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THESIS

A SYSTEMATIC APPROACH TO THE WATER SUPPLY OF A LARGE URBAN AREA

Submitted by

Alain Dérédec

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May 1972

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ABSTRACT

This study analyzes the water supply of a large urban area. The systematic aspect of water supply is stressed, along with the concept of water reuse.

The general water supply system is decomposed into a certain number of subsystems (primary supplies, secondary supplies, desalting) for which alternatives are proposed. The analysis of the physical feasibility of these alternatives is associated with the presentation, for each of them, of a cost estimation model. Particular attention has been devoted to the reliability of these cost models.

This study is considered to be the first and most important step in the process of building a computer program which would formulate and then optimize as a whole the water supply of any large urban area. In the hands of experienced planning engineers, such a program is believed to be a powerful working tool.

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INTRODUCTION

Although they may not yet be faced with the possibility of immediate water supply shortages, large urban areas feel, nevertheless, the necessity to take a long range view on the question of future water resource development. It is the purpose of this paper to conduct a comprehensive review and evaluation of the sources of water supply, both present and potential, and to determine the source development and operating practices which would best meet the growing requirements.

The complex inter-relationship among the many hydrologic, hydraulic, legal and operational parameters involved in the study indicate the desirability of a systematic approach to the analysis of the many alternatives. This approach is particularly appropriate if one wants to consider not only the traditional sources of water (reservoirs, streams, aquifers), but also a more recent and more sophisticated type of water supply, namely the reuse of waste-waters.

Respectively in the first and second part of this paper, it is determined in the most general case of a large urban area, HOW and AT WHAT COST the quality and quantity requirements of each use sector can be satisfied, knowing the qualitative and quantitative characteristics of each potential supply.

The third part of the study will consider a particular water system, that of the city of Fort Collins. How it fits in the general models derived previously, and how these models can be used for its future development, are emphasized.

The purpose of the fourth part of this study is merely to give to the user of the present work a feeling of security in using the cost

models developed in part II. Field data from two test regions, namely California and the Denver area, are compared to the values computed from the models.

Overall, this study is considered as an effort to bring together in one place most of the useful information now available, for the purpose of producing a working document useful to planners and municipal officials charged with the management and development of water resouces.

PART I: ANALYSIS OF THE GENERAL SYSTEM

A. The General Structure of the System

A-a. The Flow Chart. The most general water supply system of a large urban area is articulated in a flow chart fashion in Figure 1. Also an enlarged version of Figure 1 is available at the end of this paper, and should be kept open for a better understanding of the following analysis.

The different sector uses are represented in the lower right corner of Fig. 1 by heavy-lined boxes. The water sources are of two categories: the primary sources represented by heavy-lined boxes at the upper left part, and the secondary sources, i.e., the waste-waters, represented by heavy lines leaving the municipal, industrial and agricultural sector uses. The rest of the flow chart describes all the possible "reasonable" alternatives required to make the water of all the sources suitable to one or several uses.

A-b. The Water Demands. The water demands and their characteristics are shown on Table 1.

There are four general sectors demanding water: the municipal, industrial, recreational, and agricultural sectors. Also because the system outflow (generally dumped in a nearby running stream) is constrained both in quantity (downstream water rights, minimum low flow in streams for ecological purposes, etc...) and in quality (the bill, voted by the Senate on November 4, 1971 on pollution control, stipulates that all the water pollution problems will have to be solved by the end of 1984), it can be considered as a water demand to be analyzed in the same way as the above four demands.

Because they are the ones that are generally considered in the planning of water treatment facilities ([9], [18], [21]), each demand

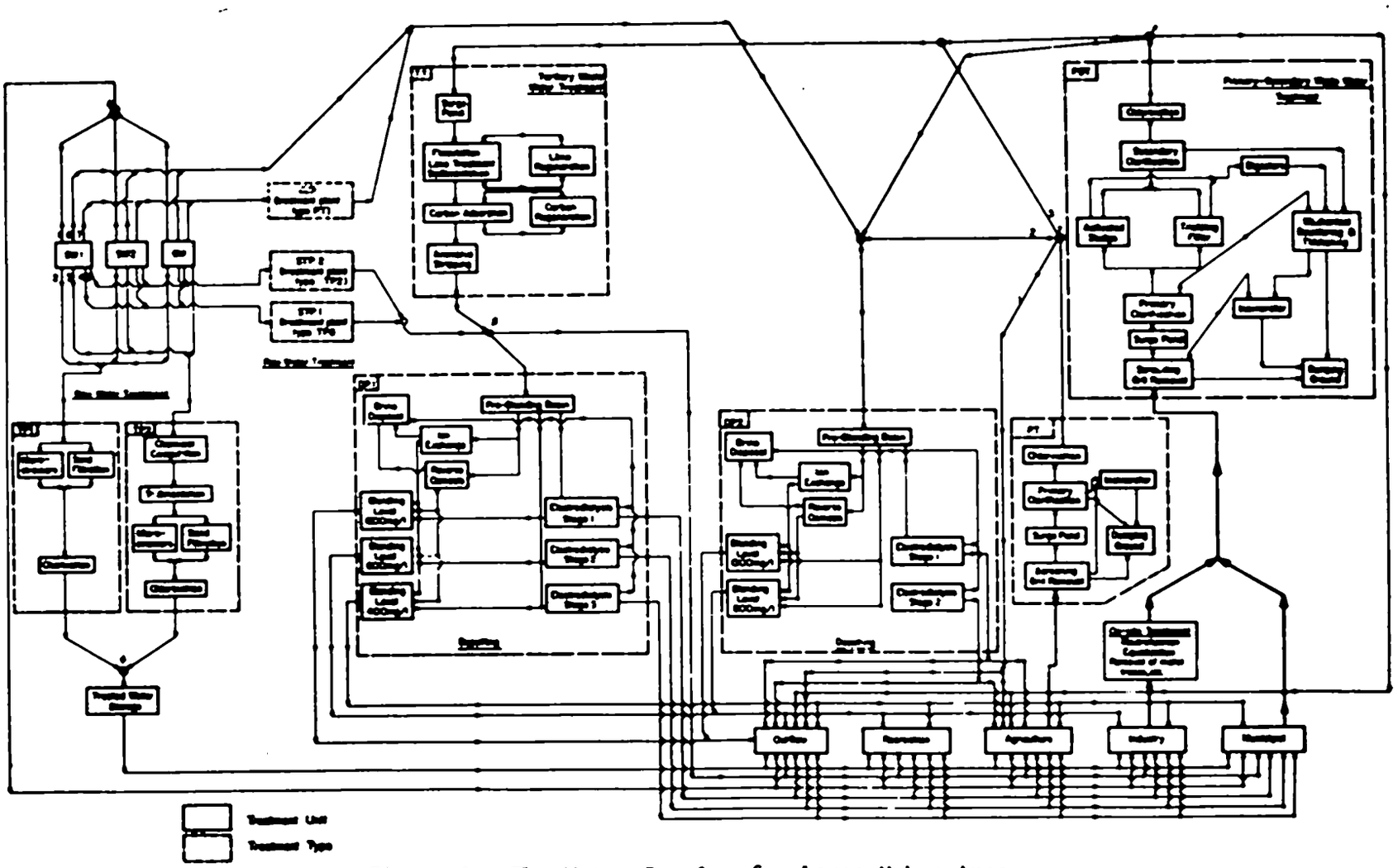


Figure 1. The Water Supply of a Large Urban Area
General Diagram

Table 1: The water demands

Type	Quality Requirements	Quantity Requirements
(1)	(2)	(3)
Municipal	MCOL MBOD MSS MTDS	MC(i)
Industrial	ICOL IBOD ISS ITDS	ID(i)
Recreation	RCOL RBOD RSS RTDS	RD(i)
Agricultural	ACOL ABOD ASS ATDS	AD(i)
System Outflow	OCOL OBOD OSS OTDS	OD(i)

is characterized by the following quality standards: the coliform count (COL), the biological oxygen demand (BOD), the weight of suspended solids (SS) and the weight of dissolved solids (TDS) (see column 2, Table 1). If some more specific criteria (for example, the concentration of sodium [Na+], or of some metal trace) would appear to be needed in some case-studies, one would just have to increase the dimension of the "quality-vector" for the corresponding use-sectors. The general understanding of this study would not be affected. It should be emphasized here that the quality-standards must not be considered as static parameters over time; they are subject to revision

as technical data, surveillance programs and technological advances make such revision desirable.

Column 3 of Table 1 shows the total quantity of water required by each sector per year. The symbols MD, ID, RD, AD, OD actually mean that, for each use, there exists a known projected demand curve (amount of water needed vs time, in years) over the study period (Figure 2). The projected demand curves are generally available from the planning office for the urban area.

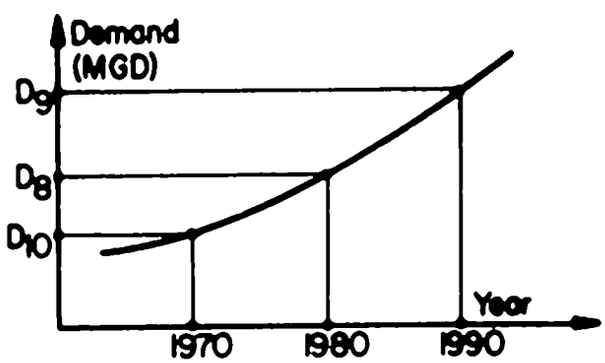


Figure 2. Demand vs. Time

A-c. The Water Supply Sources. In column 1, (of Table 2), the different types of water sources are enumerated. Because their quality characteristics and their availability are of completely different nature, the primary supplies are separated from the secondary supplies.

The primary supplies, i.e., the raw water sources, can be split into the four following categories:

-- the reservoir-type, called here Surface Water Type 1.

-- the stream-type, called here Surface Water Type 2.

-- the groundwater-type.

-- the sea-water type (or a large salted lake).

However, in this study this last type of water supply is ignored.

Table 2: The water supplies

Type	Quality	Quantity Available
(1)	(2)	(3)
Primary Supply		
Surface Water Type 1 (Reservoir-type)	COL1i BOD1i SS1i TDS1i	K1i
Surface Water Type 2 (Stream-type)	COL2i BOD2i SS2i TDS2i	K2i
Ground Water	COLGi BODGi SSGi TDSGi	KGi
Secondary Supply		
Municipal Waste Water	COLM BODM SSM TDSM	KM
Industrial Waste Water	COLI BODI SSI TDSI	KI
Agricultural Return Flow	COLA BODA SSA TDSA	KA

The secondary supplies, or waste water sources, can be:

-- the municipal waste waters (from the municipal sewers)

-- the industrial waste waters

-- the agricultural return flow

In order to compare the quality of a source with the quality requirements of the uses, one has to consider, for both sources and

uses, the same set of criteria (namely here: COL, BOD, SS, TDS). This is shown in column 2 of Table 2. Of course, the previous remarks concerning the expansion of the number of criteria needed and the possible change of standards over time, hold here.

In column 3 (of Table 2), symbols K1i, K2i,... (i, counting the number of reservoirs, or the number of streams,...) are the quantities of water available from each source. Each of these symbols is actually a vector with T components, if the study period is T years. As far as the primary supplies are concerned, we do not necessarily know all the components of these vectors. Not only are they of stochastic nature (flow in a stream, level of a reservoir, volume of an aquifer), but they depend on the water rights already owned by the city, or to be purchased or transferred. Also, one or several new reservoirs may have to be built in the future, water laws may change, especially ground water laws.

Assuming that consumptive-use ratios γm, γi, γa in municipal, industrial and agricultural sectors are known and assuming that the total supplies always meet the demands, one can consider the availabilities of waste waters as known quantities over time; they are, in year k:

$$KM(k) = (1-\gamma m)MD(k)$$

$$KI(k) = (1-\gamma i)ID(k)$$

$$KA(k) = (1-\gamma a)AD(k)$$

It is now possible to analyze the treatment alternatives offered to the primary supplies first, then to the secondary supplies in order for them to be suitable for one or more uses.

B. The Primary Supplies

Given any one of these primary supplies, say PS , seven possible routes for the water of this source are of consideration depending on its quality. They are numbered by order of increasing complexity, which will generally correspond to an order of increasing cost (see Figure 3).

Route 1. If the raw water from a given source already meets the quality standards of one or more use-sectors, it can be used directly by these sectors. On the flow chart, the "routes 1" for all the primary sources converge to point Θ . Actually, such a point should not exist, because, in general, two independent raw sources do not meet the quality standards of the same uses, and, consequently should not be mixed. For example, if source 1 meets the drinking standards and source 2 only meets the agricultural standards, it does not seem logical to blend them together. Point Θ has been introduced only in order to simplify the flow chart. Only in case of minor quality differences between two or more sources, might this node Θ exist, thus permitting a single distribution line for the water from these sources.

Routes 2 and 3. These routes correspond to the case where, regardless of the other criteria, the TDS of the raw water is less than or equal to MTDS , i.e., the maximum concentration in total dissolved solids allowed for municipal use. The two alternatives to be considered are the following.

Route 2. The raw water only needs to be filtered in order to be consumed by the municipal, industrial and recreational sectors. That may happen in the case of large reservoirs, where often it is the growth

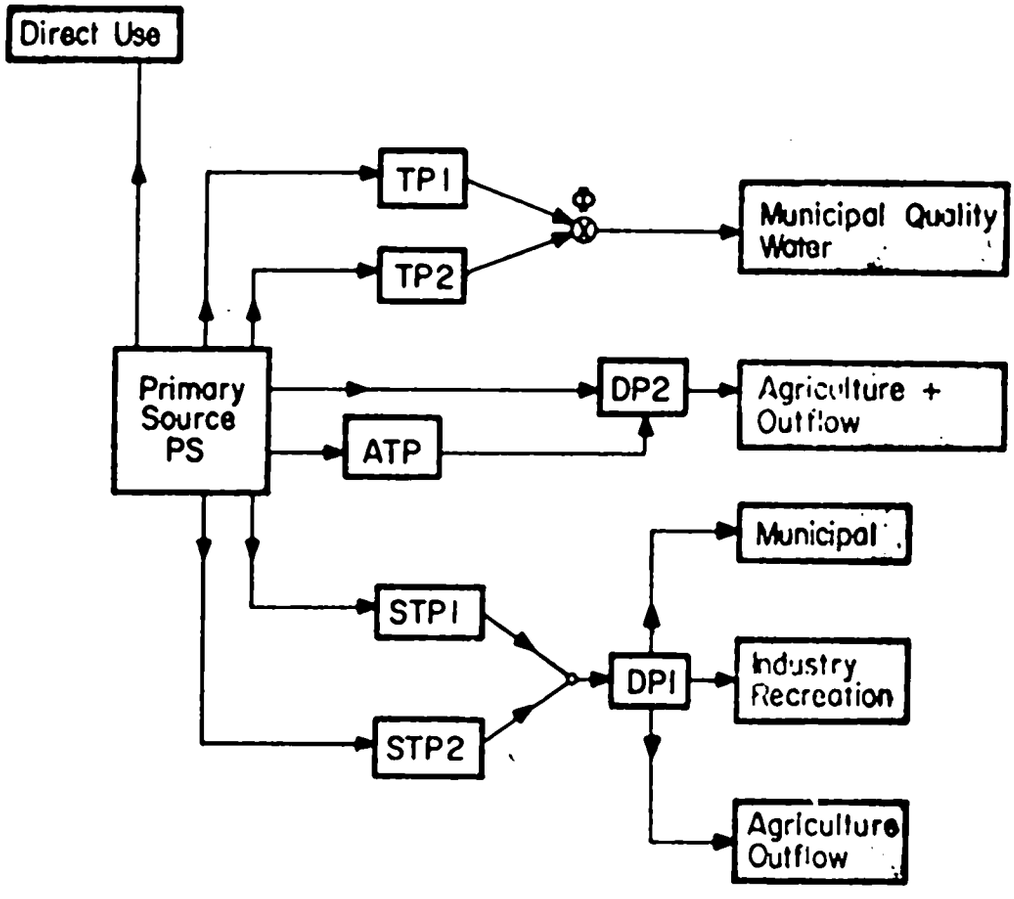


Figure 3. The Primary Sources

of some algae that prevents a direct use. Adequate treatment for such water consists of filtration through sand or micro-strainer (an economic study will make the choice), followed by a chlorination. This treatment is performed in what is called here TP1 (Treatment Plant No. 1).

Route 3. When the source of supply is a stream or an aquifer, the previous treatment is almost never sufficient. The stream water, because of upstream polluters, may contain much BOD , nutrient constituents (particularly phosphates from detergents) and suspended solids. Also the turbulence of the water flow prevents the suspended solids from settling. Ground water, on the other hand, generally needs to be softened, to have its concentration of iron ions lowered and possibly taste and odor removed. The treatment of such waters is best handled in the traditional type of municipal water treatment plant where the following operations are performed: chemical coagulation, sedimentation, filtration through sand or microstrainers, chlorination. This kind of plant is called here TP2 (Treatment Plant Type 2).

Since they are of approximately the same quality, the treated waters from TP1 and TP2 can be collected at point ϕ and distributed through a single line.

Routes 4, 5, 6, 7. These routes are concerned with the case when the primary source PS has a high TDS . In order for the water to be used, TDS must be lowered. Routes 4 and 5 are concerned with the specific case when water is needed only for agricultural or outflow purposes (almost no biological treatment needed; TDS lowered to about 1000 mg/l). Routes 6 and 7 lead the water to higher quality required by the municipal, industrial and recreational uses.

The reasons why one has to consider two desalting plants, DP1 for the municipal, industrial and recreational uses, and DP2 for the agricultural and outflow uses are the following:

1) If the water of PS is to take Routes 4 or 5, that means that it could not take either Route 1 or 3. Accordingly, not only its TDS is high, but generally the values of the other standards also are high. In order to be used in agriculture or to be dumped as system outflow, the raw water does not have to be very refined as far as COL, BOD and SS are concerned. If it were to be desalted in DP1 , it would be mixed with water biologically treated for municipal use, thus ruining the quality of the latter.
2) Also, if the agricultural return flow is to be reused, the necessity for DP2 arises anyway.

Routes 4 and 5. If the values of COL, BOD and SS of the raw water are acceptable by the agricultural and outflow demands, then it is sent to DP2 to be desalted. That is Route 4. If not, a primary treatment (grit removal, primary clarification, chlorination) is performed in ATP , before the desaltation in DP2 takes place. That is Route 5. ATP can then be used both for primary sources and secondary sources (agricultural return flow).

Routes 6 and 7. These routes correspond to the most complex alternatives with high level desaltation and high level biological treatment. The latter, as seen for Routes 2 and 3, can be either of type 1, performed in STP1 (supplementary treatment plant of type 1), or of type 2, performed in STP2 (supplementary treatment plant of type 2). The desaltation will take place in DP1 .

The alternatives related to the primary sources have been considered previously; the secondary sources are now considered.

C. The Secondary Supplies

As stated previously, the secondary supplies are of three types: the municipal sewage, the industrial waste waters and the agricultural return flow.

1. The Agricultural Return Flow [Fig. 4]

Agriculture, including the processing of agricultural products, is the source of a number of water pollutants. Among them are sediment from the erosion of crop land, animal wastes, pesticides, increase in TDS due to percolation through the ground, compounds of phosphorous and nitrogen that originate in commercial fertilizers. In order to meet the quality requirements for system outflow and possibly some low quality agriculture uses, the return flow has to go to a primary treatment plant (PT) consisting of the following operations: screening and grit removal, primary clarification, disinfection by chlorination [2]. If the pollution in animal wastes from feedlots and poultry yards is important, then each feedlot will generally have to treat first its wastes by anaerobic degradation in lagoons [11]. Although this is not mentioned on the flow chart, it may appear more feasible to have several such PT's , depending on the practicality of collecting the agricultural return flows. For instance, given a large urban area, it may be more handy to collect in PT1 the agricultural return flows from the fields north of the town, and in PT2 , those from the south. Since the cost of a primary treatment plant is generally not very high (see Part II of this study), the multiplication of such plants may

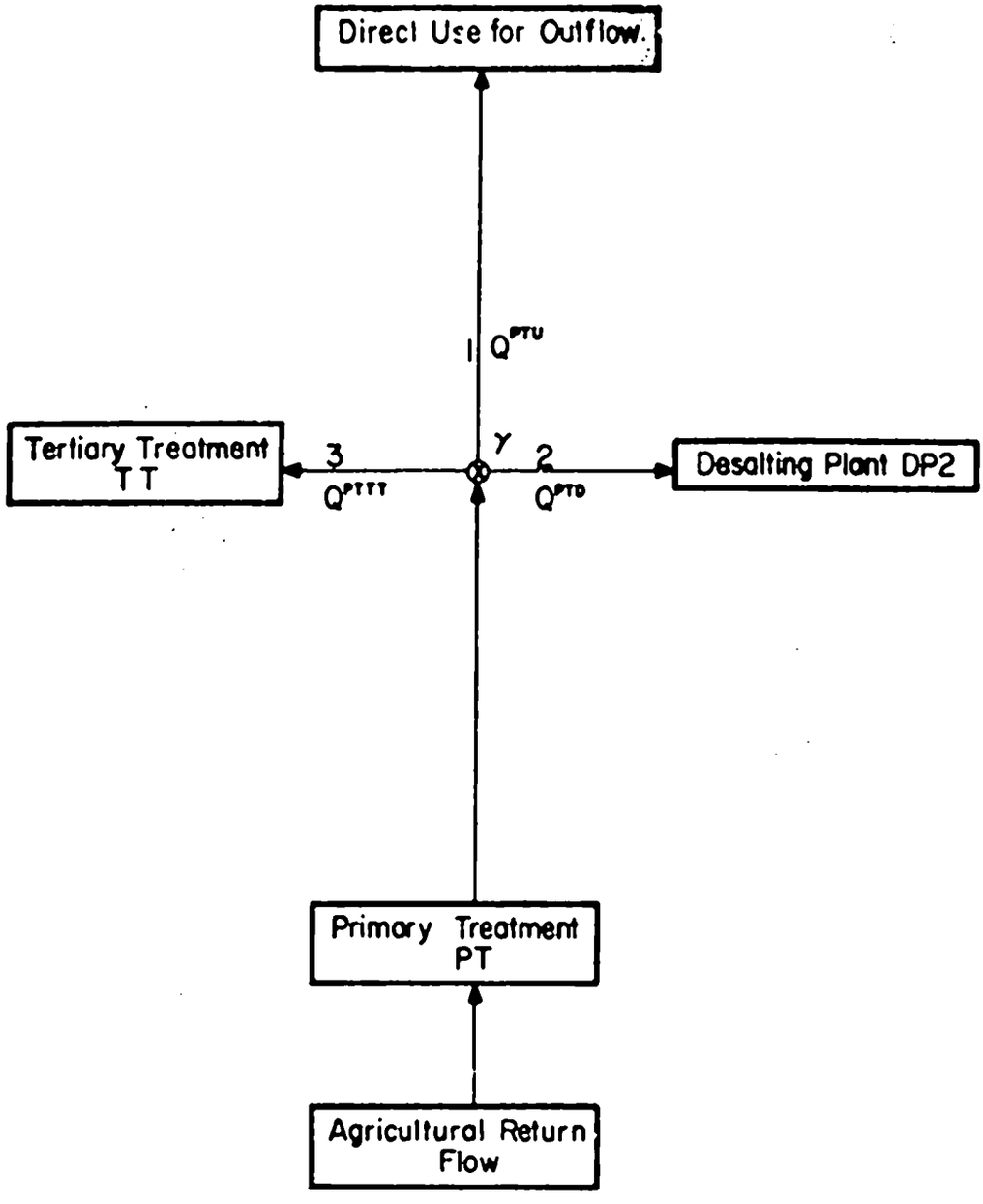


Figure 4. The Reuse of the Agricultural Return Flow

cost less in the end analysis than a sophisticated drainage system which would collect all the agricultural return flows.

From Figure 4 one can see that there are three possible routes for the water at point γ . They are numbered by order of increasing treatment complexity.

Route 1. If the TDS is in the range of 1000 to 1500 mg/l, i.e., the outflow maximum tolerance, then a volume Q^{PTU} can be sent to the outflow and, possibly, to low quality agriculture for a direct use.

Route 2. If the TDS is greater than 1500 mg/l (outflow maximum tolerance), then the water has to be desalted, at least to level 1500 mg/l, if it is to be dumped as outflow; and desalted to level 900 mg/l (standard for agriculture), if it is needed for agriculture. If the water at point γ has its TDS greater than 900 mg/l, and is needed for agriculture, then it has also to be desalted (at least to level 900 mg/l). From the previous paragraph of this study, one knows that a desalting plant (we called it DP1) may have to be built in order to supply municipal water. But because the water at point γ is of a low biological quality, it cannot be sent to DP1 , which only receives waters of high biological quality. Thus, an "agricultural desalting plant," called here DP2 , would have to be built. As previously stated, the salted primary sources to be used in agriculture or outflow could also be desalted in DP2 . From point γ , volume Q^{PTD} is sent to DP2 .

Route 3. If the agricultural return flow is to supply the industrial, recreational and municipal demands, then, from point γ , a volume Q^{PTTT} has to be sent to a tertiary treatment plant, TT , particularly to remove phosphates and nitrates.

2. The Municipal and Industrial Waste Waters [Fig. 5]

In the past, the treatment of municipal and industrial waste waters have generally been dissociated. The municipal sewage was treated in a municipal waste treatment plant, while each industrial plant had to purify its own wastes before dumping them in the nearby river. Though, much of the conventional technology used in municipal waste water treatment is used also to treat industrial waste waters. It is difficult to generalize on treatment of industrial waste waters because the sources are highly diverse and because only sketchy data are readily available on the specific materials that are treated or discharged. As a matter of fact, industrial waste waters are likely to be less amenable to conventional treatment than are the municipal waste waters because they contain substances such as trace metals and chemical compounds that resist biological degradation. But, in order to take advantage of the economies of scale, a phenomenon which is predominant in the planning of waste treatment facilities (see Part II of this study), the most valuable alternative seems to be a joint use of waste water treatment plants by industries and municipalities, with the industries being forced to handle their undegradable wastes by various kinds of on-site treatment [11].

Lawrence K. Cecil (19), gives additional support to this statement: "Most industrial wastes are high in carbon content, but deficient in other nutrients such as nitrogen and phosphorous. Sewage, on the other hand, contains a considerable excess of nitrogen and phosphorous. Combining plant waste with municipal waste eliminates the feeding of nutrients as purchased chemicals."

Also, it should be noticed that, no matter what the origin of the waste water, if it is to be reused, a great part of it has to be desalted. The desalting techniques that have been developed by the Office of Saline Water, U.S. Department of Interior, appear to be more practical on a large scale. This supports the idea of combined waste treatment, in the context of economies of scale.

Figure 5 shows the possible routes that the combined sewage may have to follow in order to supply all or part of the demands.

As stated previously, each industry has to treat on-site its waste waters for some specific pollution. This includes metal traces and non-biodegradable compounds. Also, because their pH varies over a wide range, it is necessary to neutralize these waste waters, so that the pH is driven to a value between 6 and 8, compatible with an efficient biological treatment.

In order to meet the pollution laws on discharge to receiving bodies, the combined sewage has necessarily to be treated in a primary-secondary treatment plant (PST). The primary treatment consists of the following operations: screening and grit removal, primary clarification. The two most common methods of secondary treatment are the trickling filters technique and the activated sludge process [11]. Their efficiencies are quite similar, with a little advantage for the latter (Tables 3 and 4), but its price is also a bit higher [17]. An advanced study will make the choice on each case-study. The next operation is a secondary clarification, followed by a chlorination. A major problem which arises with such a treatment is the disposal of debris and waste sludge. As far as debris are concerned, they can be merely dumped on a dumping ground; but, the pollution regulations

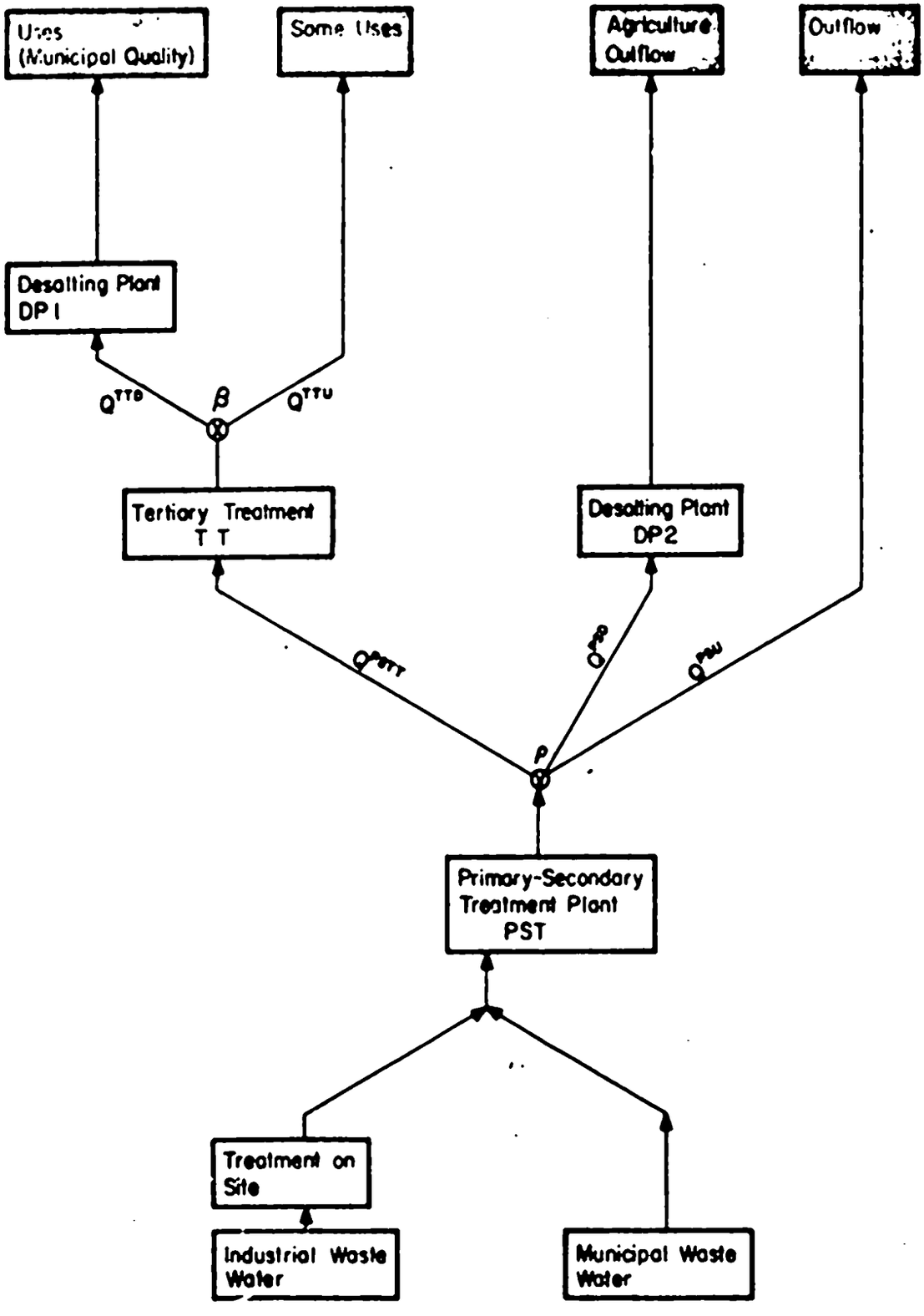


Figure 5. The Municipal and Industrial Waste Waters Reuse

becoming more stringent, it is likely that, in the future, these debris will have to be incinerated. The ashes would eventually be dumped on the previous dumping ground (that may not totally solve the pollution problem). As far as waste sludge is concerned, the majority of waste treatment plants being designed today in the U.S. are based on one of three processing schemes [11].

-- digest the sludge, dewater and thicken it mechanically, use it for landfill,

-- digest the sludge, dewater and thicken it mechanically, and incinerate it,

-- dewater raw sludge mechanically and incinerate it. This method applies mainly to the sludge from the primary and secondary clarifications. The disposing of sludge must be carefully planned since it accounts for 25 to 50% of the capital and operating costs of a treatment plant ([11], [17]).

At point ρ (in Fig. 5), three possible directions are of interest for the secondary effluent.

Route 1. At that point ρ, the water can be in most of the cases used for outflow purposes or low quality agricultural uses. If the TDS is around 900 mg/l, it can even supply the agricultural demand right away. Q^{PSU} is diverted for these direct uses.

Route 2. If the TDS is more than 900 mg/l but the water is needed for agriculture, then Q^{PSD} of it has to be sent to DP2, in order to be desalted down to level 900 mg/l.

Route 3. If the secondary effluent is needed to supply the municipal, industrial, recreational demands, then Q^{PSTT} has to be sent to a tertiary treatment plant (TT). The tertiary treatment plant

that is considered here is the most conventional type ([7], [11], [13], [18]), consisting of:

-- a floculation-lime-treatment unit, followed by a settling tank, in order to remove the phosphates,

-- an activated carbon absorption unit, in order to reduce the SS and the BOD to about 1 mg/l. These values are acceptable for drinking water,

-- an ammonia-stripping tower to lower the nitrogen concentration to drinking standards.

To make the tertiary treatment economically feasible, the spent lime and carbon must be regenerated and reused, with new carbon and lime being added as necessary to replace that lost in regeneration [11], [16].

At point β (in Fig. 5), the water is suitable to all or part of the uses, depending on the value of its TDS . Where possible, Q^{TTU} is sent for a direct use. The other part of the inflow at point β , is sent to DP1 to be desalted: that is Q^{TTD} .

D. The Desalting Processes

Because both primary and secondary sources may be concerned with desalinization, the desalting processes available at the present time and the possible structure of desalting plants DP1 and DP2 are discussed separately.

The Office of Saline Water (OSW) has given consideration to many different schemes for the separation of water from its dissolved solids; the processes which show the most promise for brackish water [25] are:

-- Electrodialysis

-- Reverse Osmosis

-- Ion Exchange

-- Vacuum Freezing

-- Other Crystallization Processes

-- Vapor Compression Distillation

The absence of multistage flash (MSF) and multi-effect falling-film (MEFF) evaporation from this list may raise a question. Without a doubt, they can be used, and, in the case of certain bruckish waters, they may be the only suitable processes. However, in general, because of the need for relatively large amounts of cooling water, MSF and MEFF are more attractive for seacoast locations. It turns out that most brackish water sources will be found in those areas where cooling water is not abundant, such as Arizona or New Mexico. Furthermore, since brackish waters will normally have a much lower range of TDS than seawater, those processes whose costs are directly related to the TDS in the feed water, such as electrodialysis, reverse osmosis, and ion exchange, should have an inherent economic advantage over those processes whose costs are unrelated to the TDS. The latter classification includes both the MSF and the MEFF processes.

The economies of vacuum freezing appear best for waters of about 5000 mg/l of TDS. Vapor compression distillation is suitable for waters containing more than 8000 or 10,000 mg/l of TDS. It is only in cases of a very "salted" primary source, that one of these methods could become attractive. Since the main concern in this study is the use of waters whose TDS is not likely to ever become more than 4000 mg/l, the two previous methods are not interesting. As for the crystallization processes, like the propane hydrate process, they also are not considered, because they so far have not shown any favorable potential for low-cost

production [25]. The study is then restricted to the three membrane processes: electrodialysis, reverse osmosis and ion exchange.

Ion Exchange. Conversion of brackish water by ion exchange has been employed to a limited extent, but, because of cost, other methods have been favored, especially for water containing more than 1000 mg/l TDS. However, recent developments producing resins of high capacity and high regeneration efficiency by using low cost regenerants have made ion exchange a competitive process for low salinity waters [39]. It should be noticed that ion exchange will produce a water of much lower TDS (about 50 mg/l) than that normally required for reuse. Therefore, water from this process could be blended with more highly mineralized water, in order to adjust the TDS to the uses requirements [27]. In this study, in order to decrease the number of blending basins and at the same time increase their capacities (this cannot be but beneficial, regarding the economies of scale), three levels of output-TDS in DP1 were selected: level 1300 mg/l, which corresponds to a possible use for outflow or low quality agriculture; level 800 mg/l, which corresponds to a use by agriculture, recreation and low quality industrial demand; level 400 mg/l, which corresponds to drinkable water. As far as DP2 is concerned, only the two first levels are of concern, since the biological quality of the input water prevents it from being used for industrial, recreational or municipal purposes.

Reverse Osmosis. This method is still at the experimental stage or used on a practical limited scale ([22], [25], [27]). It offers much promise, but its emergence as an economical demineralization process will depend entirely upon a membrane ability to produce high

quality water over extended periods of time [27]. As it is true for the ion exchange process, reverse osmosis will produce water of much lower TDS (also 50 mg/l) than that normally required for reuse. The same procedure, as far as blending is concerned, holds here.

Electrodialysis. Electrodialysis has been utilized successfully to demineralize brackish waters for many years. A single stage electrodialysis unit will remove about 35 to 45% of the mineral ions. Additional stages would be required to effect greater removal. One may notice, however, that application of a process achieving this degree of demineralization appears quite attractive, since this is approximately equal to the minerals added to waste water by one domestic water use cycle [27]. On the flow chart three stages for DP1 , and two stages for DP2 , are considered. Depending on the input TDS, they might or might not be necessary. But, if the input TDS to DP1 is very high, say 2500 to 3000 mg/l, one stage would reduce it to about 1400 mg/l (usable for outflow), a second stage to 800 mg/l (usable by agriculture, recreation, part of the industry), a third stage to about 400 mg/l (municipal and industrial water). Figure 6 shows the different alternatives that can be considered in the case of DP1 .

Regardless of the desalting process, full consideration has to be given to the problem of brine disposal. The waste product from any desalination plant must be discarded in such a way as not to cause harm to the environment. The only two economically feasible alternatives at an inland location are injection into underground formations and evaporation in solar evaporation ponds [30].

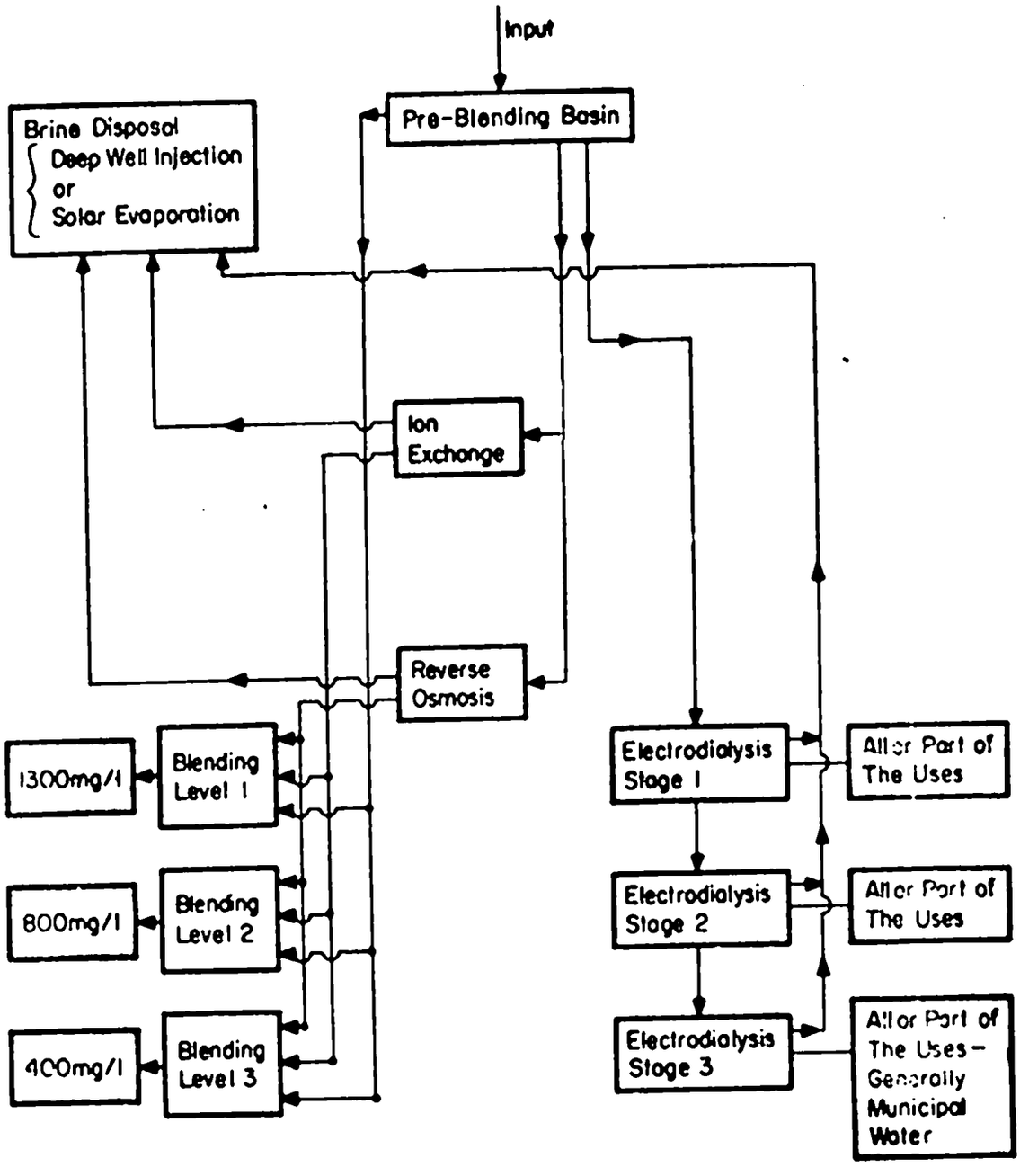


Figure 6. Desalting Plant No. 1

The deep well injection is a common practice in petroleum production. The techniques of disposal well design, construction and operation are well established. But, in a feasibility study, the short life of a well (clogging after 5 to 12 years) has to be kept in mind.

Experience and reported performance data for solar evaporation ponds come mainly from those industries which manufacture salt by the solar evaporation of seawater. Because of its extensive requirement in land, this method has not been used much, so far, for disposal of saline waste waters. However, the research conducted at Colorado State University by Dellah and Löf, on spray evaporation of brine [31], shows that this method is likely to be competitive with the deep well method for plant sizes over 5 to 7 MGD (assuming that the site of the plant is a rather dry and sunny one).

E. Removal Efficiencies

Knowing the quality characteristics of the supply sources and the quality requirements of each use-sector, it is important to know not only the eventual treatment alternatives, but also the efficiency of each treatment unit in removing the pollutants. The results of an extensive study completed at the Sanitary Engineering Research Laboratory, University of California at Berkeley [5], are shown in Tables 3 and 4.

Since the BOD is the most used overall determinant of the organic load of a water supply, its removal has to be carefully planned. Because it gives an average value of the BOD removal that one can expect from different treatment units for several values of the loading parameters, Table 3 is considered to be a very useful tool for the

Table 3: BOC removal by sewage treatment plants (Ref: [5])

Treatment Process		BOD Removal %
Type	Loading Parameters	
Chlorination of comminuted sewage	20 mg/l chlorine	10
Primary treatment	1 hr settling	26
Primary treatment	2 hr settling	35
Primary treatment	3 hr settling	38
Chemical coagulation	30 mg/l $FeCl_3$	55
Chemical coagulation	60 mg/l $FeCl_3$	80
High-rate trickling filter	1 hr, 25 lb/(1000 cu ft)(day)	82
High-rate trickling filter	1 hr, 40 lb/(1000 cu ft)(day)	78
High-rate trickling filter	1 hr, 62.5 lb/(1000 cu ft)(day)	74
High-rate trickling filter	1 hr, 187.5 lb/(1000 cu ft)(day)	60
High-rate trickling filter	2 hr, 25 lb/(1000 cu ft)(day)	84
High-rate trickling filter	2 hr, 40 lb/(1000 cu ft)(day)	80
High-rate trickling filter	2 hr, 62.5 lb/(1000 cu ft)(day)	77
High-rate trickling filter	2 hr, 187.5 lb/(1000 cu ft)(day)	65
High-rate trickling filter	3 hr, 25 lb/(1000 cu ft)(day)	85
High-rate trickling filter	3 hr, 40 lb/(1000 cu ft)(day)	81
High-rate trickling filter	3 hr, 62.5 lb/(1000 cu ft)(day)	78
High-rate trickling filter	3 hr, 187.5 lb/(1000 cu ft)(day)	67
Standard-rate trickling filter	1 hr, 12.5 lb/(1000 cu ft)(day)	87
Standard-rate trickling filter	2 hr, 12.5 lb/(1000 cu ft)(day)	89
Standard-rate trickling filter	3 hr, 12.5 lb/(1000 cu ft)(day)	89
Conventional activated sludge	6 hr, aeration	90
Conventional activated sludge	10 hr, aeration	94
Tertiary treatment		
conventional activated sludge	6 hr, aeration	
denitrification	4 hr, mixing	
chemical precipitation	200 mg/l alum	99

Table 4: Bacterial efficiencies of sewage treatment plants and operations (Ref: [5])

	Treatment Operation or Process	Bacteria Removed, %
1.	Fine screening	10 to 20
2.	Chlorination of raw or settled sewage	90 to 95
3.	Plain sedimentation	25 to 75
4.	Chemical precipitation	40 to 80
5.	High-rate trickling filtration including pre- and post-plain sedimentation	80 to 95
6.	Standard-rate trickling filtration including pre- and post-plain sedimentation	90 to 95
7.	High-rate activated sludge treatment including pre- and post-plain sedimentation	80 to 95
8.	Conventional activated sludge treatment including pre- and post-plain sedimentation	90 to 98
9.	Intermittent sand filtration	95 to 98
10.	Chlorination of biologically treated sewage	98 to 99

design of a water treatment plant. On the other hand, Table 4 gives the average values of coliform removal, which is the second most important determinant of the organic-load of a water source.

Tables 3 and 4 allow the designer to clearly enumerate, in a preliminary study, what the possible alternatives of treatment are.

For information, the Drinking Water Standards adopted by the U.S. Public Health Service specify maximum values allowed for the following quality criteria:

Criterion	Maximum Value (USPHS)
BOD	4 mg/ℓ
SS	< 2 mg/ℓ
TDS	500 mg/ℓ
COL	2 MPN/100 mℓ

In the previous section of this study a general picture of the water system of a large urban area has been proposed and analyzed. Once the possible sources of supply and the different types of demand were identified, it was possible to answer the question: HOW to meet the quality and quantity requirements of each use sector?

In Part II, the following question is going to be answered: AT WHAT COST?

PART II: ESTIMATIONS OF THE TREATMENT COSTS

This section is concerned with the establishment of a mathematical model for estimating the cost of treating water (raw-waters and/or waste-waters) in the United States. This is intended to be a reliable working tool useful to consulting engineers, municipal officials, developers, and the agencies charged with the development and management of water resources. It must be emphasized that estimates are just estimates and are in no way substitutes for actual costs. They must be used by experienced persons with a thorough knowledge of specific projects.

A. Methodology and Assumptions

Present cost estimating data, though numerous, vary widely from author to author as far as the presentation is concerned. Also, often their reliability appear to be questionable, based on the following arguments: too small a size for the sampling space (refer to Shah and Reid [28], for a study of the minimum size of the sampling space), too particular a frame of study (for example, regional study), or inconsistency in the text itself. To cope with these difficulties, the following procedure was adopted:

1) Carry out an intensive search for past and present literature
2) For each paper, analyze carefully the assumptions made, the sampling method, the consistency. That provides the basis for a first skimming.
3) By comparing the proposed models for a given item, or treatment unit, try to come up with a single cost equation (in view of a computerized scheme), and possibly with a curve (for a fast visualized estimation).

This last part of the procedure requires a few comments. Indeed, as can be seen in the tables at the end of this section, two equations for the same treatment unit are sometimes proposed. The reason is that the cost studies in the literature are often incomparable on such grounds as:

-- the definition of terms such as "capital cost," "investment cost," "operation and management cost" are not universally accepted, and vary widely from author to author. It even often happens that the components of each cost category are not clearly stated.

-- the chosen interest rates, life-time for each facility, payoff periods differ a great deal. Actually, since bonds are sold on the open market. the cost of borrowing money varies with the type of bonds sold and with the financial rating of the municipality issuing the bonds.

-- the importance of the "additional costs" (engineering fees, cost of land), when not included in one of the three previous types of costs, is differently evaluated. It is generally recommended to account for them by multiplying the investment cost by a factor α . Unfortunately, in the literature, one can expect α to vary between a low 1.10 and a high 1.40.

It is understood, in this study, that if two different models for the same item appear to have the same credibility, and if for a given set of values of the variable (two or three values) they do not differ more than 25 percent, then both of them should be kept, and the problem of choosing between them left to the experienced user.

For the sake of practicability, it is assumed here that:

1) Y , the construction cost, or investment, or capital cost (as used in the cost tables), is the amount of money needed to complete the construction of a treatment unit, at the time of construction. Y does not account for the cost of borrowing money; it does account for the "additional costs" (legal and engineering fees, interest during the construction), which were arbitrarily chosen to be 20% of the actual construction costs. However, because of its high variability, the cost of land acquisition is not included; it is recommended to account for it separately for each case study.

2) The operation and management costs account for

   -- the labor costs

   -- the power costs

   -- the purchase of chemicals

   -- miscellaneous expenses of maintenance

   The O&M costs have been expressed in two different ways: i) y is the cost per 1000 gallons of treated water, expressed in cents (value of a given year, and ii) Y_o is the total annual O&M cost, expressed in dollars of a given year.

B. How to Use the Cost Models

All the cost equations, that are included in the following tables, represent, as previously stated, national averages. They are almost useless to the planner unless he can account for regional and time differentials.

To account for these differences, the twenty U.S. regions on county line basis are considered. Each region corresponds to one of

the twenty cities used in obtaining the United States Average Engineering News-Record cost indexes ([40]); the areas of influence of these twenty major trade centers are helpful in comparing the costs on a regional basis and can be used to evaluate costs in any of the 22 U.S. river basins by taking into consideration the index or indexes which influence the area within a particular basin. However, in the case of waste-water treatment, it is recommended to use the FWPCA Sewage Treatment Plant Construction Index (WPC-STP Index, Table 5) which is more specific, hence more accurate, than the traditionally used ENR-Construction Cost Index (ENR-C Index, Table 6).

The way these indexes are used is the following: Suppose that one wants to know, in October 1971, what the investment for primary treatment of municipal waste water is going to cost at Denver (Colorado). Assume that the design flow is 10 MGD. From the equation for primary treatment presented in Table 9, one can find the U.S. average investment cost, in 1967 dollar value, i.e., Y such that:

$$\log Y = -0.46 + 0.71 \log 10.0$$

that is: $Y = 1.77$ million dollars.

From Table 5, one can pick up the following values for the Sewerage Construction Cost Index (treatment plants):

	October 1967	October 1971
U.S. Average	121	166.3
Denver	112	150.3

To adjust the value of Y to the region of Denver in October 1971, it suffices to multiply it by the ratio $\frac{150.3}{121}$, i.e.,

$$Y = 1.77 \times \frac{150.3}{121} = 2.14 \text{ million dollars (October 1971 dollar value)}$$

Table 5: Sewerage construction cost indexes in 20 cities
Water Quality Office, Environmental Protection
Agency. Base: August 1957-'59 = 100

TREATMENT PLANT

	1965	1966	1967	October 1968	1969	1970	1971	%chge 10/69-10/70	%chge 10/70-10/71
U. S. Aver.	113	118	121	127	136	148	166.3	+ 9.0	+12.3
Atlanta	103	108	111	114	122	136	149.5	+11.2	+10.1
Baltimore . . .	105	112	115	119	124	139	155.6	+11.9	+11.8
Birminghan. . .	102	105	106	112	114	121	136.7	+ 6.2	+12.7
Boston.	113	119	121	127	140	150	171.3	+ 7.0	+13.9
Chicago	116	120	123	130	137	154	173.1	+12.3	+12.4
Cincinnati. . .	112	116	120	126	136	148	167.9	+ 8.7	+13.7
Cleveland . . .	116	121	126	137	146	155	173.2	+ 6.6	+11.4
Dallas.	102	106	107	111	123	130	150.0	+ 6.1	+15.1
Denver.	106	109	112	118	126	135	150.3	+ 7.0	+11.6
Detroit	120	124	129	138	152	163	180.4	+ 6.6	+11.0
Kansas City . .	107	112	113	119	127	154	181.7	+21.8	+17.6
Los Angeles . .	116	125	130	135	138	149	175.8	+ 8.1	+18.2
Minneapolis . .	116	118	123	131	138	154	172.5	+11.8	+12.1
New Orleans . .	103	107	109	115	125	138	150.5	+10.5	+ 9.1
New York. . . .	131	134	139	148	155	169	194.5	+ 8.8	+15.3
Philadelphia. .	113	117	119	123	135	148	165.7	+ 9.5	+11.9
Pittsburgh. . .	116	119	122	128	147	154	174.3	+ 5.0	+13.0
St. Louis . . .	115	121	126	132	140	153	171.3	+ 9.0	+11.9
San Francisco .	124	129	134	139	147	157	174.3	+ 6.9	+10.8
Seattle	120	130	131	135	144	153	164.2	+ 6.3	+ 6.9

SEWER LINE

	1965	1966	1967	October 1968	1969	1970	1971	%chge 10/69-10/70	%chge 10/70-10/71
U. S. Aver.	118	122	126	132	141	154	173.3	+ 9.1	+12.2
Atlanta	102	104	109	113	122	137	150.0	+12.4	+ 9.7
Baltimore . . .	115	119	123	125	128	146	159.1	+13.8	+ 9.1
Birmingham . .	97	99	101	106	117	123	137.2	+ 4.7	+11.6
Boston.	116	122	125	133	148	156	176.3	+ 5.2	+12.9
Chicago	114	118	121	126	136	154	181.7	+13.0	+18.0
Cincinnati. . .	112	114	120	133	147	160	178.7	+ 9.1	+11.6
Cleveland . . .	118	118	126	144	154	166	177.0	+ 7.5	+ 6.8
Dallas.	98	101	103	107	118	124	143.4	+ 5.4	+15.8
Denver.	111	113	118	123	129	131	144.9	+ 1.8	+10.2
Detroit	127	131	135	145	149	172	199.4	+15.4	+16.0
Kansas City . .	112	116	119	124	135	164	190.6	+21.3	+16.2
Los Angeles . .	125	133	137	141	143	157	184.8	+16.4	+17.4
Minneapolis . .	123	126	132	141	146	163	184.3	+12.2	+12.8
New Orleans . .	103	111	114	121	133	141	149.1	+ 5.6	+ 5.9
New York. . . .	150	154	160	166	163	176	205.5	+ 7.8	+16.9
Philadelphia. .	121	128	131	133	149	155	180.4	+ 4.3	+16.2
Pittsburgh. . .	126	130	132	135	152	161	178.9	+ 5.8	+11.0
St. Louis . . .	116	120	129	136	144	160	182.8	+11.3	+14.4
San Francisco .	133	138	144	148	158	174	184.9	+10.2	+ 6.2
Seattle	131	145	142	149	159	167	176.2	+ 5.5	+ 5.3

Table 6: ENR construction index for 20 U.S. cities and Dec. '72 forecast
ENR base: U.S. average in 1913 = 100: labor and materials

December: City	'63	'64	'65	'66	'67	'68	'69	'70	'71	'72	% change 12/71-12/72
Atlanta. . . .						934	973	1101	1248	1333	+ 7
Baltimore. . .						976	1015	1215	1424	1593	+12
Birmingham . .						945	992	1098	1191	1285	+ 8
Boston						1186	1313	1434	1719	1833	+ 7
Chicago. . . .						1336	1408	1603	1837	1970	+ 7
Cincinnati . .						1341	1516	1604	1872	2098	+12
Cleveland. . .						1500	1616	1762	1977	2118	+ 7
Dallas						893	1070	1154	1283	1374	+ 7
Denver						1058	1127	1199	1352	1424	+ 5
Detroit. . . .						1399	1532	1708	1959	2091	+ 7
Kansas City. .						1115	1348	1594	1797	1941	+ 8
Los Angeles. .						1275	1309	1482	1704	1840	+ 2
Minneapolis. .						1225	1323	1542	1766	1944	+10
New Orleans. .						971	1054	1141	1270	1359	+ 7
New York . . .						1575	1628	1771	2180	2360	+ 8
Philadelphia .						1168	1188	1414	1604	1830	+14
Pittsburgh . .						1225	1321	1426	1670	1782	+ 7
St. Louis. . .						1351	1490	1647	1856	1975	+ 6
San Francisco						1413	1476	1598	1849	1983	+ 1
Seattle. . . .						1253	1335	1408	1540	1667	+ 8
U. S. Average	900	936	971	1021	1100	1204	1305	1445	1655	1789	+ 8

C. Definition of the Variables and Symbols

It is logical to relate costs of raw-water treatment and primary waste-water treatment to hydraulic loadings, and costs of secondary treatment plants to hydraulic and organic loadings, since in designing the biological unit of secondary treatment plant, consideration to organic loading is given.

The hydraulic loading has been characterized by two variables:

1) the design-flow of the plant, i.e., the maximum capacity or the size of the plant. It has been called Z and is expressed in MGD (million gallons per day). This is, of course, the main parameter in determining the investment cost of a facility.

2) the average daily flow which is treated in the plant. It has been called Q and is expressed in MGD. It is the main parameter of cost estimation for operation and management, especially if the type of treatment under consideration is more of a chemical or electrical type rather than a physical type.

A good measure of organic loading is the Population Equivalent of Waste, often simply called the PE . That reflects the contribution to organic loading from all sources within the community. It is defined by the following relation:

$$PE = \frac{8.33\ QL}{b}$$

where 8.33 is a conversion constant, Q is the average inflow of polluted water to the plant in MGD, L is the average 5-day BOD

of waste in milligrams per liter (mgl), and b is generally assumed to be 0.17 pound (lb) of BOD per capita per day.

As far as operation and management costs of a treatment plant are concerned, the management efficiency of the plant is believed to be of major importance and therefore should be accounted for in the derivation of a cost model. However, insufficient data in the literature prevents one from doing so. When mentioned in cost reports, efficiency is generally taken to be 90 percent ([16], [20]), but never is a definition of efficiency clearly stated. Also, throughout this study: i) "log" means "logarithm base 10", and ii) "E" is the estimated error, which states that one can expect the real cost of the facility under study to be E greater or less than the cost calculated from the model. E is expressed in percentage. iii) "S" is the size of the sample out of which the cost-model has been derived.

D. Analysis of the Cost Tables and the Cost Curves

For each type of treatment (raw-water treatment, primary-secondary waste-water treatment, tertiary waste-water treatment, desalting either raw brackish water or waste-waters), two tables for respectively the capital costs and the operation and management are presented. Some important points should be emphasized here.

1. Capital Costs. Often, the relation between the capital cost of a treatment unit or a total treatment type is linear, following a log-log transformation. That expresses that Y is an exponential function of the size Z :

$$Y = \alpha(Z)^{\beta} \text{ or } \log Y = \log\alpha + \beta\log Z$$

The main characteristic of these cost-models is that β is always

positive and less than 1.0: $0 \leq \beta \leq 1$. This fact points out that the concept of economies of scale for larger-capacity plants is relevant in water treatment. Actually, the smaller β , the more important the economies of scale. The following tableau allows a rough comparison of the economies of scale in the different types of treatments.

Type of treatment	β
Raw-water treatment as a whole for a TP2 type-plant	0.65
Primary treatment of waste-water	$0.61 < \beta < 0.71$
Secondary treatment of waste-water	0.22
Tertiary treatment of waste-water	$0.7 < \beta < 0.90$
Electrodialysis	0.66
Reverse osmosis	$0.80 < \beta < 0.95$

This tableau shows that the secondary treatment of waste waters is the most favorable domain for economies of scale. On the contrary, a reverse osmosis plant, which can be built easily by small segments (better elasticity to the demand), presents very small economies of scale.

2. Operation and Management Costs. Here again a log-log transformation applied on y or Y_o and the independent variables Q , Z or PE , leads to linear relations which are considered to be the most appropriate form for the estimation equation [28], that is:

$$\log y = A + B \log X \qquad (y \text{ is a cost per unit})$$

$$\log Y_o = C + D \log X \qquad (Y_o \text{ is a total cost})$$

X being the independent variable.

One notes that, in the presented models, all the coefficients of type B are found to be negative, and all coefficients of type D are positive and less than 1.0. That fact points out that just as for the

capital costs, the concept of economies of scale is relevant in O&M costs of water treatment. One can further state that: i) the O&M economies of scale are not very sensitive to the size of the plant: they do exist but are generally very small ($-0.05 > \beta > -0.36$), and on the other hand, substantial economies occur when the daily flow Q and/or the PE increase: D is found in a range of 0.55 to 0.73.

Another important consideration concerning the O&M cost is the relative importance of each of its components. A compilation by R. Michel [17] of the data from 1600 plants yields the following tableau:

Cost Component	Percentage of the Total O & M Costs
Labor	60%
Electricity	14%
Chemicals	5% to 8%
Other Supplies and other items	Determined by difference

It is crucial to note here that with 50 to 60 percent expenditures for operation and routine maintenance of municipal water treatment plants attributable to labor and supervision, changes in factors that affect manpower, such as wage escalation and training, will markedly affect plant operating cost and efficiency. On the other hand, it is remarkable that the cost of chemicals explains a very small percent of the total O&M cost. For information, the U.S. average values for the cost of the most commonly used chemicals for water treatment are found to be the following in December 1971 [29]:

Sulphate of aluminum: large deliveries $63/ton

Chlorine: $200/ton for ton containers

Lime: large deliveries $46/ton.

Following the two cost tables (capital cost table and O&M cost table) for each treatment type, most of the cost models have been plotted. Generally, not only the construction cost Y and the O&M cost y have been represented, but also A and T , which are defined as follows:

1) A , the debt service, is actually the cost of borrowing money assuming that the financing of the project is done by emission of bonds (that is generally the case), at 4.5 percent interest rate over a repayment period of 25 years. These last two figures are believed to be the average over the U.S. for that kind of financing. A is expressed in cents per thousand gallon, and clearly represents the total interest that has to be paid to the shareholders everytime that a thousand gallons of water have been treated.

2) T is the total cost of treating water, not including the construction cost. It is expressed in cents per thousand gallon of treated water and is the sum of the O&M cost y and the cost of borrowing money A .

Contrarily to Y and y , which are empirical values (averages over a sampling space), A and T are computed in the manner presented below.

For example, if one wants to know A and T for a 10.0 MGD activated sludge plant, the following values for Y and y are picked up from Figure 10 (where A and T have, of course, not been plotted yet):

$$Y = 3.0 \text{ million dollars}$$

$$y = 4.8 \text{ cents/1000 gal.}$$

From interest tables, the single payment compound factor K, for a 4.5 percent interest rate and 25 years, is found to be 3.0054. Also, the maximum total treated flow is:

$$(10 \text{ MGD})(365 \text{ days})(25 \text{ years})$$

assuming full use of the treatment plant. A, the cost of borrowing the money for construction of the plant, is then:

$$A = \frac{Y(K-1)10^8}{(10)10^3 \times 365 \times 25} \quad \text{in cents/1000 gal.}$$

$$A = \frac{(3.0)(2.0054)10^8}{365 \times 25 \times 10^4} = 6.5 \text{ cents/1000 gal.}$$

Then: $T = A + y = 6.5 + 4.8 = 11.3$ cents/1000 gal.

These plots are believed to be practical tools for the engineer or the planner in search for a quick estimation of the cost of treating water.

Also, at the end of this section, some information about the cost of brine disposal is provided. It lacks generality and should only be used with caution.

*
* *

The mathematical relations developed here are believed to present a maximum error of 25 percent (verification in Part IV of this study). The information provided in this section in the form of cost estimates can accordingly be considered reliable, at least in the initial stages

of the planning of a water treatment project, i.e., in preliminary studies, or in determining the financial need and guide bond referenda.

THE COST ESTIMATING MODELS: TABLES AND CURVES

Table 7: Capital costs for a raw-water treatment plant

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Coagulation Sedimentation	[16]	$\log Y = -1.40 + 0.89 \log Z$	Y = total capital cost	$ million (1967 dollars)	
			Z = design flow	MGD	
Microstrainers	[16]	$\log Y = -1.26 + 0.90 \log Z$			
Filtration through sand or graded media	[16]	$\log Y = -1.20 + 0.75 \log Z$			at 4 gal/mn/sq ft
Chlorination	[16]	$\log Y = -1.80 + 0.67 \log Z$			15 mn contact, 8 mg/l of chlorine, and $0.08 per pound of chlorine
Raw water treatment (as a whole)	[12]	$\log Y = -0.49 + 0.65 \log Z$	Y = total capital cost	$ million (1964 dollars)	E* = ± 20%
			Z = design flow	MGD	S** = 40

*E is the estimated error
**S is the sample size
(a) "log" stands for "logarithm base 10"

Table 8: Operation and management costs for a raw-water treatment plant

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Coagulation Sedimentation	[16]	$\log y = 0.48 - 0.038 \log Z$	y = O & M cost per 1000 gallons	¢ (1967 dollars)	
			Z = plant capacity	MGD	
Microstrainers	[16]	$\log y = -0.10 - 0.10 \log Z$			
Filtration through sand and graded media	[16]	$\log y = 0.78 - 0.36 \log Z$			
Chlorination	[16]	$\log y = -0.05 - 0.10 \log Z$			
Raw-water treatment (as a whole)	[12]	$\log y = 1.08 - 0.35 \log Z$	y = O & M cost per 1000 gallons	¢ (1964 dollars)	E* = ± 20%
			Z = design flow		S** = 40

*E is the estimated error
**S is the sample size
(a) "log" stands for "logarithm base 10"

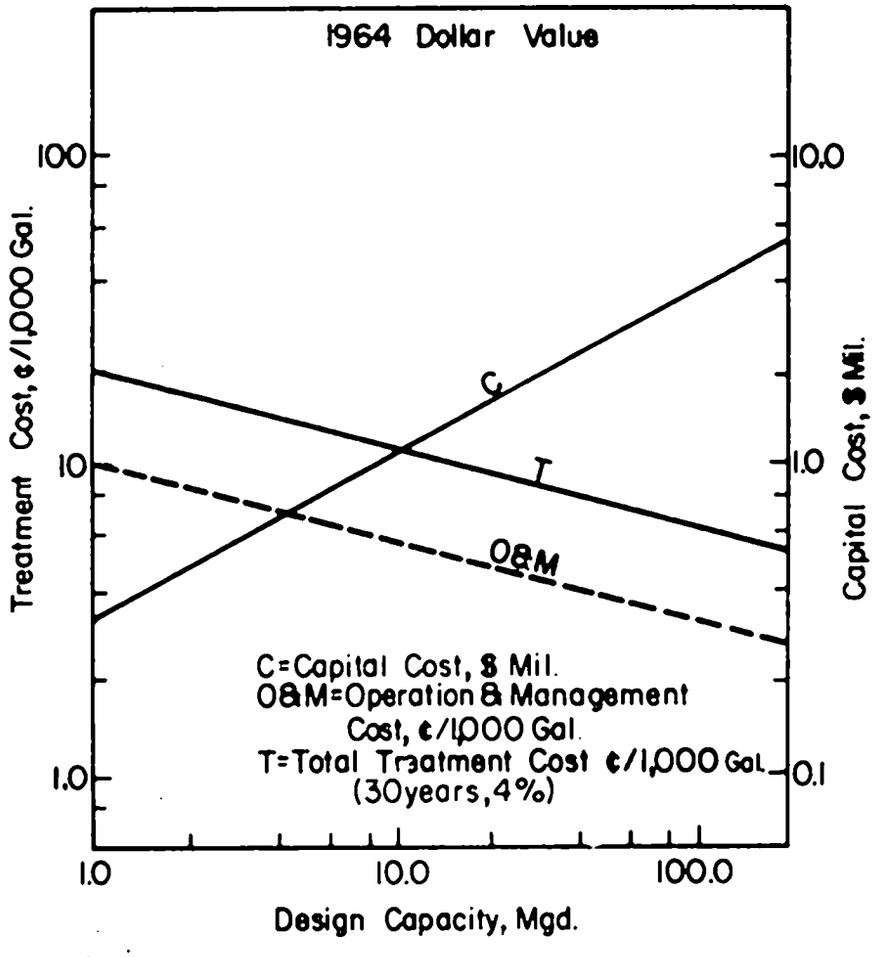


Figure 7. Raw Water Treatment [12]
Including - Coagulation
Sedimentation
Filtration through sand
Chlorination

Table 9: Capital costs for a primary-secondary treatment plant

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Equalization Neutralization Oil Separation (industrial wastes)	[20]	$\log Y = -0.57 + 0.66 \log Z$	Y :total capital cost Z :design flow	$ million (1969 dollars) MGD	
Neutralization	[20]	$\log Y = -1.22 + 0.72 \log Z$	Y :total capital cost Z :design flow	$ million (1969 dollars) MGD	
Oil separators	[20]	$\log Y = -1.24 + 0.84 \log Z$	Y :total capital cost Z :design flow	$ million (1969 dollars) MGD	
Primary treatment (pretreatment +primary clarification)	[16]	$\log Y = -0.50 + 0.71 \log Z$	Y :total capital cost Z :design flow	$ million (1967 dollars) MGD	
	[28]	$\log Y = -0.48 + 0.61 \log Z$	Y :total capital cost Z :design flow	$ million (1959 dollars) MGD	
Pretreatment (screening + grid removal)	[20]	$\log Y = -1.72 + 0.63 \log Z$	Y :total capital cost Z :design flow	$ million (1969 dollars) MGD	
Primary clarification	[20]	$Y = 10^{-3}[A(17.3 + 6.7/A^{0.9})]$	Y :total capital cost A :surface area of the settling basins, for overflow of 800 gpd/ft^2	$ million (1969 dollars) 1000 sq ft	
Secondary Treatment (as a whole)	[28]	$\log Y = -3.88 + 0.47 \log (PE) + 0.22 \log Z + 0.035\ D1 + 0.275\ D2$	Y :total capital cost PE :population equivalent Z :design flow D1 and D2 are: *D1 = 0, D2 = 0 for SRTF plants *D1 = 1, D2 = 0 for IRTF plants *D1 = 0, D2 = 1 for activated sludge plants	$ million (1959 dollars) MGD	
Trickling Filters	[16]	$\log Y = -0.3 + 0.76 \log Z$	Y :total capital cost Z :design flow	$ million (1967 dollars) MGD	
Standard Rate Trickling Filter (SRTF)	[28]	$\log Y = -2.56 - 0.50 \log (Pb) + 0.05 \log Z$	Y :total capital cost PT :population equivalent Z :design flow	$ million (1959 dollars) MGD	

Table 9: Capital costs for a primary-secondary treatment plant (continued)

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
High Rate Trickling Filters	[28]	$\log Y = -1.9 + 0.36 \log (PE) + 0.36 \log Z$	Y :total capital cost PE :population equivalent Z :design flow	$ million (1959 dollars) MGD	
Activated Sludge	[16]	$\log Y = -0.26 + 0.71 \log Z$	Y :total capital costs Z :design flow	$ million (1967 dollars)	
	[28]	$\log Y = -2.30 + 0.46 \log (PE) + 0.36 \log Z$	Y :total capital cost PE :population equivalent Z :design flow	$ million (1959 dollars) MGD	
Secondary Clarification	[20]	$Y = 10^{-3}.A(16.2 + 6.9/(A)^{1.13})$	Y :capital cost A :surface area of the settling tanks for overflow of 750 gpd/ft^2	$ million (1969 dollars) 1000 sq ft	
Chlorination	[20]	$\log Y = -4.94 + 0.47 \log Z$	Y :capital cost Z :flow design	$ million (1969 dollars) MGD	
	[16]	$\log Y = -1.80 + 0.67 \log Z$	Y :capital cost Z :flow design	$ million (1967 dollars) MGD	
Sludge Return Pumps	[20]	$Y = 4.7 \times 10^{-3} + 10^{-3} \times 1.45 Z$	Y :capital cost Z :design flow	$ million (1969 dollars) MGD	
		$Y = 3.7 \times 10^{-3} + 10^{-3} \times 1.12 Z$	Y :capital cost Z :design flow	$ million (1965 dollars) MGD	
Vacuum Filters	[20]	$Y = 10^{-3}(16.5 + 48.0 \times (\frac{A}{100}))$	Y :capital cost A :filter area, for loading of 6 lb/ft^2/hr	$ million (1969 dollars) sq ft	
	[16]	$Y = 10^{-3}(13 + 37.0 \times (\frac{A}{100}))$	Y :capital cost A :filter area, for loading of 6 lb/ft^2/hr	$ million (1960 dollars) sq ft	
Digesters	[16]	$Y = [1.04 + 10.7 (\frac{1}{V})^{0.87}] \times (V)$	Y :capital cost V :digester volume	$1000, (1960 dollars)	
	[20]	$Y = [1.34 + 13.8 (\frac{1}{V})^{0.87}] \times (V)$	Y :total capital cost V :digester volume for a 20 days-retention at high rate digestion	$1000, (1969 dollars) 1000 sq ft	
Control house (excavation + building + lab + shop)	[20]	$\log Y = -1.29 + 0.7 \log x$	Y :capital cost Z :plant capacity	$ million (1969 dollars) MGD	
	[16]	$\log Y = -1.40 + 0.7 \log x$	Y :capital cost Z :plant capacity	$ million (1967 dollars) MGD	

Table 10: Operation and management costs for a primary-secondary treatment plant

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Conventional Primary	[17]	$\log Y_o = 4.34 + 0.59 \log Q$	Y_o = annual O & M cost	$ (1967 dollars)	S* = 163
	[17]	$\log Y_o = 1.37 + 0.74 \log PE$	y = O & M cost per 1000 gal.	¢ (1967 dollars)	S = 163
	[16]	$\log y = 0.65 - 0.17 \log Z$	Q = average daily flow	MGD	S = 120
			Z = plant size	MGD	E** = ± 15%
			PE = population equivalent		
Standard Rate Trickling Filter Plant	[17]	$\log Y_o = 4.23 + 0.57 \log Q$			S = 104
	[17]	$\log Y_o = 1.43 + 0.69 \log PE$			
	[16]	$\log y = 0.78 - 0.28 \log Z$			E = ± 15%
High rate Trickling Filter Plant	[17]	$\log Y_o = 4.32 + 0.62 \log Q$			S = 36
	[17]	$\log Y_o = 1.79 + 0.64 \log PE$			
Activated Sludge Plants	[17]	$\log Y_o = 4.50 + 0.73 \log Q$			S = 118
	[17]	$\log Y_o = 1.45 + 0.75 \log PE$			
	[17]	$\log y = 0.95 - 0.107 \log Z$			E = ± 15%

*R is the correlation coefficient
**S is the size of the sample
***E is the estimates error
(a) "log" stands for "logarithm base 10"

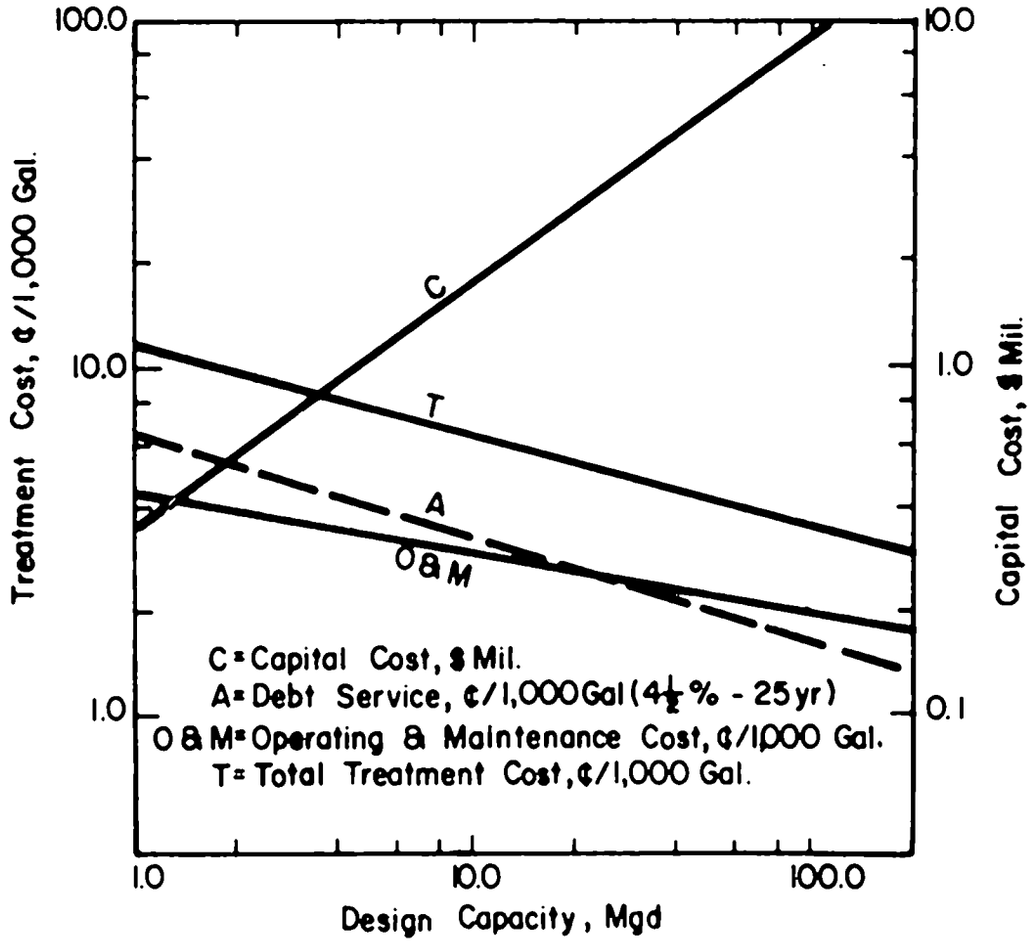


Figure 8. Primary Treatment Plants (as from R. Smith [16]) - Includes Screening, Grit Removal and Primary Clarification - 1967 dollar value

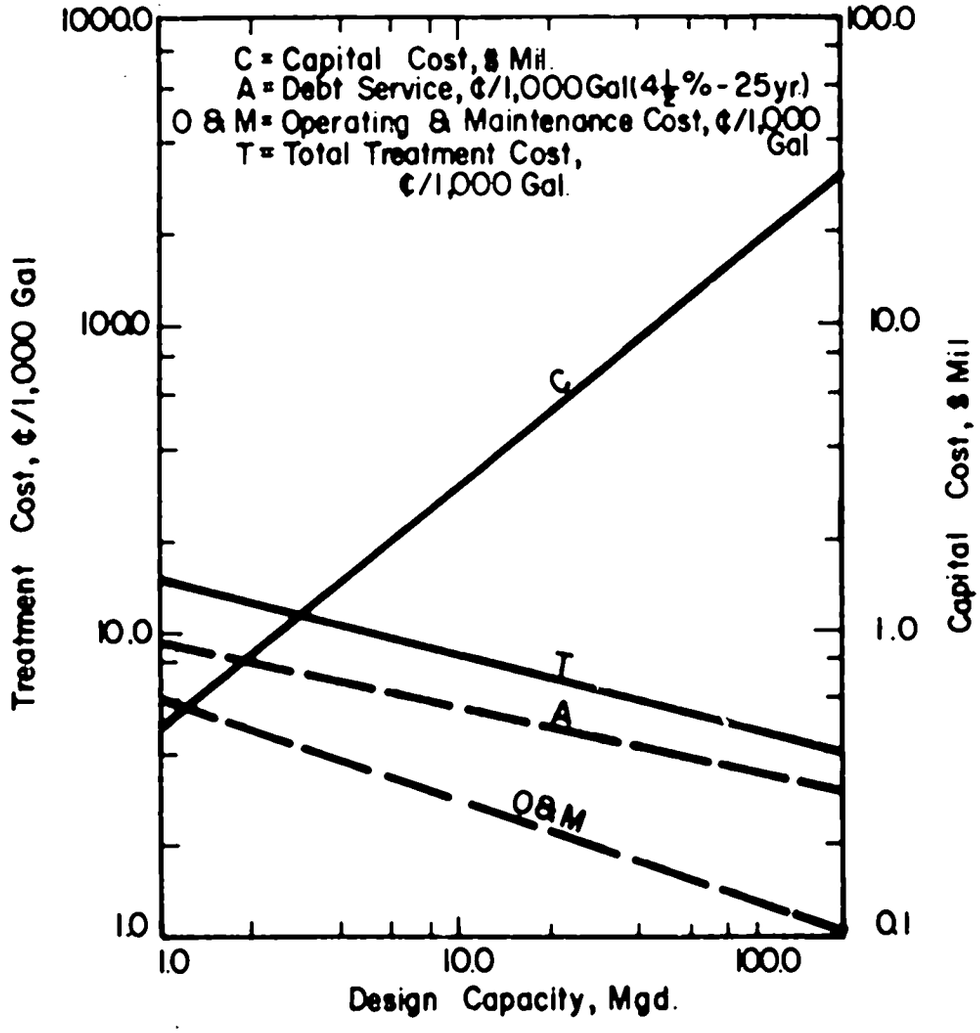


Figure 9. Trickling Filter Process (as from R. Smith [16]) - 1967 dollar value

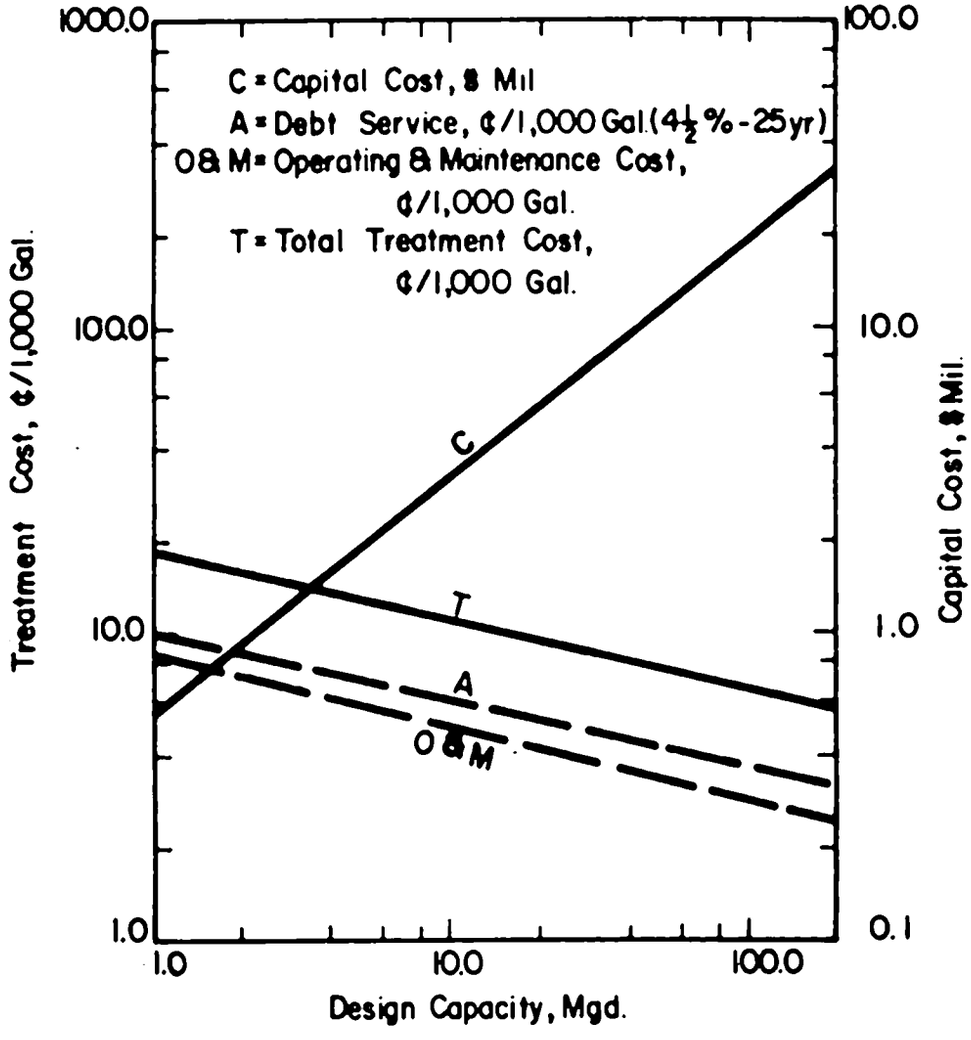


Figure 10. Activated Sludge Process (as from R. Smith [16]) - 1967 dollar value

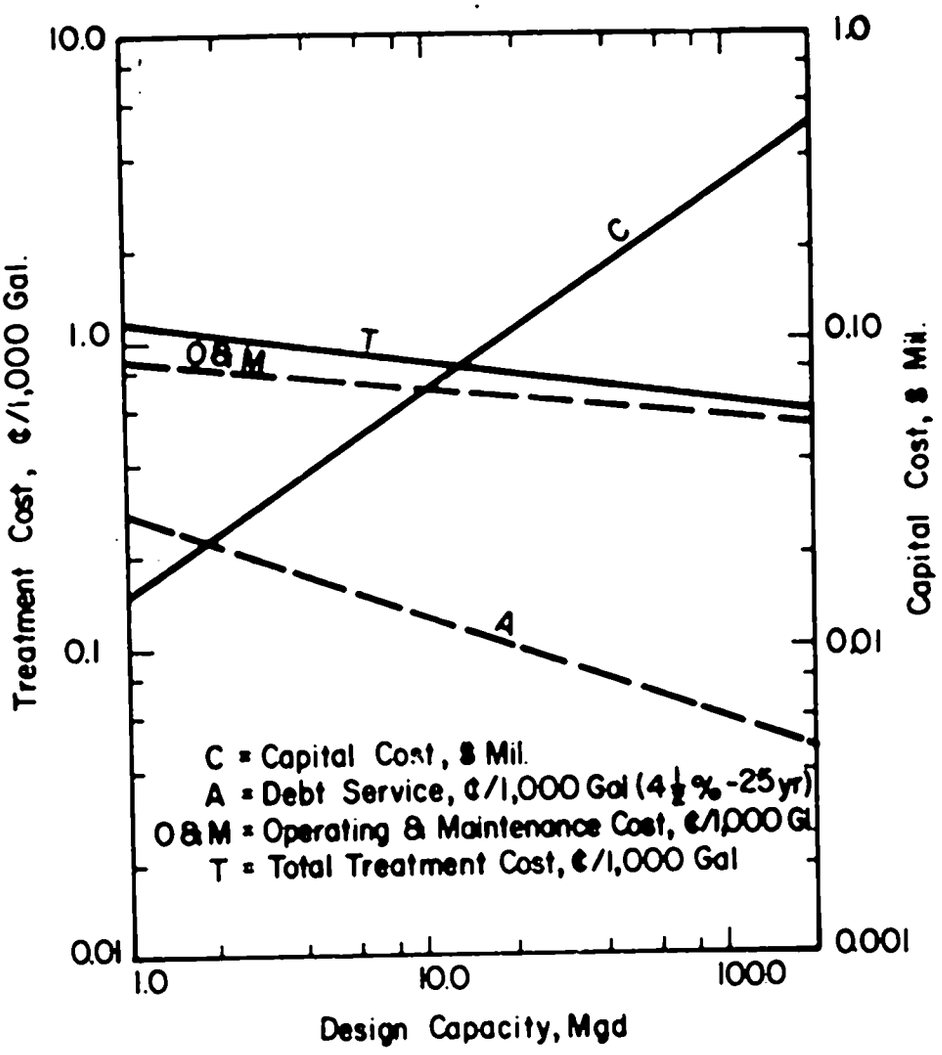


Figure 11. Chlorination (as from R. Smith [16]) - Design Parameter: 15 mn Contact at Rate of 8 mg/l - 1967 dollar value

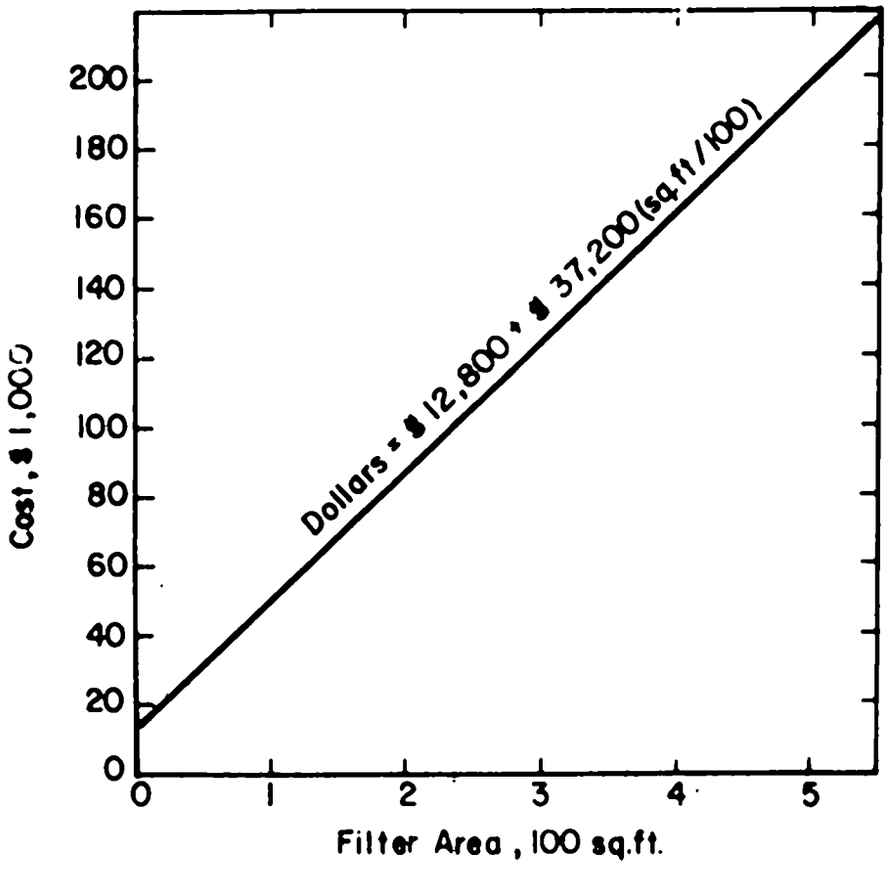


Figure 12. Vacuum Sludge Filter (as from R. Smith [16]) - Includes Housing and Installation - 1960 dollar value - ENR-C Index = 812

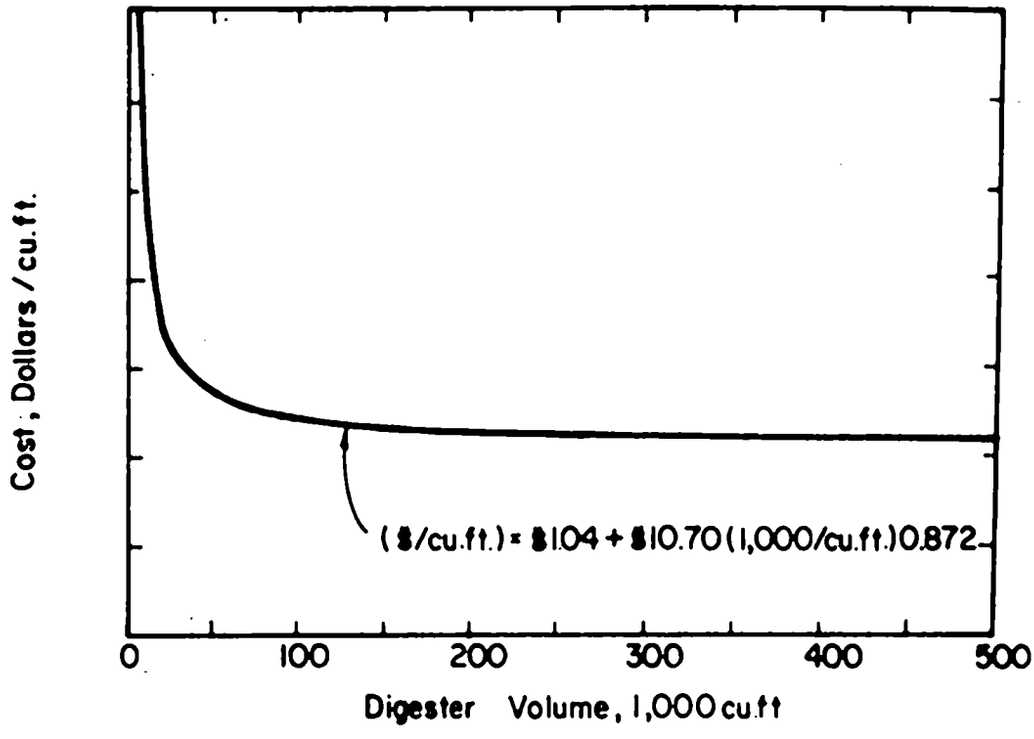


Figure 13. Digesters (as from R. Smith [16]) - 1960 dollar value - ENR-C Index = 812

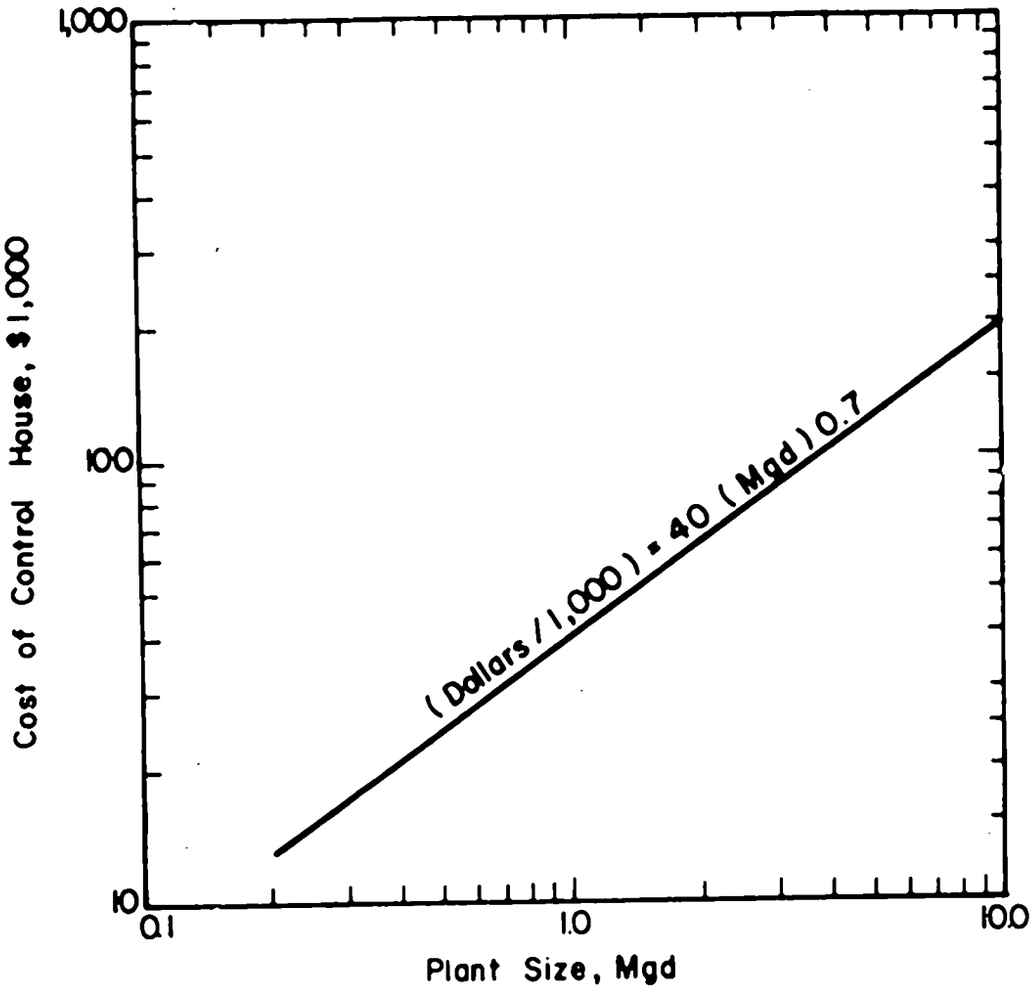


Figure 14. Control House (as from R. Smith [16])- Includes Excavation, Building, Laboratory and Shop - 1967 dollar value

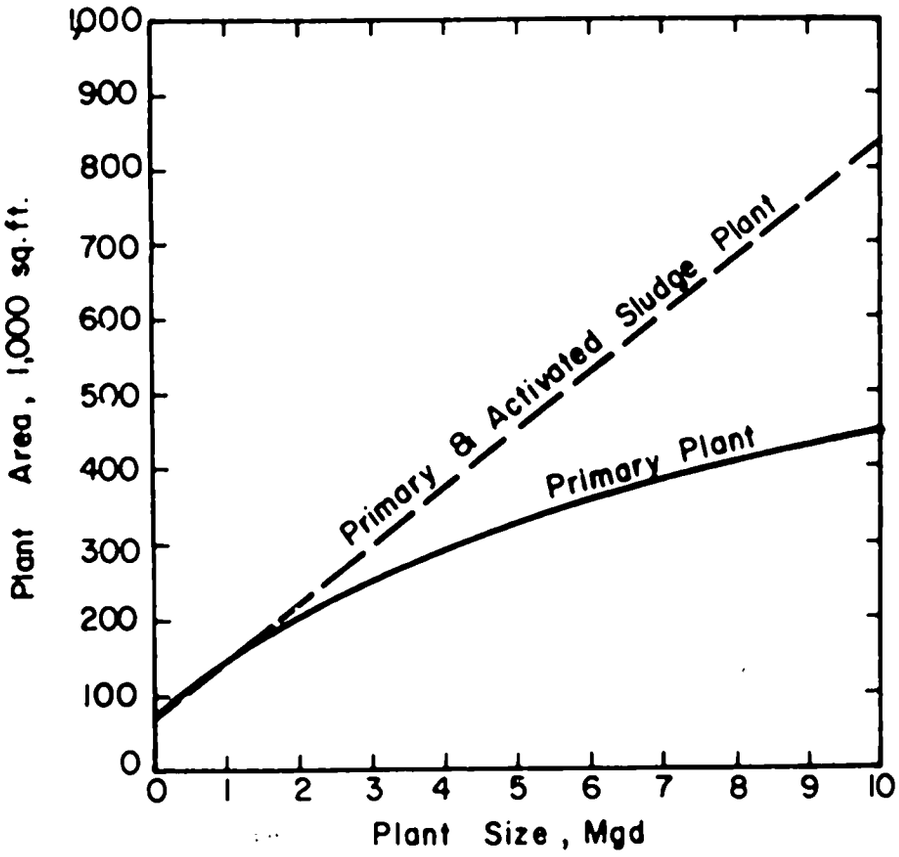


Figure 15. Land Requirements for Primary and Primary with Activated Sludge Plants (as from R. Smith [16]) - 1967 dollar value

Table 11: Capital costs for a tertiary treatment plant

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Floculation +Lime treatment +Sedimentation	[16]	log Y = -1.30 + 0.89 log Z	Y = total capital cost	$ million (1967 dollars)	
			Z = plant capacity	MGD	
Granular carbon	[16]	log Y = -0.40 + 0.65 log Z	Y = total capital cost	$ million (1967 dollars)	
			Z = plant capacity	MGD	
	[20]	log Y = -0.70 + 0.86 log Z	Y = total capital cost	$ million (1969 dollars)	
			Z = plant capacity	MGD	
Ammonia Stripping	[16]	log Y = -1.40 + 0.90 log Z	Y = total capital cost	$ million (1967 dollars)	Z>5 MGD
			Z = plant capacity	MGD	
	[16]	log Y = -1.05 + 0.35 log Z			Z<5 MGD

(a) "log" stands for "logarithm base 10"

Table 12: Operation and management costs for a tertiary treatment plant

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Floculation + Lime Treatment + Sedimentation	[16]	$\log y = 0.476 - 0.038 \log Z$	y = O & M cost per 1000 gallons Z = plant capacity	¢ (1967 dollars)	Lime: 300 mg/l
Granular Carbon Adsorption	[16]	$\log y = 1.0 - 0.28 \log Z$			7.5% Carbon Loss
Ammonia Stripping	[16]	$\log y = 0.2 - 0.30 \log Z$			Z < 3 MGD
		$\log y = 0.08 - 0.04 \log Z$			Z > 3 MGD

(a) "log" stands for "logarithm base 10"

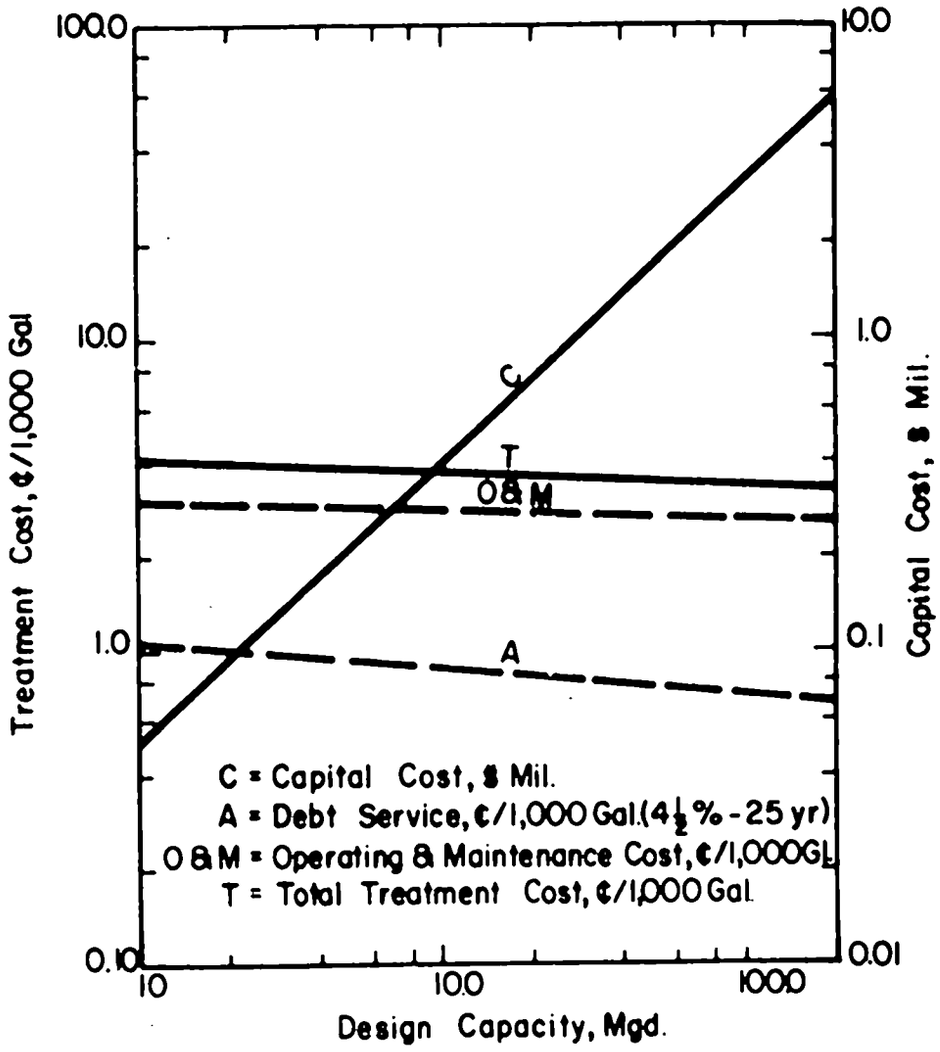


Figure 16. Coagulation and Sedimentation (as from R. Smith [16]) - Chemicals Used: 300 mg/l of Hydrated Lime
50 mg/l of Ferrous Sulfate
1967 dollar value

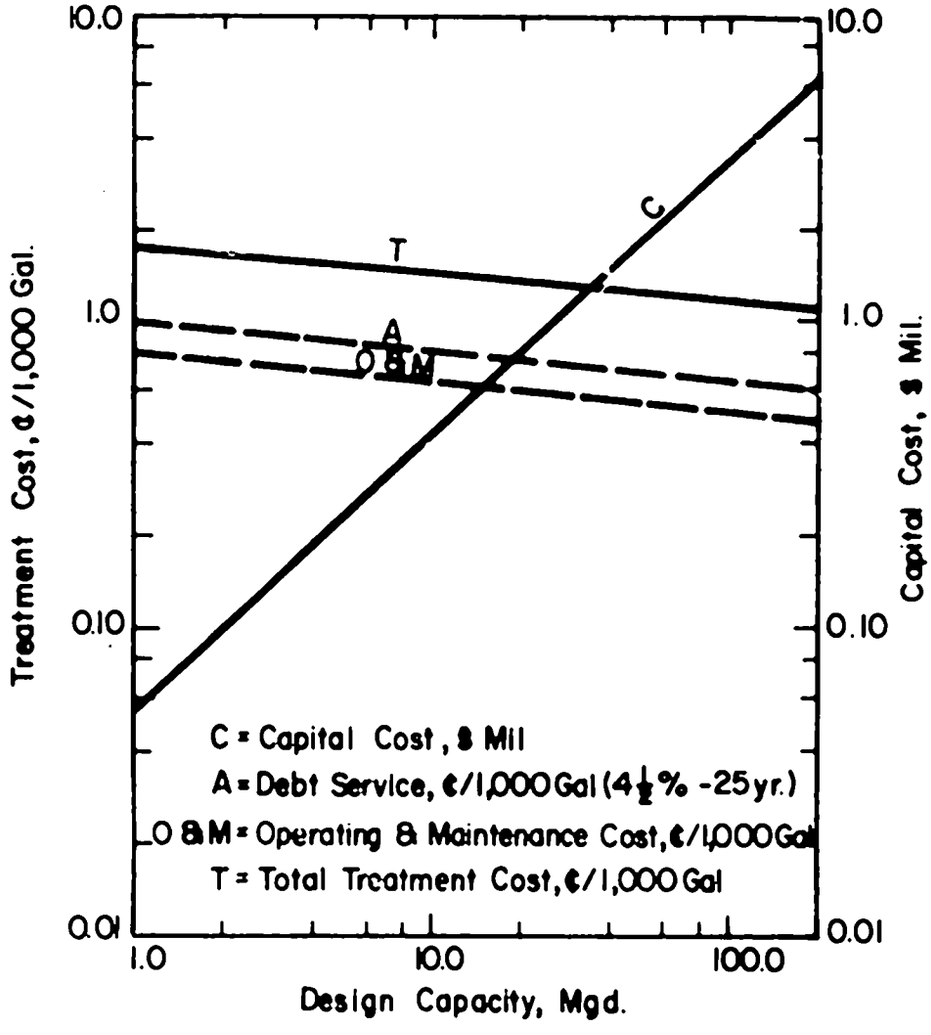


Figure 17. Microstraining (as from R. Smith [16]) - 1967 dollar value

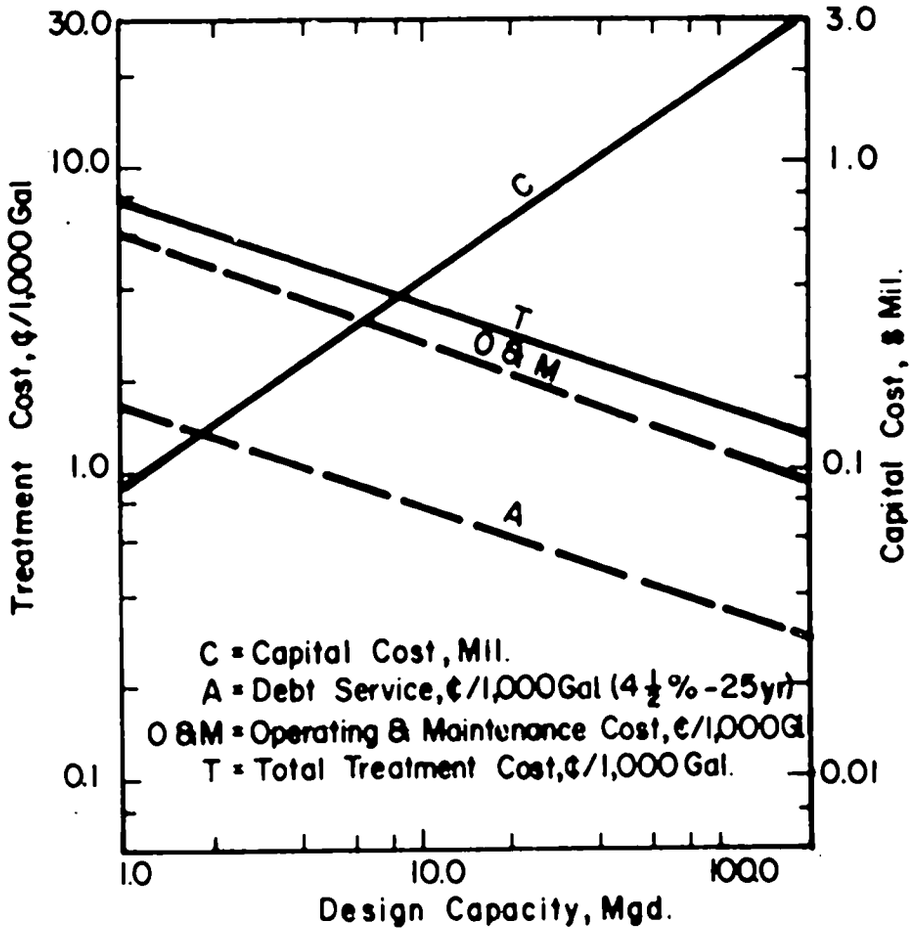


Figure 18. Filtration Through Sand or Graded Media (as from R. Smith [16]) - Design Parameter: 4 gallons/minute/sq ft - 1967 dollar value

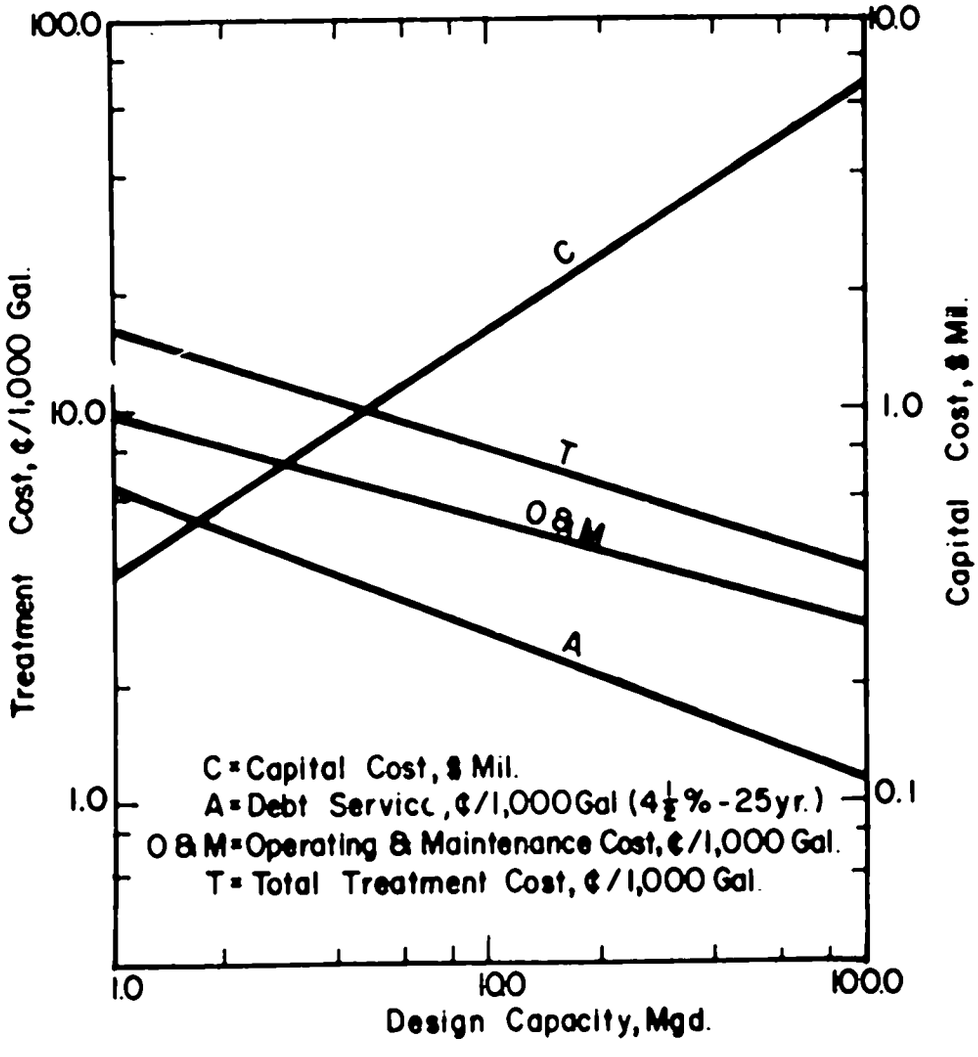


Figure 19. Granular Carbon Adsorption Process (as from R. Smith [16]) - 1967 dollar value

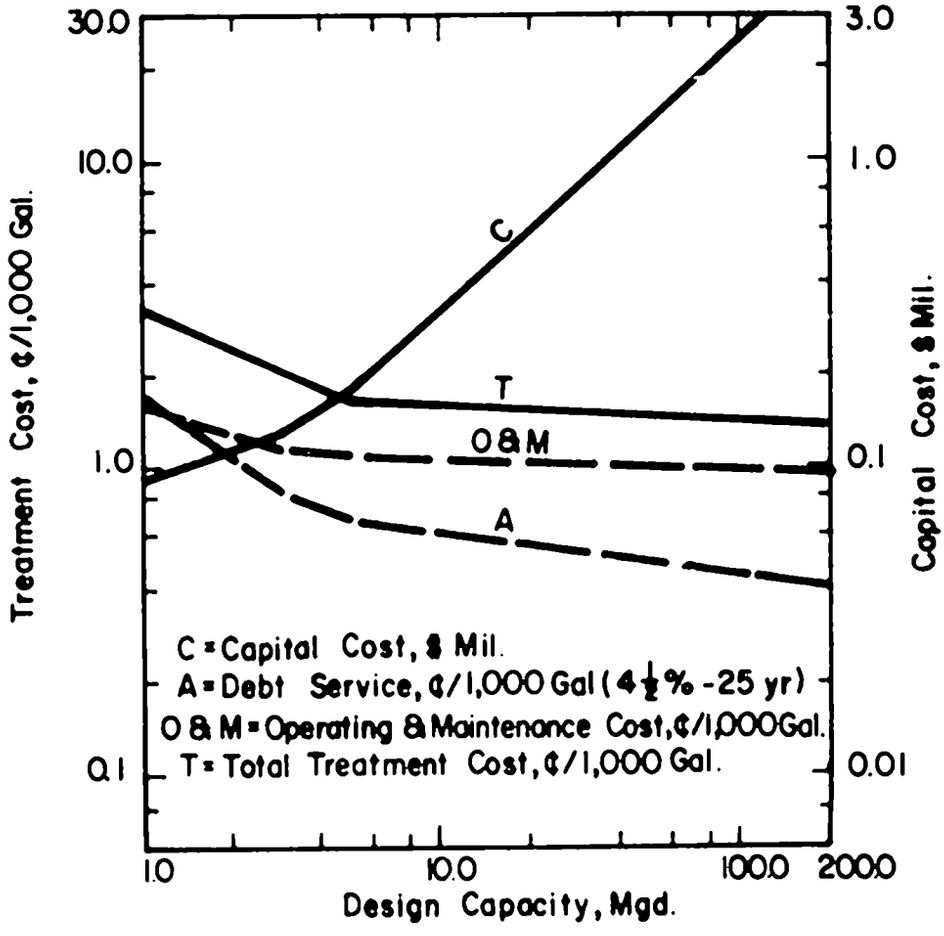


Figure 20. Ammonia Stripping Process (as from R. Smith [16]) - 1967 dollar value

Table 13: Capital costs for desalting plants

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Electrodialysis	[13]	log Y = -0.66 + 0.66 log Z	Y = capital cost Z = plant capacity	$ million (1968 doll.) MGD	for removal of 500 mg/l
	[16]	log Y = -0.30 + 0.67 log Z	Y = capital cost Z = plant capacity	$ million (1967 doll.) MGD	for removal of 90% of TDS
Reverse Osmosis	[13]	log Y = -0.525 + 0.96 log Z	Y = capital cost	$ million(1968 doll.)	
	[30]	log Y = 1.78 - 0.2 log Z	Y = total product-water cost per 1000 gallons	¢ (1970 dollars)	1 year-life membranes (present technology)
	[30]	log Y = 1.66 - 0.14 log Z	Z = plant capacity	MGD	2 year-life membranes (present technology)
		log Y = 1.64 - 0.13 log Z			5 year-life membranes (projected technology)
Ion Exchange	[32]	Z = 1 MGD 0.13<Y<0.18	Y = capital cost	$ million (1968 doll.)	10 year-life equipment
		Z = 5 MGD 1.3<Y<2.0	Z = design size of the installation	MGD	3 year-life resin
Brine Disposal by evaporation ponds (traditional type)	[30]	1 MGD Brine: 5.5<Y<15.5 at Fort Morgan (Colo.): Y = 9.2 5 MGD Brine: 27<Y<77 at Fort Morgan (Colo.): Y = 46	Y = capital cost	$ million (1970 do l.)	6 year lining life Ratio of desaltation = 1/1
Brine Disposal by Spray Evaporation	[31]	The total capital cost is found to be from 15% to 40% less than that of usual evaporation ponds.			6 year lining life 15 to 18 nozzles per 1000 ft^2

(a) "log" stands for "logarithm base 10"

Table 14: Operation and management costs of desalting plants

Treatment Unit	Source	(a) Regression Equation	Variables	Units	Limitations
Electrodialysis	[13]	log y = 0.90 - 0.210 log Z	y = O & M cost per 1000 gallons	¢ (1968 Dollars)	for removal of 500 mg/l of TDS
			Z = plant capacity	MGD	
	[16]	log y = 1.01 - 0.12 log Z	y = O & M cost per 1000 gallons	¢ (1967 dollars)	
			Z = plant capacity	MGD	
Reverse Osmosis	[13]	y = 31	y = O & M cost per 1000 gallons	¢ (1968 dollars)	1 year-life membranes
Ion Exchange	[32]	Z = 1 MGD 14 < y < 17	y = O & M cost per 1000 gallons	¢ (1968 dollars)	H_2SO_4:24$/ton
		Z = 5 MGD 58 < y < 78	Z = plant capacity		CO_2:60$/ton
		the range of y corresponds to a range of 40$/ton to 90$/ton for the price of [illegible]			

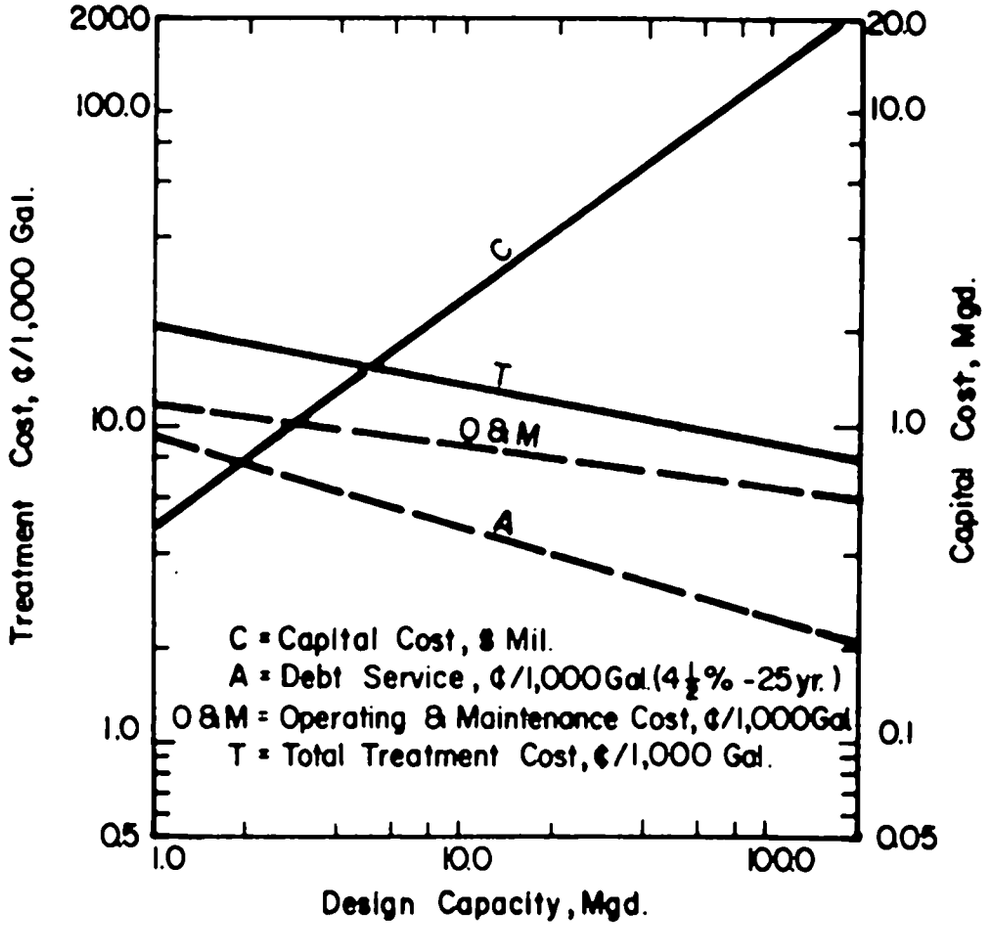


Figure 21. Electrodialysis Process (as from R. Smith [16]) - 1967 dollar value

Supplement: Cost of Brine Disposal

No average cost model is available at the present time; however, the figures presented below are believed to be of interest in the rough evaluation of brine disposal costs.

a) Deep Well Disposal

The cost of brine disposal by deep-well injection is primarily dependent upon disposal volume and well-depth as far as capital costs are concerned, and on injection pressure as far as O & M costs are concerned. This is, assuming that the geology at the well location is favorable, i.e., no particularly tough engineering problems are likely to arise.

The Office of Saline Water, U.S. Department of the Interior, selected seven inland United States communities as examples for the design of deep well disposal facilities for desalination waste brines. The design bases and the cost summaries are shown in Table 15. If the geology is favorable, and if the deep well is carefully designed and operated in a non-negligent manner, this disposal method is generally considered to be a feasible and practical one [30].

b) Solar Evaporation Ponds

First, one should note that the building of solar evaporation ponds is, but for a few exceptional cases, forbidden by many state regulations. When ponds are permitted, they are required to be lined with no less than a 30 mm thickness of plastic sheeting (Texas, Colorado, New Mexico). The 30 mm liner has, accordingly, been adopted as the basis for design and cost estimates, with an installed cost estimated at 30 cents per square foot (1969 dollar-value). Considering the extensive acreage needed for efficient evaporation, one understands

Table 15: Summary of estimated costs of deep-well disposal of desalination wastes [1]

	Clinton (Okla.)	Freer (Texas	Ark. City (Kansas)	Midland (Texas)	Kermit (Texas)	Ft. Morgan (Colo.)
Design Bases:						
Product volume (MGD)	10.0	0.65	7.0	5.0	20.0	3.0
Waste volume (MGD)	4.41	0.214	1.27	0.8	8.65	1.0
Well depth (Ft)	5000	2700	4000	5000	4050	6150
Injection pressure(psi)	1365	300	400	850	900	2200
Costs: (2)						
Well construction cost ($ x 10^6)	1.549	0.075	0.195	0.157	0.650	0.787
Annual operation and maintenance cost ($/Yr x 10^6)	0.492	0.031	0.034	0.082	0.394	0.219
Total annual cost ($/Yr x 10^6)	0.914	0.057	0.063	0.120	0.653	0.384
Product Water Bases:						
Total unit capital cost ($/gal/day)	0.480	0.369	0.080	0.080	0.121	0.591
Total unit operating cost ($/1,000 gal)	0.251	0.238	0.025	0.066	0.089	0.351

(1) All costs in 1969 dollar-value
(2) Interest rate between 4% and 5%; life of a well = 8 years.

why the brine disposal by solar evaporation is considered to be unfeasible in almost every case. Roughly, one can expect the cost of disposing of brines by the means of ponds to be between $1.0 and $3.0 per thousand-gallon of brine, while the cost of water desalination per se is generally less than 50 cents per thousand-gallon [30].

However, recent reports ([30], [31]) show that, if the method of spray evaporation is used, substantial economies appear, as shown in the following tableau [30]:

Station	Brine evaporation cost $/1000 gal. (1969 dollar-value)	
	Spray Evaporation	Solar Evaporation
Phoenix (Arizona)	0.41	0.77
Roswell (New Mexico)	0.67	0.87
Midland (Texas)	0.66	0.94
Aberdeen (S. Dakota)	1.80	2.90

It is anticipated, [31], that this method of disposal, when technically bettered, might compare favorably with the deep well method of disposal.

PART III: THE FORT COLLINS WATER SYSTEM

A. General

Fort Collins is the county seat and largest city in Larimer County, and eighth largest city in the State. It is located on the Cache La Poudre River along the historic Overland Trail near the foothills of the Rockies, five miles west of Interstate 25 on U. S. Highway 287 and Colorado Highways 1 and 14.

The City is a trade center in one of Colorado's major agricultural areas. Its economy is heavily oriented towards agriculture, though the mineral and industrial contribution is significant. It is the home of Colorado State University, which also has a strong influence on local economy and business. Fort Collins is a home-rule City with a Council-Manager form of government and has both City and regional planning commissions to aid in planning for orderly growth and development of the area. Since 1950 the City has experienced a rapid growth in population which is expected to continue in the future. In 1971, the population is estimated to be around 45,000.

Population forecasts used in this report are those of the City's consultants. There are some indications at present that the forecasts may be on the low side. New forecasts, however, would only be additional guesswork at this time. Acceleration of growth can be at least partially absorbed by the high per capita demands assumed for the future in the city water requirement figures. The rapid growth experienced by the City in the past decade has made it necessary for the City to develop and carry out an ambitious and expanded plan of water plant development.

B. Source of Supply

Water for the Fort Collins municipal system comes from direct flow diversion rights on the Cache La Poudre River and from releases from Horsetooth Reservoir, a unit of the Colorado-Big Thompson (C-BT) Project, the latter obtained through ownership of Northern Colorado Conservancy District units. Water from Horsetooth Reservoir can be released to the Cache La Poudre River in exchange for water diverted from the River or can be released directly to a new 15 million gallon per day (MGD) treatment plant constructed in Soldier Canyon west of the City. There is no groundwater supply nor waste-water reuse for the time being.

The Colorado-Big Thompson Project comprises a system of diversion, storage and conveyance works providing for the annual diversion of up to 310,000 acre feet of supplemental irrigation water for lands within the Northern Colorado Water Conservancy District whose offices are in Loveland, Colorado. Principal features of the Project include Lake Granby which stores water diverted from the Colorado River and tributaries, Shadow Mountain Reservoir which receives water pumped from Lake Granby, and Grand Lake, adjacent to Shadow Mountain Reservoir, which diverts water through the Alva B. Adams Tunnel. On the east slope the diverted water is released from the tunnel near Estes Park and conveyed through a series of power plants, storage reservoirs, and canals to users within the District. The Green Mountain Dam on the Blue River is also part of the Project, providing replacement storage for Colorado River users and power generations for repayment of project cost (Fig. 26).

Water from the Colorado-Big Thompson Project is delivered on the basis of estimated requirements of the eastern slope, estimates of runoff for the season and availability of water to the project. The right

to receive project water is signified by ownership of C-BT acre-feet units. There are 310,000 units, corresponding to the maximum amount of water than can be delivered to users in any year. The annual yield of a unit is termed its "quota" and may be as high as one acre foot, or as low as 0.60 acre foot. The quota is established by the District each year prior to the irrigation season and can be adjusted in either direction after the season starts if conditions are materially different than anticipated.

Analysis of District operations since 1954 indicates that the average declared quota is 0.78 acre feet, which represents the yield of one unit of C-BT water. The average delivered yield is 0.74 acre feet per unit, slightly less than the declared quota because of non-payment of assessments by some users and the fact that some users do not request all of their water to be released in some years. In the case of Fort Collins, for example, it has been noted that the yield is greatest in dry years such as 1958, 1963, and 1966, whereas it is lowest in wet years such as 1957, 1961, and 1965. This reflects the advantages derived from the operations of carry-over storage facilities in which years of abundant supply are used to fill the reservoirs so that supplemental water is available in years of low surface runoff. To the City of Fort Collins, it means that storage water is available to supplement direct flow rights during periods of high demand when direct flow rights are not adequate.

Water rights owned by the City, including C-BT, direct flow rights from the Cache La Poudre River at the existing diversion structure, and ownership in various ditch companies which provide water to the Fort Collins area, are summarized in Table 16.

Water from the North Poudre Canal Company can be used by exchange from Horsetooth Reservoir or by release from mountain reservoirs. Water available through ownership in the other ditch companies must be transferred or pumped for use in the municipal system, excluding that which is presently used for parks, cemetery and golf course irrigation.

Table 16: Fort Collins water rights (as of July 1971, from the City Engineer's office)

1. Colorado-Big Thompson Project (C-BT)

Number of shares: 9,378
Yield (AF/share): .78

Volume (AF): 7277

2. Direct Flow from Cache la Poudre River

Name	Date	Priority No.	Amount cfs
Yeager	June 1, 1860	1	3.50
Pioneer	March 1, 1862	5	2.15*
Boyd & Freeman	March 15, 1862	6	7.00
Pioneer	September 15, 1864	12	2.78*
Brown	May 1, 1865	14	4.50
Total			19.93

*Diversion period limited to April 15 to October 15 of each year without replacement. From October 15 to April 15 of next year, water may be diverted on these rights provided it is replaced with an equal amount of water from sources other than waters which are naturally tributary to the Cache La Poudre River.

Volume (AF): 8240

3. Ownership in Irrigation Ditch Companies

	Shares	Yield AF/Share	Volume AF
Mountain & Plains	31.0000	1.720	53.32
North Poudre	769.7500	5.570	4287.51
Pleasant Valley and Lake Canal	47.4547*	55.550	2636.11
Taylor & Gill	.9625	48.500	46.68
New Mercer	9.7205	28.800	279.95
Larimer County No. 2	8.5646	44.580	381.81
Arthur Ditch	145.2404	4.420	641.96
Harmony Lateral	1.7500	4.4571	7.80
Warren Lake	12.7133	10.0000	127.13
Dixon Lateral	3.8000	4.4474	16.90
Sherwood Lateral	.4375	4.3429	1.90
		Volume (AF):	8481.07

TOTAL WATER RIGHTS VOLUME (AF): 23,998

*this figure includes 286.75 assigned to North Poudre

C. Physical Facilities and Treatment Plants (Figure 22)

Diversions from the Cache La Poudre River to the present treatment plant are made by a concrete diversion structure located in Section 32, Township 9 North, Range 70 West and conveyed approximately one quarter mile to a 17 million gallon per day (MGD) treatment plant situated on the south bank of the river. This treatment plant is of the type TP2 (see flow chart in Part I), where the following operations take place: a chemical coagulation, a sedimentation, a filtration through both sand beds and microstrainers, and finally a chlorination.

Treated water from this plant is carried from the plant to the Bingham Hill and Soldier Canyon clear water storage reservoirs and directly to the distribution system. The Bingham Hill reservoir is a rectangular reinforced concrete clear water storage structure with a capacity of 4.5 million gallons located approximately 3.5 miles northwest of the City near La Porte.

At Soldier Canyon, about two miles west of the City, there are three concrete storage facilities with a combined capacity of 37 million gallons. Additional storage, as well as some surge control, is provided by a steel standpipe located about 1.5 miles northwest of the city limits having an operating depth of 64 feet, providing 150,000 gallons of storage. The total clear water storage available to the city is 41.65 million gallons.

Since September 1970, a second treatment plant is operational at Soldier Canyon. It is of the type TP1 (see flow chart in Part I) with only microstrainers and chlorinators. This plant is supplied by releases from Horsetooth Reservoir through the Soldier Canyon outlet works and can be operated at a rate of 24 MGD during periods of peak

demand. Treated water goes to the Soldier Canyon reservoirs and then directly to the distribution system through a recently constructed primary transmission line. Operation of the new Soldier Canyon Plant brings about two significant advantages to the City; the elimination of winter diversion problems and the minimizing of the minor losses incurred by conveying Colorado-Big Thompson water in the Cache La Poudre River.

Studies of the Fort Collins water supply system made in 1964 by J. T. Banner and Associates indicated that in several major portions of the system adequate operating pressures were not being sustained under normal and peak demand conditions. To remedy these deficiencies the city in 1965 installed a series of primary transmission lines from the Soldier Canyon storage reservoirs to the distribution system along with several other primary feeders. These improvements provide adequate operating pressures under present and anticipated future peak demand conditions for a population of approximately 70,000.

The sewage of the city of Fort Collins is treated in two separate waste water treatment plants. The older one, situated on the Cache La Poudre River, just north of Highway 14, is a 5 MGD primary secondary treatment plant (PST type); it operates with trickling filters. The newer one, east of the Cache La Poudre River, on Drake Road, is also a 5 MGD primary secondary treatment plant, but it operates with the activated sludge principle. In both cases, the sludge is mechanically thickened and digested, dried on beds, and is finally disposed of for land-fill. No incineration takes place. There is no pretreatment of the industrial waste waters. Also, it should be noted that, in some times of the year, in summer, the sewage is directly dumped into an

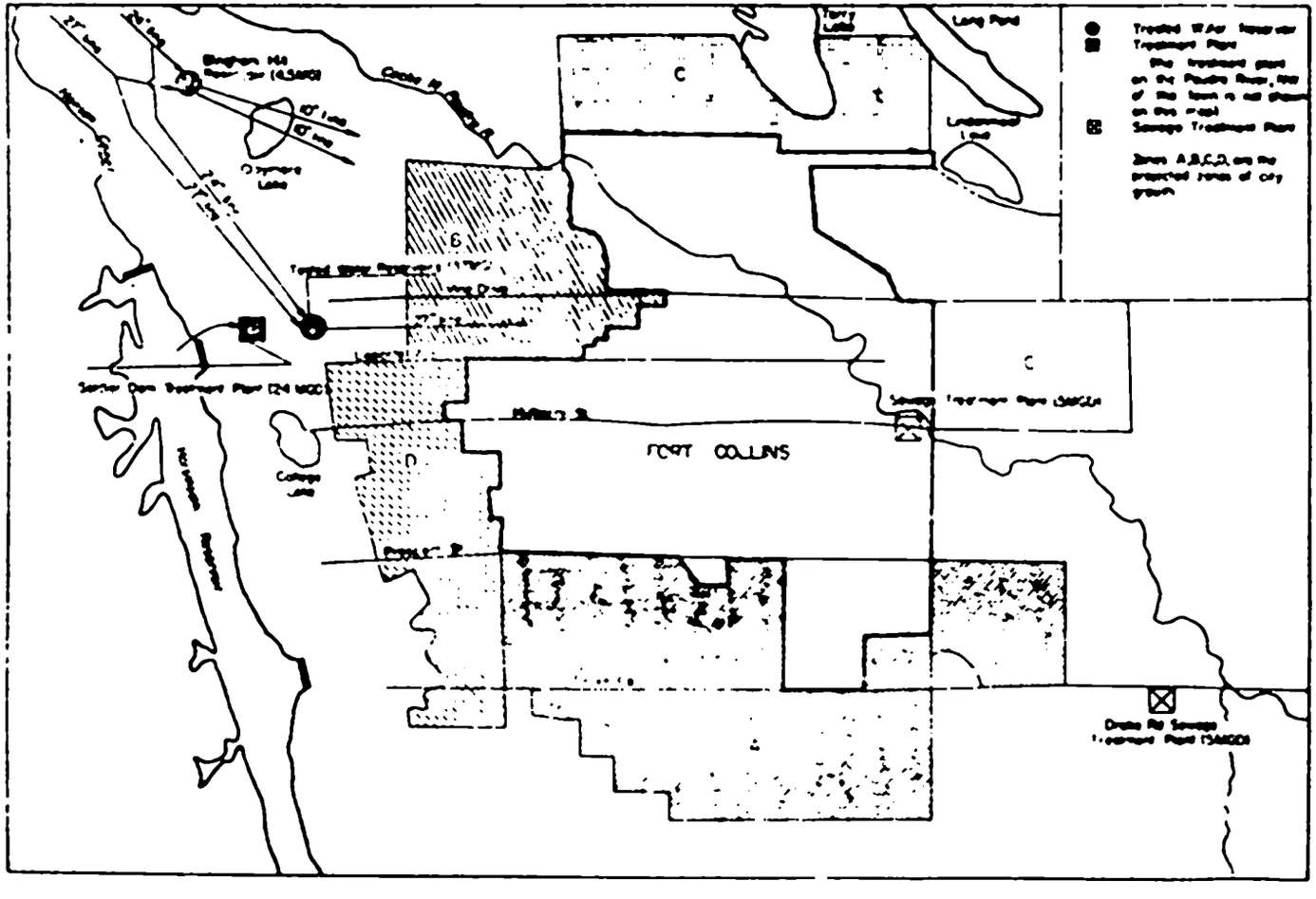


Figure 22. The Present System of Fort Collins

irrigation canal, the Fossil Creek. With the quality regulations on sewage disposal becoming more stringent, this permissiveness might become illegal soon.

D. Water Requirements

It is first interesting to note from Table 17 that the per capita consumption of water in Fort Collins is among the highest in the state, a condition which is the result of distribution system losses, a high percentage of flat rate users and the transient student population. This point is reassuring; in case of necessity, this consumption could be lowered easily, without major harm to the population.

Table 17: Water consumption by Colorado municipalities (as from the Wheeler Report [36])

	Population Served 1000	Water Consumption in Gallons Per Capita Per Day					
		1960	1961	1962	1963	1964	Average
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Denver	664	215	183	222	222	220	212
Aurora	55.7	117	101	125	115	128	117
Englewood	55.7	187	142	184	184	194	178
Boulder	47.0	229	203	230	196	186	209
Arvada	33.9	144	118	145	160	161	146
Fort Collins*	32.8	292	213	263	251	249	254
Longmont	15.7	-	-	-	-	239*	-
Golden	13.2	149	132	142	144	140	141
Broomfield	6.0	129	102	137	146	148	132

* Water use and population served outside City not included.
** Use by the sugar manufacturing plant not included. When the total use is included, the use is 209 g.p.c./day.

From the records of average water use through the year (Table 18), a peak-day demand is determined given the estimated population curve;

it is then possible to predict the maximum daily demand, i.e., the product of the expected population of a given year by the peak day demand. Knowing the present treatment capacity and the treated water storage, one can see from Figure 23 and Table 19 that in 1980 or 1981 the present treatment facilities will no longer be sufficient to meet the peak-day demand. However, they still will be able to meet the peak 7-day daily demand up to year 1986 (that was the conclusion of the Wheeler report [36]).

Table 18: Water use through the year
(as from the City Engineer's Office - 1971)

Period	Water Use in Gallons Per Capita Per Day (gpcd)
Average	300
November through March, inclusive	160
April and October	250
May and September	400
June, July and August	500
Peak 7-day	600
Peak day	700

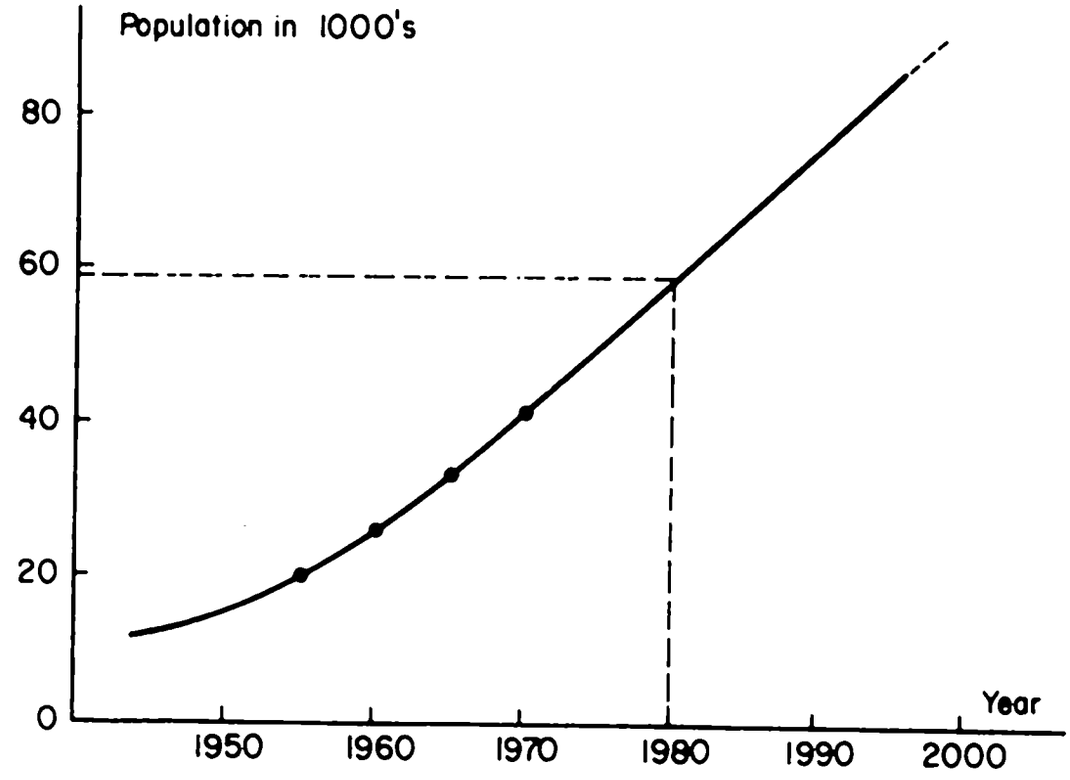
In addition to providing a dependable source of potable water to domestic and industrial users the municipal system must at all times be capable of supplying large quantities of water under pressure sufficient to meet anticipated fire protection requirements. Although the total amount of water used in a year for extinguishing fires may be a negligible part of the annual usage, the rate of demand required during a fire may be so great as to be the deciding factor, in all but the largest systems, in determining the size of some elements of the distribution system and in deriving operating criteria.

Table 19: Future water requirements and present capabilities[1]

Year	Estimated Population	Maximum Daily[2] Demand (MG)	Present Capabilities	
			Peak Daily (MG)	Treated Water Storage (MG)
1970	42,000	29.4	41	41.65
1975	50,000	35.0	41	41.65
1980	59,000	41.3	41	41.65
1985	68,000	47.6	41	41.65
1990	76,000	53.1	41	41.65
1995	84,000	58.8	41	41.65

(1) Source: City Engineer's Office

(2) Based on 700 gpcd on peak day ([36], [37])



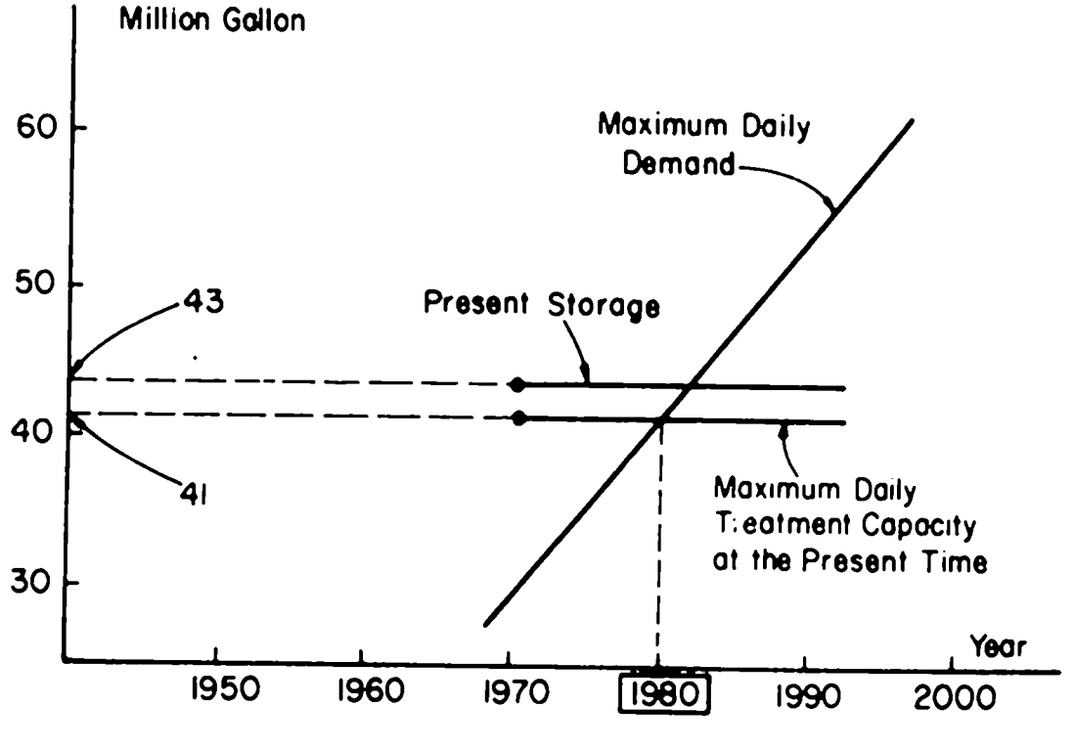


Figure 23. Future Water Requirements vs Present Capabilities

The National Board of Fire Underwriters recommends fire flows which should be available. The required flows vary with the population served and the general structural condition of the system, and for the City of Fort Collins, should provide enough water for a 10-hour fire with not less than 20 pounds per square inch pressure at the hydrant. The rates and amounts of water required under these criteria for the Fort Collins system with various populations are given in Table 20. Studies of the system under present and future conditions have shown that these demands can be met ([36], [37]).

Table 20: Fire demand requirements(1)

Population	Flow (gpm)	Total Quantity Required for 10 Hour Duration (million gallons)
40,000	6000	3.6
60,000	7300	4.4
80,000	8300	5.0
100,000	9200	5.5

(1) As from the Banner Report [37].

As far as water quality is concerned, the City of Fort Collins has adopted the limits recommended by the U.S. Public Health Service along with the Colorado State Department of Public Health [21]. These limits and a range of values representative of a desirable supply are given below:

Characteristics	U.S.P.H.S. Maximum	Colorado Public Health Maximum	Desirable Range
Turbidity	5	5	0 - 1
Color	15	15	0 - 5
Odor	3	3	0
Total Hardness (Calcium Carbonate, mg/ℓ)			10 - 200

Characteristics	U.S.P.H.S. Maximum	Colorado Public Health Maximum	Desirable Range
(Continued)			
pH	-	-	6 - 7.5
BOD (mg/ℓ)	4.0	4.0	-
TDS (mg/ℓ)	500	500	< 400
COL (MPN/100 mℓ)	2	2	-
Nitrate (mg/ℓ)	45	45	-
Chlorides (mg/ℓ)	250	250	-

As a conclusion, one can think that the City of Fort Collins will be supplied with water up to year 1980 on a reliable basis. This reliability is emphasized by the facts that the raw water is obtained from at least two independent sources of supply, that the treated water is supplied to the system through more than two transmission lines designed so that failure of either one would not reduce the total supply capacity by more than 60 percent, and that the provision of clear water storage capacity is equivalent to several days average supply and is at an elevation sufficient to distribute the water by gravity.

However, as stated before, this good supply will not be sufficient for ever and the City is now considering several new potential supply sources for the future.

E. Potential Supplementary Sources

The "plan of augmentation" of the City of Fort Collins has selected five potential supplementary sources. They are:

-- The Michigan Ditch - Joe Wright Reservoir project

-- The Windy Gap project

-- The purchase of additional C-BT units

-- The purchase of additional Ditch Company stock

-- The building of a downstream storage (Fossil Creek Reservoir)

It must be noted that never have groundwater nor reuse of wastewaters been ever considered as potential supplies.

Figure 24 shows the relative situation of these alternatives.

1. Michigan Ditch - Joe Wright Reservoir Project. W. W. Hansen [32] considers this potential supply as "the key to the entire resource development program," based on the following analysis.

"The Michigan Ditch is a collector ditch located in the drainage of the North Platte River, and transports the water collected across the Divide to the Cache La Poudre basin, tributary to the South Platte. Although it is permitted by court decree to divert 6,000 acre-feet per year in this manner, the North Poudre Irrigation Company (NPIC) from whom the right was acquired, historically has diverted an average of about 2,500 acre-feet per year. When combined with the 800 acre-feet of native water which it has the right to store in Joe Wright Reservoir, the company had avilable about 3,300 acre-feet annually from this "source." Consequently, the city was able to acquire title to the ditch and reservoir by guaranteeing the company 3,300 acre-feet annually from whatever sources the city had available. The sources under consideration are several:

(a) Water brought into the basin via the Michigan Ditch,

(b) C-BT water owned by the city and stored in Horsetooth Reservoir,

(c) Shares owned by the city in other irrigation companies. The city owns shares of stock in other companies. By legal proceedings the city's fractional ownership of

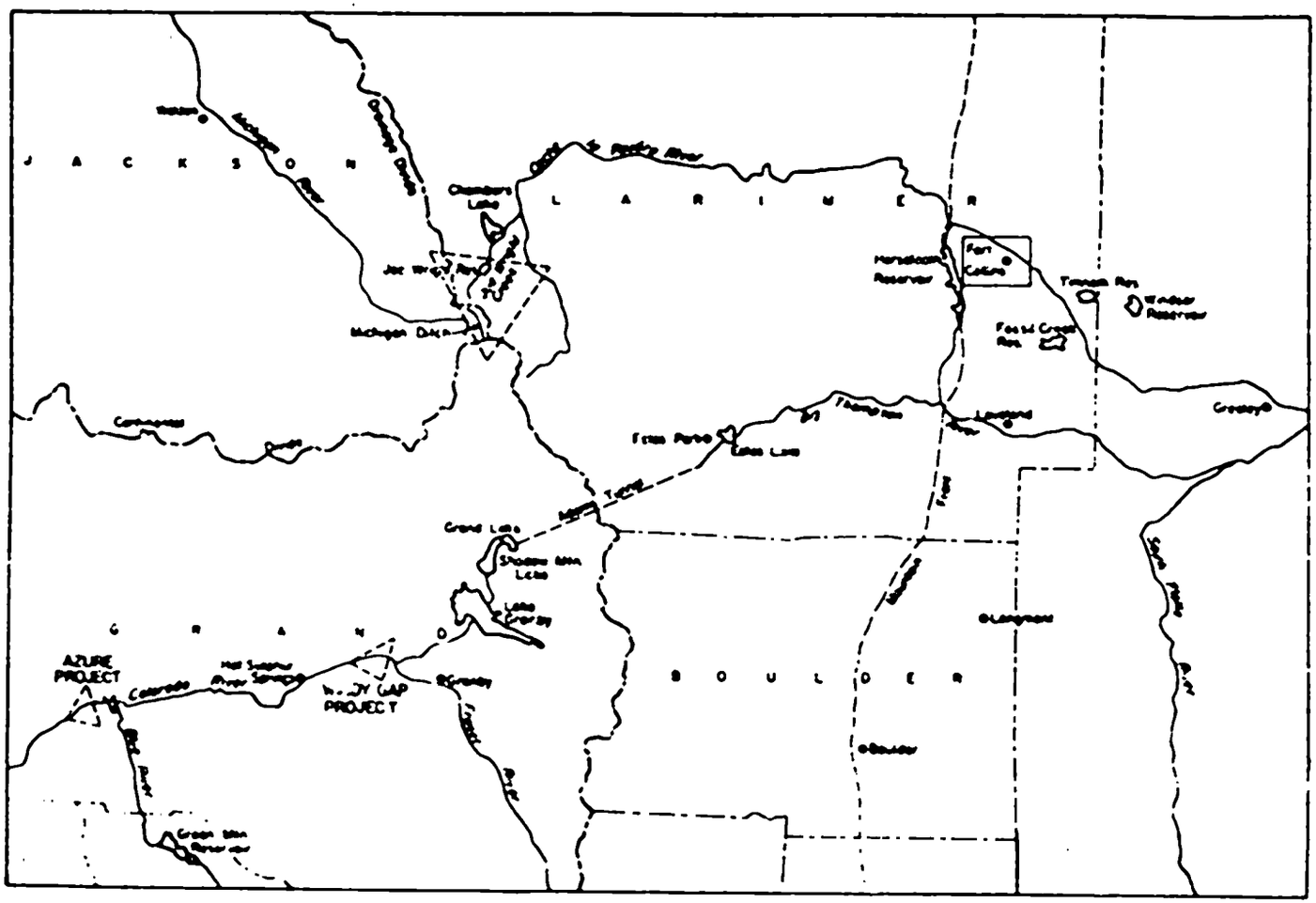


Figure 24. The Water Supply Augmentation Alternatives

the water-rights of one company may be transferred to NPIC; the point at which this portion of the water is diverted may be changed."

This analysis is visualized in Figure 30.

Recent surveys and investigations have shown that the present capacity of 800 acre-feet of the Joe Wright Reservoir can be enlarged to 8,000 acre-feet (provided that the right of storage for the capacity is granted to the city). The inflow to Joe Wright Reservoir would be the sum of the diversion flow from Michigan Ditch and the regular flow of the Joe Wright creek.

2. The Windy Gap Project. The Windy Gap Project is concerned with the common water supply of what is called the "six cities," i.e., Fort Collins, Loveland, Greeley, Longmont, Estes Park and Boulder. The water would be tapped from the Upper Colorado River, at a spot just downstream from its junction with the Fraser River. It then would be pumped back into Lake Granby and, through the use of the Colorado-Big Thompson Project facilities, would be delivered on the Eastern slope for the use of the six cities (see Figure (26)).

A recent economic feasibilitity study by Engineering Consultants, Inc., from Denver ([34], [35]) has drawn the following conclusions:

(a) The project is technically feasible.

(b) An average of up to 58,000 acre-feet of water can be diverted annually from the Colorado River for use by the six cities at a reasonable cost. The cost covers construction of the necessary diversion facilities and provisions of replacement water as required. The Colorado-Big Thompson storage and transportation

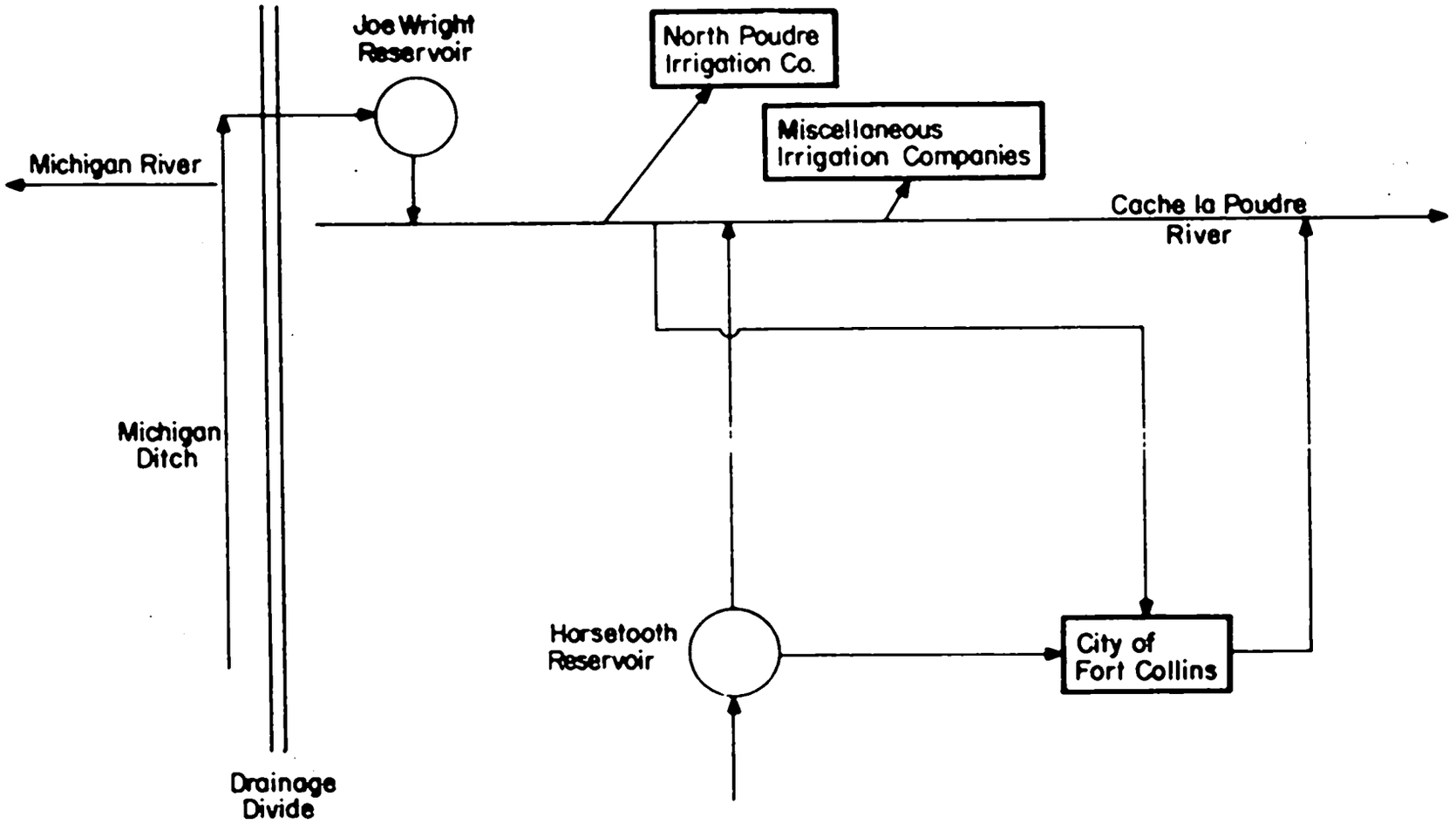


Figure 25. The Michigan Ditch-Joe Wright Reservoir System
(as from W. W. Hansen [32])

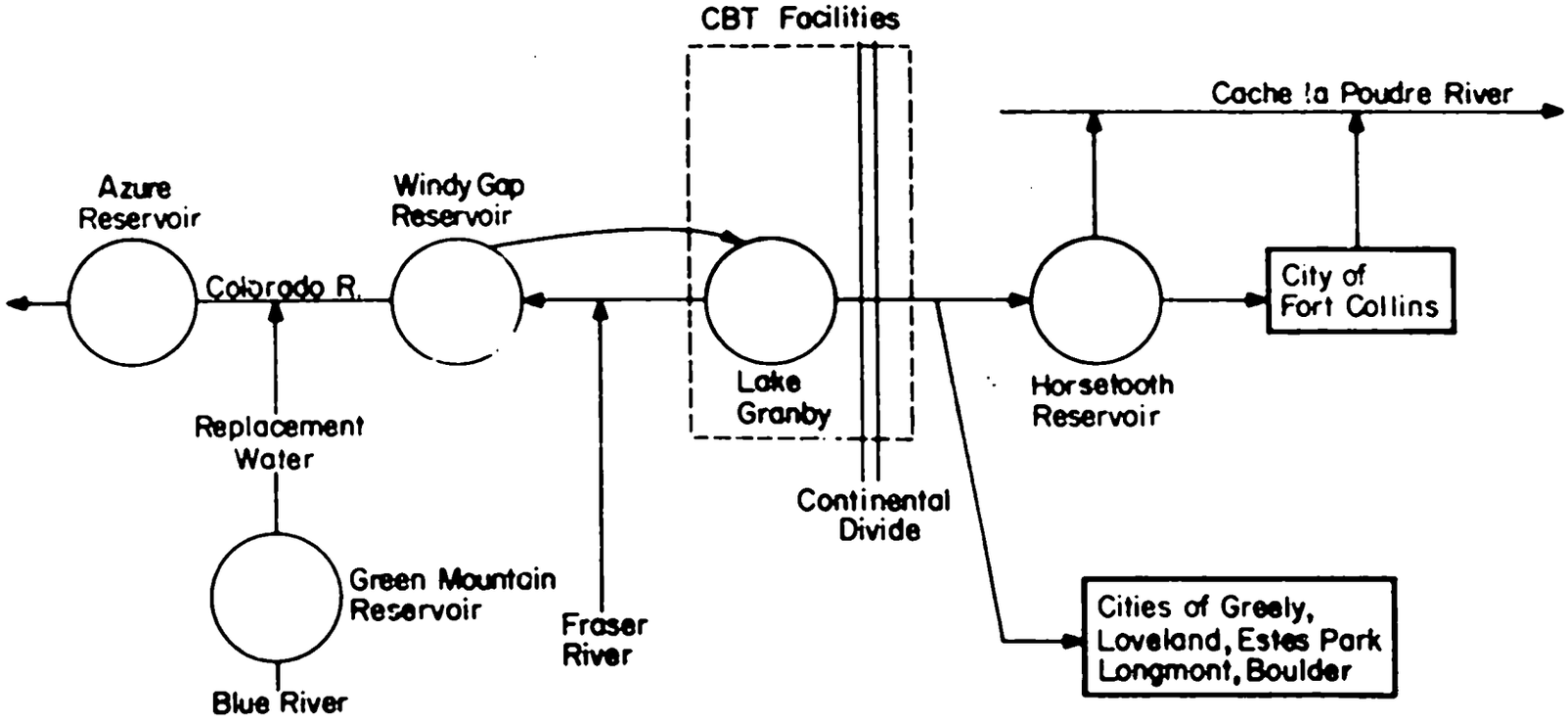


Figure 26. The Windy Gap Project

facilities normally have excess capacity which allows transmission of an annual average of up to the 58,000 acre-feet of water in addition to the CBT water presently carried in the system.

(c) The project can be constructed in either two or three stages, thus making it possible to more nearly match expenditures with growing needs.

(d) The stage one development would provide 25,000 acre-feet of water per year at a cost per acre-feet of water delivered of about $14.50.

(e) The atage two development would provide 33,000 acre-feet of water per year in addition to the 25,000 acre-feet of stage one at a cost of $32.80 per acre-feet. The average cost per acre-feet for the entire 58,000 acre-feet is $24.50. The costs of supplying the necessary replacement water and pumping energy are included in these costs. No benefit from additional power generated in CBT facilities is included.

(f) If unused storage capacity in Green Mountain Reservoir, which is allocated for replacement water for CBT diversions, can be used for six cities replacement, no additional replacement water is required for the first stage development.

(g) The necessary amount of replacement storage required for the second stage development is dependent upon the existence and use of the Azure Project. The Azure Project would be built on the Colorado River, downstream

of its junction with the Blue River, for the water supply of the Northwestern Colorado, if the economy of this region were to develop rapidly in the future (because of the oil industry for instance). With respect to the six cities, the Azure Project has senior water rights. Without the Azure Project, the amount of replacement capacity required is approximately 21,000 acre-feet. That amount is estimated to cost $220,000 per year. With the Azure Project, the required replacement capacity is 50,000 acre-feet, which has an estimated cost of $550,000 per year.

(h) The amount of additional energy generated as a result of the Windy Gap Project water passing through the CBT Eastern Slope generating facilities is approximately 117 million kilowatt hours per year. This compares with 45 million kilowatt hours of energy required to pump the water from the Windy Gap Reservoir to Grand Lake.

3. The Purchase of Additional C-BT Units. The purchase of additional units in the Colorado-Big Thompson Project would provide the city with added flexibility in water supply planning. In particular, this water could be either used directly by the City or traded with the North Poudre Irrigation Company for additional direct rights in the river.

4. Additional Ditch Company Stock. "For a variety of reasons, the city continually adds to its portfolio of shares of stock in small irrigation companies. Although legally permissible, public relations and goodwill considerations have made the city reluctant to use this

water directly by transfer to the citt's point of diversion. Since transfer to other irrigators is slightly more palatable to the other shareholders in these small companies, the most expedient use of this water is transfer to the North Poudre Irrigation Company, in order to reduce the proportion of the compensation delivered from sources more useful to the city" (as from W. W. Hanson [32]).

5. Downstream Storage. At the present time, the city does not have storage rights in Fossil Creek Reservoir, and the connection to the reservoir inlet ditch is only for temporary by-pass of sewage effluent overflow. However, the fullest use of "reusable" water for irrigation can only be obtained by some provision for storage, in order for an exchange of water upstream to take place.

"Since preliminary feasibility studies of enlargement of Fossil Creek Reservoir have proved discouraging, another alternative has been considered which would create space in the reservoir for the city. Three important irrigation companies won the whole storage-right in Fossil Creek Reservoir and have a direct connection to several other reservoirs," among them the Timnath Reservoir (Figure 27). The alternative, then, is enlargement of these two reservoirs and transfer of the three companies storage to them. This procedure would make space available in Fossil Creek Reservoir for the city's use - that would be beneficial for both parties, the irrigators and the city.

6. Water Reuse. Although this alternative has been constantly rejected by the city planners, it should be kept in mind as a potential supplementary supply. A recent study on the sanitary situation of the Larimer County [42] actually points out its importance for the future

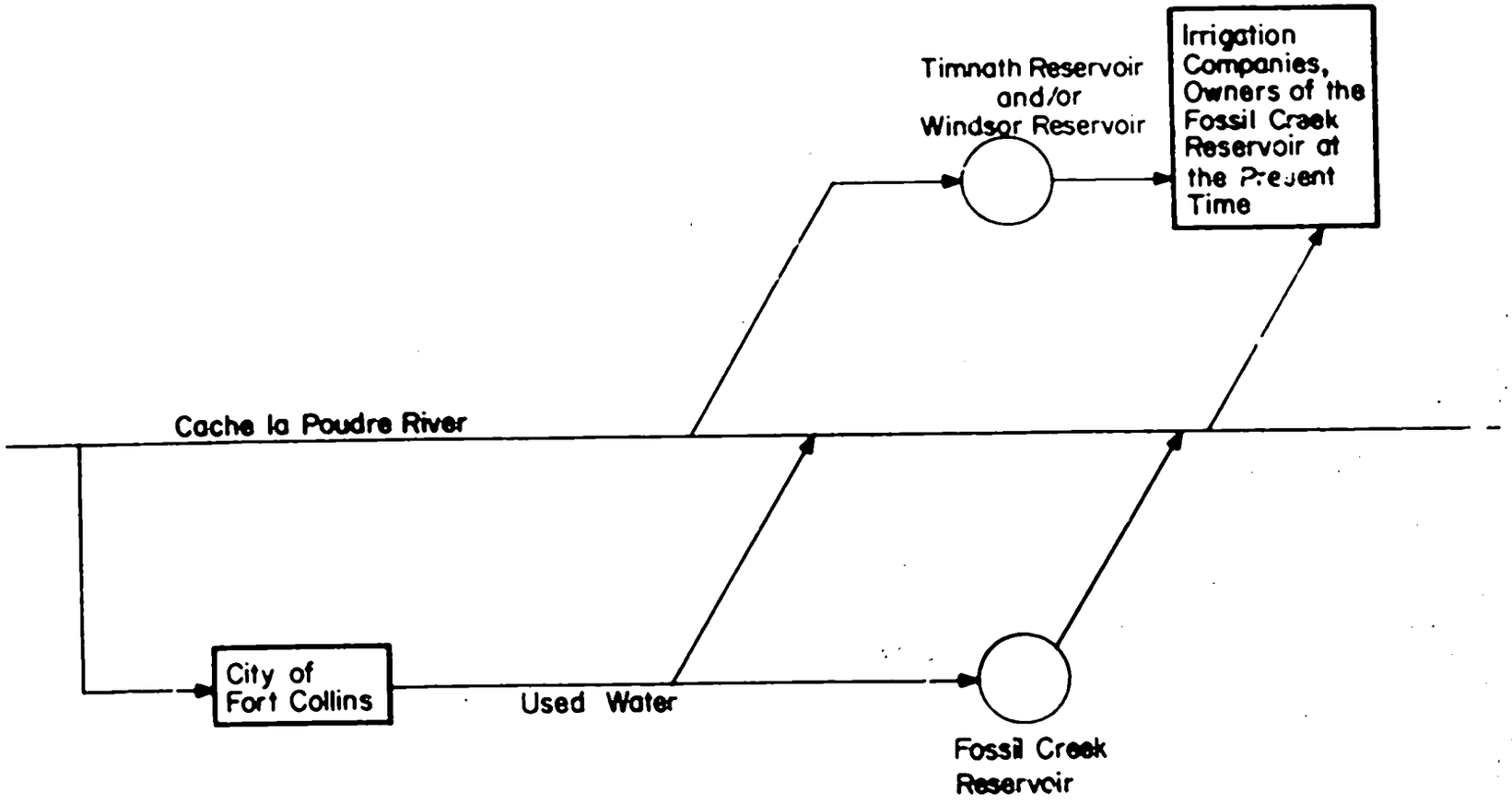


Figure 27. The Fossil Creek Project (downstream storage)
(as from W. W. Hansen [32])

of the region and suggests that the waste waters should be soon used for groundwater replenishment.

*

* *

In this section entitled "The Fort Collins System" the present physical water system is first described: the present supply sources, the treatment facilities and the water requirements, the similarity with the general model of Part I of this study. One conclusion is that, up to year 1980, no major problem of supply is likely to arise. The potential supplementary supplies have then been examined and it was found that quite many interesting alternatives are of consideration. It must be emphasized, though, based on past and present experience, that the final choices and decisions are likely to be of a political and legal nature, rather than a technical one. Should that prevent the engineer from trying to determine the best plan of development, based on technical and economical criteria? The answer is no. On the contrary, with the help of the three first parts of this study (general physical model, cost models, particular system of Fort Collins), it can be possible for the city engineers to compute optimal plans of augmentation of the water supply, each of them corresponding to each foreseeable set of political decisions.

PART IV: MODELS VS. ACTUAL DATA

In order to make the eventual user of the cost models, (as developed in Part II of this study), feel more confident, these models are confronted with some actual data. A maximum discrepancy of 30 percent between the values computed from the models and the actual values, is generally considered a criterion for the reliability of the models. The two control areas that have been chosen are Southern California, because data was readily available, and Denver, because of its proximity to Fort Collins.

A. Capital Costs in Southern California

Data source is "an Economic and Engineering Analysis of Municipal Wastewater Renovation" by Porter et. al., [18], where costs are those of the Los Angeles area in 1968.

The Sewerage Construction Cost Index is found to be (see Table 5):

-- 135 at Los Angeles in 1968

-- 121 for the U.S. average in 1967

In order to adjust these actual costs to the U.S. average in 1967, they have to be multiplied by a factor of 0.89 ($= \frac{121}{135}$). Once this is done, it is possible, as shown in the following tableau, to compare actual costs and values computed from the models, for several types of treatment or treatment units. The size of the plant is arbitrarily chosen to be 30 MGD. Also "%" refers to the difference between the actual cost and the computed cost, expressed as a percent of the computed cost.

Capital costs ($ million)

Type of Treatment	Los Angeles Area	Model	%
(1)	(2)	(3)	(4)
Primary	1.7	1.4	21

Capital costs ($ million)- (Con't)

Type of Treatment (1)	Los Angeles Area (2)	Model (3)	% (4)
Primary and Secondary (Activated Sludge)	9.1	7.9	15
Carbon Treatment (Tertiary Treatment)	2.25	3.0	25
Electrodialysis	5.4	5.6	4

From column 4 of the above tableau, the maximum error between the actual data and the costs computed from the models is found to be 25 percent. This value is very acceptable in most feasibility studies or preparation of bond elections; accordingly, the models should be considered reliable, at least for such purposes.

B. The O&M Costs of Sewage Treatment at Denver

Data source is the 1972 annual budget for the Metropolitan Denver Sewage Disposal District No. 1 [38]. The costs are expressed in 1970 dollar value, at Denver.

The ENR Construction Cost Index (labor and materials) is found to be (Table 6):

-- 1100 for the U.S. average in 1967

-- 1199 for the Denver area in 1970

In order to be adjusted to the U.S. average in 1967 these actual costs at Denver have to be multiplied by a factor 0.915 ($= \frac{1100}{1199}$) . Denver costs and costs computed from the models are shown in the following tableau. Here, again, "%" refer to the difference between actual and computed costs, expressed as a percent of the computed value. It is remarkable that this difference is found to be less than

10 percent, a fact that should reinforce the reader's confidence in the developed models. Also, as stated in Part II of this study, it can be seen from column 2 that labor accounts for about 60 percent of the total O&M costs.

	O&M costs (cents/1000 gal)		
Type of Treatment	Denver	Model	%
(1)	(2)	(3)	(4)
Primary (26 MGD)			
Labor	1.38		
Others	1.00		
Total	2.38	2.50	5
Secondary Activated Sludge Plant (113 MGD)			
Labor	1.38		
Others	1.28		
Total	2.66	2.90	9

One last remark concerning sludge disposal is of interest here. In 1970, at the Metropolitan Denver Sewage Disposal District No. 1, the total O&M expenditures for sludge disposal have been 2.165 million dollars; at the same time, the total O&M expenditures for the district have been 3.490 million dollars. In other words, sludge disposal represents about 60 percent of the total O&M costs of the plant! That emphasizes the importance of a very careful planning of sludge disposal.

PART V: CONCLUSION

This study is intended to be a working tool in the hands of experienced engineers and planners in charge of the operation or development of the water supply of large urban areas. The systematic aspect of water supply has been stressed, along with the concept of waste water reuse.

In Part I, the general structure of a water supply system is considered. Particular attention has been devoted to the presentation of this system, in order to facilitate its eventual programming. It was then tested, in Part III, how an actual water supply system, namely that one of Fort Collins, can fit in the general structure analyzed in Part I. The choice of Fort Collins as a control area also permitted to collect, in one place, most of the available information that would be needed if the present study were used to plan the development of this rapidly growing region.

It is the purpose of Part II of this study to give reliable estimates of the capital and O&M costs involved in water treatment, where economies of scale are found to be a predominant phenomenon. These models were then confronted, in Part IV, with some actual cost data, and a maximum error of 25 percent was found.

It can be the subject of a further study to write a computer program for the planning of the water supply of a large urban area. The optimization criterion could be the minimum cost of facilities construction and overall operations, over a long period of time, say 50 years. The decision variables may be, for example, the types of treatment, the timing of expansion, the magnitude of capacity expansions of the plants, etc. In order to account for the economies of scale, a nonlinear programming method of optimization would necessarily have to be used.

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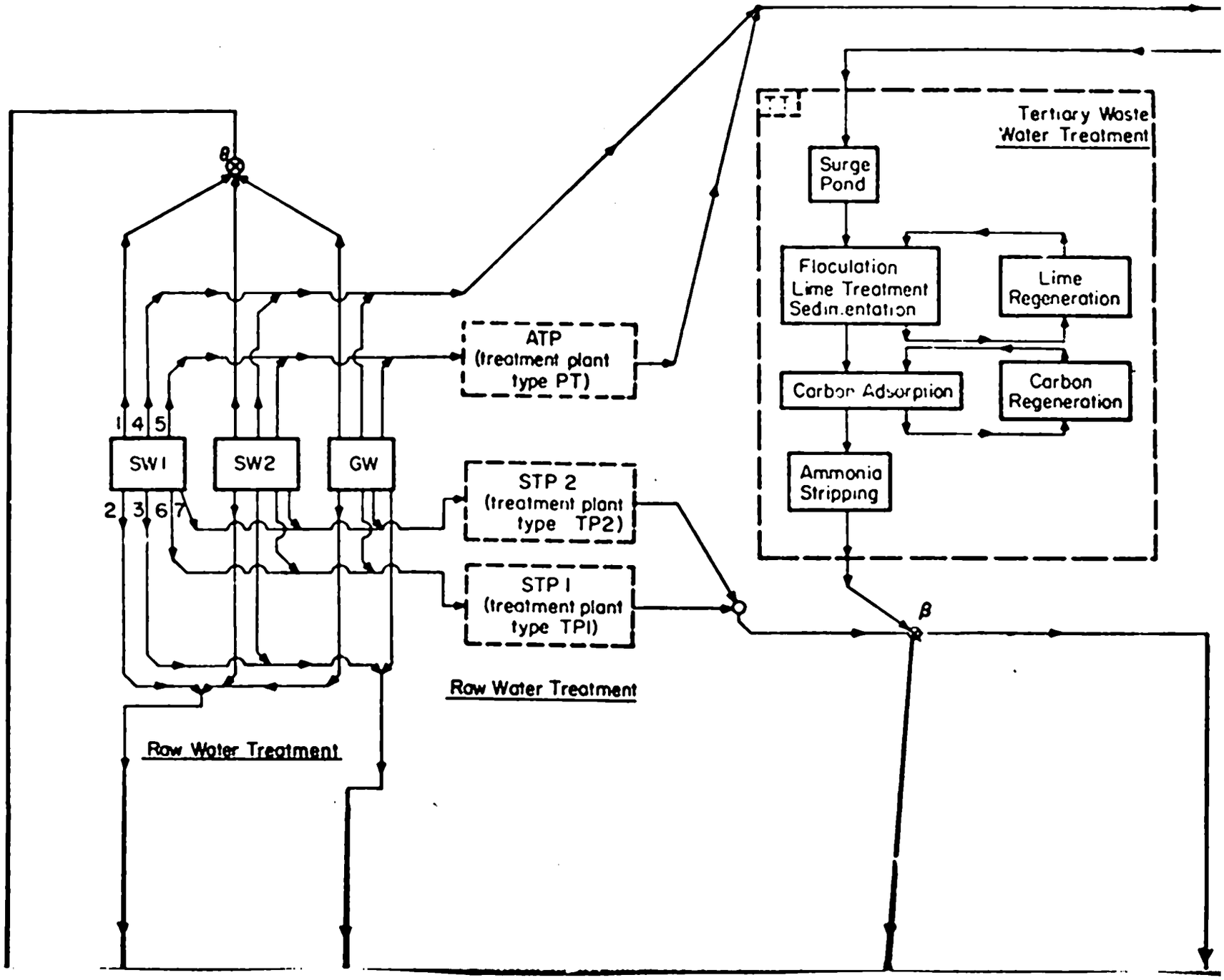
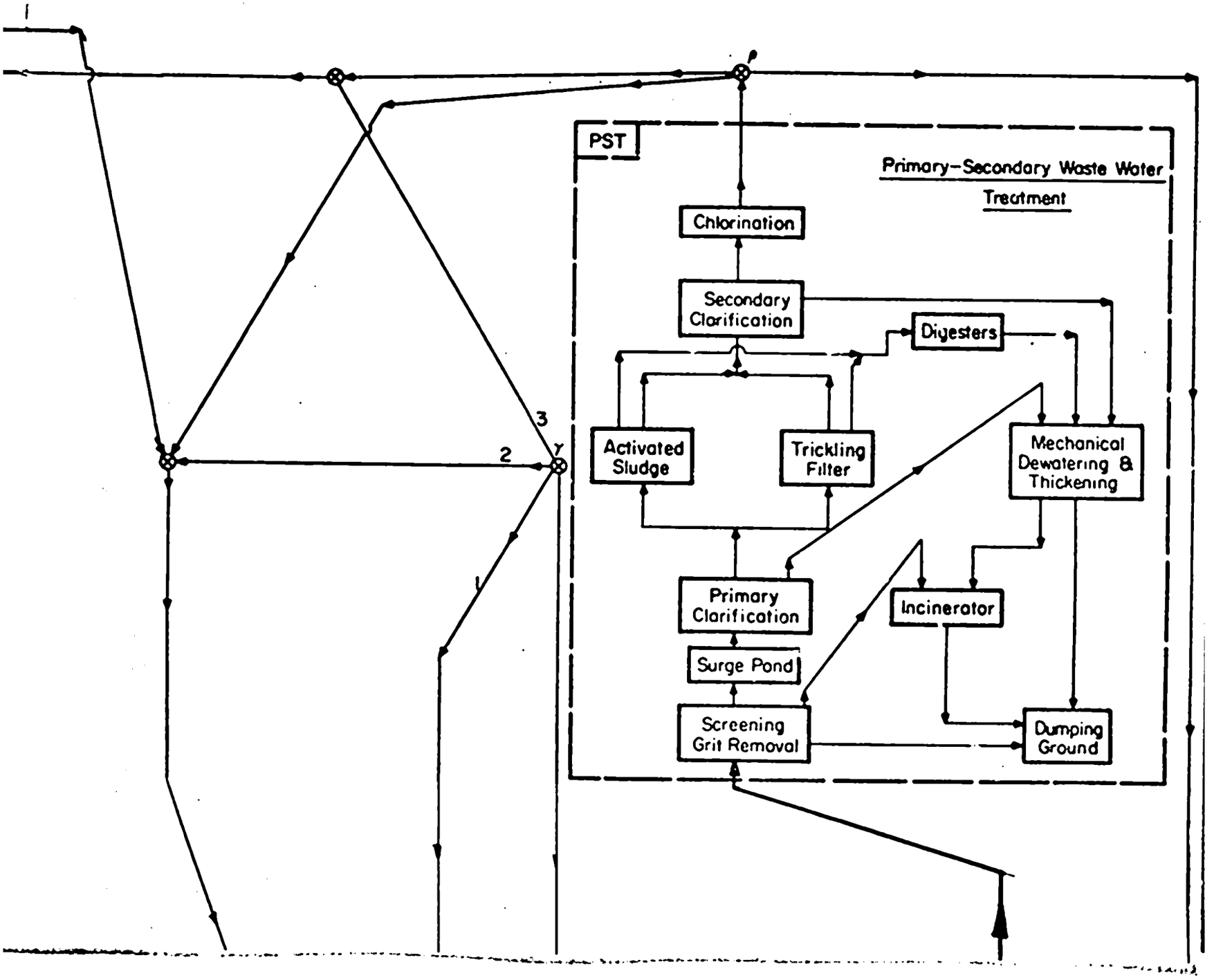
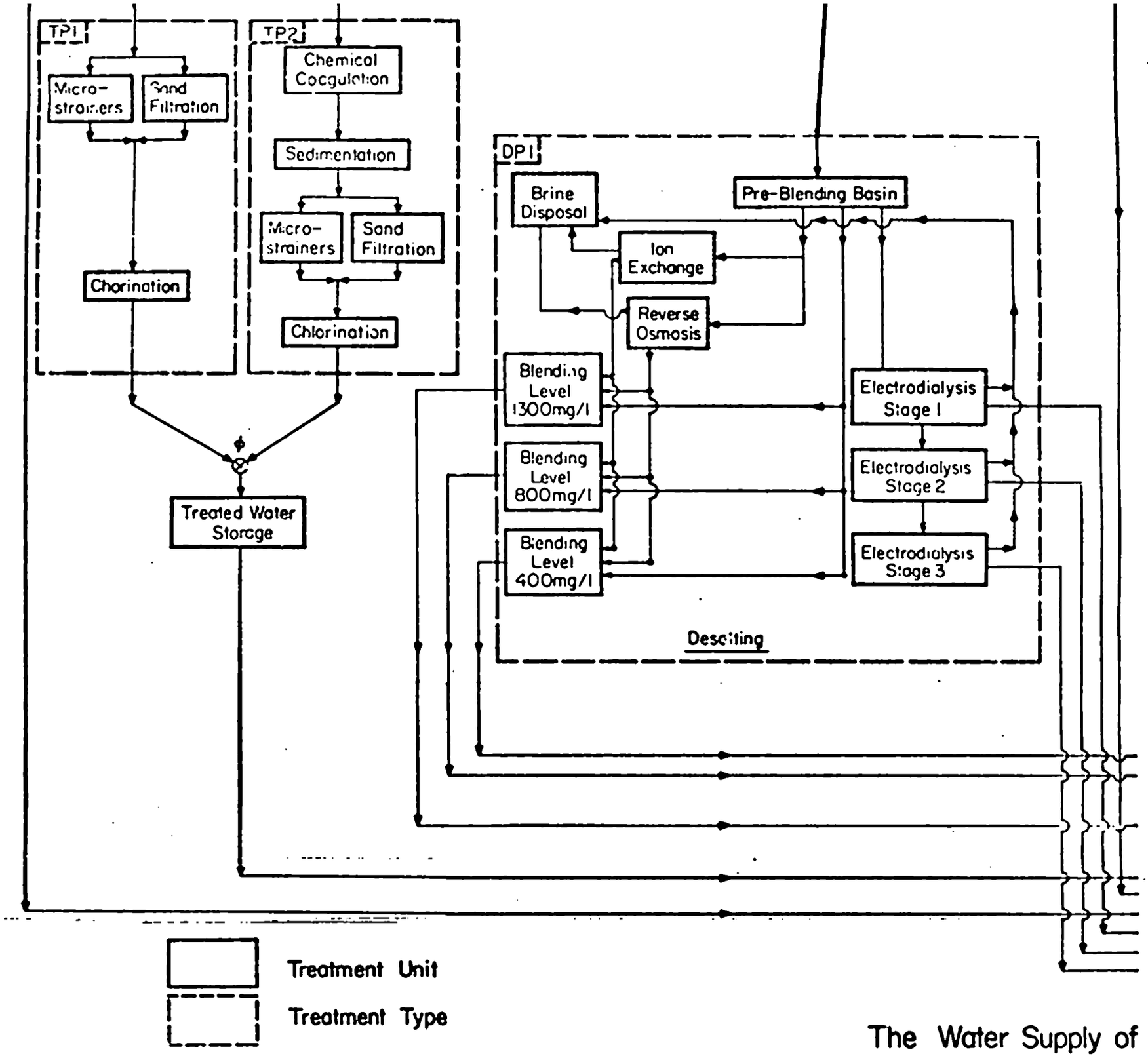
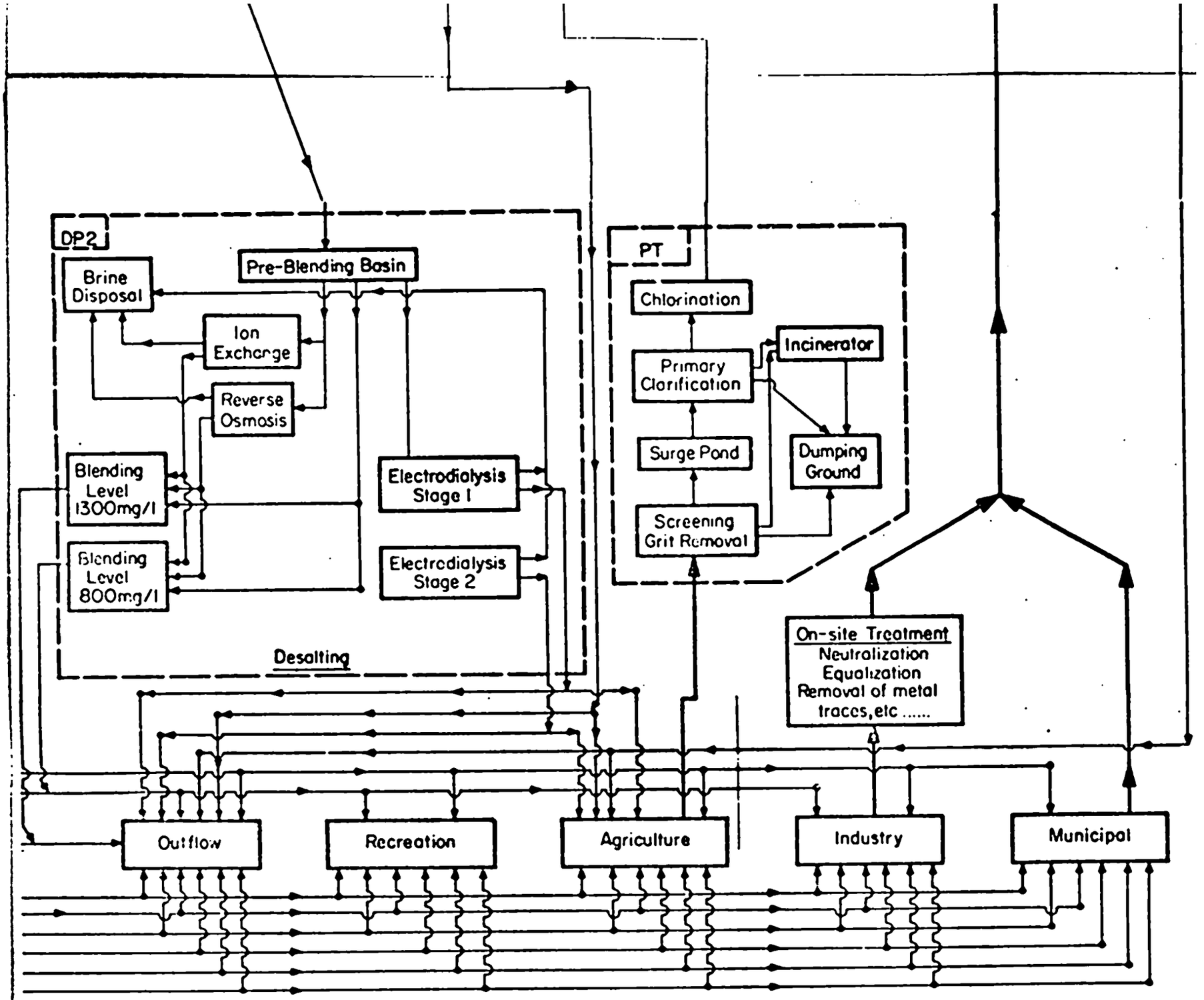

α
1
4
5
SW1
SW2
GW
2
3
6
7
ATP
(treatment plant
type PT)
STP 2
(treatment plant
type TP2)
STP 1
(treatment plant
type TP1)
Row Water Treatment
Row Water Treatment
TT
Tertiary Waste
Water Treatment
Surge
Pond
Floculation
Lime Treatment
Sedimentation
Lime
Regeneration
Carbon Adsorption
Carbon
Regeneration
Ammonia
Stripping
β


1
2
3
Y
PST
Primary–Secondary Waste Water Treatment
Chlorination
Secondary Clarification
Digesters
Activated Sludge
Trickling Filter
Mechanical Dewatering & Thickening
Primary Clarification
Incinerator
Surge Pond
Screening Grit Removal
Dumping Ground



The Water Supply of

General I



a Large Urban Area

Diagram