by a layer of diatomaceous earth supported by a filter element termed a septum, This layer of diatomaceous earth is about I/8-inch thick at the beginning of filtration and must

be maintained during filtration by a constant feed of diatomaceous earth (body feed) to the influent water. At the conclusion of a filter run, the layer of diatomaceous earth will have increased in thickness to about 1/2 inch. Filtration rates generally vary from 0.5 to 2.0gpm per square foot. The principal use of diatomite filters has been for swimming pool waters, but some have been installed for the treatment of potable water.

3) PRESSURE FILTERS

Pressure filters are similar in construction and operating characteristics to rapid sand filters. However, in a pressure filter the media, gravel bed, and underdrains are enclosed in a steel shell (refer figure below).
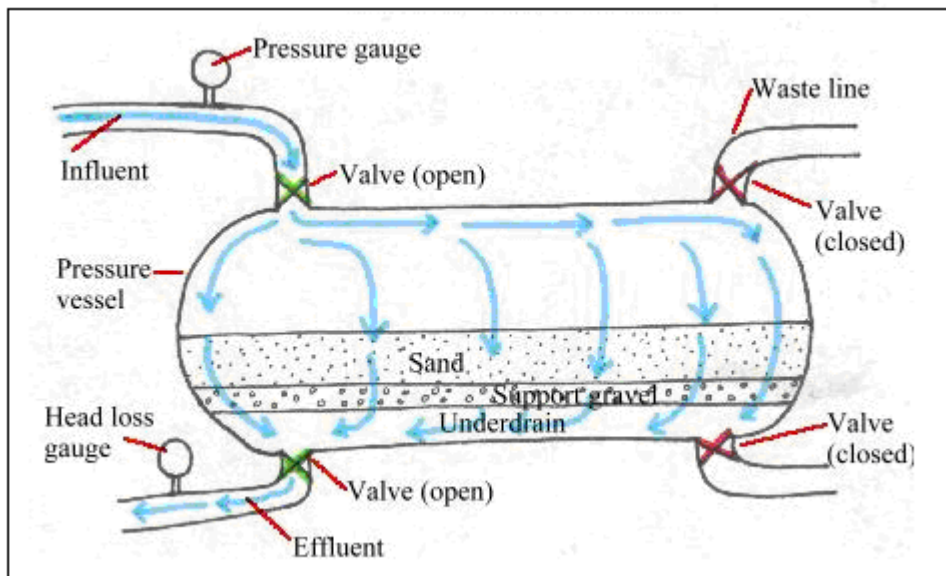
There are a variety of new pressure filters in use today. The most common of these are the conventional downflow filter, the high-rate downflow filter and the up flow filter. An advantage of any pressure filter is that any pressure in waterlines leading to the filter is not lost, as in the case of gravity filters, but can be used for distribution of the filter effluent. Between 3 and 10 feet of pressure head are lost through the filter. The primary disadvantage of a pressure filter is that, due to the filter being enclosed in a steel shell, access to the filter bed for normal observation and maintenance is restricted. Also, the steel shells require careful periodic maintenance to prevent both internal and external corrosion. The use of pressure filters is not advantageous in most systems. However, if the pressure requirements and conditions in a particular system are such that re-pumping of filtered water can be eliminated, cost savings will be realized.

a) *Conventional downflow filters* - Conventional downflow pressure filters consist of a bed of granular media or multi-media and are good in removing suspended solids comprised of floe. The

advantages over gravity filters include lower installation cost and adaptability to different piping systems. Hydraulic loadings range from 1 to 4gpm/sq- ft.

b) *High-rate downflow filters* - High-rate downflow filters have filtration rates of 10-20gpm/sq. ft. The higher downflow velocities require coarser media which allow suspended solids to penetrate deeper into the medium. As a result, more solids can be stored in the filter bed before backwashing is required. Many units exhibit 1-4 lbs/sq- ft. solids-loading capacity. The higher filtration rates also allow smaller or fewer filters to be used over conventional filters. However, the high solids-loading capacity of this filter requires higher backwashing flow rates and hence larger backwashing water storage tanks.

c) *Upflow filters* - Upflow multi-media filters allow filtration of high solids-loaded liquids in concentration up to 1,000 mg/L. The advantage of upflow multi-media filters is that the coarser material at the inlet collects the heavier particles, while the finer material collects the smaller particles, thus efficiency of the filter is increased.

d) *Upflow continuous backwash sand filters* - Upflow continuous backwash sand filters continuously clean the filter medial by recycling the sand internally through an air lift pipe and sand washer. The regenerated sand is then redistributed to the top of the sand – bed. Once the sand migrates down to the bottom of the bed it is again airlifted and repeats the cycle. Upflow continuous backwash sand filters require no backwash valves, storage tanks, or backwash pumps; therefore their operation is greatly simplified.

DISINFECTION

Disinfection involves destruction or inactivation of organisms which may be objectionable from the standpoint of both health and safety. In as much as the health of water consumers is of principal concern to those responsible for supplying water, design of facilities for disinfection must necessarily be carefully executed.

1) CHLORINATION

The application of chlorine to water is the preferred method of disinfecting water supplies at majority installations. Gaseous chlorine is 2.5 times heavier than air and is non-flammable.

a) Definitions

Terms frequently used in connection with chlorination practice are defined as follows:

❖ *Chlorine demand* - The difference between the concentrations of chlorine added to the water and the concentration of chlorine remaining at the end of a specified contact period. Chlorine

demand varies with the concentration of chlorine applied, time of contact, temperature, and water quality.

❖ *Chlorine residual -* The total concentration of chlorine remaining in the water at the end of a specified contact period.

❖ *Combined available residual chlorine -* Any chlorine in water which has combined with nitrogen — The most common source of nitrogen is ammonia, and compounds formed by the reactions between chlorine and ammonia are known as chloramines. The disinfecting power of combined available chlorine is about 25 to 100 times less than that of free available chlorine.

❖ *Free available residual chlorine -* That part of the chlorine residual which has not combined with nitrogen.

b) *Chlorination practice*

❖ *Combined residual chlorination -* Combined residual chlorination entails the application of sufficient quantities of chlorine and ammonia, if ammonia is not present in the raw water, to produce the desired amount of combined available chlorine (chloramine) in water. If enough ammonia is present in raw water to form combined chlorine residual, only chlorine need be added to the water. Combined residual chlorination is generally used only when maintaining an adequate free chlorine residual in the distribution system is difficult or when objectionably high levels of TTHMs would be formed as a result of free residual chlorination. Due consideration of other TTHM control alternatives should be made before using chloramines.

❖ *Breakpoint chlorination -* If water contains ammonia or certain nitrogenous organic matter which reacts with chlorine, the addition of chlorine causes the formation of chloramines until the ratio of elemental chlorine to ammonia compounds is about 5 to 1. Further addition of chlorine results in the oxidation of chloramines to gaseous nitrogen and nitrogen oxides, which decreases the quantity of chloramines present. After all of the chloramines have been oxidized, additional chlorine added to the water forms only free available chlorine. The point at which all of the chloramines have been oxidized and only free chlorine is formed is called the "breakpoint. " If no ammonia is present in the water, there will be no breakpoint. The chlorine required to reach the breakpoint is usually about 10 times the ammonia nitrogen content of the water. However, in certain waters, because of the presence of other chlorine consuming substances, as much as 25 times the ammonia nitrogen concentration may be required. If you receive complaints on chlorine smell in the drinking water turn the chlorine up to reach break point chlorination. Enough chlorine should be added past the breakpoint to ensure adequate free chlorine residual.

❖ *Marginal chlorination -* Marginal chlorination involves the application of chlorine to produce a desired level of total chlorine residual regardless of the relative concentrations of free or

combined chlorine present. In marginal chlorination the initial chlorine demand has been satisfied but some oxidizable substances remain.

❖ *Chlorine dosages* - Waterborne bacteria are the major concern at fixed installations. Even at lower pH levels, free chlorine residuals should not fall below 0.2 to 0.5 mg/L and combined chlorine residuals should not fall below 2.0 mg/L. If marginal chlorination is practiced, the total chlorine residual must not be less than 2.0 mg/l. Whenever epidemiological evidence indicates an outbreak of a nonbacterial waterborne disease such as amebiasis, infectious hepatitis, or schistosomiasis, cysticidal free chlorine residuals shall be maintained in the water supply.

c) *Other effects of chlorination*

In addition to the disinfection achieved with chlorination, other beneficial effects should be noted. Since the oxidizing power of chlorine is high, in the presence of free chlorine, hydrogen sulfide is oxidized, nitrites are oxidized to nitrates, and soluble iron and manganese are oxidized to their insoluble oxides. Free chlorine also reacts with naturally occurring taste, odor and color producing organic substances to form chloro-organic compounds, e.g., trihalomethanes. The US EPA, after much discussion over costs/ benefits, has chosen a maximum contaminant level for TTHM's of 100 micrograms per liter (µg/L) for plants serving above 10000 persons and has indicated a future goal of 10-25 µg/L. The present maximum contaminant level of 100 µg/L was a compromise with the water treatment industry to avoid costly modifications to existing plants. To reach the US EPA's future maximum contaminant level for TTHM's, more significant changes in disinfection practices will be required.

d) *Application of chlorine*

Chlorine may be applied to water of two forms: As gaseous elemental chlorine or as hypochlorite salts. Gaseous elemental chlorine shall be used for water disinfection at all fixed installations. The cost of hypochlorite salts is prohibitive in all plants larger than 0.5 mgd. For remote sites at fixed installations, some well sources require 5gpm or less. These sources with small demands can use hypochlorite for disinfection.

❖ *Point of application* - Chlorine may be applied to water in a variety of locations in the water treatment plant, storage facilities, or distribution system. It is absolutely essential that the chlorine applied to the water be quickly and thoroughly mixed with the water undergoing treatment. If required, special chlorine mixing facilities should be provided. In conventional water treatment plants, chlorine may be applied prior to any other treatment process (pre-chlorination), following one or more of the unit treatment process (post-chlorination), and again in the more distant points of the distribution system (de-chlorination).

o Pre-chlorination – Pre-chlorination has often been used so the water would maintain chlorine residual for the entire treatment period, thus lengthening the contact time. The

coagulation, flocculation, and filtration processes were thought to be improved by pre-chlorination of the water, and nuisance algae growths in settling basins were reduced. In pre-chlorination, the chlorine was usually injected into the raw water at or near the raw water intake. Pre-chlorination was the most accepted practice of disinfection in the past. However, since many surface waters contain THM precursors that will combine with the free chlorine during pre-chlorination and form potentially carcinogenic THMs, such as chloroform, the point of application has been shifted further down the treatment process to take advantage of precursor removal during treatment.

o   Post-chlorination – Post-chlorination generally involves the application of chlorine immediately after filtration and ahead of the clear well. The design and construction of water treatment plants will include the necessary provisions for changing the locations of chlorine applications as may later be desirable for improving treatment or disinfection processes.

o   De-chlorination -. Dechlorination is the practice of adding chlorine to water in the distribution system to maintain minimum chlorine residual throughout the system.

e) *Chlorination equipment*

Hypochlorite salts must be applied to the water in solution form. Hypochlorite solutions are pumped by a diaphragm pump through an injection system into the water to be chlorinated. If elemental chlorine is used for disinfection, it shall be injected by solution-type chlorinators. Since chlorine solutions are acidic, many components of a chlorination system must be constructed of corrosion resistant materials such as glass, silver, rubber, or plastics. Maintaining the chlorination apparatus in a trouble-free state is essential, Key spare parts and repair kits for chlorination systems must be kept on hand. Critical components of the chlorination system shall be installed in duplicate.

f) *Automatic control*

If automatic chlorination control is utilized, the chlorine feed rate should be controlled primarily by the rate of flow of water, with a signal from a downstream residual chlorine analyzer used to trim the feed rate. Provision for manual control during emergency situations must be included.

g) *Super chlorination and de-chlorination*

Super chlorination may be necessary if there are large variations in chlorine demand or if available contact time is brief. Water which has been super chlorinated generally requires de-chlorination before discharge to the distribution system. Dechlorination may be achieved through the application of sulfur dioxide, sodium bisulfite, or sodium sulfite, or by passing the water through granular activated carbon filters. The de-chlorination process (and subsequent de-chlorination, if necessary) shall be controlled so that the free residual chlorine remaining in the water is at least

0.2 mg/L. Careful monitoring must be practiced to assure that potentially harmful levels of TTHMs are not exceeded. A summary of TTHM regulations are presented below:

Summary of TTHM Regulations

Maximum Contaminant Level (MCL)	0.10 mg/l (100 micrograms per liter) Total Trihalomethanes
Applicability	Community water systems that add disinfectant to the treatment process (ground and surface)
Effective	Systems > 75000: Nov 29, 1981 System 10-75000: Nov 29, 1983 System < 10000: State discretion
Monitoring Requirements	Running annual average of a minimum samples per quarter per plant taken on same day. Systems using multiple drawing raw water from a single aquifer may, with State approval, be considered one treatment plant for determining the required number of samples.
Sample Locations	25% at extreme of distribution system; 75% at locations representatives of population distribution.

e) Safety precautions for chlorination

The AWWA manual "Safety Practice for Water Utilities" contains safety recommendations regarding the use of chlorine. These recommendations shall be followed at all water treatment facilities.

2) ALTERNATE DISINFECTANTS

 If the use of chlorine as a disinfectant causes unacceptably large concentrations of chlorinated organic compounds, and if all other methods for reducing TTHM's have been exhausted, such as moving the point of chlorination, aeration, and special coagulant (as shown in table below for chloroform which is the main constituent of TTHMs in many cases) and if an alternate raw water source, such as a ground water source, is not available, an alternative disinfectant must be considered. Any alternate disinfectant system installed as the primary means of water disinfection shall have chlorination facilities available and operative for stand-by use. Five alternative disinfectants are discussed below; ozone, chlorine dioxide, chloramines, ultraviolet (UV) radiation, and UV and Ozone combined. While chlorine is the least costly disinfectant, considering dosage and energy consumption basis, alternate disinfectants are not significantly more expensive.

Effective of Various Unit Processes for Reducing Chloroform Formation Potential

Process	Chloroform Formation Potential µg/l	Chloroform Formed, µg/l	Remarks
Aeration followed by chlorination	66 (a)	66 (a)	Diffused air aeration with air to water ratios up to 20:1 did not reduce chloroform formation potential (10 min contact time)
Coagulation, Sedimentation and Dual media filtration followed by chlorination	48 (b)	13 (b)	
Coagulation, Sedimentation, Filtration/Adsorption by Granular Activated Carbon (5min contact time) followed by chlorination	48	$\leq$1 $\leq$10	GAC would be effective for 3 weeks GAC would be effective for 8 weeks
Powdered Activated Carbon added after Coagulation and Settling followed by Chlorination	27 (a)	20 (a) 9	At PAC dosage = 8 mg/l At PAC dosage = 8 mg/l PAC contact time = 2-20min
Ozone Only	48	None Found	O_3 neither forms trihalomethanes, nor removes precursors at disinfection doses
Ozone followed by chlorination	48	48	Disinfection doses $\leq$ 1mg/l
Chlorine Dioxide Only	74	<1	ClO_2 does not form trihalomethanes
Coagulation, Sedimentation and Filtration followed by Chlorination Chlorine dioxide	 17 (c) 3 (d)	 1.3 mg/l ClO_2 and 1.5 mg/l Cl_2 Trihalomethane with chlorine formation decrease as the ratio of ClO_2 at Cl_2 increases	

All tests performed on Ohio River water. Chloroform Formation Potential is the amount of chloroform formed when raw water is chlorinated past break-point and stored at 25C for a specified contact time.

a – Chlorine contact time = 48 hours

b – Chlorine contact time = 96 hours

c – Chlorine contact time = 22 hours

d – Chlorine contact time for combination of chlorine dioxide with chlorine = 22 hours

a) Ozone

Ozone is an extremely powerful disinfectant that has been used in Europe either as a sole disinfectant, or in conjunction with post-chlorination to impart a persistent chlorine residual in the water distribution system. United States potable water plants have in the past used ozone to control taste and odor. Today ozonation is being increasingly used as a primary disinfectant prior to rapid mixing, flocculation and filtration. Ozonation does not produce THMs; it is reduced to oxygen and does not leave any residual disinfectant; hence the need for post-chlorination. Ozone is generated electrically, as needed using the electric discharge gap (corona) technique. Air or oxygen stream, a cooling water stream and alternating electric current are required. Efficient cooling is essential to reduce thermal decomposition of ozone. Bubble diffusers appear to be the most economic ozone contractors available.

b) Chlorine Dioxide

Chlorine dioxide is a highly effective disinfectant producing minimal THMs in the presence of their precursors. Chlorine dioxide uses in the United States have been limited to taste and odor control although it has been used elsewhere as a primary disinfectant and is presently receiving more attention in the United States. The common method of chlorine dioxide production is to react chlorine gas from a conventional chlorinator with a sodium chlorite solution. Following the mixing of the chlorine and sodium chlorite streams and prior to introduction into the main stream the mixed stream is passed through a packed column contactor to maximize chlorine dioxide production. A major disadvantage of chlorine dioxide is the formation of chlorate and chlorite which are potentially toxic.

c) Chloramines

The use of chloramines as a disinfectant fell into disuse after the introduction of breakpoint chlorination. To achieve the same disinfection ability of chlorine, 10 to 15 times the amount of chloramines are needed or longer contact time is required. More chloramines are needed if high concentrations of organic material are found in the influent water, Chloramines are easy to generate, feed, and produce a persistent residual that will remain through the water distribution system. Chloramines may be produced by introducing ammonia to the water stream prior to the

addition of free chlorine. This process can be optimized for minimum THM production and maximum disinfection. Recently however there has been some concern over chloramine toxicity.

d) *Ultraviolet Radiation*

Ultraviolet (UV) radiation has undergone development, but has not been used on a large scale for drinking water supply disinfection. Most of its uses include product or process water disinfection where high purity, sterile water is needed. UV radiation has been used to disinfect drinking water at remotely located hotels and on cruise ships. Few large scale water processing plants use UV disinfection, although its application is feasible. UV disinfection does not leave a disinfectant residual and should be accompanied by post-chlorination. Ultraviolet irradiation is also effective in oxidizing organic compounds in water; water turbidity will inhibit the effectiveness of UV disinfection.

e) *UV and Ozone*

Recently there has been some experimentation in a combined UV and ozone contactor. Results from these tests show promise. However, there is no known water treatment plant operating with this method of disinfection.

FLUORIDE ADJUSTMENT

1) HEALTH EFFECTS

An excessive fluoride concentration will damage the teeth of children using the water for extended periods. On the other hand, moderate concentrations, 0.7- 1.2 mg/L, are beneficial to children's teeth. Most natural waters contain less than the optimum concentration of fluoride. Upward adjustment of the fluoride concentration can be achieved by application of a measured amount of a fluoride chemical to the water. For installations where it is desirable and feasible to add fluoride, control limits and optimum concentrations are as follows:

Annual Average of Maximum Daily Air Temperatures °F	Control Limits, Fluoride Concentration mg/l		
	Lower	Optimum	Upper
50.0 – 53.7	0.9	1.2	1.7
53.8 – 58.3	0.8	1.1	1.5
58.4 – 63.8	0.8	1.0	1.3
63.9 – 70.6	0.7	0.9	1.2
70.7 – 79.2	0.7	0.8	1.0

Annual Average of Maximum Daily Air Temperatures °F	Control Limits, Fluoride Concentration mg/l		
	Lower	Optimum	Upper
79.3 – 90.5	0.6	0.7	0.8

2) FLUORIDATION CHEMICALS

Chemicals most frequently used for fluoridation are – Sodium silicofluoride (Na_2SiF_6), sodium fluoride (NaF) and fluosilicic acid (H_2SiF_6). For a particular installation, the choice of chemical will depend principally on delivered cost and availability.

    a) Sodium fluoride - This chemical is commercially available as a white crystalline powder having a purity of 95 to 98 percent. (Sometimes it is artificially colored nile blue.) Volubility is approximately 4 percent at 77°F. The pH of a saturated solution is 6.6. The 100% pure material contains 45.25% fluoride. It is available in 100-pound bags, 125 to 400 pound drums, and bulk.

    b) Sodium silicofluoride - This compound is commercially available as a white powder with a purity of 98 to 99 percent. Volubility is only about 0.76 percent at 770 F. The pH of a saturated solution is 3.5. The 100 percent material contains 60.7 percent fluoride. It is available in 100 pound bags, 125 to 400 pound drums, and bulk.

    c) Fluosilicic acid - This chemical is commercially available as a liquid containing 22 to 30 percent by weight of fluosilicic acid. It is sold in 13 gallon carboys, 55 gallon drums, and in bulk. The 100 percent pure acid contains 79.2 percent fluoride. The pH of a 1 percent solution is 1.2, and the use of fluosilicic acid as a fluoridation agent in a water of low alkalinity will significantly reduce the pH of the water. It should not be used for fluoride adjustment of waters of this type unless pH adjustment is also provided.

3) POINT OF APPLICATION

It is essential that all water pass the point of injection of the fluoridation chemical and that the flow rate past this point be known with reasonable accuracy. At a water treatment plant, the preferred application point is usually the combined effluent of all filters. The fluoride chemical can be fed at an earlier stage of treatment, for example, the combined filter influent, but part of the fluoride applied will be removed by the filtration process. Coagulation and lime softening will also remove a small amount of the applied fluoride. A larger dose is required to offset treatment process losses. If ground water is the supply source, the fluoride chemical should be injected into the discharge pipe of the well pump. Where the supply is from several wells, each pumping independently to the distribution system, it will be necessary to provide an injection point at each well. If flow past the injection point is variable, automatic equipment that will feed fluoride chemical at a rate proportional to flow is a requirement.

4) FLUORIDE FEEDERS

Volumetric or gravimetric dry feeders equipped with dissolvers are suitable for sodium fluoride or sodium silicofluoride. Feeders should be equipped with weighing devices that will accurately measure the weight of chemical fed each day and the feed equipment should be designed to minimize the possibility of free flow (flooding) of chemical through the feeder. Normally, the feed machine's supply hopper should hold no more than 100 to 200 pounds of chemical. Large extension hoppers holding much greater quantities of dry fluoride chemical increase the danger of flooding and overfeeding and are not recommended for most installations. Solutions of sodium silicofluoride are acidic and corrosion-resistant dissolvers and solution piping must be provided where this chemical is employed. If fluosilicic acid is used, it can be applied by means of a small metering pump into an open channel or a pressure pipe. Storage tanks, feeders, and piping for fluosilicic acid must be made of corrosion-resistant material. The acid is slightly volatile and the feed system should be enclosed. If not enclosed, special exhaust ventilation should be provided to protect personnel from fluoride fumes.

5) FLUORIDE REMOVAL

Fluoride removal can be accomplished by passage of the water through beds of activated alumina, bone char, or tricalcium phosphate. When the capacity of the bed to remove fluoride is exhausted, it can be regenerated by treatment with a caustic soda solution followed by rinsing and acid neutralization of the residual caustic soda. Other methods of fluoride removal include electrodialysis, reverse osmosis and ion exchange. Some fluoride reduction can be obtained by water softening using excess lime treatment. Fluoride reduction by this method is associated with magnesium precipitation and the extent of fluoride removal is a function of the amount of magnesium precipitated from the water. All removal processes produce liquid wastes and suitable provision must be made for their disposal. Guidance as to the fluoride removal process to be employed can be obtained from laboratory studies of process effectiveness and fluoride removal capacity, using samples of the water that is to be treated.

TASTE AND ODOR CONTROL

Most taste and odors in surface water are caused by low concentrations of organic substances derived from decomposing vegetation, microscopic organisms, sewage and industrial waste pollution, etc. Treatment for taste and odor removal involves destruction of the odorous substance by chemical oxidation or its removal by aeration or adsorption or activated carbon.

1) CHEMICAL OXIDATION

Chemical oxidizing agents which have been found effective and which can be used in the treatment of potable water are chlorine, chlorine dioxide, potassium permanganate, and ozone. No single chemical is completely effective under all operating conditions.

2) AERATION

Aeration is helpful in eliminating odor caused by hydrogen sulfide, but is ineffective in significantly reducing odor associated with dissolved organics.

3) ADSORPTION

Powdered activated carbon is commonly used for removal of tastes, odor and color by adsorption. The carbon can be applied to the water at any point in the treatment plant prior to filtration, but it is usually advisable to apply it early in the treatment process to prolong contact. For maximum effectiveness, carbon should be applied well ahead of chlorine, and preferably in advance of lime softening. The influent to a pre-sedimentation basin is normally an effective carbon application point. Powdered carbon dosages usually range from 5 to 10 mg/L, but as much as 50 mg/L may be required. The use of powdered activated carbon adds more suspended solids and increases the amount of sludge, thereby creating a sludge disposal problem. Powder activated carbon is marginally effective in reducing TTHMs. Granular activated carbon (GAG) has also been used for taste and odor removal. It has been employed as a separate treatment step in the form of carbon columns and as a substitute for sand in the filtration process. Used in this way, the granular carbon serves in a dual capacity as a filtration medium and for taste and odor removal. Granular activated carbon is also excellent at reducing TTHMs. Granular activated carbon must be reactivated on a regular basis to keep its absorptive abilities. Because of the cost of reactivation of GAC, other methods of taste-and-odor control and reduction of TTHMs should be considered. Aeration is generally more cost effective than GAC contractors.

SOFTENING

Whether water softening is provided will depend entirely on the type of project and the uses to be made of the water. Two general types of processes are used for softening: The "lime-soda ash" process and the "cation ion exchange" or "zeolite" process.
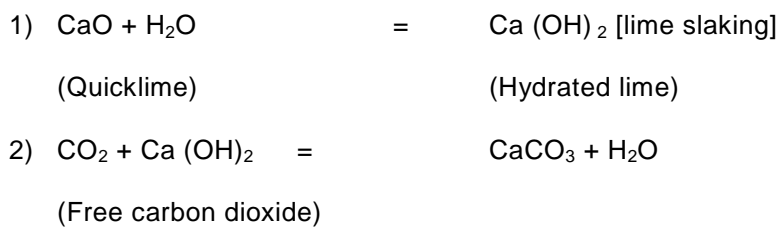
1) APPLICATIONS

a) *Permanent posts or bases* - Softening of the entire supply for a permanent post or base may be considered if the hardness exceeds 200 mg/l, with hardness expressed as equivalent $CaCO_3$. Softening of a post water supply to a total hardness of less than 100 mg/L is not required, however, softening to less than this amount is justified for the special purposes and services given in paragraphs (c), (d), (e), and(f) below.
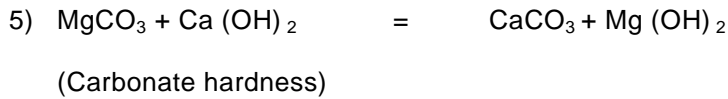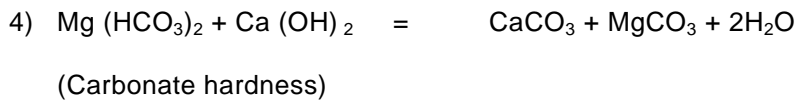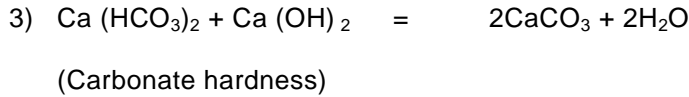
b) *Nonpermanent bases* - For Army temporary construction and for Air Force bases not in the permanent category, the entire supply will not be softened unless the total hardness exceeds 300 mg/L. However, when a treatment plant is constructed for the removal of turbidity or iron, the plant may also be designed to accomplish partial softening.

c) *Laundries* - Water for laundries shall have a hardness of 50 mg/l or less. Installation of cation ion exchange water softeners to reduce the hardness to zero is recommended.

d) *Boiler water* - Boiler water for power plants and heating plants may require softening, but satisfactory results can often be obtained by application of corrosion and scale inhibitors. Depending on the pressure at which the boiler is to operate, partial water-demineralization may also be necessary.

e) *Dining facilities* - The installation of softeners for small dining facilities, latrines and bathhouses is not recommended. However, water softeners to reduce hardness to 50 mg/L maybe justified for large central dining facilities to protect equipment and to insure satisfactory washing of dishes. Each such instance will be justified separately.

f) *Hospitals* - When the water supplied to a hospital has a hardness of 170 mg/L or more, the water will be softened to approximately 50 mg/L. Where critical equipment requires water having a hardness of less than 50 mg/L, as special study will be made to determine the most feasible means of obtaining water of the necessary hardness. Zero hardness water may be piped from the main softener or maybe supplied from small individual softeners, whichever is the more feasible. The sodium content of the treated water must be taken into account when selecting a softening method for hospitals.

2) *LIME-SODA ASH PROCESS*

a) *Softening chemicals and reactions* - The principal chemicals used to effect softening are lime, either hydrated lime ($Ca(OH)_2$) or quicklime (CaO), and soda ash (Na_2CO_3). These chemicals are added to the water to be softened and react with the calcium carbonate and magnesium to form insoluble compounds of calcium carbonate and magnesium hydroxide. If quicklime is used, it is usually converted to slurry of hydrated lime by slaking with water prior to application. The chemistry of the process can be illustrated by the following equations:

Lime Treatment

1) $CaO + H_2O$ = $Ca(OH)_2$ [lime slaking]

   (Quicklime)   (Hydrated lime)

2) $CO_2 + Ca(OH)_2$ = $CaCO_3 + H_2O$

   (Free carbon dioxide)

3) $Ca(HCO_3)_2 + Ca(OH)_2$ = $2CaCO_3 + 2H_2O$

(Carbonate hardness)

4) $Mg(HCO_3)_2 + Ca(OH)_2$ = $CaCO_3 + MgCO_3 + 2H_2O$

(Carbonate hardness)

5) $MgCO_3 + Ca(OH)_2$ = $CaCO_3 + Mg(OH)_2$

(Carbonate hardness)

Soda Ash Treatment

6) $CaSO_4 + Na_2CO_3$ = $CaCO_3 + Na_2SO_4$

(Non carbonate hardness)

7) $MgSO_4 + Ca(OH)_2$ = $Mg(OH)_2 + CaSO_4$
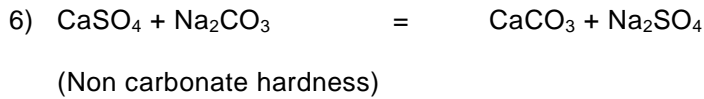
All of the above reactions can be accomplished in a single stage of treatment. Lime and soda ash can be added at the same point and will react with each other; however, the net effect will be as illustrated by reactions 2 through 7.

b)   *Chemical requirements*

   ❖   *Lime* - A reasonably accurate estimate of lime requirements for softening can be computed from a water analysis which gives free CO_2, bicarbonate alkalinity and, magnesium. Requirements of quicklime or hydrated lime can be computed as follows:

   Lbs quicklime per million gallons =

$$\frac{[10.6]\,[CO_2] + [4.68]\,[Alk + MgH] + [35]\,[8.34]}{\dfrac{\%\ CaO\ in\ quicklime}{100}}$$

   (Quicklime purity usually ranges from 90 to 95% CaO)

   Lbs hydrated lime per million gallons =

$$\frac{[14.0]\,[CO_2] + [6.18]\,[Alk + MgH] + [46]\,[8.34]}{\dfrac{\%\,Ca(OH)_2\ in\ hydrated\ lime}{100}}$$

   (Hydrated lime purity usually range from 95 to 98% ($Ca(OH)_2$)

   Where

   o CO_2 = mg/L carbon dioxide expressed as CO_2

- o Alk = mg/L bicarbonate alkalinity expressed as CaCO$_3$

- o MgH = Magnesium hardness expressed as CaCO$_3$, mg/L

- o 35 = excess of pure quicklime required to insure effective precipitation of magnesium mg/L

- o 46 = excess of pure hydrated lime required to insure effective precipitation of magnesium mg/L

- ❖ Soda Ash – To estimate Soda ash requirements, it is only necessary to know the non-carbonate hardness of the raw water and to establish the amount of noncarbonated hardness to be left in the finished water. The latter is termed residual noncarbonated hardness. Inasmuch as most commercial soda ash is 990/0+ Na2CO3, it is normally not necessary to correct for the purity of this chemical.

  Lbs soda ash per million gallons = [8.34] [NCH-R]

  Where

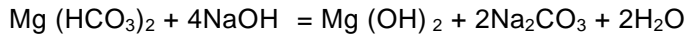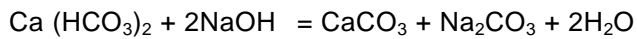 - o NCH = mg/L of noncarbonated hardness

  - o R = mg/L of residual noncarbonated hardness

  – (The term [NCH-R] is the mg/L of noncarbonated hardness removed)

c) *Characteristics of lime-softened water* - The carbonate hardness of the water, after application and reaction of the softening chemicals plus sedimentation and filtration, should be approximately 50 mg/L. The total hardness will consist of the carbonate hardness, 50 mg/L, plus the residual noncarbonated hardness that was intentionally allowed to remain in the water. It is not advisable to reduce the carbonate hardness to the lowest possible value because such water will be corrosive. In lime softened wasters, it is desirable that the magnesium hardness be reduced to 40 mg/L or less. The residual calcium hardness should be approximately 50 mg/L and the alkalinity also about 50 mg/L. Some ground water supplies contain no noncarbonated hardness. For such waters, lime treatment alone will suffice for softening.

d) *Sludge production* - The lime-soda ash softening process produces chemical sludge composed principally of calcium carbonate and magnesium hydroxide. As withdrawn from sedimentation basins equipped for mechanical sludge removal, the proportion of dry solids in the sludge will generally fall within the range of 2 to 10 percent. The weight of dry solids produced by softening reactions will average approximately 2.5 times the weight of commercial quicklime used. For hydrated lime, softening solids produced will be roughly twice the weight of commercial hydrated lime employed. Fairly accurate values of total solids production at an operating plant can be developed utilizing a mass balance which takes into consideration the suspended solids in the raw water, the quantity of dissolved calcium and magnesium in the raw

and finished water, the quantity and purity of lime applied, the quantity of coagulant used, and the stoichiometry of the softening and coagulation reactions. Means of disposal of waste solids from softening plants must receive careful consideration at an early stage of treatment plant design.
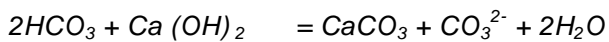
e)   *Lime-caustic soda process -* An alternative softening process, sometimes used, is the lime-caustic soda process. The process is worth consideration when considerable reduction in noncarbonated hardness is required. Application of the process involves substitution of caustic soda (sodium hydroxide) for soda ash and part of the lime. The remaining lime reacts with carbonate hardness constituents as previously indicated. The caustic soda also reacts with carbonate hardness as follows:

$Ca (HCO_3)_2 + 2NaOH = CaCO_3 + Na_2CO_3 + 2H_2O$

$Mg (HCO_3)_2 + 4NaOH = Mg (OH)_2 + 2Na_2CO_3 + 2H_2O$

The above reactions generate soda ash (Na_2CO_3) which will reduce the noncarbonated hardness as previously indicated. All of the reaction products are chemically identical to those obtained by the use of lime and soda ash. The amount of caustic soda required can be calculated from the theoretical quantities of pure lime and soda ash required. Less calcium carbonate sludge is formed with the lime-caustic soda process. This may be an advantage if softening sludge disposal is a problem. For water softening purposes, caustic soda should be purchased as a 50 percent solution containing 6.38 pounds of pure NaOH per gallon. A 50 percent solution must be stored at temperatures above about 600 F. to prevent freezing. As a storage alternative, the 50 percent solution may be diluted to 25 to 30 percent strength which has a freezing point in the approximate range of 0C to 30F. Caustic soda in any form must be viewed as a hazardous substance, capable of causing serious burns. Personnel responsible for handling and feeding the chemical must understand its potentially dangerous nature, know what precautions should be taken and be supplied with appropriate protective clothing, safety showers, etc.

f)   *Re-carbonation* – Re-carbonation involves the introduction of carbon dioxide and/or bicarbonate ion into softened water for the purpose of neutralizing excess hydroxide alkalinity and relieving calcium carbonate and magnesium hydroxide super saturation. Carbon dioxide should either be purchased as liquefied carbon dioxide, which must be stored at the plant in a refrigerated storage tank, or generated at the water treatment plant by the combustion of coke, oil, or gas. Re-carbonation can also be achieved by utilizing carbon dioxide and bicarbonate available in the raw water. This is the "split" treatment process.

❖   *Chemical reactions:* The following reactions illustrate the chemistry of the re-carbonation process:

Neutralization of excess lime

$CO_2 + Ca\,(OH)_2 \qquad = CaCO_3 + H_2O$

Conversion of magnesium hydroxide to magnesium carbonate

$CO_2 + Mg\,(OH)_2 \qquad = MgCO_3 + H_2O$

Conversion of carbonates to bicarbonates

$2CO_2 + Ca\,CO_3 \qquad = H_2O + Ca\,(HCO_3)_2$

$2CO_2 + Mg\,CO_3 \qquad = H_2O + Mg\,(HCO_3)_2$

Use of bicarbonate derived from raw water

$2HCO_3 + Ca\,(OH)_2 \qquad = CaCO_3 + CO_3^{2-} + 2H_2O$

The above reactions are accompanied by important changes in the pH of the softened water, and the pH value is used as a re-carbonation control parameter. Re-carbonation can be practiced in a single-stage or two stage configuration. If re-carbonation is accomplished in two stages, the first stage is devoted to neutralization of most of the excess lime. This involves conversion of excess lime to calcium carbonate and a pH change from about 11 to approximately the 9.5-10 range. Following the first stage of re-carbonation, the water must be flocculated and settled to remove excess calcium carbonate. Coagulant such as silica, starch, polymer or ferric sulfate may be employed to assist in coagulation and settling of the calcium carbonate particles. The second stage of re-carbonation, usually just ahead of filtration, serves principally as a "trim" stage in which final pH adjustments are made, as necessary. Guidance as to the correct pH can be obtained through calculation of the saturation index . For softened waters of low alkalinity, a plus index is generally advisable. Carbon dioxide added in the second stage converts carbonates to bicarbonates. If only a single stage of re-carbonation is employed, the carbon --- dioxide feed must be adjusted so that the previously described reactions take place to the extent necessary at the single point of re-carbonation. Single stage re-carbonation should be followed by facilities for removal of precipitated calcium carbonate. A single stage of re-carbonation provides no opportunity for further pH adjustment prior to filtration. Application of polyphosphate compound, such as sodium hexametaphosphate, may be desirable following re-carbonation. Approximately 0.5 mg/L of a polyphosphate will inhibit scale formation and prevent incrustation of the filter media with hard, crystalline deposits of calcium carbonate.

❖ Carbon dioxide requirements- Carbon dioxide requirements are established by the amount of hydroxide alkalinity converted to carbonate and the amount of bicarbonate formed. For plant control purposes, alkalinity values are generally expressed in terms of calcium carbonate ($CaCO_3$). Total carbon dioxide requirements are calculated as follows, where (OH^-) is the

hydroxide alkalinity in mg/L, as $CaCO_3$ and (HCO_3^-) is the desired bicarbonate alkalinity in mg/L as $CaCO_3$ in the finished water:

Total mg/L CO_2 = $(.44)[(OH^-) + (HCO_3^-)]$

Lbs CO_2 per million gallons = $3.67[(OH^-) + (HCO_3^-)]$

The approximate CO_2 yields of common fuels are:

Fuel	Basis	CO2 Yield (lbs)
Natural gas	1000 cu-ft	115
Coke	1 lb	3
Kerosene	1 gal	20
Fuel Oil (No. 2)	1 gal	23
Propane	1000 cu-ft	141
Butane	1000 cu-ft	142

Carbon dioxide injection equipment must be carefully designed to ensure maximum absorption efficiency. This is especially important if pure (99. 5% and above) carbon dioxide is used. Equipment is available which will essentially 100% absorption of pure CO_2 when injected as a gas. An alternative feed method for pure CO_2 is use of a chlorinator as a feeding device. With this feeding arrangement, the CO_2 is in solution at the feed point and the problems associated with inter-phase gas transfer are avoided.

g) Split Treatment – Split treatment is accomplished by excess lime treatment of part of the raw water and then, after removal of most of the precipitated magnesium hydroxide and calcium carbonate by sedimentation, adding the balance of the raw water and again mixing and settling. Employment of excess lime treatment in the first stage ensures efficient magnesium removal. The excess lime is neutralized in the second stage by carbon dioxide and bicarbonate in the un-softened, bypassed water. Conventional re-carbonation facilities are not normally required when split treatment is used. Split treatment is often applicable to well waters where taste, odor, turbidity and color are not a problem. While the process can be employed with surface waters, it is generally preferable to follow conventional procedures in which all of the water receives tow-stage treatment. The principle operating problems associated with split treatment are correct adjustment of chemical feeds and bypass flow. The calcium carbonate equivalent of the HCO_3 and CO_2 content of the bypassed water must equal the calcium carbonate equivalent of the OH- content of the first stage softened water plus the desired bicarbonate alkalinity in the finished water, all expressed in consistent units.

This is shown in the following equation:

$$Q_B (A + 2.27C) + (Q_R - Q_B) (H) + (Q_R) (F)$$

Where

- Q_R = Rate of flow of raw water

- Q_B = Rate of flow of bypass raw water

- $Q_R - Q_B$ = Rate of flow of softened water

- P = % of raw water bypassed = $100Q_B/Q_R$

- A = Bicarbonate alkalinity of raw water, mg/L as $CaCO_3$

- C = Carbon dioxide in raw water, mg/L as CO_2 (2.27C = CO_2 expressed as $CaCO_3$)

- H = Hydroxide alkalinity of the first stage softened water, mg/L as $CaCO_3$

- F = Bicarbonate alkalinity desired in finished water, mg/L as $CaCO_3$

Solving this equation and recalling this P, the percentage of total raw water bypassed = $100Q_B/Q_R$ gives the following:

$$P = \frac{100 (H + F)}{H + A + 2.27C}$$

The magnesium hardness of the finished water can be estimated from the following:

$$MgH = \frac{(P)(MgR) + (100 - P)(MgS)}{100}$$

Where

- MgH = magnesium hardness of finished water in mg/L

- MgS = magnesium hardness of the first stage softened water in mg/L

- MgR = magnesium hardness of the raw water in mg/L

- P = % bypass water

h) *Incidental benefits of lime softening*

- ❖ *Disinfection* - Excess lime provides excellent bactericidal treatment, especially at pH values above 10.5. Lime treatment, while not a substitute for chlorination, is an effective supplement,

- ❖ *Reduction of dissolved solids* - Removal of carbonate hardness by lime treatment results in reduction in the total dissolved solids content of the water. All reaction products of lime

softening are relatively insoluble. The lime added to the water, as well as the carbonate hardness constituents in the water, are largely precipitated.

❖ *Iron and manganese removal* - Lime softening is also highly effective as a means of iron and manganese removal. The high pH achieved insured essentially complete precipitation of any iron and manganese present in the raw water.

❖ *Clarification* - Lime softening provides excellent coagulation and clarification as a result of the precipitation of magnesium hydroxide plus a large amount of calcium carbonate.

i)   *Softening plant design* - The equipment, basins, and filters required for lime, lime-soda ash, lime caustic, or split treatment softening are generally similar to the facilities used in conventional coagulation- filtration plants. Two stages of treatment are usually advisable. The design of a lime-soda ash or similar softening plant is a complex and difficult task requiring the services of engineers experienced in projects of this kind. Their assistance should be sought in early stages of project planning.

❖ *Mixing equipment* - One problem encountered at softening plants is vibration of rapid mixing devices due to non uniform deposits of calcium carbonate scale._ Frequent cleaning of the mixer may be required. The frequency of such cleaning can be reduced by recirculation of previously precipitated calcium carbonate sludge from the settling basin to the rapid-mix chamber. Parshall flumes can serve as mixing devices.

❖ *Flocculation and clarification* - Each separate stage of flocculation and clarification should have a total detention time at design flow of about 2.5 hours, 30 minutes for flocculation and 2 hours for clarification. Average depths of both flocculation and clarification units should be 8 to 15 feet. The overflow rate in clarifiers at design flow should be about 0.75gpm per square foot.

❖ *Sludge removal and recirculation* - First-stage settling basins shall have mechanical sludge removal equipment. Such equipment is also desirable in the second- stage basins which follow re-carbonation. Sludge recirculation is generally desirable except during occurrences of severe taste and odor problems. Recycling of a portion of the settled sludge, which is high in calcium carbonate, to the rapid-mix chamber is effective in promoting the softening reactions, especially carbonate precipitation. Where pre-sedimentation is employed, recycling sludge to the pre-sedimentation basin influent will enhance the performance of the pre-sedimentation basin.

❖ *Solids contact units* - Solids contact type basins may be used at many softening plants, particularly those treating ground water, These basins provide the functions of mixing, sludge recirculation, sedimentation and sludge collection in a simple compact unit. Basins of this type, if properly sized, will provide effective softening and clarification treatment.

Overall basin depths of 10 to 15 feet should be used, and the unit should be designed so that the softening slurry is recirculated through the center chamber at a rate of flow 3 to 5 times as great as the rate of flow through the entire unit. The upflow rate at the slurry separation level in the clarification zone should not exceed approximately 1.5gpm per square foot.

❖ *Chemical application and storage* - Lime feeders and slakers are key items of equipment at a softening plant and must be selected on the basis of reliability. Another important item requiring careful consideration by the designer is chemical storage. Depending on the size of the plant, bulk or bag unloading and storage for lime and soda ash must be provided. Storage equivalent to at least 30 days average use shall be provided. Caustic soda, if used, will generally be purchased as a 50 percent solution and appropriately sized storage tanks must be provided for this chemical.

❖ *Sludge disposal* - A disadvantage of any lime softening process is the production of a large mass of sludge of high water content. Provision for its disposal in an environmentally acceptable manner must be made and this problem must be carefully considered in connection with softening plant location and design.

3) CATION EXCHANGE SOFTENING

Hardness is caused principally by the cations calcium and magnesium, and cation exchange softening is accomplished by exchanging these ions for a cation, usually sodium, which does not contribute to hardness. This exchange is achieved by passage of the water through the bed of a granular sodium cation exchanger. This reaction is reversible and the exchanger can be regenerated with a strong solution of sodium chloride (common salt).

a) Media used for ion exchange softening – Ion exchange change water softeners typically use polystyrene resins as the softening media. Such resins must have a hardness exchange capacity of at least 25,000 grains of hardness per cubic foot of resin.

b) *Regeneration of ion exchange softeners* - The regeneration process generally involves three steps: (1) backwashing, (2) application of regeneration solutions, and (3) rinsing.

❖ *Back washing* - The purposes of water softener backwashing are generally the same as the purposes of filter backwashing. Any turbidity particles filtered out of the water during softening are removed by the backwashing process. For polystyrene resin media, bed expansions of from 50 to 100 percent are normally required, which involves backflow rates of 4 to 10 gallons per minute per square foot of bed area. Backwash periods generally range from 2 to 5 minutes. Ion exchange water softeners which operate upflow rather than downflow will not require backwashing, but the water to be softened must be virtually free of suspended matter.

❖ *Application of salt brine -* After the unit has been backwashes, a salt solution is applied to the medium in order to regenerate its softening capabilities. Regeneration brines should be 10 to 15 percent solutions of salt. The more salt used in the regeneration of a softener, the more complete the regeneration will be, and the greater the exchange capacity of the regenerated medium will be. The costs of the extra salt required to obtain the added exchange capacity must be weighed against the advantages of the higher exchange capacity in order to determine which salt dosage to use. Salt consumption commonly ranges from about 0.3- to 0.5-pound of salt per 1,000 grains of hardness removed. The contact time of the brine with the softening medium also has a direct effect on the exchange capacity of the regenerated medium. Contact times of 20 to 35 minutes will generally be used.

❖ *Rinsing -* After regeneration, the brine must be rinsed from the unit before softening is resumed. Disposal of backwash water, spent regenerant, and rinse water must be carefully considered.

c) *Ion exchange water softeners -* Although most ion exchange softeners will be downflow pressure softeners, softening can also be achieved upflow. Larger ion exchange softening facilities are often operated upflow in order to avoid the necessity of backwashing. In general, ion exchange softeners are of two types; open gravity softeners and pressure softeners.

❖ *Open gravity softeners -* Open gravity softeners are constructed in much the same manner as rapid sand filters, and the modes of operation are very similar. However, the ion exchange medium used in open gravity softeners is much lighter than the sand used in filters, so backwash rates for open gravity softeners may also be operated upflow, but the softener will not achieve any filtering effects so the influent water must be virtually free of suspended matter.

❖ *Pressure softeners -* A polystyrene *resin* medium used for pressure softening shall have a minimum bed depth of 24 inches and physical properties approximately the same as the following:

Shipping weight	45 - 55 lb/cu-ft net
Density	48 – 53 lb/cu-ft
Moisture content	42 – 48%
Void volume	40%
Screen grading	16 – 50 mesh
Effective size	0.45 – 0.55mm

Uniformity coefficient	<1.70
Fines through 50 mesh screen	<1%

If downflow softening is used, the flow rate through the softening medium may vary from 2 to 8gpm per square foot but must not exceed 10gpm per square foot under the most severe loadings. Severe reductions in exchange capacity are experienced if the softener operates at rates of flow in excess of 10gpm per cubic foot for sustained periods of time. With upflow softening, the rate of flow should be adjusted to maintain a bed expansion of from 40 to 60 percent. The degree of bed expansion is a function of both the flow rate and the temperature of the influent water, so the flow rate must be decreased as water temperature decreases if a constant bed expansion is to be maintained.

d) *Blending* - An ion exchange softener operating properly will produce water having a hardness approaching zero. Inasmuch as it is not generally economical or desirable to soften all water to this low hardness level, provisions, for blending the softened water with the un-softened water are desirable.

e) *Other factors affecting ion exchange softening*

❖ *Turbidity* - Turbidity particles present in the water influent to the softener are deposited on the softening medium and may cause losses of exchange capacity and excessive head losses through the softener. If turbidity levels are excessive, the particles must be removed from the water prior to softening or special backwashing procedures must be implemented.

❖ *Bacterial slimes* - Unless proper disinfection is practiced, bacterial slimes can form in the softening medium and cause excessive head losses and loss of exchange capacity. These slimes can be prevented or removed through chlorination of' feedwater or regeneration water.

❖ *Temperature* - The loss of head through a water softener is strongly affected by water temperature, with lower head losses occurring at higher temperatures. For example, at similar flow rates the head loss through a softener at 122°F is only about 35% of what it would be at 37°F. Also the water temperature affects the exchange capacity of the softener, with a 10 to 15 % increase at high operating temperatures (>86°F.) over the exchange capacity at low temperatures (32 to 50°F)

❖ *Iron, manganese and aluminum* - If iron, manganese, and aluminum are present in the influent water, precipitates may be formed which coat the medium particles and cause a loss of exchange capacity. This problem can be avoided through treatment to remove the iron, manganese, and aluminum from the water prior to softening. If iron fouling occurs it may be possible to overcome it by periodic applications of sodium bisulfite, sodium

hydrosulfite, hydrochloric acid, or sulfuric acid to the softening media. However, these treatments should be implemented only after a thorough study of the problem by someone experienced in this area.

❖ *Total hardness and sodium concentration* - If the total hardness exceeds 400 mg/L or the sodium salts, expressed as mg/L $CaCO_3$, exceed 100 mg/L; the softener should be sized on the basis of the "compensated total hardness" rather than the total hardness.

Compensated hardness is calculated as follows:

$$TH_C = \frac{(TH)\ (9000)}{9000 - TC}$$

Where

o TH_C = compensated hardness in mg/L as $CaCO_3$

o TH = total hardness in mg/L as $CaCO_3$

o TC = total cations in mg/L, all expressed as $CaCO_3$

Compensated hardness (THC) in mg/L is converted to grains per gallon by multiplying by 0.0584 or dividing by 17.1.

f) Removal of noncarbonated hardness following lime softening - In some cases, it is more economical to remove noncarbonated hardness in cation exchanges than by application of soda ash. This method involves the use of lime for reduction of carbonate hardness. Following re-carbonation, the water is filtered. Then all or part of the water, depending on the final hardness desired, is treated in cation exchange softeners for the removal of noncarbonated hardness. The technique is most suitable to those areas where regeneration salt can be obtained at a low cost.

g) Comparison on lime-soda ash and cation exchange processes - Although the purpose of both the lime-soda ash process and the cation exchange process is to achieve removal of calcium and magnesium ions, the modes of operation and the quality of the resultant water are somewhat different.

❖ *Turbidity, iron, and manganese* - Lime-soda softening also effects removal of turbidity and iron and manganese, whereas cation exchange softening may have to be preceded by conventional treatment for removal of suspended matter and iron and manganese.

❖ Bacteria – The lime-soda ash process will also result in the removal or destruction of most bacteria in the water. In contrast, the water entering a cation exchange softener must be disinfected in order to prevent the growth of bacterial slimes within the softening resin.

❖ *Total dissolved solids* - Total dissolved solids concentrations of water are usually lowered by lime soda ash softening, especially if most of the hardness initially present is carbonate hardness. However, application of soda ash to remove noncarbonated hardness results in a slight increase of TDS concentrations. Softening of water by cation exchange processes always results in an increase in TDS levels, because the sodium required to replace calcium and magnesium in the water has a mass 1.15 times as large as the calcium replaced and 1.89 times as large as the magnesium replaced.

IRON AND MANGANESE CONTROL

Iron (Fe) and manganese (Mn) can be considered the two most troublesome minerals to be found in water supplies. These elements cause stains on porcelain plumbing fixtures and laundry and cause coffee or tea to be cloudy and unpalatable. In addition, they can cause diarrhea.

The most common way of treating iron and manganese is aeration, followed by the addition of potassium permanganate ($KMnO_4$), chlorination, addition of an alkali, flocculation and settling, and then filtration. The addition of potassium permanganate is the only aspect of this process which is unlikely to be part of a typical water treatment process when iron and manganese are absent.

1) OCCURRENCE OF IRON AND MANGANESE

Dissolved iron and manganese are encountered principally in ground waters devoid of dissolved oxygen, Normal, oxygenated surface waters do not contain significant concentrations of these metals; however, stagnant water, found in the bottom of thermally-stratified reservoirs, sometimes contain dissolved iron and manganese. Their presence in solution is associated with anaerobic conditions near the bottom of the reservoir.

2) EFFECTS OF IRON AND MANGANESE

Dissolved iron in excess of 1 or 2 mg/L will cause an unpleasant taste, and on standing, the water will develop a cloudy appearance. Iron concentrations appreciably greater than 0.3 mg/L will cause red stains on plumbing fixtures and laundry. Similarly, manganese will cause black stains if present to the extent of more than about 0.05 mg/L. Deposits of iron and manganese can build up in water distribution systems and periodic "flushouts" of these deposits result in objectionable color and turbidity at the consumer's tap.

3) REMOVAL BY OXIDATION AND FILTRATION

Oxidation can be accomplished with dissolved oxygen, added by aeration, and by the addition of an oxidizing chemical, such as chlorine, chlorine dioxide, potassium permanganate, or ozone. Manganese is more difficult than iron to oxidize and precipitate. In the absence of manganese, iron can often be

removed with minimum treatment, consisting of aeration followed by direct filtration. In general, aeration alone will not oxidize manganese unless the pH is raised to about 9.5. Strong oxidants, such as chlorine or potassium permanganate, are effective at lower pH values. To insure oxidation, precipitation and agglomeration of iron and manganese and their essentially complete removal, at least three treatment steps are usually necessary: aeration, contact time, and filtration. An aerator containing trays of coke, limestone, etc., as mentioned in paragraph 2-3c is commonly used. Reaction time is provided by a contact or contact-sedimentation basin having a detention period of at least 30 minutes. Filtration is accomplished by conventional single or multimedia filters designed for a filtration rate of at least 3.0 gpm per square foot - The aeration step is frequently supplemented by a chemical oxidant, such as chlorine or permanganate. Flocculation is advantageous in the contact basin, particularly if iron exceeds about 2 mg/L.

4) REMOVAL BY ION EXCHANGE

The cation exchange (sodium zeolite) softening process, under proper conditions, is capable of removing limited amounts of dissolved (unoxidized) iron and manganese. For application of this process, it is essential that the raw water and wash water contain no dissolved oxygen and that the sum of the iron and manganese concentrations not exceed about 0.5 mg/L. The presence of oxygen or higher concentrations of iron and manganese will cause rapid fouling of the exchange resin with consequent loss of removal capacity. If fouling occurs, treatment of the resin with sodium bisulfite solution and dilute hydrochloric or sulfuric acid will be required to restore capacity.

5) REMOVAL BY LIME-SODA SOFTENING

Lime-soda softening is an effective means of removing both iron and manganese.

6) STABILIZATION OF IRON AND MANGANESE

Under some circumstances, stabilization of iron and manganese by application of a polyphosphate compound may be acceptable. The iron and manganese in the water are maintained in a dispersed state through the completing action of a polyphosphate compound. Dosages of about **5** mg/L of sodium hexa-meta-phosphate for each mg/L of iron and manganese are reasonably effective however the total polyphosphate dosage should not exceed 10 mg/L, expressed as PO_4^{3-}. The polyphosphate stabilizing compound *must be* added to the water prior to chlorination. If the chlorine is applied first, it will oxidize the iron and manganese to insoluble forms rendering the stabilizing agent ineffective. Stabilization of concentrations of iron and manganese in excess of approximately 1.0 mg/L is generally not satisfactory. Also, stabilization will not persist if the water is heated because heating coverts polyphosphates to orthophosphates which have no stabilizing power. Stabilization, although helpful, is not a substitute for iron and manganese removal, and, in general, should be viewed as a temporary expedient to be used pending installation of removal facilities.

CORROSION AND SCALE CONTROL

"Corrosion" can be defined as the deterioration of metal by direct chemical or electrochemical reaction with its environment. "Scale" refers to an accumulation of solids precipitated out of the water. In water treatment, corrosion and scale are closely associated. Both must be considered in connection with the design and operation of treatment works. This scale may be desirable because it can provide a measure of protection against corrosion. However, thick layers of scale are detrimental in both hot and cold water systems. It is essential to produce a "balanced" water that is neither highly corrosive nor excessively scale forming.

1) **CORROSION**

   a) The extent and nature of corrosion reactions depend upon many factors. Among the most important are the chemical and physical nature of the water, its velocity, pipe metallurgy and pipe coating. In existing systems, where corrosion is a problem, most of these factors, with the exception of the chemical nature of the water, are not readily susceptible to change. Consequently, for these situations, emphasis must be placed on adjustment of the water's chemical quality as the only practical means of corrosion control in an existing system. Controllable factors are principally calcium content, alkalinity and pH. Certain corrosion inhibitors can also be used, but relatively few are suitable for potable water systems.

   b) Treatment to insure deposition and maintenance of a thin layer of calcium carbonate on the pipe interior is one widely used means of corrosion control. This control method, while not infallible, has been fairly successful in minimizing the corrosion rate of iron pipe. The rate of formation of calcium carbonate is favored by high concentrations of calcium and bicarbonate and carbonate alkalinity. Protection of this type cannot be attained in waters containing very low concentrations of calcium and alkalinity.

   c) Corrosion rates may also be reduced by the use of certain inhibitors. For potable water systems, the most practical inhibitors are silicates and certain polyphosphate compounds. Sodium silicate can be used to a limited extent in very soft water. Polyphosphates can be applied for scale as well as corrosion control. They are considered most effective for corrosion control in the pH range 5.0 to 8.0 and their effectiveness is greatly influenced by velocity. Low velocity, such as encountered in dead-end mains, reduces the effectiveness of all corrosion control methods.

   d) Dissolved oxygen and carbon dioxide have a significant effect on corrosion rates. Carbon dioxide lowers the pH and makes the water more aggressive. Carbon dioxide can be removed chemically, but it is generally not feasible to attempt chemical removal of oxygen from potable water supplies. Most surface waters are normally saturated with oxygen while ground waters, initially free of oxygen, usually absorb some during treatment and distribution. When

considering the removal of carbon dioxide by aeration, it should be kept in mind that while efficient aeration will remove most of the carbon dioxide, it will, in doing this, practically saturate the water with oxygen.

e) Corrosion rates are influenced to some extent by all mineral substances found in water, but corrosion effects are so interrelated that it is not possible to isolate the quantitative influence of individual ions. It is known that high concentrations of chloride and sulfate ions will produce increased corrosion rates. However, their adverse effects are somewhat mitigated by alkalinity (carbonate, bicarbonate) and calcium ions. To obtain appreciable benefit from alkalinity and calcium, the total alkalinity, expressed as calcium carbonate, should be at least 50 mg/L, preferable in the range of 50 to 100 mg/L. The calcium concentration, calculated as calcium carbonate, should also be at least 50 mg/L. In general, the higher the concentrations of alkalinity and calcium, the greater is the water's capacity for corrosion retardation. On the other hand, excessive calcium and alkalinity will often result in objectionable scale formation. It is, therefore, necessary to seek a compromise between corrosion on the one hand and scale formation on the other. (6) Based on the corrosion accelerating effects of chloride and sulfate and the corrosion inhibiting effects of alkalinity, the following ration, termed the "Corrosion Index," has been developed.

$$\text{Corrosion Index} = \frac{[0.02821]\,[Cl^-] + [0.02083]\,[SO_4^{2-}]}{[0.01639]\,[HCO_{3-}]}$$

$[Cl^-]$ = mg/L chloride ion as Cl^-

$[SO_4^{2-}]$ = mg/L sulfate ion SO_4^{2-}

$[HCO_{3-}]$ = mg/L bicarbonate ion as HCO_{3-}

For a pH range of about 7 to 8, and in the presence of dissolved oxygen, an index below about 0.1 indicated probable general freedom from corrosion. An index higher than 0.1 is indicative of corrosion tendencies; the higher the index, the greater the probability of corrosion.

2) SCALE

Scale problems in distribution systems are caused principally by calcium carbonate, magnesium hydroxide, aluminum hydroxide and the oxides and hydroxides of manganese and iron.

a) Aluminum – Aluminum hydroxide deposits can result from excessive alum use for coagulation and/or improper coagulation practice, such as poor mixing and flocculation and incorrect coagulation pH. Aluminum hydroxide can create a soft, white deposit having a rippled surface which will produce reductions in pipe carrying capacity as measured by the Hazen- Williams "C" value. The problem is one of "after precipitation" of aluminum hydroxide; i.e., aluminum

remains in solution until after filtration. Chlorination, which often follows filtration, will reduce the pH slightly and the chemical nature of aluminum is such that a slight reduction in pH will result in significant reduction insolubility.

b) Magnesium - Magnesium hydroxide deposits have caused serious difficulties in distribution systems and hot water heaters. Magnesium volubility is highly sensitive to pH and temperature and failure to exercise careful control over its stabilization following softening will usually lead to deposition problems. In the absence of detailed information regarding the scaling tendencies of given water, it is advisable to maintain magnesium hardness below 40 mg/L and pH below 9.8. Hot water heaters should be operated so that water temperatures will not exceed 140° F.

c) Iron and manganese - Hydrous oxide deposits of iron and manganese are inevitable in distribution systems handling water containing more than about 0,3 mg/L of iron and 0.05 mg/L of manganese. The severity of the problem is directly related to the concentration of iron and manganese and the best solution is to remove them at the source. A less satisfactory procedure is to attempt to prevent their precipitation by polyphosphate treatment at the source. Iron deposits may also be caused by corrosion reactions which form loose scale or tubercles. In severe cases, cleaning and lining of the pipe may be required. Tubercle formation can be minimized through corrosion control.

3) *CHEMICAL CONTROL OF CORROSION*

a) Calcium carbonate saturation

❖ One means of chemical control that has been reasonably successful is treatment of the water to ensure deposition and maintenance of a coating of calcium carbonate. Prediction of the tendency of a water to precipitate or dissolve a protective coating of calcium carbonate can be based on computation of what is termed the "Langlier Index" (LI). This index is calculated as follows:

LI = pH – pHs

Where

LI = Langlier Index

pH = Actual pH of the water

pH_s = Calculated pH of water corresponding to saturation with calcium carbonate

The value of pH_s is calculated on the basis of no change in the water's alkalinity in the water's alkalinity, calcium content, dissolved solids, or temperature. If the computed pH_s is less than the actual pH, the LI will be positive, indicative of a calcium carbonate deposition tendency. If pH_2 is greater than the actual pH, the LI will be negative and this indicates saturation or a tendency toward dissolving calcium carbonate, and corrosivity. An LI value

of O indicates exact saturation with calcium carbonate and no tendency toward deposition or solution.

❖ The complete equation for the exact calculation of pHs is quite complex and a simplified form, covering the pH range 6.5 to 9.5 may be used. The simplified equation is as follows:

pHs = A + B – log (ca2+) – log (alkalinity)*

Where

pHs = pH of calcium carbonate saturation

A = constant which is a function of water temperature

B = constant which is a' function of total dissolved solids concentration (TDS, mg/L) log (Ca^{2+}) = logarithm to the base 10 of the Ca^{2+} concentration in mg/L log (Alkalinity) = logarithm to the base 10 of the total alkalinity expressed as CaCO3, in mg/L.

Values of Ca^{2+} and Alkalinity are obtained from analytical data.

The values of A and B are obtained from the tables below:

Constant A as a Function of Water Temperature

Water Temperature	A
32	2.60
39	2.50
46	2.40
54	2.30
61	2.20
68	2.10

Constant B as a Function of Total Dissolved Solids

TDS mg/L	B
0	9.70
100	9.77
200	9.83
400	9.86

TDS mg/L	B
800	9.89
1000	9.90

* Source - Reprinted from Standard Methods, for the Examination of Water and Wastewater; 14th edition.

Examples

Examples of the calculation procedure to be followed in determining the LI are as follows:

Water No 1

TDS mg/L	340
Calcium (Ca^{2+}), mg/L	72
Alkalinity, mg/L as $CaCO_3$	80
pH	8.5
Temperature, F	65

$LI = pH - pH_s$

$pH_s = A + B - \log (Ca^{2+}) - \log (\text{alkalinity})$

$pH_s = 2.10 + 9.85 - 1.86 - 1.90 = 8.20$

$LI = 8.5 - 8.2 = +0.3$

Water No 2

TDS mg/L	490
Calcium (Ca^{2+}), mg/L	32
Alkalinity, mg/L as $CaCO_3$	20
pH	7.5
Temperature, F	75

$LI = pH - pH_s$

pH_s = A + B – log (Ca^{2+}) – log (alkalinity)

pH_s = 2.00 + 9.87 – 1.51 – 1.30 = 9.07

LI = 7.5 – 9.1 = -1.6

❖ The LI is not a quantitative index in the sense of providing a numerical measure of the amount of calcium carbonate that will be precipitated or dissolved. Rather, it merely indicates a tendency in the direction of precipitating or dissolving calcium carbonate. If the water is extremely soft (deficient in calcium ions) and has a low alkalinity, the water's capacity for protection will be minimal even though a high pH and a positive LI are consistently maintained. The water should contain at least 50 mg/L of alkalinity and at least 50 mg/L of calcium hardness in order to take advantage of calcium carbonate protection. For softened waters, maintain an LI of about + 1.0, and, in addition, apply about 0.5 mg/L of polyphosphate to the filtered water in order to prevent excessive deposition in pumps and mains near the treatment plant.

❖ The maintenance of a positive Langlier Index does not preclude the possibility of corrosion. Conditions may be such that only a partial coating of calcium carbonate is deposited, resulting in a type of corrosion at the uncoated areas, known as "pitting." Pitting corrosion results in loss of metal from relatively small areas of the pipe rather than informly over the entire surface. As a consequence, the pipe may fail fairly quickly because of corrosion penetration of the pipe wall.

SPECIAL PROCESSES

In some cases it will be necessary to use raw water supplies containing unacceptably large concentrations of constituents that cannot be removed by conventional treatment processes. The most common of these objectionable constituents are mineral salts, such as sulfates and chlorides, and volatile organic compounds, (VOCs). Special treatment processes are necessary to remove these materials,

1) DEMINERALIZATION

The presence of excessively high concentrations of dissolved minerals in water is indicated by high chloride (Cl-), sulfate (SO_4^{2-}), and total dissolved solids (TDS) levels. The recommended limits for these substances are 250 mg/L, 250 mg/L, and 500 mg/L, respectively. These limits are based on esthetic considerations and considerably higher concentrations, while not desirable, can be tolerated. Where demineralization is required, processes commonly employed are electrodialysis, reverse osmosis, distillation, and ion exchange. Disposal of waste brine solutions derived from these processes often poses a serious problem and must be carefully considered at an early stage in project development. All demineralization processes are energy intensive, and alternative water sources should be thoroughly

investigated before a commitment to a demineralization project is made. If the demineralization process selected requires large inputs of electricity, consideration should be given to its operation principally during "off-peak" hours with storage of desalted water until needed,

2) REMOVAL OF VOLATILE ORGANIC COMPOUNDS

VOCs can be either halgonated naturally occurring organic substances (trihalomethanes), or synthetic organic compounds (SOCs).

a) *Trihalomethanes* - Naturally occurring organic substances (precursors), such as humic and fulvic acids are derived from leaf and soil extract and are not themselves volatile. When the precursors (usually found in surface waters) enter the treatment facility in the raw water they react with the free available chlorine injected for purposes of disinfection. These halgonated organic compounds are known as trihalomethanes (THMs). Other THMs can be produced by exposing precursors to other halogens, such as bromine or iodine. This grouping of total trihalomethanes (TTHMs) is generally comprised of four primary constituents: trichloromethane (chloroform), bromodichlormethane, chlorodibromomethane, and tribromomethane (bromoform). Three basic approaches to control THMs are:

   ❖ Use of a disinfectant that does not generate THMs in water. (Ozone, chlorine dioxide)

   ❖ Treatment to reduce the concentrations of precursor material prior to chlorination (coagulation, flocculation, filtration),

   ❖ Treatment to reduce THM concentrations subsequent to their formation (aeration, carbon adsorption). These three methods have been presented throughout this technical manual.

b) Synthetic Organic Compounds (SOCs) - are VOCs some of which have been found in many groundwater sources used for potable water supplies. SOCs are found in groundwater due to improper disposal of spent industrial-type solvents, paint thinners, cleaning agents and some household chemicals, Two common SOCs are trichloroethylene (TCE) and tetrachlorethylene. Some VOCs are rather soluable and have little affinity for soil materials, and therefore can travel great distances to an aquifer from an industrial waste lagoon, industrial, commercial on domestic septic system, landfill, accidental spill or illegal disposal.

c) Removal Technologies for VOCs - Three different technologies are available for the removal of VOCs: aeration, carbon adsorption, or resin absorption. All of these methods have been presented in previous sections of this technical manual, with the exception of resin absorption. Resin absorption involves the physical separation of the organic compounds from water by using a polymeric absorbent or resin filled unit. The resin is specific to the VOC it will remove, therefore great care must be taken in the selection of the resin. The resin-filled units also require frequent regeneration with a low pressure backwash and an alcohol-wash. The waste

from the backwash will contain high concentrations of VOCs and may be classified as hazardous waste.

d) Selection of a removal technology - Important parameters for removing VOCs are the concentrations concerned, the type of VOC, and the cost of the removal method.

❖ The higher the concentration of VOCs the more expensive removal will become, Higher concentrations of VOCs will normally require larger equipment, e.g. counter-current packed column aeration towers must increase in either volume or blower and pump horse power for increased removal of VOCs. Low TTHM concentrations may be handled by simply changing the point of chlorination and allowing coagulation and flocculation to remove THM precursors. High TTHM concentrations may require the addition of an aeration tower or a GAC contactor and at the extreme an alternate disinfectant such as ozone.

❖ The type of VOC to be removed may dictate the method of removal. Most VOCs can be reduced to meet Federal maximum contaminant levels through airstripping by an aeration tower. However, some VOCs, such as bromoform cannot be easily removed through airstripping and a more expensive method of removal such as carbon adsorption must be used.

❖ Airstripping through counter-current packed column aeration towers appear to be a cost-effective method for reducing VOCs. Preliminary analyses suggests that it may be more economical than GAC or resin absorption treatment. Predicted capital costs and overhead and maintenance expenditures for aeration towers are less than other treatment technologies. However, pilot testing must be performed to prove the feasibility of any solution to the removal of VOCs. Pilot testing will allow enhancement of a selected method, once that method has been proven feasible, allowing a maximum removal of VOCs for a minimum of cost.

Conclusion

The design of treatment facilities will be determined by feasibility studies, considering all engineering, economic, energy and environmental factors. All legitimate alternatives will be identified and evaluated by life cycle cost analyses. Additionally, energy use between candidate processes will be considered. For the purpose of energy consumption, only the energy purchased or procured will be included in the usage evaluation. All treatment process systems will be compared with a basic treatment process system, which is that treatment process system accomplishing the required treatment at the lowest first cost. Pilot or laboratory analysis will be used in conjunction with published design data of similar existing plants to assure the optimal treatment. It is the responsibility of the designer to insure that the

selected water treatment plant process complies with Federal Environmental Agency, State or local regulations, whichever is more stringent.

****