

CHAPTER 5W. - DISINFECTION

The disinfection of water involves specialized treatment for the destruction of harmful and otherwise objectionable organisms. Classically, disinfection has been practiced for the purpose of destroying or inactivating disease (pathogenic) organisms, more particularly, bacteria of intestinal origin. Pathogenic organisms other than bacteria that merit attention in connection with water disinfection include a variety of viruses, intestinal protozoa, and some few macro organisms. Important among the factors affecting the survival of these organisms in water, insofar as disinfection is concerned, is their natural resistance or imposed resistance to the disinfectants used.

The disinfection process, as practiced in water treatment, should be differentiated from the process of sterilization. Sterilization is the complete destruction of all living organisms, including bacteria amebic cysts, algae, spores and viruses. Disinfection does not provide for destruction of all microorganisms; not even all pathogenic organisms. The hepatitis virus and the polio virus, for example are not completely inactivated by most current disinfection techniques (except for ozone).

In the disinfection of water a group of bacteria called Coliforms are used as the major indicators of pollution in drinking water. Standard Methods defines the coliform group as "including all aerobic and facultative anaerobic, gram negative, non spore forming rod shaped bacilli which ferment lactose with gas formation within 48 hours of incubation at 35°C. Some coliforms particularly *Escherichia coli* (*E.coli*) are characteristically found in the intestinal tracts of humans and other animals. These coliforms are excreted in the feces and are better known as fecal coliforms. Some coliforms also exist in soil and other habitat not associated with human digestive tracts.

The presence of any type of coliform in treated drinking water suggests either inadequate treatment or contamination of the water after treatment. Because of their relative ease of detection, the coliform organisms are used as the major indicators of pollution in drinking water.

The most common form of disinfection for water treatment is chlorination. Other forms of use are iodine, fluorine bromine, ozone, ultraviolet light, hydrogen peroxide, and permanganate. The ASCE·AWWA·CSSE Water Treatment Plant Design Manual states that "Except for chlorine,. . ., most other water disinfectants herein mentioned have one or more serious limitations that preclude their general acceptance and adaptability for municipal potable water treatment operations. (8)

In light of new and continued research with ozone as a disinfectant, the strict requirements set forth in the new Primary Drinking Water Standards, and the ability of ozone to remove color, we propose to discuss two methods of disinfection, chlorine, and ozone.

Chlorine

Chlorine has been used as a disinfectant for water since 1896, in England. Since then, as of 1962, more than 10,000 U.S. Water treatment facilities serving about 135 million people, employ chlorine disinfection. (16)

For purposes of disinfection of municipal supplies, chlorine is primarily used in two forms; as a gaseous element or as a solid or liquid chlorine containing hypochlorite compound.

1. Gaseous chlorine

In a gaseous state chlorine is greenish yellow in color and about 2.48 times as heavy as air; the liquid is amber colored and about 1.44 times as heavy as water. Chlorine is only slightly soluble in water, its maximum solubility being about 1% at 49.2 F.

2. Hypochlorites

Calcium hypochlorite, a dry bleach, has been used widely since the late 1920's. Present day commercial high test calcium hypochlorite products contain at least 70% available chlorine. Calcium hypochlorite is readily soluble in water, varying from about 21.5 g/100ml @ 0°C to 23.4 g/100ml @ 40°C, (15)

Commercial sodium hypochlorite usually contains 12 to 15% available chlorine and is available only in liquid form.

When chlorine dissolves in water, it reacts to form hypochlorous and hydrochloric acids. Chlorine has a strong affinity for other materials particularly for reducing agents which sometimes exert a considerable chlorine demand. In these reactions, the chlorine atom manifests a great tendency to lose electrons and form chloride ion, or organic chlorides. It has lately been found that a few of these organic chlorides or "chlorinated hydrocarbons", are carcinogenic, i.e., cause cancer. This is the case where there are large quantities of organic compounds in the water, especially heavily polluted rivers. For this reason, many researchers feel that chlorination practices, will soon be abandoned in search for a safer method of disinfection.

The destruction of pathogens by chlorination is dependent upon water temperature, pH, time of contact, degree of mixing, turbidity, presence of interfering substances, and concentration of chlorine available. The order of difficulty in disinfection of waterborne pathogens generally is bacteria < virus < cysts.

Researchers have found many relationships between the kill rate of the microorganisms to be removed, and chlorine residual, temperature and pH. It has been generally recommended that thirty minutes of contact time at 0.4 mg/l chlorine residual be required for disinfection of most microorganisms. Where colder waters are encountered, longer contact time with higher residuals are required for adequate disinfection. Longer contact time requires increased storage usually underground, and increased costs. A thorough testing program comparing residuals to kill rate is recommended to determine the required dosage rates for the new water treatment plant.

Ozone

Ozone, a faintly blue, pungent smelling, unstable gas, is an allotropic form of oxygen in which three atoms of the element are combined to form a molecule, O_3 . Weber states "Ozone, O_3 is a particularly powerful oxidizing agent, from both thermodynamic and kinetic standpoints. The immediate bacteriacidal properties of ozone are superior to those of chlorine and are to a large extent independent of pH." Ozone is used extensively in Europe for water

treatment for disinfection and concurrent with removal of odor, color, iron and manganese.

Ozone is much faster acting as a disinfectant than chlorine, thus reducing the need for long contact periods and large storage facilities. Disinfection with ozone has the advantage of being effective against some chlorine resistant pathogens like cysts, and certain virus forms. Ozone unlike chlorine does not impart offensive tastes and odors to water, nor has it been shown to produce toxic substances such as chlorinated hydrocarbons, which can result from chlorination practices.

Water Quality and Treatment⁽⁸⁾ lists some but not all of the advantages and disadvantages of ozone as a disinfectant. These are:

<u>Advantages</u>	<u>Disadvantages</u>
1. Complex taste, odor, and color problems are effectively reduced or eliminated.	1. No lasting residual disinfecting action is provided.
2. It is a powerful oxidant that rapidly oxidizes organic impurities.	2. Electric energy requirements and capital and operating costs are high (about 10 to 15 times higher than chlorine).
3. Disinfecting action is effective over a wide temperature and pH range.	3. The ozone-air mixture produced by necessary on-site generation is only slightly soluble in water, and production is complicated when temperature and humidity are high.
4. Bactericidal and sporicidal action is rapid (said to be from 300 to 3,000 times quicker than chlorine), and only short contact periods are required.	4. Process is less flexible than chlorine in adjusting for flow rate and water quality variations.
5. Odors are not created or intensified through formation of addition or substitution complexes.	5. Analytic techniques are not sufficiently specific or sensitive for ready and efficient control of the process.
6. There is no possibility of danger or harm from over-treating.	6. Waters of high organic and algal content usually require thorough pretreatment to satisfy the ozone demand.
7. Ozone is much more soluble in cold waters than chlorine.	

As stated previously, ozone is a multipurpose chemical that can be used for color reduction, iron and manganese oxidation, and taste and odor control, besides being a powerful disinfectant.

Stumm ⁽¹⁵⁾ reports that "two major deterrents to the use of ozone for disinfection of public water supplies in the U.S. have been the initial cost of ozonation equipment, and the fact that it provides no residual protection against recontamination, and must therefore be followed by post-chlorination. Improved water quality, higher chlorine stability, and the prevention of the formation of malodorous compounds such as the chlorophenols and other potentially toxic chlorinated compounds must be considered as significant benefits in the use of ozone for disinfection of water supplies."

A typical ozone application is shown in Fig. 5W.-1

A complete system is composed of three major parts as described below:

1. Ozone generation in which ozone is generated by passing air or oxygen through a highly charged electrical field.
2. Ozone dissolution, in which the air/ozone gas mixture is diffused through the water.
3. Ozone destruction, in which any off gas or undissolved ozone is thermally or catalytically destroyed before it can enter the atmosphere.

Ozone must be bubbled through the water, and as it rises, will make contact with the compounds in solution.

The instability of ozone generally necessitates its generation at the time and on the site of its application.

Ozone is generated either from air or pure oxygen. Ozone generated from air is about 1% concentrated while ozone generated from pure oxygen is about 1.7 - 2% concentrated. ⁽¹⁸⁾

Briefly ozone generation can be described as follows. ⁽¹⁵⁾

1. Filtered air is blown through a cooler to maintain the air at a constant temperature.
2. The air then passes through desecating beds which remove all the moisture from the air.
3. Ozone is generated then by passing the air past 5000 to 30,000 volt electric discharge.
4. The air and ozone gas mixture leaves the ozonator and is injected at a fairly high pressure, 90 p.s.i., into the solution to be treated.

The major item of operating expense in the generation of ozone is the cost of electric power. The first cost of an ozone plant depends on when and where it is built, the size of ozonator required, and the amount of color, etc. in water to be oxidized. Most water applications are in the dose range of .5 - 1.5 p.p.m.

Modern ozonators require very little maintenance beyond a semi-annual cleaning and occassional diaelectric replacement. The stainless steel structure of the ozonator has a life expectancy of at least 20 years.(16)

As pointed out before, ozone is very unstable, and leaves no residuals in the system. Therefore, in all ozone practices, ozonation is followed by slight chlorination so that residuals can be measured and the consumer protected against contamination occurring in the distribution system.

Comparative operating costs of using chlorine and ozone are shown in Tables 5W.-1 and 5W.-2. Taking only disinfection into consideration, the cost for ozone greatly outweighs the cost of a chlorine system.

For reasons that will be pointed out in the next chapter, ozone will be investigated for its ability to oxidize color, and taste and odor causing compounds.

It is recommended that ozone be used primarily as an oxidizing agent for bleaching out the color causing compounds, and secondly as a disinfectant. The ozone levels required for color removal are in excess of those required for maximum disinfection, and

100% disinfection is expected. Chlorination is recommended to compliment the ozonation process and provide for a residual to be maintained in the distribution sytem. Thirty minutes of contact time at a concentration of one p.p.m. is recommended.

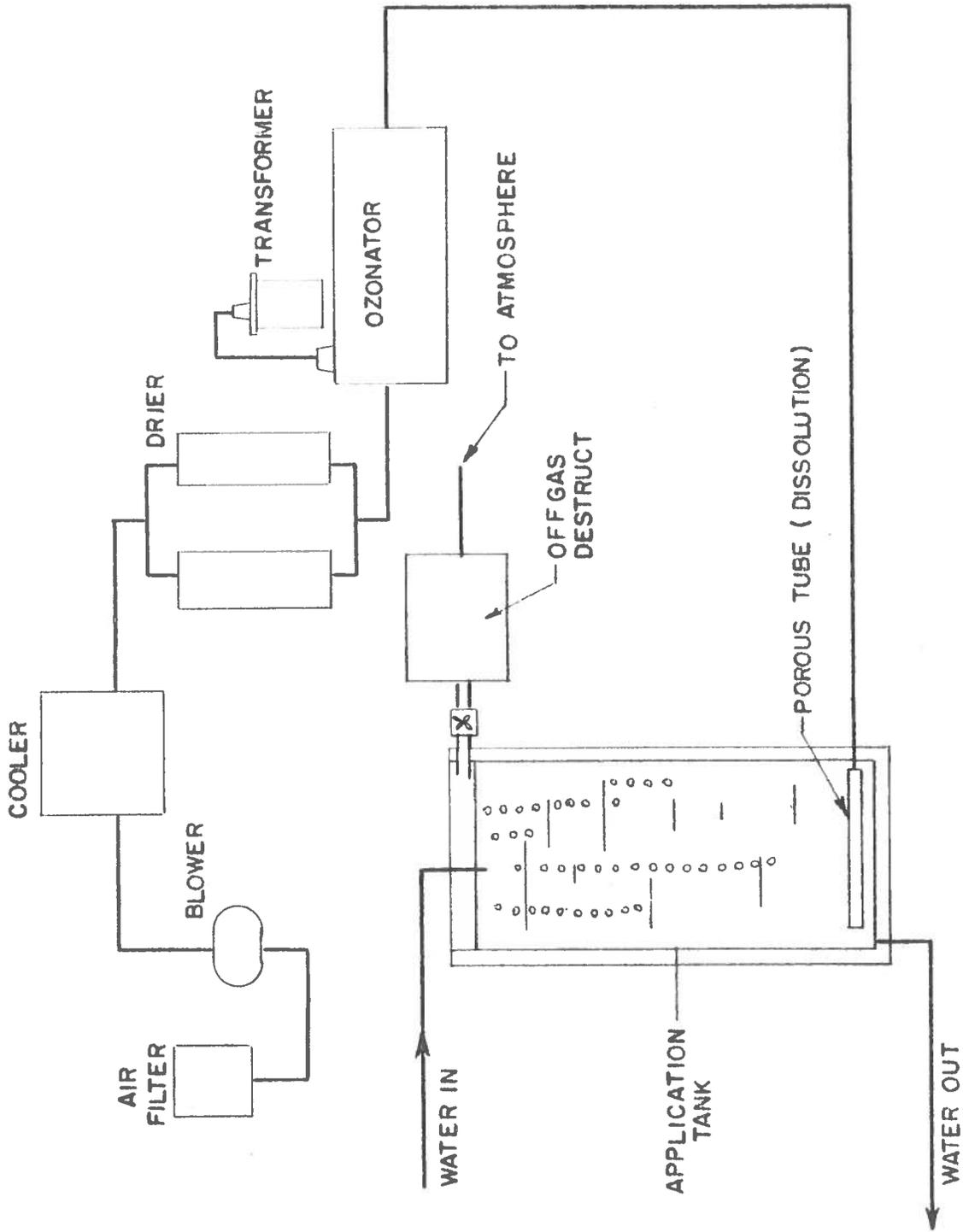


FIG. 5W-1
 SCHEMATIC OZONE PLANS FOR WATER TREATMENT

T A B L E 5W-1

ESTIMATING COSTS AND SIZES FOR OZONATORS
(based on PCI equipment)

DOSAGE P.P.M.	CAPACITY from DRY AIR	EQUIPMENT COST	POWER REQUIREMENT KWH/HR.	POWER COST PER MO.	POWER COST*/ MIL. GAL.	SIZE REQ'D. L. W. H.
.24	7 lb./da.	\$15,000	4.8	64.86	\$.83	96x48x70
.49	14 lb./da.	20,000	7.2	88.74	1.14	96x48x70
.73	21 lb./da.	25,000	9.6	112.63	1.45	96x48x70
1.73	50 lb./da.	45,000	24	254.91	3.28	9'x 5' x8'
3.47	100 lb./da.	90,000	48	494.80	6.36	10'x 6' x 8'

* This figure is based on pumping 18 hours per day at 2400 g.p.m. or 2,592,000 gal. per day, and a power cost as shown below:

CHARGES PER KWH FOR SARATOGA, AS OF 5/15/77

1st 100/mo. = 5.21¢/KWH
 next 100/mo. = 4.21¢/KWH
 next 800/mo. = 2.21 /KWH
 next 1000 = 1.71/KWH
 next = 1.51/KWH

Total x 18.5% surcharge

Note: Based on 20 year operation .73 P.P.M. dosage, the total cost per year for ozone equipment on power cost is:

$$\frac{\$50,000 + (\$112.63 \times 12 \text{ mo.}) (20 \text{ yr.})}{20 \text{ yrs.}} = \$3,851.56 = \$321/\text{mo.}$$

T A B L E 5W-2

ESTIMATED CHLORINE COSTS AS OF 5/16/77

I DOSAGE PPM	II P.P.D.**	III LBS./MO.	IV CYLINDERS/MO.	V. CL ₂ COST/ MO.	VI*** SHIPPING COST/MO.	VII TOTAL COST PER MO.	VIII COST/ MIL. GAL.
.76	21.62	648.60	4	155.40	53.40	208.80	2.69
1.51	43.23	1297.20	9	349.65	120.15	469.80	6.04
3.78	108.1	3243.0	21	815.85	280.35	1096.20	14.10
6.04	172.74	5188.2	35	1359.75	467.25	1827.00	23.50
7.56	216.2	6486.0	43	1670.55	574.05	244.60	28.87

* Based on 30 day month

** Based on 2400 g.p.m. pumping rate for 18 hours per day, or
2,592,000 G.P.D.

*** Based on shipment of 250 lb. cylinder from Denver via Salt Creek
Frtways.

Note: Based on 20 year operation and 2 p.p.m. dosing, the total cost
per year for equipment and chlorine is

$$\frac{\$10,000.00 + (469.80 \times 12) \times 20}{20 \text{ yrs.}} = \$ 6,137.60/\text{yr.} = \$ 511.47/\text{mo.}$$

CHAPTER 6W. - COAGULATION, FLOCCULATION & OZONATION

Coagulation may be described as that process which is employed to remove foreign particles that are difficult to settle out of water quickly and easily, either because the particle is too small, or because the particles are suspended in solution. Coagulation involves the use of coagulants which chemically speed the removal process. Coagulants may be described as those substances which are capable of aiding in the removal of impurities from water. The purpose of coagulation is to neutralize or destabilize the charged particles in water which cause turbidity and thereby transform them into a particle which is more easily settleable.

Colloids are usually defined as charged particles which fall into the 1 μm to 1 nm size. Particles larger than this usually merge into suspensions and emulsions which may be separated more or less readily by physical means. The charge on the particle is what makes it difficult to settle, as most particles are like charged and tend to repel and thus are capable of staying in solution indefinitely.

Some examples of particles classified as colloids are clays, viruses and some bacteria, and because these materials are not removeable through sand filters, they must be removed from solution in some other way.

Removal mechanisms of colloids have been theorized by many researchers. These theories are too complex for this type of report, but are listed below. (15) (31) (27) (33):

1. Double layer compression theory.
2. Adsorption and Charge Neutralization
3. Enmeshment in a Precipitate
4. Adsorption and Interparticle Bridging

Some common coagulants being used now in the water industry are aluminum salts and ferric salts.

Flocculation is described as that process whereby the neutralized particles are brought together to form a mass which is more easily settleable.

The rate of aggregation is determined by the rate at which collisions occur between colloidal particles and coagulants, and the effectiveness of these collisions in permitting attachment between the particles. These contacts can be increased in several ways, but the most easily controllable method is mixing.

Many factors affect the rates of coagulation. Colloids may be precipitated out of solution by ions of opposite charge. Generally the precipitating power of a bivalent ion (++) will be 50-60 times as great as a monovalent (+) ion and the precipitating power of a trivalent ion (+++) will be 600-700 times as great as that of a monovalent ion. It is this remarkable fact which throws most light on the action of soluble aluminum and ferric salts as coagulants. Donley (34) demonstrated that a polyelectrolyte (multicharged coagulant) can have a significantly greater precipitating power than even a trivalent ion.

Researchers have shown that the electrokinetic or zeta potential of the particle, pH, and other chemical characteristics of the solution all effect coagulation. (33), (35), (36) The chemical reactions are carried out in extremely low concentrations and good coagulation plant operation requires ingenuity, resourcefulness and scientific ability. The high color and turbidity periods are prevalent often for only a two month duration and rarely for more than three months. Our experience and that of the water works industry has shown that a good coagulation process may take one to two weeks to be operating properly. The result of this slow start up could mean that about the time the coagulation units are working properly the supplies will be clear. Recent process discoveries such as the Roberts Manhattan Process have greatly aided proper plant control and should be considered for any coagulation-filtration plant design. (20) In addition, jar testing must be incorporated into normal plant operating routine.

Jar testing is a process in which the plant operator can analyze the water for proper coagulant dosing, mixing, pH monitoring, etc. Jar

testing is essential for maintaining the efficiency of the coagulation-flocculation process. Each time the water quality changes, jar testing must be performed to obtain the correct dose of coagulant for that particular period.

Costing was done for coagulation-filtration versus the recommended processes and these studies showed that coagulation-filtration is the most expensive of possible alternatives.

Color Removal

As mentioned before, color removal in the Spring and early summer months is a major concern of this report. The National Secondary Drinking Water Regulations set the maximum limit for color at 15 NTU and 3 NTU as the desirable limit. Measurements have been as high as 80 NTU. Unfortunately, color is one of the more difficult particles to remove, since it is usually dissolved in solution, or too small to filter. 90% of color compounds are 3.5 - 10 μ in size where clay particles are more apt to be 1 μ .⁽⁸⁾

Color particles are invariably negatively charged and are hydrophilic ("water loving") in nature. These properties of small size, hydrophilic nature and strong negative charge indicate a mechanism of removal entirely different from coagulation of turbidity, may be required.

Color removal through coagulation is generally best in the pH range of 4 to 6. The relationships between pH, alum dosage (used to raise the pH) color removal and zeta potential are given in a series of graphs presented here as Figures 6W-1, 6W-2, and 6W-3.⁽³⁶⁾ The chemical characteristics of the raw water will often adversely affect color coagulation. The water may prove difficult to coagulate as it is quite cool, soft and will have a minimum of turbidity present.

Other forms of color removal besides coagulation have been tried with varying degrees of success. One of the more successful methods has been to heavily chlorinate (often break point chlorinate) the colored water. The City of Miami and Fort Lauderdale have successfully treated water having 55 and 80 color units respectively by chlorinating with about 8 mg/l dosages,⁽¹⁴⁾ Chlorination followed by activated

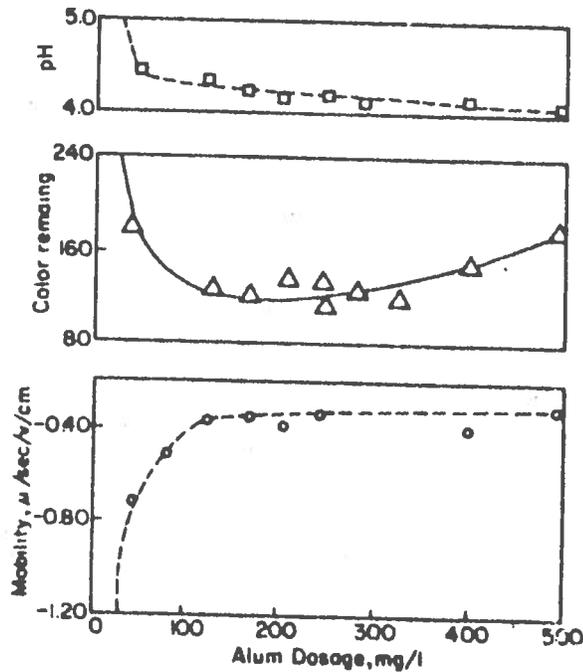


Fig. 6W-1

Effect of Alum Dosage on Coagulation of Colored Water. Curves Show the Effect of Alum Dosage on Floc Mobility, Color Removal, and pH. Color Removal was Poor with All Dosages, the Best Sample Showing a Residual Color of 120 Units.

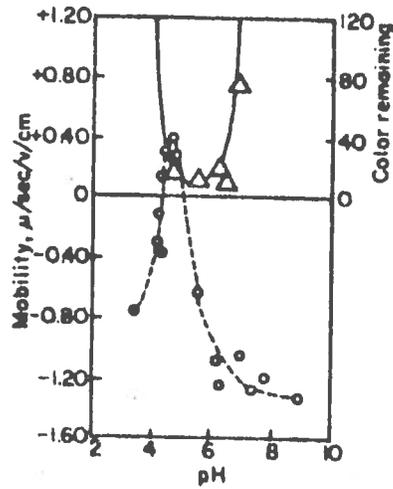


Fig. 6W-2

Effect of pH on Coagulation of Colored Water with Constant Alum Dosage. A Constant Alum Dosage of 120 mg/l was Used, and pH was Adjusted with $\text{Ca}(\text{OH})_2$. Excellent color Removal was Obtained within a Narrow pH zone of 5.0-5.7. Particle Charge First Decreased, then Reversed Almost Exactly at the Beginning of the pH Zone of Optimum Color Removal.

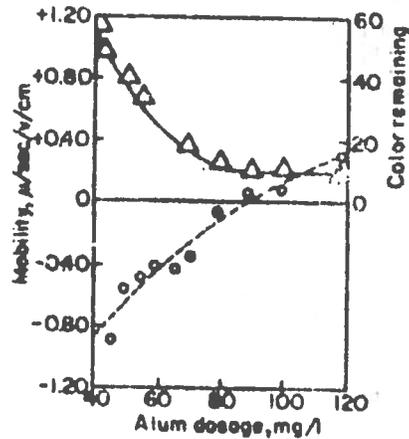


Fig. 6W-3

Effect of Alum Dosage on Coagulation of Colored Water at Constant pH. The pH was Maintained Constant at 4.85 with Addition of HCl or $\text{Ca}(\text{OH})_2$, While Alum Dosage was Varied. Color Reduction was Increased as Alum Dosage was Increased. Particle Charge Reversal Occurred Almost Exactly at the Alum Dosage for Optimum Color Removal.

carbon filtration was tested in June, 1972 in the Yellowstone National Park Study with a high degree of success on Old Faithful and Canyon water supplies. The activated carbon absorbed most of the color from the raw water sources and further removed the chlorine tastes. However, unfortunately, super chlorination followed by A. C. Filtration was not very effective in oxidizing color in the raw waters tested for Saratoga. Table C-4 in the appendix lists the results of the chlorination-activated carbon filtration tests performed in the Spring of '77. These results and the costs in Tables 5W.-1 and 5W.-2 led us to search for an alternate treatment solution for the color.

Ozone (O_3) as well as being an excellent disinfectant as mentioned previously, has been documented as an excellent color removing agent. Weber states that, "In general, ozone is found useful in the following areas of water treatment:

1. color removal
2. taste and odor removal
3. disinfection
4. iron and mangangese removal
5. phenol, oxidation
6. cyanide oxidation"

Many other researchers have found that ozone can be very effective in color removal. (8), (15), (16), (18)

The solubility of ozone in water depends mainly on the water temperature, and the partial pressure of O_3 in the gas phase. Solubility factors for different temperatures are given in Table 6W.-4. It should be pointed out that the average water temperature during the color periods is less than $10^{\circ}C$; and that at these temperatures, ozone is the most soluble.

TABLE 6W.- 4
STABILITY OF OZONE IN WATER (16)

<u>Temperature</u> <u>(C°)</u>	<u>Bunsen</u> <u>Coefficient</u>	<u>Henry</u> <u>Coefficient</u>
0	0.49	3.95
5	0.44	3.55
10	0.375	3.0
20	0.285	2.29
30	0.2	1.61
40	0.145	1.17
50	0.105	0.85

A typical ozone system is shown in Fig. 5W.-1. As depicted, the initial capital costs for an ozonation system are disadvantageous. The high costs are in the generator, dissolution equipment, destruct system, underground contact chambers and operating costs and must be considered.

On June 2, 1977 a sample of raw water from the North Platte behind the existing water treatment plant was taken to the University of Colorado at Boulder, to determine the ability of ozone to oxidize the color from the water. The facility in Boulder is one of the few in the region that has a bench scale, or laboratory ozonator, that was immediately available for our use. The results from that study serve as the basis for the design we recommend and show that 99% color removal and total organic carbon (T.O.C.) reduction is achievable by dosing the water with an equivalent of 10 p.p.m. ozone concentration for a period of 2 minutes, see Appendix Section C.

The study involved ozone dosing of raw water and filtered water. Raw water with a color level of 60 J.T.U. and T.O.C. level of 10 p.p.m. was used for testing. The raw water was ozonated with 10 p.p.m. for 2 minutes, and resulted in a reduction of color to 25 and T.O.C. of 6.3 p.p.m., but the color remaining was still yellow in appearance. It appeared that the turbidity was interfering with the ozonation process.

A lab scale Diatomaceous Earth Filter (D.E.) was used to filter the interfering turbidity from the colored water. The D.E. filter alone reduced the color to 20 J.T.U and a T.O.C. level below 4 p.p.m. The filtered sample was then ozonated with 10 p.p.m. ozone for 2 minutes, and the resulting color was less than 5 J.T.U. and T.O.C. less than 4 p.p.m. The treated water was not differentiable from distilled water with the naked eye. Results are summarized in the Appendix.

The conclusion drawn from the study and the recommendation of this report is that the problem of color removal can be eliminated by fine filtration followed by ozonation. We recommend dose rates of 2-5 p.p.m. of ozone and contact times of 8-10 minutes, through a water column depth of 15-18 feet. Phase I of construction should include construction of a detention system large enough to treat 6.5 M.G.D. at a detention time of 8-10 minutes since the underground work is very costly to perform at a later date. This structure, however, will allow for a detention period of 15-20 minutes until the Town expands it's system from Phase I.

Turbidity Removal

Turbidity in surface waters consists largely of clays and other mineral particles. The size range of most of these particles is 0.2 to 5.0 μ . which places them in the upper size range of true colloids and in a size range which would settle under gravity in sufficient time.⁽¹⁵⁾ Coagulation of such suspensions in contrast to color is accomplished relatively easily, once the proper pH and chemical dosages are determined. In general, coagulation of dilute clay suspensions is controlled more by the coagulants and the chemical composition of the water than by the nature of the suspended solids. Figures 6W.-5 and 6W.-6 show the effect of pH and strength of coagulant on turbidity removal.⁽³⁷⁾ However, the river supply should not require major turbidity removal as a backwashable infiltration gallery is recommended.

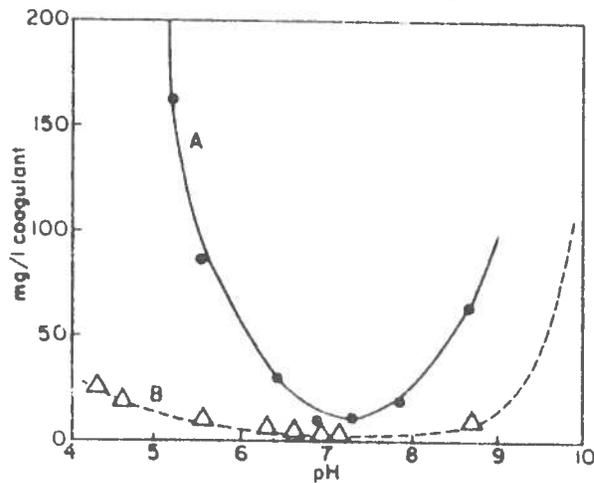


Fig. 6W-5 Coagulation of 50mg/l Kaolin with Aluminum Sulfate and Ferric Sulfate. Comparison of pH Zones of Coagulation of Clay Turbidity by Aluminum Sulfate, Curve A, and Ferric Sulfate, Curve B. Points on the Curves Represent the Coagulant Dosage Required To Reduce Clay Turbidity to One-Half its Original Value. (27)

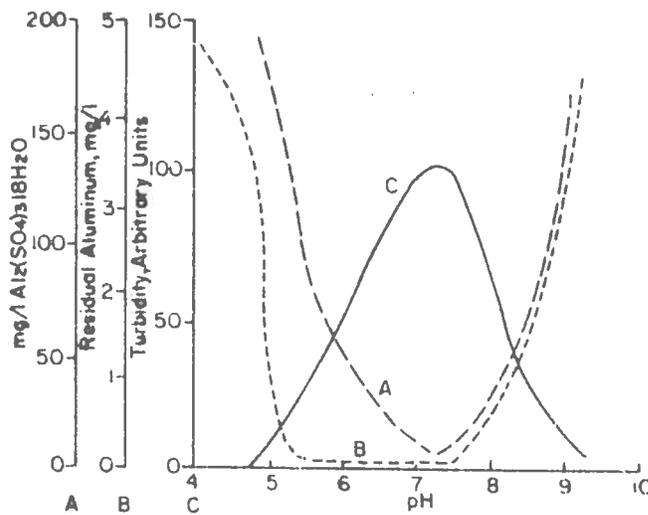


FIG. 6W-6 Comparison of the Effect of pH on: Curve A, Dose of Aluminum Sulfate Required to Halve Turbidity of 50mg/l Clay Suspension; Curve B, Aluminum Solubility as Indicated by Residual Aluminum After Coagulation with 200 mg/l $Al_2(SO_4)_3 \cdot 18 H_2O$. Curve C, Flocculation of $Al(OH)_3$ as Indicated by the Turbidity 1 Minute After Mixing the Reagents. (27)

CHAPTER 7W. - FILTRATION

Table 7W.-1 presents dual media, mixed media, high rate sand, diatomaceous earth and rapid sand filters, and some physical characteristics of each type of filter. Although all of these filter systems were examined as potential systems for use, we recommend the use of D.E. filtration, as will be discussed further on.

Dual and Mixed Media Filters

Conventional sand filters are of two types, rapid sand, and dual media. Dual media filters are normally composed of two layers of filter media, anthracite and sand. The normal operating flow rate expected through a dual media filter is 2-4 g.p.m./ft². For a town of 10,000 people, a dual media filter system alone would require 2250 square feet or 4 filter beds of approximately 576 square feet. The problem with using dual media filters is that normally most of the actual filtration is accomplished in the top 3-6 inches of the filter bed and this necessitates more frequent backwashing than if the entire bed depth were utilized.

Mixed media filters, are composed of 3 different layers of graded filter media. These layers are a porous top layer of anthracite coal, a middle more dense less porous layer of sand and a bottom very dense layer of garnet. Gradation of this bed not only allows more efficient use of the entire filter bed depth, it also allows filtration rates of 6-10 g.p.m./ft², a much more economical system. The mixed media system is a system which has been patented by Neptune Micro Floc, and is sometimes referred to as a "Micro Floc Filter." It has both the advantage of lengthening filter runs while operating at higher flow rates, and conserving space in a design. Using a flow rate of 6 g.p.m./ft², the filter bed size for a mixed media system would be 750 square feet for a population of 10,000.

It should be pointed out that neither dual or mixed media filters have proven effective in bacteria or virus removal.

Diatomaceous Earth Filters

The filter medium used in diatomite filters is manufactured from diatomaceous earth deposits which consist of siliceous skeletal remains of diatoms. Diatoms are algae, single celled plants, found in both

fresh and marine waters. Figure 7W.-1 shows the skeleton of a typical diatom, and how their angular structure lend themselves as excellent filter material. The diatom skeleton is almost 100% silica. Diatoms are in a size ranging from 5 to 50_u but more commonly 10-20_u. The particle size of diatoms assures that when operated properly, the final turbidity of the filtered water should be less than 1.0 J.T.U.

D.E. filters have been in operation for many decades. The majority of their modern day applications have been for water purification in the brewing and canning industries and swimming pools. Only recently have they become attractive for municipal use. Their use is recommended where there are smaller space requirements, lower capital investment and decreased need for extensive pretreatment. Commercial D.E. costs about 10 to 12 cents per pound and characteristics of some types of filter media are shown in Table 7W.-2. The filters may be pressure or vacuum type, with normal flow rates of 1.0 g.p.m./ft² and 0.5 g.p.m./ft² respectively. (8)

The filter cycle consists of four steps: (1) precoat application; (2), (3) filtration of water usually accompanied by the application of body feed; and (4) the removal of the spent filter cake. Figure 7W.-2 shows three of the four steps involved in D.E. filtration. The fourth step is backwashing.

Precoating consists of the application of a thin, 1/8 inch thick layer of D.E. to the septum to form the filter medium. The septum material varies from manufacturer but is generally a rigid or semi-rigid, stainless steel mesh, or a plastic mesh material covered by a cloth "sock". About 0.1 to 0.2 lb. of D.E. per square foot of septum is used to give a 1/8 inch coating and cycled through the septum until all of it is deposited as a cake.

During filtration, the suspended solids are removed on the precoat surface resulting in increased pressure drop across the filter. Due to hydraulic compression of the solids on the precoat cake, the filter cycles may be very short unless additional D.E. (body feed) is added during the filtration period. Use of body feed in a filter cake of solids mixed with D.E. filter aid provides a more porous cake and results in longer filter cycles.

Removal of the spent filter cake can be accomplished either by compressed air "bumping" or backflushing with water. The filter cake is knocked off of the septum and drained away to waste. However, the D.E. material is inert, and will not combine with other materials, and recent innovation now makes it possible to reuse the filter cake instead of wasting it. The cake together with the impurities are bumped off the septum, broken up in the bottom of the pressure vessel, and then redeposited. All of the cake and impurities are cycled through the system and redeposited on the cake, as in the precoat process. However, small impurities passing through the septum are caught by the precoat on its next cycle through the septum and this continues until all of the impurities are redeposited with the D.E. in the precoat cake. This regenerative ability makes for a very inexpensive operation since the same D.E. may be used over and over again, before being wasted. It also allows for a shorter down time than that required by a complete backwash cycle. Experience with the ten year old D.E. filtration system on Lookout Mountain above Golden, Colorado, has shown that filter runs as long as 3 months without backwashing are common, and that no body feed system is required during this period. The raw water is of similar quality as the raw river water in Saratoga, and the turbidity of the filtered water is less than 0.2 J.T.U.

When the filter runs become progressively shorter, and the regeneration process becomes inefficient, the D.E. is wasted, and a new coating of D.E. is deposited in the precoat cycle. No spent filter cake should be discharged into sanitary or storm sewers. A separate lagoon or filter should be provided for spent filter cake dewatering. Ultimate filter cake disposal should be to a sanitary landfill.

In addition to being a very fine polishing filter, the D.E. filter is the only conventional filter system capable of removing 99% of bacteria, and most virus. Early U.S. Public Health Service Bulletins⁽³¹⁾ established the relationship of various treatment processes to coliform removal. These studies showed that slow sand filters would reduce coliform counts to between 30 and 40 per 100 ml when the raw water

count was 5000. The same studies showed that with a raw coliform loading of 1000 the filter effluent would approximate 10. Total treatment including coagulation and settling would result in effluent coliform loadings of between 20 and 30 and 6 respectively when using the same raw water loading. Recent studies^(19,38) show that D.E. filters can give "complete" coliform removals up to influent coliform levels of several thousand per 100 ml and further that the D.E. filter has excellent results in virus removal. By using D.E. filtration, ozonation with 10 minute contact time, and residual chlorination, all residual bacteria and virus should be eliminated, from the finished water.

As discovered in the study at Boulder, D.E. filters are ineffective in removing the color causing compounds found in the Saratoga raw water. However, taste and odor compounds, and any other compounds capable of removal via carbon adsorption are removeable with D.E. filters. It is possible to premix powdered activated carbon into the precoat solution⁽¹⁵⁾ and add this to the filter septum during the precoating cycle. Activated carbon will adsorb many of the organic compounds that may cause taste and odor, and some color causing compounds.

From a maintenance point of view, the D.E. filter system can be completely automated. When the raw water quality changes, the only thing affected are the length of the filter runs. No jar testing or chemical manipulation is required, making for less operator input.

Recommendation

It is recommended that D.E. filtration shall be used for turbidity removal, fine polishing, and some color removal in conjunction with the backwashable infiltration gallery. Ozonation shall follow filtration primarily for color, taste and odor, and iron and manganese removal, and secondarily for disinfection. Chlorination shall follow ozonation so that a residual chlorine level can be measured and maintained in the system. It is our conclusion, that if this water treatment

system be followed and operated as proposed, the finished water quality will exceed all the new national primary and secondary drinking water standards and be as fine of quality of water produceable with the technology available today.

TABLE 7W-1 - FILTERS

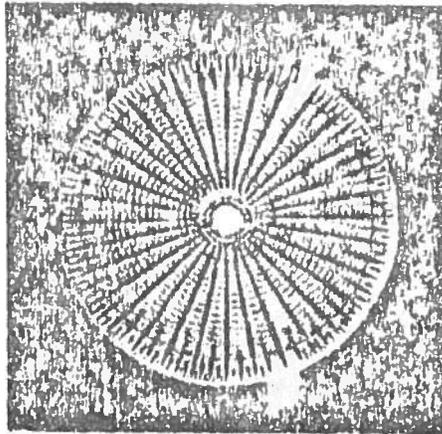
FILTER TYPE	GRAVITY OR PRESSURE	MEDIA **	PARTICLE SIZE **	FLOW RATE GPM/ft ²
1 Dual Media	Normally gravity	Anthracite Sand	microns-u 800-1200 300	2 - 4
2. Mixed Media	Normally Gravity	Anthracite Sand Garnet	800-1200 300 100	6 - 10
3. Diatomaceous Earth	Pressure Or Vacuum	Diatoms with Septum	10-20	0.5 - 1
4. Rapid Sand	Normally Gravity	Sand Gravel	300	2

* Micro Floc Patent

** Media and Particle size are examples only - and can be varied

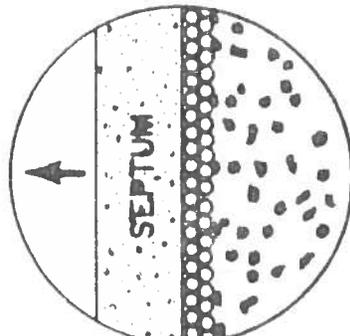
TABLE 7W-2 -TYPICAL VALUES FOR
 VARIOUS COMMERCIAL GRADES OF
 FILTER MEDIA⁸

Product	Mean particle size, microns
Johns-Manville	
Celite 560	50
Celite 545	21.0
Celite 535	16.2
Celite 503	12.8
Hyllo Super-Cel	9.5
Great Lakes Carbon	
Dicalite 5000	21.8
Dicalite 4200	19.3
Speedex	13.0
Speedplus	10.4
Eagle Picher	
FW-80	32.0
FW-50	19.3
FW-20	17.0
FW-14	12.0



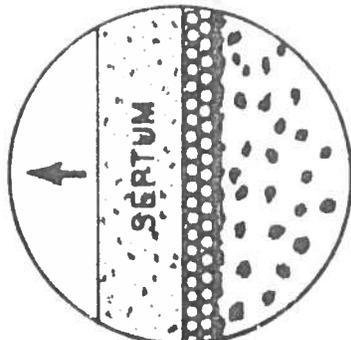
Microscopic enlargement of a typical diatom structure

Fig. 7W-1 (41)



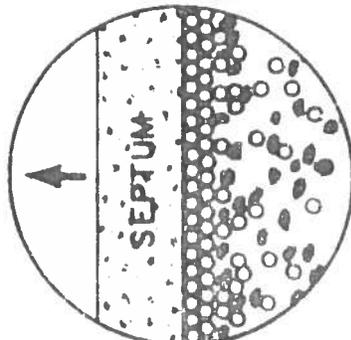
STEP 1

**Diatomite Precoat on Septum
(Normal Flow.)**



STEP 2

**Filtered Solids Clogging Precoat
(Reduced Flow.)**



STEP 3

**Cake Kept Open by Continuous
Feed. (Normal Flow)**

FIG. 7W-2 DIATOMACEOUS EARTH FILTER

P A R T I I
WATER-DETAILED ANALYSIS,
EVALUATION, AND PROPOSED CONSTRUCTION

CHAPTER 8W. - GENERAL INTRODUCTION

EXISTING WATER SYSTEM

Scope

Chapters 8W. - 13W. will describe and evaluate the existing water treatment and supply system, and make firm recommendations for improvements. Some detail covered elsewhere in the report will not be restated. Some general statements are applicable and are stressed here:

1. A full time licensed operator should be employed who will be responsible and competent in maintaining both the existing and proposed treatment plants in maximum operating efficiency. This point cannot be stressed enough as will be seen later.
2. The distribution system should be metered as this will cut water useage by at least 33 per cent and allow the Town to reach a population of 5000 before expanding the proposed facilities.
3. The Town must adopt codes, and standards to be followed for construction of any water improvements, and these codes must be enforced. Any plans for construction should be approved by the Wyoming Department of Environmental Quality and a permit to construct issued prior to the Towns review for construction permits. All new underground construction should have qualified full time inspection and proper testing.
4. The system proposed hereinafter for construction is for a population of 5000, and will be recommended as Phase I of construction. Phase II of construction will be initiated when the Town reaches 5000 people and will accommodate a population of 10,000. The proposed improvements are designed for 5000, and made easily expandable for Phase II.

Existing Treatment System

The existing treatment system was designed by Mr. Harold Hoskins of Omaha, Nebraska, and built in 1957. The plant is an Infilco-Accelator type of facility. The existing plant is shown from the outside in Fig. 8W.-1. Treatment involves a surface type river intake, chlorination, coagulation, flocculation and sedimentation through an upflow

clarifier, sand filtration and a clear well storage area. The controls for pumping and recording of tank levels are also located in the treatment plant. A schematic of the existing facility is shown in Fig. 8W.-2. Details of each operation will be covered in the chapters that follow.

The capacity of the existing facility is 0.82 M.G.D. when operated and maintained in maximum operating efficiency. Using the peak day demand figure of 650 g.p.c.p.d. as determined in Chapter 2W, the equivalent population capacity of the treatment plant is 1258 people. It has long been known and documented that the plant has not been able to meet summer demands, and this is the reason why. The design capacity for the system is about 1250 people and when this is combined with the losses through leakage, poor maintenance, etc., the actual supply capacity of the plant is more likely 1200 people.



FIG. 8W-1 Western Exposure of
the Existing Water
Treatment Plant

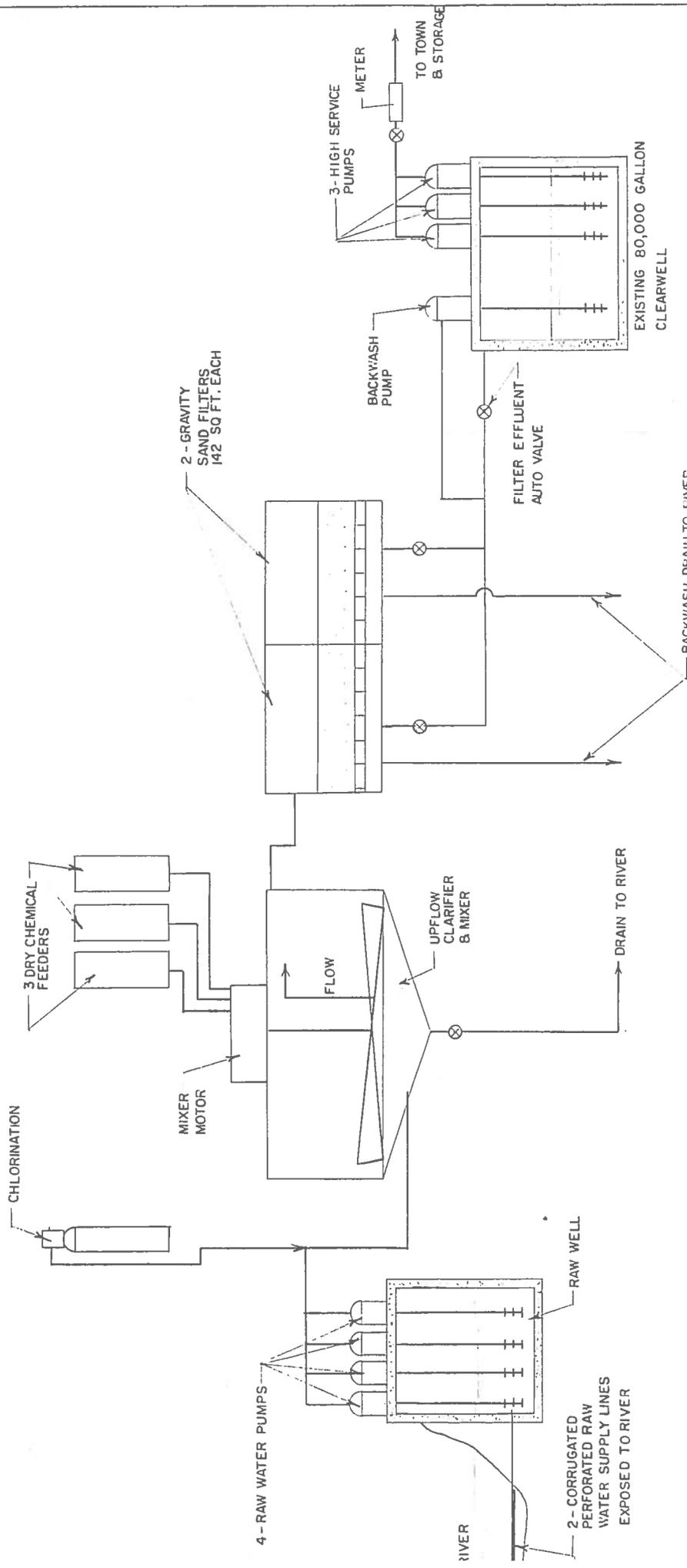


Fig 8W-2
 SCHEMATIC - EXISTING TREATMENT
 PLANT

CHAPTER 9W.-COLLECTION

Scope

The scope of this chapter is to examine and evaluate the existing water collection system and make recommendations for improving it.

Description of Existing Collection System

The present supply system prior to construction of the new intake system was built in 1972 and is composed of two 60 feet long, 20 inch diameter perforated corrugated metal pipes lying partly buried under a small diverted channel of the North Platte River, see also Fig. 8W-2. These two pipes gather raw water directly from the river and it flows by gravity through a 12 inch diameter collector line into the raw water well in the treatment plant. The quality of the water varies seasonally as has been discussed previously.

Evaluation

The intake pipes have 3/8 inch diameter perforations and are open on the ends. They were originally buried under the channel bed but through time, the cover over the pipes has been washed away, partly exposing the pipes.

The exposed open ends and perforations present a serious problem by allowing any large object to enter the intake lines, and make its way into the raw well. Sand, rock, sticks, rags, fish and even muskrats have been found in the raw well. These objects seriously damage the raw water pumps that push the water through the treatment processes. In addition, this type of intake provides no pre-treatment function at all.

The system as it exists is subject to plugging and increasingly reduced capacity. There is no way to backwash or clean the system, and the damage and maintenance costs will continue if this system is not improved.

Recommended Construction

The recommended construction is schematically shown in Fig. 9W.-1. The system recommended is a backwashable infiltration gallery and is designed to supply 2400 g.p.m. as discussed in Chapter 4W. and shown in Fig. 4W-1. Two 100 feet long stainless steel well screens will be placed 4 feet deep under the river bottom and two more

pipes stubbed out for construction Phase II. The slot openings of the screen should be non-clogging, and sized to keep out sand and particles larger than .02 inch dia.

The sand on top of the well screens shall be graded similar to a rapid sand filter. The entire structure and river bank shall be riprapped with large rock to prevent erosion.

The concrete raw water well structure shall be designed with adequate piping, storage and pumping controls to provide for backwashing the system semiautomatically. Backwash water storage shall be at least 36,000 gallons. Chlorinated water cannot be used for backwash water, as it may adversely interfere with the aquatic life of the river. This structure shall be put into use as soon as possible to supply the existing system, and the existing system can be used as a backup intake system.

As mentioned before, the Town should obtain a surface water right for a volume of 8.70 C.F.S. in addition to the 2 C.F.S. already appropriated.

Features of this plan include:

1. Construction of an adequate raw water supply that is back-washable. It will clog, but if routinely backwashed, it will provide adequate flows for a population of 5000.
2. It is designed to resist erosion from the rapid spring runoffs and ice flows which occur in the late winter months.
3. The infiltration gallery will provide a good quality of raw water for treatment, as most of the turbidity will be removed in the graded sand. No sticks, rocks or animals will have access to the raw water well or pumps.
4. By constructing a new intake system, the old system can be used as a standby thus giving the system two sources of supply.
5. Estimated cost of construction is \$275,000.

Since this report is being published after construction was completed by Teton Construction Company of Cheyenne, Wy., in 1977 on the intake structure, some photographs are included for future reference.

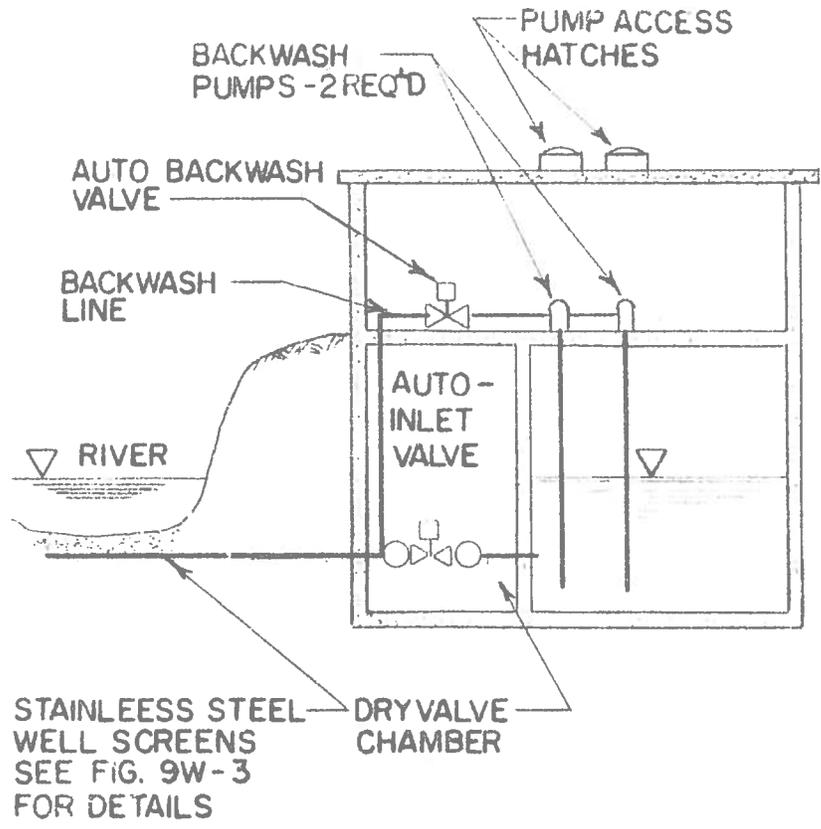


FIG. 9W-1
 SCHEMATIC - PROPOSED
 INTAKE SYSTEM

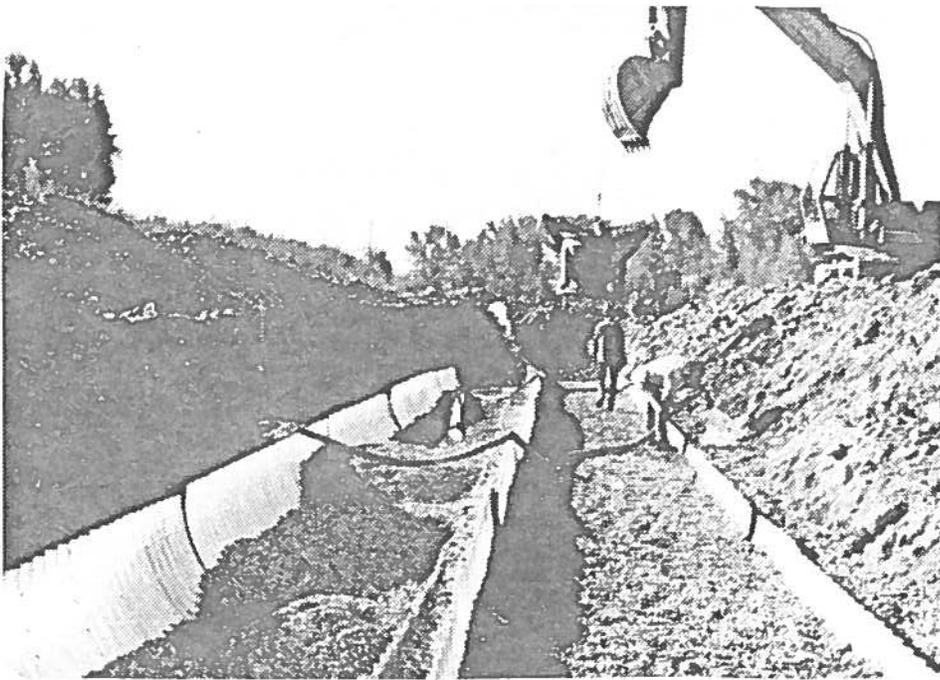


Fig. 9W-2

Intake Collector Lines
Joined to Well
Screens-1977



Fig. 9W-3

Graded Gravel Being
Placed and Bedded in
Corrugated Aluminum
Pipe-1977

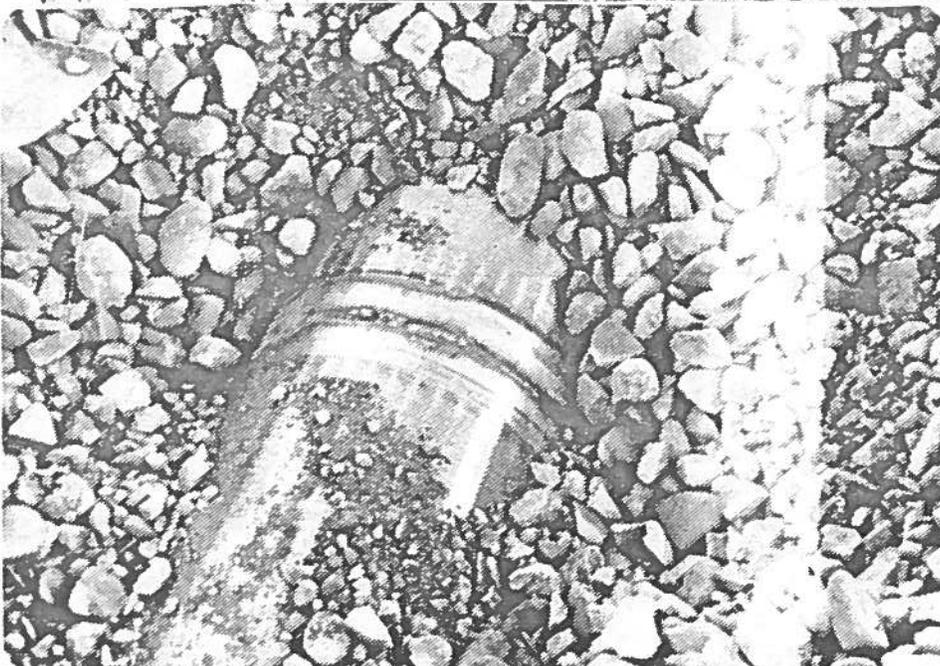


Fig. 9W-4

Close-up view of 10"
Dia. Stainless Steel
Screen Joined by
Stainless Steel Weld-
1977

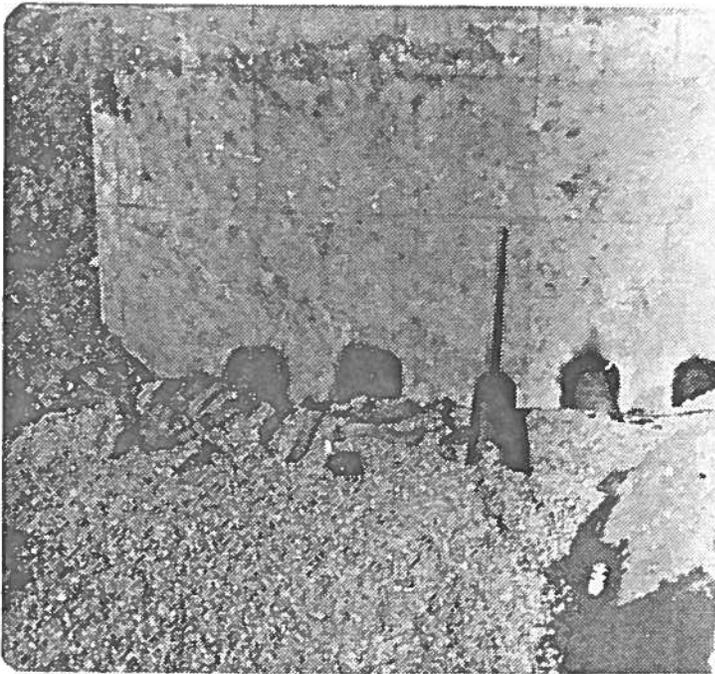


Fig. 9W-5

Two 12 Inch Diameter
M.J.D.I.P. Collector
Lines Entering Dry Valve
Chamber on Left Side;
the 12 Inch Diameter
Water Line Crossing the
River in the Middle, and
two 12 Inch Lines
Stubbed Out of Chamber
for Future Expansion-1977

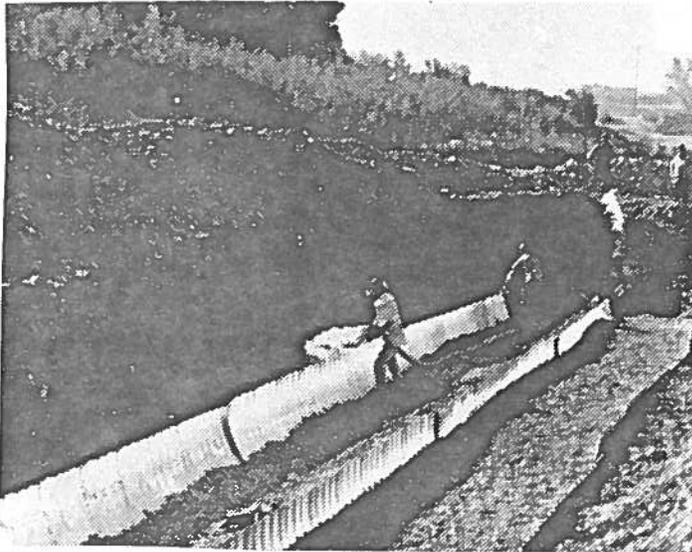


Fig. 9W-6

Comparative Location of
Intake Lines and Depth
Below River Bottom-1977



Fig. 9W-7

12 Inch Deep Riprap Being
Placed Over Intake
Screens-1977