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February 29, 1988

Mr. Stan Arlt
Director, Water and Waste Utilities
CITY OF RICHLAND
505 Swift Boulevard
Richland, WA 99352

Dear Mr. Arlt:

I am pleased to transmit this final report on the Water Filtration Plant and North Richland Well Field Evaluation.

The comments on the draft report provided by you and your staff have been incorporated into the report and into the ICF Northwest report in Appendix A.

Thank you for the opportunity to work with the City on this enjoyable project.

Very truly yours,

Brian W. Hemphill

Brian W. Hemphill, P.E.

BWH:ljs

pc: Mr. Chuck Miller, ICF Northwest

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SECTION 1
INTRODUCTION

BACKGROUND

The City of Richland provides drinking water to a population of about 30,000. The supply system consists of a series of wells and a large filtration plant. The City has been supplying water since the Wellsian Way well field was started up in 1943. All water was obtained from wells until the startup of the Columbia River filtration plant in 1963.

Today, the primary water sources are the filtration plant and the North Richland well field. Together, these sources represent 83 percent of the total system capacity. They also are the objects of this study. This study was not done in response to any significant problem at either location; both have provided high-quality water for a long period of time, and continue to do so. In the case of the filtration plant, it was decided to study the current operation and attempt to determine the cause and remedy of a gradual decline in plant capacity. The well field was chosen for study in order to optimize its operation in terms of water quality and in cost of operation and maintenance.

OBJECTIVES

The following objectives were established at the onset of this study, as summarized from the scope of work in the contract:

Well Field

1. Evaluate existing data relating to hydrogeologic conditions and operation of the recharge basins.

2. Perform physical inspection of the recharge basins, including analysis of surface soils.

3. Measure surface infiltration rates using a ring infiltrometer.

4. Characterize the performance of the well field through conductance of a pumping test that will help to establish the transmissivity and storage capacity of the aquifer.

5. Evaluate the well field and recharge system in order to assist in planning for the optimum operation of the recharge system.

6. Provide recommendations for future operations that will ensure continued productive use of the well field while minimizing operating costs.

Filtration Plant

As described above, the primary focus of the study of the filtration plant related to the decline in system capacity. It was suspected that the cause of the problems were related to gradual failure of the filter media and underdrain gravel system. Evaluation of this theory took a high priority. Other aspects were added to the project that would also help in providing improved operation of the plant in other ways. These are summarized below:

1. Review of records and plant operation in order to quantify the various parameters relating to system performance.

2. Perform a physical inspection of the filters to evaluate their condition and the need for rebuilding.

3. Evaluate alternatives and provide recommendations for replacement filter media if required.

4. Perform a capacity analysis to determine the limitations on plant performance, to include considerations of effluent quality and operating costs. This includes analysis of the costs and benefits of constructing additional filters.

5. Evaluate the potential for improved operation through the use of cationic polymer coagulants as an alternative to alum. This evaluation includes

conductance of jar tests in order to determine the effectiveness of various products.

6. Examine the existing system in terms of the location and order of chemical feed and mixing conditions, and provide suggestions for possible improvements.

Water Management Strategy

The results of the evaluations of the filtration plant and well field were to be developed into an overall strategy for future operation of the two major sources.

1. Evaluate the viability of the well field in light of impending federal and state regulations regarding the classification of certain recharge-type systems as surface water systems.

2. Evaluate the advisability of constructing additional filtration facilities.

3. Discuss the impacts of the recent amendments to the Safe Drinking Water Act as they may relate to the City of Richland.

SECTION 2
SUMMARY

The report analyzes a variety of subjects relating to the filtration plant and well field. Each is summarized below.

FILTRATION PLANT

Filter Media

An inspection of the filter media found that the media and gravel are in a deteriorated condition and in need of replacement in order to restore the full capabilities of the system. A specification for the recommended replacement media is included in the Appendix.

Capacity Evaluation

The plant capacity is analyzed from the standpoints of hydraulics, process capability, and operating costs. It is concluded that (with rebuilt filters) the plant will be capable of producing the 30 mgd rated capacity at reasonable operating costs throughout the year. Even higher rates are feasible.

Plant Expansion

The economics of expanding the filtration system were examined. It is not cost-effective to construct additional filters. Savings in operating costs are insignificant compared to the capital cost. Plant expansion should be considered only if the North Richland well field becomes unavailable and water demands increase.

Chemical Feed Evaluation

A series of jar tests were run in the plant laboratory to evaluate various aspects of the coagulation process. In tests using alum, it was found that there was no benefit to practicing pH control, and that extended flocculation time did not improve treatment.

Tests with various cationic polymers showed a possibility of reduced operating costs if they are used in lieu of alum. Preliminary estimates show a reduction in chemical costs of $1.00 per million gallons treated when raw water turbidity is low. This rate of savings would amount to about $5,000 per year. Additional savings of about $5,000 could also be expected due to longer filter run lengths. Final conclusions cannot be drawn on the basis of these tests, though, since the low raw water turbidity made it difficult to distinguish results. Further tests are recommended in the spring, when raw water conditions are poorer.

The factors to be considered in choosing the order of chemical feed are discussed; the present practice is logical. It is recommended that an alternate alum injection point on the discharge side of the Parshall flume be tested.

WELL FIELD EVALUATION

An analysis of the well field hydraulic and geological characteristics was made. It is recommended that the well pumps be rearranged to take full advantage of the aquifer capacity. It is also recommended that the recharge basins be covered with a layer of sand in order to provide a surface that can be maintained to sustain the proper surface characteristics for maximum infiltration. By making the recommended modifications, the pumping rate of the recharge water could be reduced significantly.

A review was also made of the water quality aspects of the well field. The field is producing water of high quality. Particle counts and hardness measurements indicated that the soil is providing a high level of particle removal. Approximately 90% of the water being pumped out appears to have come from the recharge basins.

The amendments to the Safe Drinking Water Act may impact the well field; however, the criteria for classifying water sources as surface or as groundwater sources have not yet been established. Based on the historical quality and reliability of the well field product water, and on the results of particle analyses performed on samples from the field, it appears unlikely that the well field would be subjected to surface water filtration requirements; but a final determination must await finalization of SDWA regulations.

WATER MANAGEMENT

SDWA Amendment Impacts

The SDWA (Safe Drinking Water Act) amendments are discussed, and their relevance to Richland are presented. The most significant impact will relate to increased monitoring requirements. There is no evidence at this time that additional treatment processes will be needed.

Relative Costs of Supply Alternatives

Costs are presented to show how the filtration plant compares to the well field. Generally, the filtration plant is less costly, except when demand is high and raw water conditions are at their worst.

Plan for Future Operation

The current operating practices should be continued, with the filtration plant being the preferred source. Long range plans will depend on population trends and on whether the well field is classified as a surface or ground water. Expansion of the treatment plant is not economically attractive unless the well field becomes unavailable and water demands increase.

SUMMARY OF COSTS

Estimated capital costs for the recommended modifications are as follows:

Item	Cost
Filter media replacement	$125,000
Line recharge basins with sand	50,000
Repair fence around recharge basins	48,000
Relocate well pumps	5,400
TOTAL	$228,400

SECTION 3
DESCRIPTION OF EXISTING FACILITIES

OVERALL SYSTEM DESCRIPTION

The Richland water supply system has been developed over a period of more than 40 years to where it now includes five well sources in addition to the filtration plant. A listing of the sources, along with their respective capacities, is shown in Table 3-1. Figure 3-1 shows the location of the facilities. The total rated capacity of all sources is 49 mgd. Peak day water demands are about 31 mgd.

Use of Sources

The supply to the distribution system is a blend from several sources. The decisions as to which source to operate at a given point are based on a series of conditions, which include the relative costs of operation (based primarily on costs for energy); water quality, with particular emphasis on hardness; the need to operate wells on a routine basis in order to maintain the viability of the equipment; and the maintenance of an adequate chlorine residual throughout the distribution system.

In general, the filtration plant is used to provide the majority of the water. This is because it provides a high-quality, soft water at a relatively low cost. Most of the wells are used throughout the year to supply water when the plant is out of service and to augment the capacity during peak flow demands. The Duke well field is used only sparingly, however, due to the very high hardness of its water.

Water Quality

The Richland water supply system has a good history of providing high-quality water. The only water-quality problems have related to high hardness and to taste and odor problems, neither of which is a health concern. Appendix B contains typical water analyses.

TABLE 3-1

WATER SUPPLY SOURCES*

Well Fields - Groundwater		No. Wells	Capacity, mgd
Columbia		1	0.8
Duke		2	2.0
Wellsian Way		3	4.0
Willowbrook		1	1.4
Well Field - Recharge			
North Richland/D-5		11	11.0
Columbia River			
Water Treatment Plant		--	30.0
	TOTALS:	18	49.2

* Source: City of Richland Water System Plan, September, 1987; pp. 8-9

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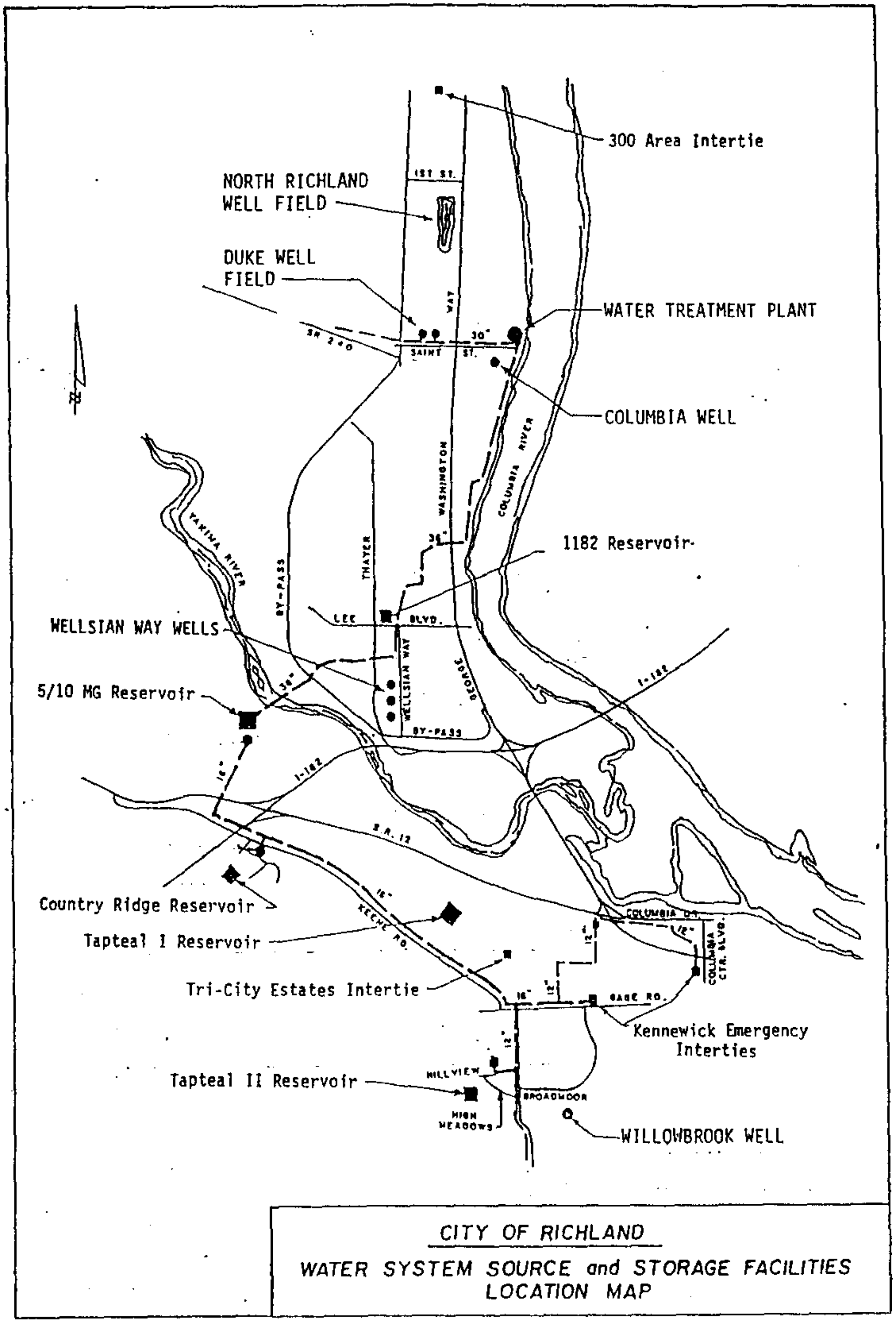


Figure 3—1. System Map (From: Water System Plan; September, 1987)

WATER FILTRATION PLANT

Treatment Process Description

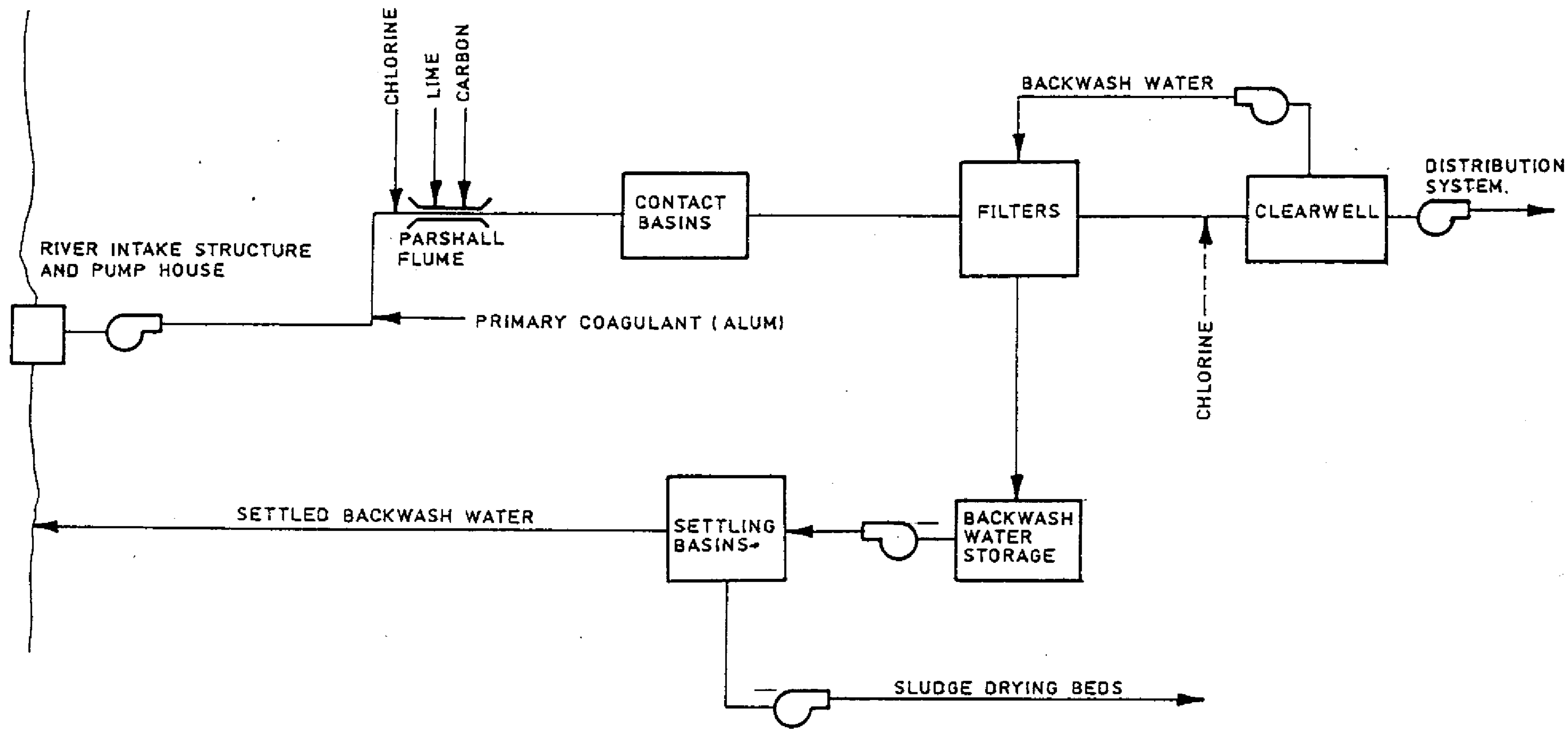
The Columbia River water treatment plant when completed in 1963 had a design capacity of 15 mgd. The original capacity was increased to 30 mgd in 1977 by adding pumping capacity at the intake and with hydraulic improvements at the filters. The hydraulic capacity of the plant is 45 mgd.

The plant can be classified as a direct filtration plant although it does not include flocculation (Figure 3-2). The raw water enters the plant through a 36 inch line that discharges into a Parshall flume. About 10 feet upstream of the flume the coagulant is metered into the raw water line. Chlorine is added in the channel immediately ahead of the flume except when powdered activated carbon is used to control taste and odor. When carbon is added to the diverging suction of the flume the chlorine injection point is moved to the filter influent. Lime addition is used only at times of low raw water pH and is then added to the throat of the flume.

The Parshall flume is primarily a flow measuring device. It also serves to provide mixing energy, although without a hydraulic jump very little mixing is provided in the flume itself. The flash mix occurs when the water from the flume discharges into the small stilling basin at the entrance to the rectangular conduit which carries the water to the contact basins. The detention time in the contact basins is about 40 minutes at 30 mgd.

A small stream of coagulated water is diverted to a pilot control filter equipped with a continuous turbidimeter. The coagulant dosage can be optimized by monitoring the effluent turbidity from the control filter. The advantage of this system is that the coagulant requirements can be determined up to 40 minutes in advance of the water reaching the plant filters. This provides time for dosage adjustments without possible adverse effects on the product water quality.

Before the water enters the four mixed media filters, a nonionic polymer is added as filter aid to prevent premature floc breakthrough. The filters are equipped with



Water Filtration System Flow Schematic

CWC-HDR, INC.

FIGURE 3—2

Wheeler bottoms and Palmer rotary surface washers. At 15 mgd, the original design capacity, the filtration rate is 5 gpm/sf. However, the filters have been operated successfully at twice that rate. Backwash water is supplied by a vertical turbine pump with a maximum capacity of 10,500 gpm. Normally the filters are backwashed when the headloss reaches 10 to 12 feet.

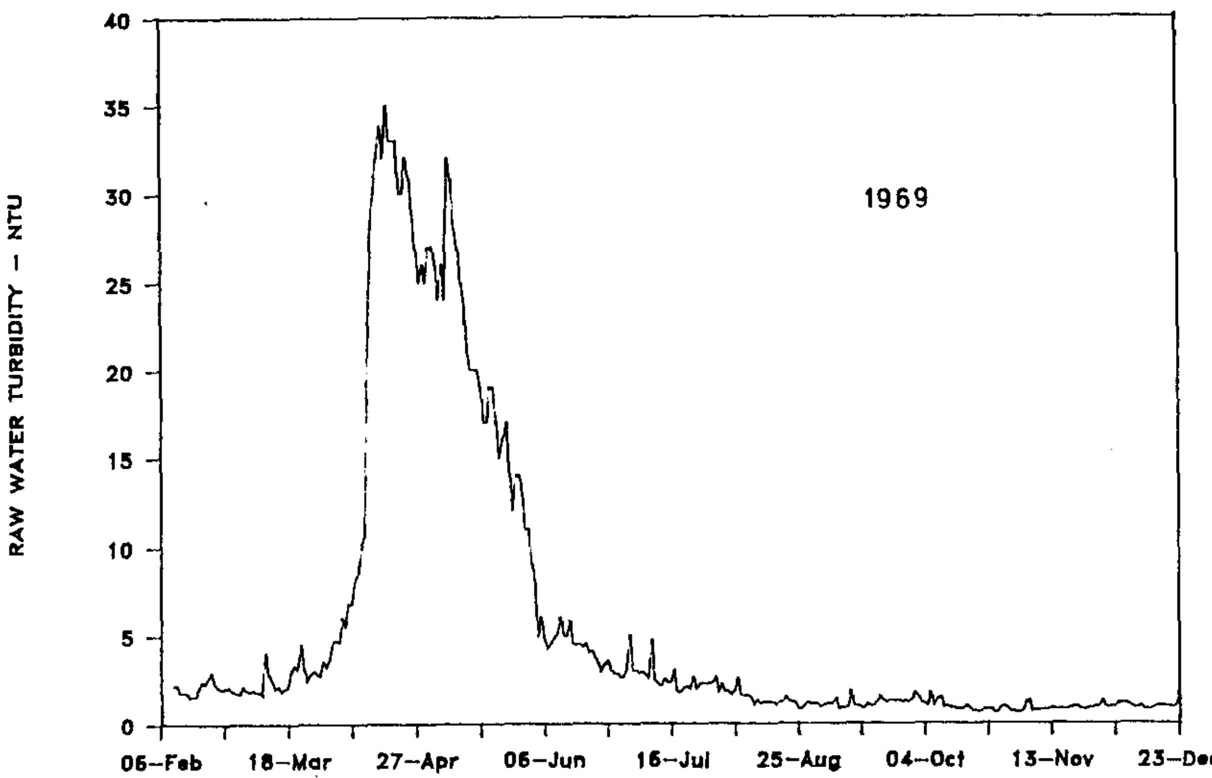
An existing 2.2 million gallon reservoir at the plant site was converted to a clearwell. It provides equalization between plant output and high service pumpage, storage of water for backwash, and additional chlorine contact time before the water enters the distribution system.

The water from each backwash first goes to a storage basin. The purpose of the storage basin is to provide flow equalization to avoid shock loadings on the backwash water settling basins. From the storage basin the water is pumped to the settling basins. The effluent was intended to be discharged to the Columbia River and the sludge pumped to drying beds. However, because the effluent from the settling basins does not meet the NPDES permit, the entire volume of backwash water is pumped to the drying beds.

Raw and Filtered Water Quality

Sudden and dramatic changes in raw water quality can be one of the most difficult challenges a water treatment operator has to face. Fortunately, the variations in the raw water composition at Richland are relatively gradual due primarily to the absence of major tributaries close to the City's intake.

The raw water turbidity can vary widely from one year to the next. A typical low turbidity would be 0.7 NTU occurring in late fall and the early winter months. The maximum readings are normally recorded in April and May during spring runoff. A typical seasonal variation is shown in Figure 3-3. According to plant records, maximum turbidity typically varies from about 4 NTU to 7 NTU, with a high of 35 NTU recorded in 1969. The variations sometimes observed from one year to the next are primarily a function of the intensity of the spring runoff. There is no indication that the dams upstream on the Columbia River contribute to the annual turbidity fluctuations. The dam closest to the Richland intake is Priest Rapids



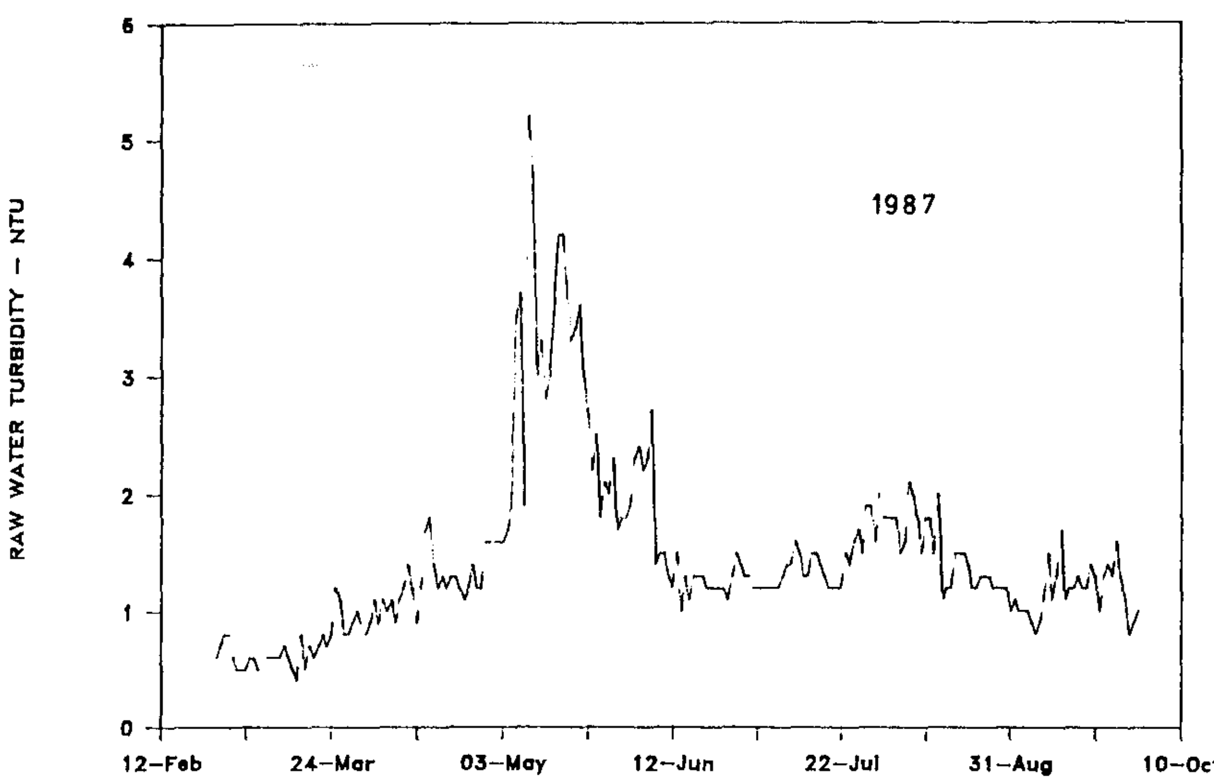


Figure 3—3. Seasonal Variations in Raw Water Turbidity

operated by the Grant County PUD. Like the other dams on the river, its main purpose is power generation and not to regulate the river flow.

The filtered water turbidity is generally around 0.2 to 0.3 NTU. When higher values up to 0.7 NTU have occurred, they do not reflect an inability of the treatment plant to perform but rather shows that the chemical conditioning of the water is not optimum. Excellent filtered water quality has been produced even at extreme conditions with high raw water turbidity and flow rates.

Objectionable tastes and odors sometimes occur in the fall. This condition is caused by algal blooms on the river. The algae are probably also responsible for the diurnal pH fluctuations, which can be as great as 0.8 units. The tastes and odors are controlled by addition of powdered activated carbon in the Parshall flume. Lime is available for pH adjustment. Generally, however, the composition of the raw water is such that there is no need for lime to optimize coagulation. If corrosion control is needed, lime can be added to the filter influent.

Chemical Feed

Liquid alum has been the sole coagulant at the Richland water filtration plant. In addition to the alum, polymer is added to the filter influent to strengthen the alum floc and prevent premature breakthrough. Lime is available for pH adjustment and corrosion control and powdered activated carbon for control of tastes and odors. Disinfection is accomplished by chlorine gas added to the plant influent. When algae blooms on the river necessitate the use of activated carbon to eliminate undesirable tastes and odors, a chlorine injection point at the filter influent is used because of the proximity of the carbon and chlorine addition points to each other at the head of the plant.

With the exception of 1981, there is a relatively good correlation between the alum dosage and raw water turbidity. According to Figures 3-4, 3-5, 3-6 and 3-7 the alum dosage for a typical year varies from a low of 1 to 2 mg/L to a high of 11 to 13 mg/L. Although there is an appreciable degree of scatter in the data, each NTU turbidity increase up to about 5 NTU, typically requires about 2 mg/L increase in alum dosage. In reference to the data from 1969, which is the only year on

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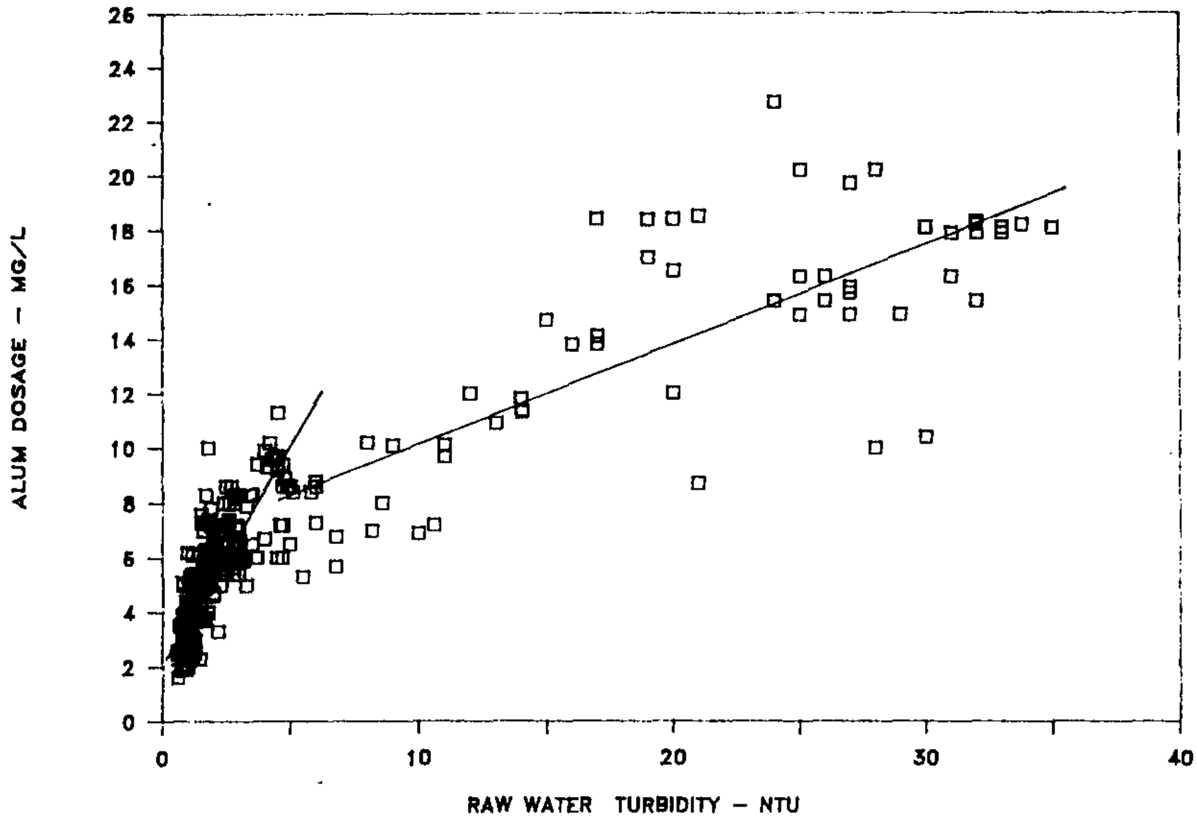

Figure 3–4. Raw Water Turbidity vs Alum Dosage for 1969

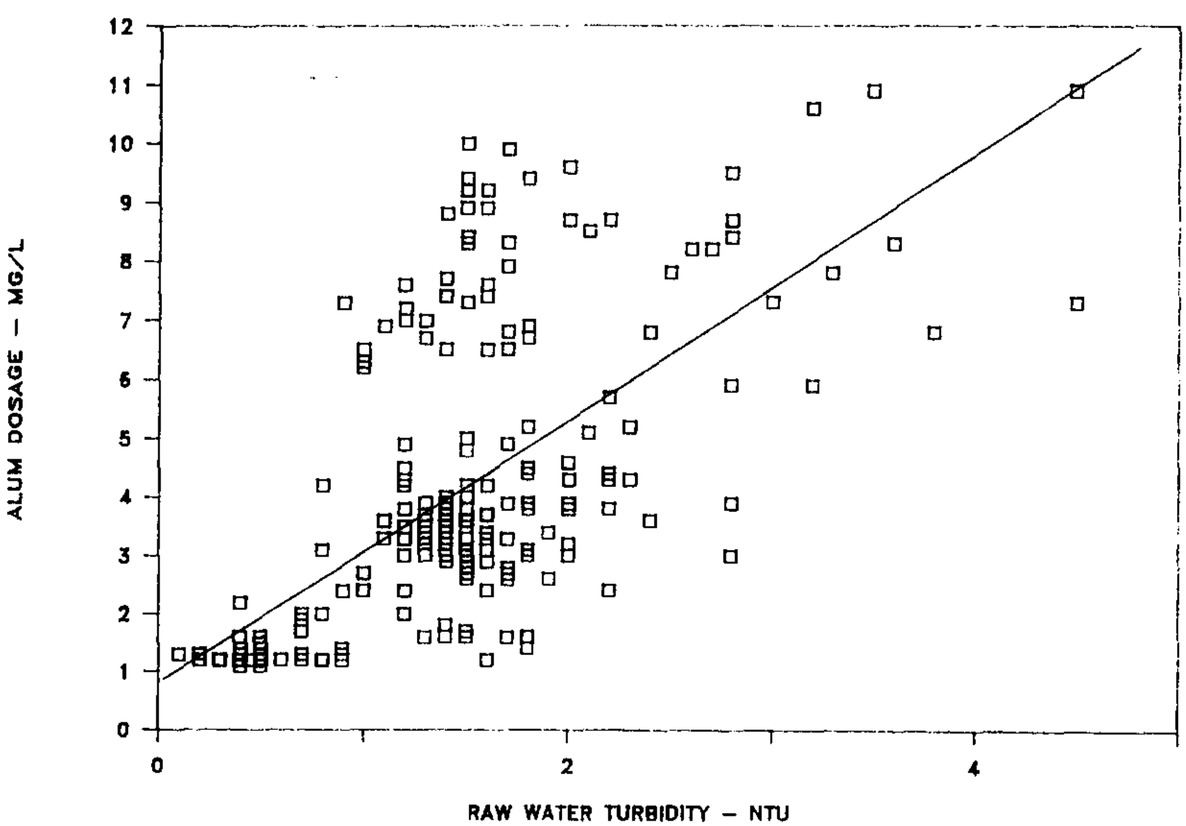

Figure 3–5. Raw Water Turbidity vs Alum Dosage for 1981

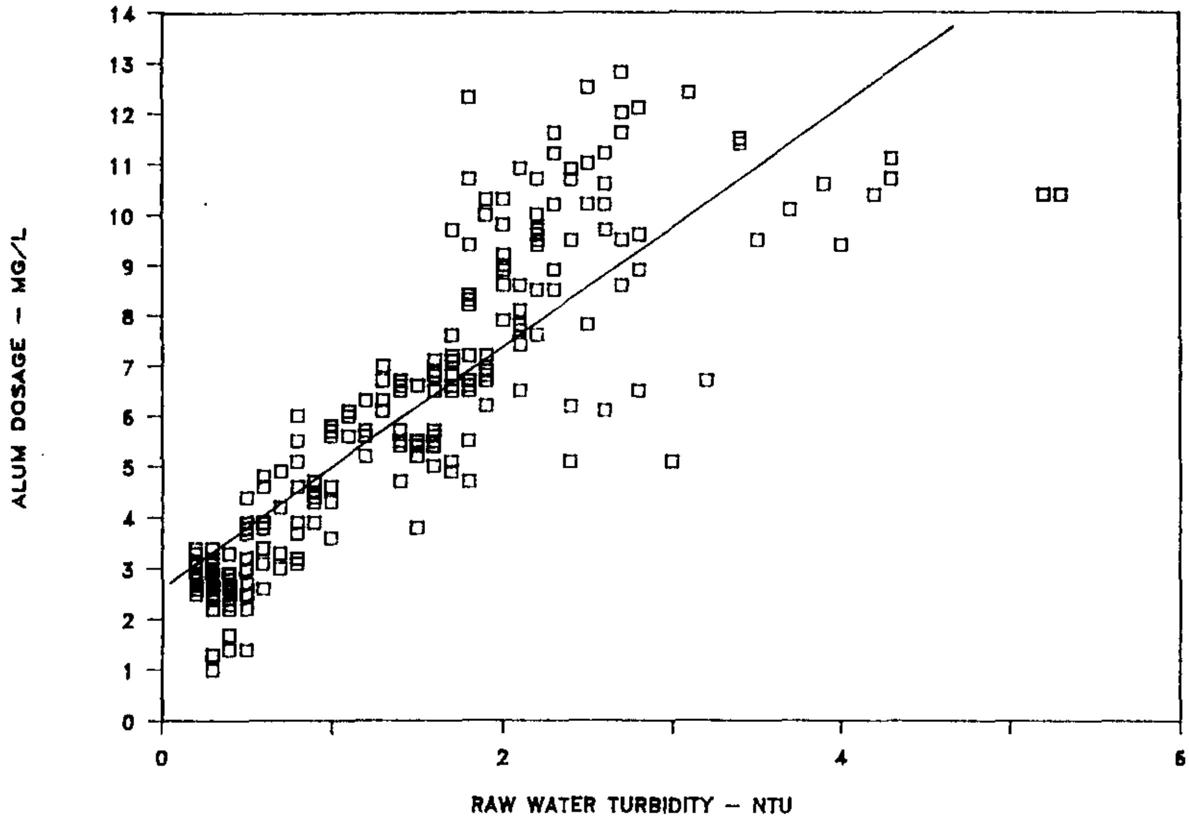

Figure 3–6. Raw Water Turbidity vs Alum Dosage for 1982

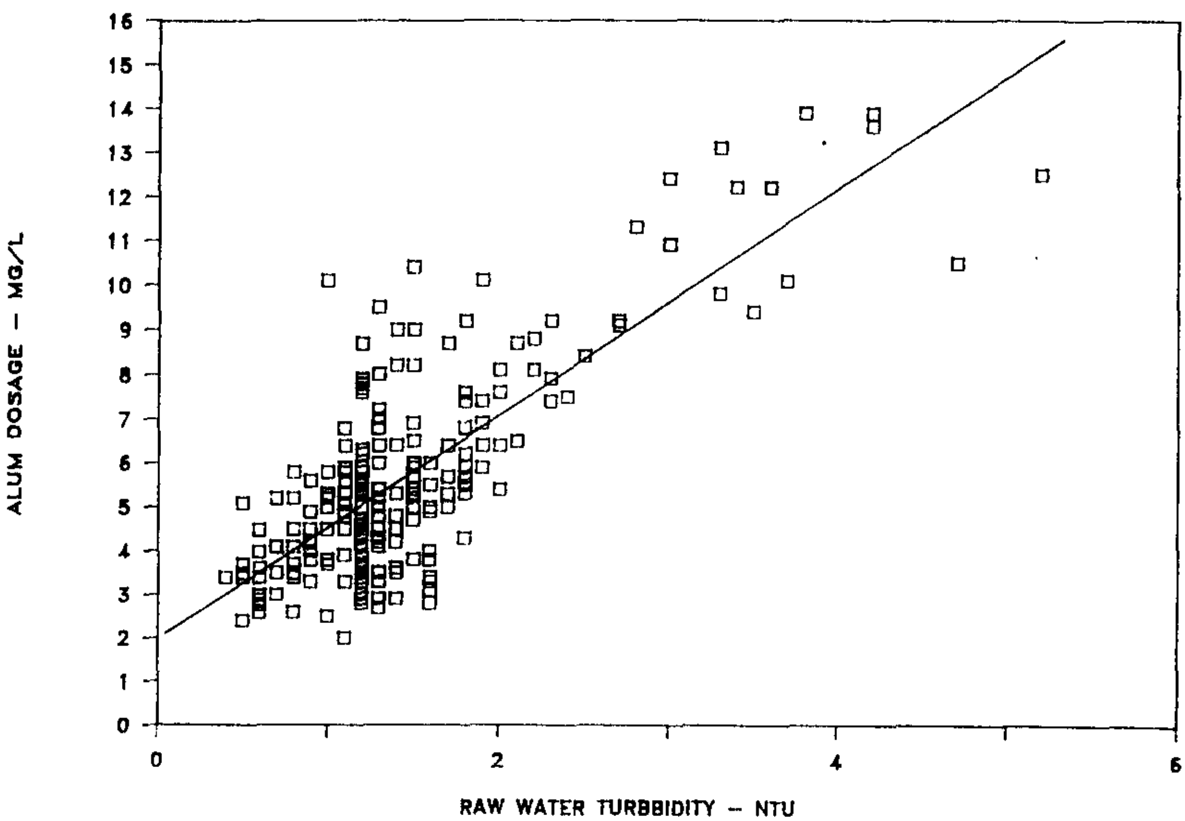

Figure 3–7. Raw Water Turbidity vs Alum Dosage for 1987

record where the raw water turbidity remained in the 20 to 35 NTU range for more than one month, it appears that the relationship between the alum dosage and raw water turbidity changes when the turbidity reaches 5 NTU. Above this turbidity level only about 0.5 mg/L of additional alum is needed for each NTU increase in turbidity. This is most likely due to the interaction between the alum and the colloidal particles. The mechanisms involved in the formation of floc at the pH typical for the Columbia River water, are adsorption - destabilization and sweep coagulation. At low turbidity, the concentration of alum is, by itself, primarily responsible for the amount of $Al(OH)_3(s)$ precipitate that is formed. As the turbidity increases more $Al(OH)_3(s)$ is needed to enmesh the colloidal particles. However, when the concentration of colloidal particles reaches a certain level, the particles themselves can serve as nuclei for the formation of the precipitate. This can result in a decrease in the amount of alum needed to remove a given quantity of colloids.

The polymer is added in dosages ranging from 0.004 to 0.116 mg/L. A typical dosage is 0.014 mg/L. Ideally the filter aid is used to assure that terminal headloss is reached just before turbidity breakthrough. The dosage requirements can best be achieved through operational experience.

Over the last several years, lime has been used only on a few occasions. The range has been from 3 to 10 mg/L with 5 mg/L being the most common dosage. Carbon is used primarily in the fall when the algae blooms occur. The dosage can vary from about 0.5 to 2.5 mg/L depending on the severity of the taste and odor problem. The chlorine dosage is normally around 2 mg/L which gives a residual of approximately 0.1 mg/L. A typical THM level is 0.025 mg/L which is well below the MCL of 0.1 mg/L.

OPERATING HISTORY

Filter Rates and Run Lengths

The large difference between the winter and summer water usage in Richland is typical of other communities in this region of the state. The increase experienced during the summer is primarily due to watering of lawns although typical domestic activities such as laundering of clothes tend to become more frequent and thereby consume more water during the summer.

When the plant was brought on line in 1963, the design capacity, was 15 mgd which corresponds to a filtration rate of 5 gpm/sf. In the first years of operation the plant was rarely operated at this rate. By about 1975, the demand for water had increased to where the plant was producing at its design capacity with some regularity. In the ensuing years, it was not uncommon that the plant design capacity was exceeded in late summer. The most recent years of operation have experienced flow rates up to 30 mgd.

The duration of a filter run depends on a variety of factors. Some of these are related to characteristics of the filter media (effective size, porosity) and the design (terminal headloss). Others are a function of the raw water quality (turbidity) and the plant operation (chemical conditioning, filtration rate). The parameters that are a function of the design, the plant operator has no control over. The operational factors, however, can be, controlled to some degree. Good floc penetration in the filter bed will extend the filter run whereas an increase in the filtration rate will shorten it.

There are no records that show the duration of the filter runs prior to 1981. However, information such as flow rates, amount of water used for backwash, and raw and filtered water turbidities are available. Comparing these data with the complete records of recent years, an estimate of the run lengths in the early years of operation can be made.

Model for Predicting Filter Run Length

To help in analyzing past records and in predicting future performance, a series of multiple regression analyses was performed on operating data from the years 1969 and 1981. This resulted in the following equation to predict run length based on raw water turbidity and filtration rate:

$$\text{Run length (hrs)} = 416 \times (\text{NTU})^{-0.69} \times (q)^{-1.59}$$

In this equation, NTU is the raw water turbidity, and q is the applied flow in gpm/sf.

A comparison of the variations in run length analyzed using this model for 1981 and 1987 is shown in Figure 3-8. For comparison the estimated values for 1969 are also presented. These were based on information on raw and filtered water turbidities, flow rates, and backwash water usage. The difference in run lengths between the three years are primarily due to the deterioration of the filters over these six years; higher dosages of alum and polymer were needed to produce an acceptable effluent quality, particularly in late spring and early summer when the raw water turbidity was at its highest level.

Figure 3-9 is a graphical depiction of the model and shows how the predicted run length varies with filtration rate and raw water turbidity. Figure 3-10 shows how these convert to net production values. Net production is the fraction of the raw water that actually ends up as product water, after accounting for loss during backwash. It is interesting to see how at high flow rates, even seemingly short runs result in reasonable net production values. For example, at 10 gpm/sf and turbidity of 2 NTU, a run length of about 7 hours is predicted, with a net production of 94 percent, which is an acceptable level.

NORTH RICHLAND WELL FIELD

The report on the North Richland well field, prepared by ICF Consulting Engineers, is bound in the appendix of this report.

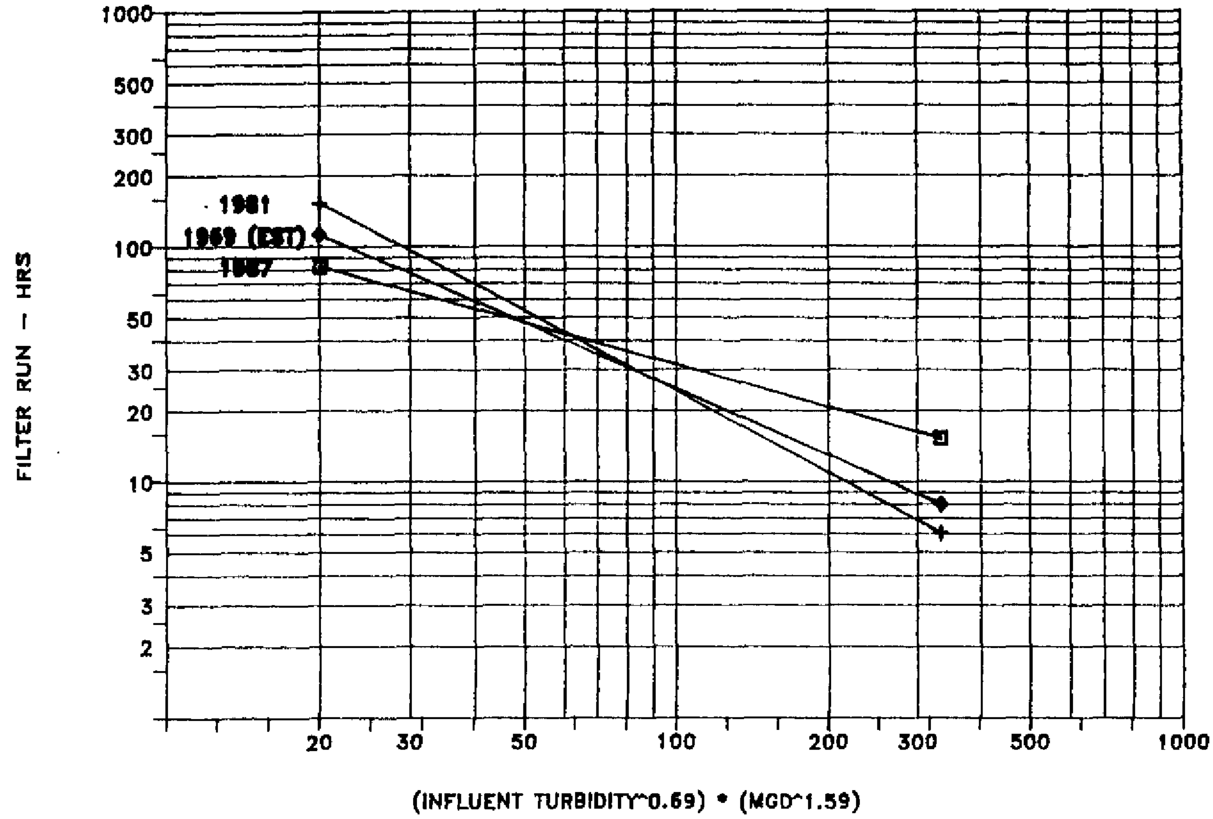

Figure 3-8. Historical Run Length Data

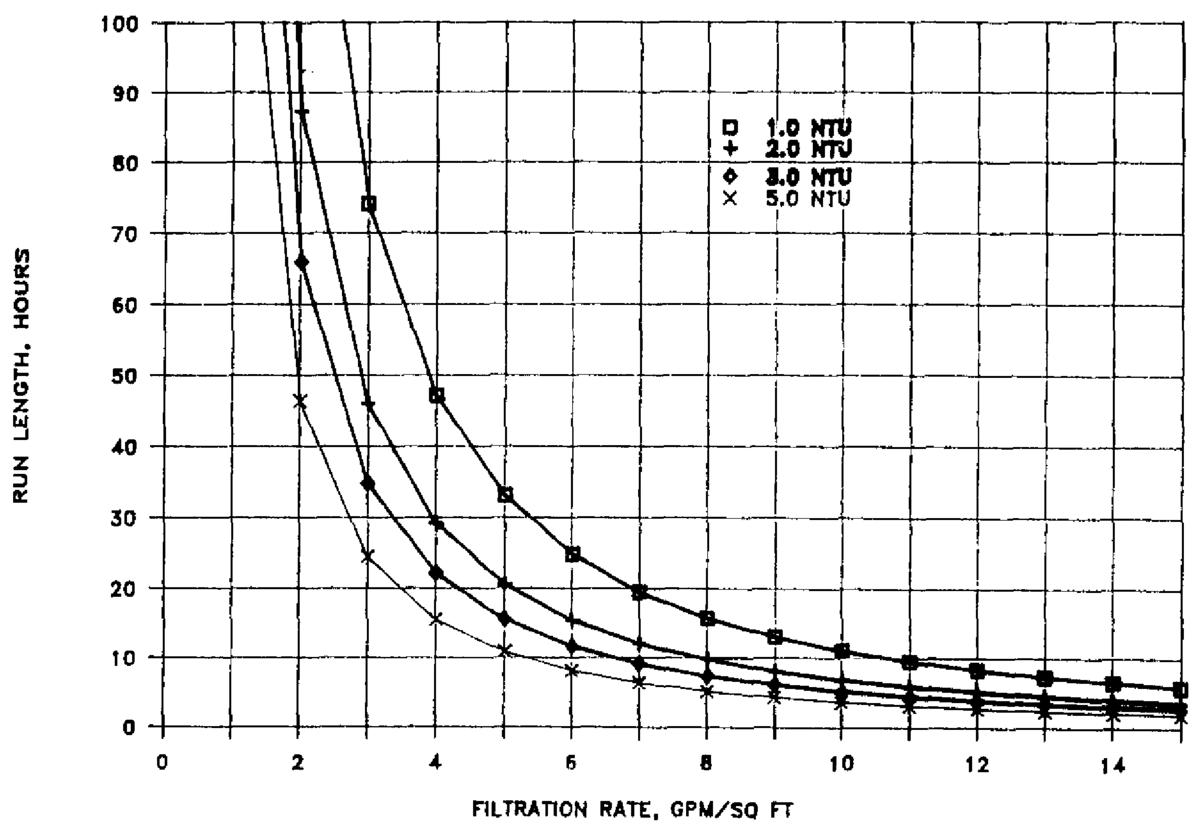

Figure 3-9. Projected Run Length at Various Flows and Turbidities

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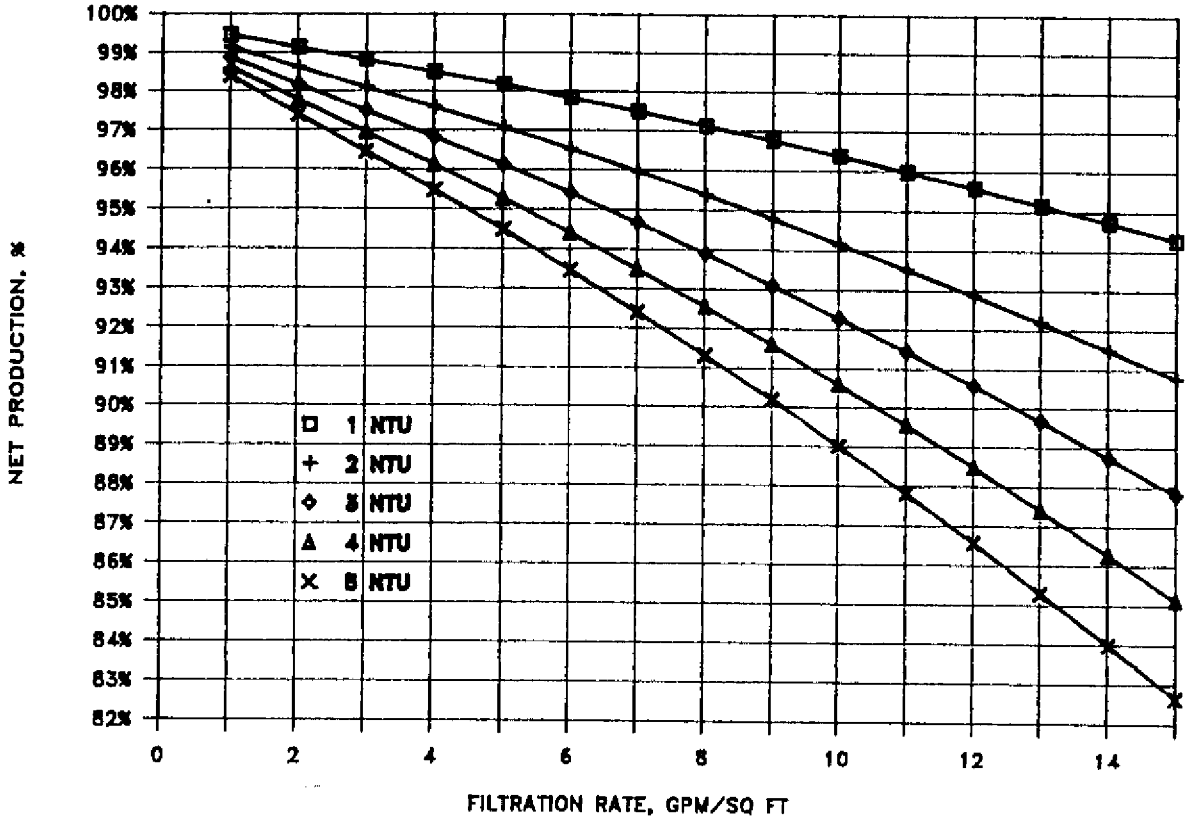

Figure 3—10. Predicted Net Production Levels at Various Flows and Turbidities

SECTION 4
WATER FILTRATION PLANT EVALUATION

This chapter includes discussion of a variety of subjects relating to the filtration plant. The integration of the filtration plant into the supply system as a whole is discussed in Section 6.

RESULTS OF FILTER INSPECTION

Since the results were critical to the remainder of the study, the first task undertaken in this project was an inspection of the filters. A separate technical memorandum, bound in this report as Appendix C, presents the results of the inspection.

In summary, it was found that the filters are in a deteriorated condition, caused by the disruption of the filter support gravel. This has resulted in the loss of significant quantities of media and in other operational problems. It was recommended that the filters be rebuilt as soon as possible in order to maintain the integrity and reliability of the filtration plant. With a life of 25 years, the filters have given exceptional service; lives of high-rate filters of 5 to 10 years are typical.

REPLACEMENT MEDIA EVALUATION

Existing Media

The existing filters have a dual media, originally installed as 10 inches of sand underneath 20 inches of anthracite coal. The media was supplied by Microfloc, Inc. This was one of the early installations of multi-media, and predated the availability of tri-media designs, which include a layer of fine high-density sand at the bottom. Currently, the filters have only about 20 inches of media, due to the loss of materials through the support gravel, as well as some probable loss of coal through abrasion and removal by the normal backwashing process.

The media depth of 30 inches is typical of most high-rate filters, and the available space does not restrict the choice of materials, except to preclude the use of deep, mono-media filters.

Alternative Media Types

Appropriate alternatives for consideration for use in a high-rate filtration system such as that at Richland include the various types of multi-media designs. Single media, such as sand or coal, do not provide the solids retention capacity that is achievable with the multi-media systems, and are not suitable for this application. The only exception to this may be deep, coarse coal filters (e.g. 6-8 feet deep), which are excluded based on the system configuration as discussed above.

The multi-media designs may be classified as dual- or mixed-media systems. Both include a top layer of coarse anthracite coal over a layer of silica sand. The mixed-media systems employ a third layer, consisting of 1.5-4.5 inches of fine high density sand, which is either garnet or eliminate, depending on availability. Material sizes are carefully selected to balance with the corresponding specific gravity to result in the desired coarse-to-fine configuration after backwash. The total bed depth for all systems is typically 30 inches.

In the multi-media designs, the anthracite is designed to capture the bulk of the solids; its coarse, angular shape provides very high storage capabilities for large amounts of solids with low headloss. The lower, finer, layers provide the additional surface area needed to provide the high level of polishing necessary in drinking water filtration systems. The greater the surface area, the better the performance. This explains the benefit of the relatively thin layer of fine, high density sand that is present in the mixed-media beds.

Comparison of Media

Table 4-1 presents a summary of the available media beds, including estimates of installed cost per square foot in Richland. Also listed are relative performance information for the beds. As can be seen, the more high-density sand is used, the higher the cost, the better the capture, and the higher the headloss. What is necessary is to balance the various parameters in order to determine the lowest-cost design that will provide the needed level of performance at Richland.

TABLE 4-1

ALTERNATIVE MEDIA COMPARISON

	MEDIA TYPE		
	Dual Coal/Sand	Mixed MF-162	Mixed MF-186
Layers			
Coal	18 in	18 in	16.5 in
Silica sand	12 in	9 in	.9 in
High density sand	---	3 in	4.5 in
Relative Performance			
Effluent quality[1]	1.2	1.0	0.95
Chemical usage[2]	1.1	1.0	0.95
Run length[1]	1.0	1.0	0.9
Surge resistance	Fair	V. Good	V. Good
Start-up lag time	2.0	1.0	1.0
Relative Cost, $/SF	50	60	65

(1) Assuming equal chemical dosage
(2) At equivalent effluent quality

It would be, of course, possible to run pilot studies to evaluate media performance. However, in order to be conclusive, they would need to be run over a long period. They would also require intensive monitoring. The resulting great expense would outweigh the benefits. There is enough experience with these systems to enable comparisons without testing.

In most side-by-side comparisons of dual- and mixed-media filters in water treatment applications, the mixed-media systems have demonstrated better performance than the dual-media designs. The difference is generally greater with increasingly difficult filtration applications, such as direct filtration as practiced at Richland. The improved performance is generally seen in terms of improved effluent quality at a given chemical feed condition, or, alternatively, equivalent effluent quality at lower chemical doses. This in turn leads to longer filter runs.

Another advantage that has been observed with the mixed-media filters is improved resistance to breakthrough of floc during flow surge conditions, such as occurs when a filter is stopped and restarted. This also is evidenced by shorter lag time for the filter to meet effluent turbidity goals following backwashing.

It has been found that there is usually negligible improvement in effluent quality with the addition of high-density sand beyond about three inches. The additional head loss induced by more fine sand is not justified. The best filter bed for most applications, when taking into account first cost and operating costs, has been found to be the MF-162 bed design, which uses the three inch high density sand layer.

In order to assess the cost benefits that may be achieved through the improved performance of the mixed-media system, the plant performance model (described later in this report) was run using the expected relative performance of the two alternatives. The comparison is based on a conservative assumption of 10 percent additional chemical consumption with a dual media filter, and a cost difference of $10 per square foot. The net effect is an annual cost saving of about $2,400 for the mixed-media.

Recommendation

It is recommended that the filters be replaced with an "MF-162" mixed-media bed having 3 inches of high-density sand, along with 9 inches of silica sand and 18 inches of anthracite coal.

Filter Media Specifications

Appendix D contains a specification for the recommended filter media.

FILTRATION PLANT EXPANSION

As discussed earlier, the filtration plant was originally designed with the provision to add two banks of filters, of the same size as the existing filters, when needed to expand plant capacity. The original concept was based on an ultimate capacity of 45 mgd, with a filtration rate of 5 gpm/sq ft. With the upgrades made in the late 1970's, the system is now rated at 30 mgd at a rate of 10 gpm/sq ft. As it now stands, the system demand exceeds this amount only during a few days each year; the balance is made up by wells. It is known, however, that the cost of producing water from the filtration plant is less than that of the well field. It is also known that the operating costs of producing water from filtration systems is generally lower at lower filtration rates (excluding capital costs). This is due to less frequent backwashing and higher net production. Lastly, the design filtration rate of 10 gpm/sq ft is high compared to most municipal systems. For these reasons, it was decided to consider the cost-effectiveness of adding filter basins.

When the existing filters are rebuilt, and can be relied upon to produce the full 30 mgd, additional filters will not be needed to meet anticipated demands in the foreseeable future, since the wells within the system have sufficient capacity. This analysis, therefore, will consider only the potential cost benefits.

Description and Cost of Plant Expansion

Description--
Since the original design anticipated the installation of additional filters, the plant expansion could be made without a great deal of expense. The existing influent supply, chemical feed, backwashing, surface wash, and contact basins are adequate and would need no modification. The only work would involve construction of the filter basins, along with the associated piping, valves, and controls. For this analysis, two filters, each 22 ft by 24 ft, with a total surface area of 1056 sq ft is anticipated. This would add 50 percent to the existing system, for a total filter area of 3168 sq ft. The plant capacity would be 45 mgd at a rate of 10 gpm/sq ft.

Cost--

A cost estimate for the expansion was included in the 1987 Water System Plan. The cost in 1977 was estimated to be $384,000. If this is escalated to 1987 using the ENR "Treatment Plant Equipment" cost index, the equivalent current cost would be $717,000. As a check, a cost estimate was also generated with the CWC-HDR computerized estimating program "Water Cost". This predicts a total current construction cost of about $835,000. Using the more conservative latter value, adding 20 percent for engineering, administrative, and sales tax costs gives an estimated total capital cost of $1,000,000. The annualized cost for this project, based on an interest rate of 8 percent and a term of 30 years, is $89,000 per year.

Cost Effectiveness

The effect of a filter expansion on overall plant operating costs is discussed below.

FILTRATION PLANT CAPACITY EVALUATION

Introduction

In this section, evaluations are made of the effective capacity of the filtration plant. This includes consideration of hydraulic capacity; process capacity; and operating costs. Included is consideration of the cost-effectiveness of constructing additional filters.

Hydraulic and Process Capacity

As described earlier, the existing system is rated at a hydraulic capacity of 30 mgd. Although the system hydraulics have not been analyzed as part of this study, it is probably feasible to operate at even higher rates.

Process capacity in recent years has been limited by the deteriorated filters. Short filter runs at high rates have limited the throughput to about 25 mgd. However, the plant records show that the filters have been operated at 10 gpm/sf and produced excellent water quality in the past. New filter media would enhance the plant's ability to operate at high flow rates.

Projections of Performance and Costs Under Various Conditions

Analysis of the cost-effective capacity of the filtration must include all of the costs that are affected by changes in plant flowrate. In this section, a spreadsheet-based model is described that allows analysis of the costs of production under a wide variety of conditions, including a range of raw water quality conditions and plant flows.

Spreadsheet Construction--

The spreadsheet is designed to allow computation of the cost components described below. The framework is based on operation through the year 2000. The current annual demand of 5,300 mg is escalated at a rate which may be changed, and which was set at 2.7%/year, which is the rate used in the City's Water System Plan. The costs are computed on a monthly basis, with relevant variables changed throughout the year in order to obtain a representative annual cost.

System Monthly Flows. Flows for each month are taken as a percentage of the total annual demand. The rates are based on averages calculated from the years 1984-1987. Also indicated for each month is a representative raw water turbidity, which is used in the calculation of run length.

The flowrate from the filtration plant is calculated on the total system demand, while taking into account a maximum filtration plant flow, and a minimum flow from the wells within the system. The basic analysis includes a minimum well flow of 1.0 mgd, and a maximum filter effluent flowrate of 9.5 gpm/sq ft. The latter figure results in a filter influent rate of about 10 gpm/sq ft after losses during backwashing are taken into account. Flows demanded in excess of this capacity are assumed to be supplied by production from the wells.

Run Lengths. For each condition of filter flowrate and raw water turbidity, a projection is made of the filter run length, based on a mathematical relationship developed from the historical operating data.

Variable Factors. Within the spreadsheet, provision is made to make changes to various parameters in order to determine the sensitivity of the costs. These include a factor for the raw water turbidity (turbidity multiplier); a run length multiplier, that is used to assess possible changes in run length that may occur in response to changes in chemical feed or for differing media types; as well as input cells for filter area, demand growth rate, interest rate (which affects only the media replacement cost calculation), and maximum filter rate.

Cost Components--

Table 4-2 is a summary of the various cost factors that are incorporated in the model. Each is described in further detail below, including the assumptions that have been incorporated into each.

Power Costs. The City's electricity rate schedule incorporates a stepped rate structure and a demand charge, with the provision for credits. In this analysis, all power costs are assessed at the rate of $0.0172/kWh, which is the rate applied to consumption over 20,000 kWh per month, which is always exceeded. No demand charges are considered; these can generally be balanced by proper system operation, and are not expected to differ significantly over the ranges considered in the analysis.

Influent Pumping. Influent pumping for the filtration plant is based on the static lift of about 47 ft (at a river level of 340 ft, which is representative of typical conditions), and a friction loss of 23 ft, for a total pumping head of 70 ft. An overall wire-to-water efficiency of 63 percent is assumed. This results in a cost of $6/mg pumped.

Backwash Pumping. Pumping of backwash water involves three separate steps as the water is moved from the clearwell through the filters to the storage basin; pumped into the settling basins; then repumped to the drying beds. Additional power is consumed by the agitation pumps that are run while the settling basin contents are pumped to ensure that on overly heavy sludge layer does not form. Until the performance of the settling basins is improved, all backwash water will continue to be pumped to the drying beds. Table 4-2 summarizes the values used for the calculations. These result in an overall cost of $68/mg of backwash water. Calculations assume a total backwash volume of 120 gallons per square foot of filter.

TABLE 4-2

SUMMARY OF COST FACTORS USED IN MODEL

Parameter	Factor
Power cost	$0.0172/kWh
Influent Pumping	
Total dynamic head	47 ft.
Overall efficiency	63%
Backwash Pumping	
Backwash volume	120 gal/sf media
Backwash pumps	
Total pumping head	50 ft
Overall efficiency	70%
Transfer pumps	
Total pumping head	20 ft
Overall efficiency	50%
Settling basin sludge pumps	
Total pumping head	70 ft
Overall efficiency	35%
Percent pumped to drying beds	100%
Agitation pumps	
Assumed running during sludge pumping	
Sludge pumping rate	280 gpm
Agitation pump size	8 @ 8.2 hp ea.
Media Replacement	
Media life	6,000 backwashes
Replacement cost	$60/sf
Chemical Costs	$4-$8/mg
Well Field Operation	
Influent pumping	
Total pumping head	100 ft
Overall efficiency	63%
Influent/production	150%
Well pumps (to surface)	
Total pumping head	21 ft
Overall efficiency	63%
Chlorination	$2/mg

Media Replacement. As evidenced by the current need, filter media must be replaced periodically as the media/gravel system becomes no longer serviceable. The frequency of this replacement is difficult to predict with any accuracy. For this analysis, it was assumed that the existing filters provided a total useful life of 20 years. It is also assumed that the useful life is based on a certain number of backwash cycles, since it is the backwash process that leads to disruption of the gravel support layers.

Based on an analysis of backwash data for the years 1969 and 1981, it is estimated that over the 20-year life, the filters were backwashed a total of 6,000 times. For each year, the equivalent annual cost for media replacement is then based on calculation of an equivalent annual cost based on an interest rate of 8 percent, and a life that is based on the predicted number of backwashes that take place in that year. For example, if 600 backwashes are predicted for a given year, the cost for that year for media replacement is calculated on the basis of an annual cost for a 10 year life (6,000/600), at which time an expenditure of $60 per square foot of media is required.

Chemical Costs. It was assumed that the current costs for chemicals would be maintained in the improved plant. These costs now vary by month, as raw water conditions change. They range from about $4/mg to $8/mg. It is possible that these costs may drop as improved treatment is realized from the new filters.

Well Field Operating Costs. In the summary (Table 4-4), a column is included that shows the cost of operating the well field. This was computed based an estimated 100 ft. total pumping head (70 ft static and 30 ft friction loss) for the raw water supply pumps, and on a wire-to-water efficiency of 63%. At the unit cost of $0.0172/kWh, the cost is $8/mg of recharge flow. In addition, it was assumed that the recharge water flow is 150% of the produced flow, as suggested by the ICF analysis of the recharge system. The net cost of pumping recharge is therefore 1.5 x $8, or $12 per mg produced . Added to the recharge pumping costs is the cost of lifting the groundwater back up to the surface, which amounts to about $2/mg produced, for a total pumping cost of $14/mg produced. Finally, chlorination cost is added at $2/mg, for a total cost of production of $16/mg. Note that if recharge water is pumped at a rate of 3 times the produced flow, as is now commonly practiced, this cost becomes $28/mg produced.

Total System Costs. The total of the filtration system and wellfield costs is shown in a column in the summary (Table 4-4).

Example Complete Spreadsheet Printout--

Table 4-3 is an example printout of an entire spreadsheet. The case shown in that table is the "base case", in which the existing plant continues to operate as-is. Table 4-4 shows the summary of the costs as computed in that run. For other cases that were examined, only the summary sheet is included in this report.

RESULTS OF COST ANALYSES

Base Case

Referring to Table 4-4, it can be seen that under current conditions, the annual average cost of production from the filtration plant is about $14.20/mg. In Table 4-3, it is shown that the cost varies from $10.44/mg in winter months to $17.57/mg in June. The difference is primarily due to chemical cost ($4/mg higher in June), with the remainder due to higher costs for backwash pumping.

Of the total cost of production, influent pumping and chemical feed costs represent about 85 percent of the total, on an annual basis. These two items are relatively insensitive to operational variables such as filtration rate and run length. The sensitivity becomes significant only when net production drops below normal values.

Backwash pumping and media replacement costs are directly related to filtration rate, since it bears directly on run length and frequency of backwashing. Since these costs represent only about 15 percent of the total, however, it can be seen that filter run length is not a critical parameter in terms of operating costs, as long as reasonable net production levels are maintained.

Table 4—3. Filtration Plant Analysis Spreadsheet

BASE CASE - EXISTING FACILITIES — 15-Dec

Filter area, sf: 2112 — Growth rate: 2.70% — Turb. multiplier: 1.0

Max effluent rate, gpm/sf: 9.5

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOTAL
Percent annual flow	3.80%	3.91%	4.20%	8.30%	11.46%	14.35%	15.76%	15.18%	9.97%	5.69%	3.78%	3.60%	100%
Days	31	28	31	30	31	30	31	31	30	31	30	31	365
Typical turbidity	0.7	1.1	1.4	2.8	2.2	1.6	1.6	1.5	1.3	0.8	0.9	0.8	
Chemical cost, $/mg	4	5	6	7	7	8	6	6	6	4	4	4	

Total monthly system demand, mg

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOTAL
1988	202	207	223	440	607	761	835	804	529	301	200	191	5300
1989	207	213	229	452	624	781	858	826	543	310	206	196	5443
1990	213	218	235	464	641	802	881	848	558	318	211	201	5590
1991	218	224	241	476	658	824	905	871	573	327	217	207	5741
1992	224	230	248	489	676	846	929	895	588	335	223	212	5896
1993	230	237	254	502	694	869	954	919	604	344	229	218	6055
1994	237	243	261	516	713	893	980	944	620	354	235	224	6219
1995	243	250	268	530	732	917	1006	969	637	363	241	230	6387
1996	250	256	275	544	752	941	1033	995	654	373	248	236	6559
1997	256	263	283	559	772	967	1061	1022	672	383	255	243	6736
1998	263	270	290	574	793	993	1090	1050	690	393	262	249	6918
1999	270	278	298	589	814	1020	1119	1078	709	404	269	256	7105
2000	278	285	306	605	836	1047	1150	1107	728	415	276	263	7297

Total daily demand, mg

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1988	6.5	7.4	7.2	14.7	19.6	25.4	26.9	25.9	17.6	9.7	6.7	6.2
1989	6.7	7.6	7.4	15.1	20.1	26.0	27.7	26.6	18.1	10.0	6.9	6.3
1990	6.9	7.8	7.6	15.5	20.7	26.7	28.4	27.4	18.6	10.3	7.0	6.5
1991	7.0	8.0	7.8	15.9	21.2	27.5	29.2	28.1	19.1	10.5	7.2	6.7
1992	7.2	8.2	8.0	16.3	21.8	28.2	30.0	28.9	19.6	10.8	7.4	6.9
1993	7.4	8.5	8.2	16.7	22.4	29.0	30.8	29.6	20.1	11.1	7.6	7.0
1994	7.6	8.7	8.4	17.2	23.0	29.8	31.6	30.4	20.7	11.4	7.8	7.2
1995	7.8	8.9	8.6	17.7	23.6	30.6	32.5	31.3	21.2	11.7	8.0	7.4
1996	8.0	9.2	8.9	18.1	24.3	31.4	33.3	32.1	21.8	12.0	8.3	7.6
1997	8.3	9.4	9.1	18.6	24.9	32.2	34.2	33.0	22.4	12.4	8.5	7.8
1998	8.5	9.7	9.4	19.1	25.6	33.1	35.2	33.9	23.0	12.7	8.7	8.0
1999	8.7	9.9	9.6	19.6	26.3	34.0	36.1	34.8	23.6	13.0	9.0	8.3
2000	9.0	10.2	9.9	20.2	27.0	34.9	37.1	35.7	24.3	13.4	9.2	8.5

Filter plant effluent flow; maximum = 28.9 ; assuming minimum well production = 1.0 mgd

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1988	5.5	6.4	6.2	13.7	18.6	24.4	25.9	24.9	16.6	8.7	5.7	5.2
1989	5.7	6.6	6.4	14.1	19.1	25.0	26.7	25.6	17.1	9.0	5.9	5.3
1990	5.9	6.8	6.6	14.5	19.7	25.7	27.4	26.4	17.6	9.3	6.0	5.5
1991	6.0	7.0	6.8	14.9	20.2	26.5	28.2	27.1	18.1	9.5	6.2	5.7
1992	6.2	7.2	7.0	15.3	20.8	27.2	28.9	27.9	18.6	9.8	6.4	5.9
1993	6.4	7.5	7.2	15.7	21.4	28.0	28.9	28.6	19.1	10.1	6.6	6.0
1994	6.6	7.7	7.4	16.2	22.0	28.8	28.9	28.9	19.7	10.4	6.8	6.2
1995	6.8	7.9	7.6	16.7	22.6	28.9	28.9	28.9	20.2	10.7	7.0	6.4
1996	7.0	8.2	7.9	17.1	23.3	28.9	28.9	28.9	20.8	11.0	7.3	6.6
1997	7.3	8.4	8.1	17.6	23.9	28.9	28.9	28.9	21.4	11.4	7.5	6.8
1998	7.5	8.7	8.4	18.1	24.6	28.9	28.9	28.9	22.0	11.7	7.7	7.0
1999	7.7	8.9	8.6	18.6	25.3	28.9	28.9	28.9	22.6	12.0	8.0	7.3
2000	8.0	9.2	8.9	19.2	26.0	28.9	28.9	28.9	23.3	12.4	8.2	7.5

Table 4-3. (Continued)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOTAL
Estimated run length, hours		RL Mult=		1.00 Turb mult=			1.0						
1988	206.9	119.3	106.8	18.7	13.6	11.0	9.9	11.1	23.3	90.7	165.6	209.2	
1989	196.8	113.6	101.6	17.9	13.0	10.5	9.5	10.6	22.3	86.6	157.5	198.9	
1990	187.3	108.2	96.8	17.1	12.4	10.1	9.1	10.1	21.3	82.6	149.9	189.2	
1991	178.3	103.1	92.2	16.4	11.9	9.6	8.7	9.7	20.4	78.8	142.7	179.9	
1992	169.7	98.2	87.8	15.6	11.3	9.2	8.4	9.3	19.5	75.2	135.9	171.2	
1993	161.6	93.6	83.7	15.0	10.9	8.8	8.4	8.9	18.6	71.8	129.4	162.9	
1994	153.9	89.2	79.8	14.3	10.4	8.4	8.4	8.8	17.8	68.5	123.3	155.1	
1995	146.6	85.0	76.0	13.7	9.9	8.4	8.4	8.8	17.0	65.4	117.4	147.7	
1996	139.6	81.1	72.5	13.1	9.5	8.4	8.4	8.8	16.3	62.5	111.9	140.6	
1997	133.0	77.3	69.1	12.5	9.1	8.4	8.4	8.8	15.6	59.6	106.6	134.0	
1998	126.8	73.8	65.9	11.9	8.7	8.4	8.4	8.8	14.9	57.0	101.6	127.6	
1999	120.9	70.4	62.8	11.4	8.3	8.4	8.4	8.8	14.3	54.4	96.9	121.6	
2000	115.2	67.1	59.9	10.9	8.0	8.4	8.4	8.8	13.7	52.0	92.4	115.9	
Net production													
1988	0.994	0.990	0.989	0.966	0.961	0.959	0.956	0.960	0.976	0.990	0.992	0.993	
1989	0.994	0.990	0.989	0.965	0.960	0.958	0.955	0.959	0.975	0.990	0.992	0.993	
1990	0.993	0.990	0.988	0.964	0.959	0.957	0.954	0.958	0.974	0.990	0.992	0.993	
1991	0.993	0.990	0.988	0.963	0.958	0.955	0.952	0.956	0.974	0.989	0.992	0.993	
1992	0.993	0.989	0.988	0.962	0.957	0.954	0.951	0.955	0.973	0.989	0.992	0.993	
1993	0.993	0.989	0.988	0.961	0.955	0.953	0.951	0.954	0.972	0.989	0.991	0.993	
1994	0.993	0.989	0.987	0.960	0.954	0.951	0.951	0.953	0.971	0.989	0.991	0.992	
1995	0.993	0.989	0.987	0.959	0.953	0.951	0.951	0.953	0.971	0.988	0.991	0.992	
1996	0.992	0.988	0.987	0.958	0.951	0.951	0.951	0.953	0.970	0.988	0.991	0.992	
1997	0.992	0.988	0.986	0.956	0.950	0.951	0.951	0.953	0.969	0.988	0.991	0.992	
1998	0.992	0.988	0.986	0.955	0.949	0.951	0.951	0.953	0.968	0.987	0.990	0.992	
1999	0.992	0.987	0.986	0.954	0.947	0.951	0.951	0.953	0.967	0.987	0.990	0.991	
2000	0.992	0.987	0.985	0.953	0.946	0.951	0.951	0.953	0.966	0.987	0.990	0.991	
Filter plant influent flow, mgd (qp/np)													
1988	5.5	6.5	6.2	14.1	19.3	25.4	27.1	26.0	17.0	8.8	5.7	5.2	
1989	5.7	6.7	6.4	14.6	19.9	26.1	27.9	26.8	17.5	9.1	5.9	5.4	
1990	5.9	6.9	6.6	15.0	20.5	26.9	28.7	27.5	18.0	9.4	6.1	5.5	
1991	6.1	7.1	6.9	15.5	21.1	27.7	29.6	28.3	18.6	9.6	6.3	5.7	
1992	6.3	7.3	7.1	15.9	21.7	28.5	30.4	29.2	19.1	9.9	6.5	5.9	
1993	6.5	7.5	7.3	16.4	22.4	29.4	30.4	30.0	19.7	10.2	6.7	6.1	
1994	6.7	7.8	7.5	16.9	23.1	30.2	30.4	30.3	20.3	10.5	6.9	6.3	
1995	6.9	8.0	7.8	17.4	23.7	30.4	30.4	30.3	20.8	10.8	7.1	6.5	
1996	7.1	8.3	8.0	17.9	24.4	30.4	30.4	30.3	21.5	11.2	7.3	6.7	
1997	7.3	8.5	8.2	18.4	25.2	30.4	30.4	30.3	22.1	11.5	7.6	6.9	
1998	7.6	8.8	8.5	19.0	25.9	30.4	30.4	30.3	22.7	11.8	7.8	7.1	
1999	7.8	9.0	8.7	19.5	26.7	30.4	30.4	30.3	23.4	12.2	8.0	7.3	
2000	8.0	9.3	9.0	20.1	27.5	30.4	30.4	30.3	24.1	12.6	8.3	7.5	

Table 4-3. (Continued)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOTAL
Filter plant influent flow, mg/month													
1988	172	181	194	424	600	762	841	806	511	273	172	161	5096
1989	177	187	200	437	618	784	866	829	526	281	177	166	5248
1990	183	192	206	450	636	807	891	854	541	290	183	172	5405
1991	189	198	213	464	655	831	917	879	557	299	189	177	5567
1992	195	205	219	477	674	856	941	905	574	308	195	183	5730
1993	201	211	226	492	694	881	941	931	590	317	201	189	5873
1994	207	217	233	506	715	907	941	939	608	326	207	195	6001
1995	214	224	240	521	736	911	941	939	625	336	213	201	6101
1996	220	231	248	537	758	911	941	939	644	346	220	207	6201
1997	227	238	255	553	780	911	941	939	662	357	227	213	6303
1998	234	245	263	569	803	911	941	939	682	367	234	220	6409
1999	241	253	271	586	827	911	941	939	702	378	241	227	6517
2000	249	261	279	604	852	911	941	939	722	389	248	234	6628
Backwash volume, mgd	gal/sf=	120	sf=	2112									
1988	0.029	0.051	0.057	0.324	0.449	0.553	0.611	0.550	0.261	0.067	0.037	0.029	
1989	0.031	0.054	0.060	0.339	0.469	0.578	0.639	0.575	0.273	0.070	0.039	0.031	
1990	0.032	0.056	0.063	0.355	0.491	0.604	0.668	0.600	0.286	0.074	0.041	0.032	
1991	0.034	0.059	0.066	0.372	0.513	0.631	0.698	0.627	0.299	0.077	0.043	0.034	
1992	0.036	0.062	0.069	0.389	0.536	0.660	0.725	0.655	0.312	0.081	0.045	0.036	
1993	0.038	0.065	0.073	0.407	0.561	0.689	0.725	0.685	0.327	0.085	0.047	0.037	
1994	0.040	0.068	0.076	0.426	0.586	0.720	0.725	0.693	0.342	0.089	0.049	0.039	
1995	0.042	0.072	0.080	0.445	0.613	0.725	0.725	0.693	0.357	0.093	0.052	0.041	
1996	0.044	0.075	0.084	0.466	0.640	0.725	0.725	0.693	0.373	0.097	0.054	0.043	
1997	0.046	0.079	0.088	0.487	0.669	0.725	0.725	0.693	0.390	0.102	0.057	0.045	
1998	0.048	0.082	0.092	0.509	0.699	0.725	0.725	0.693	0.408	0.107	0.060	0.048	
1999	0.050	0.086	0.097	0.532	0.731	0.725	0.725	0.693	0.426	0.112	0.063	0.050	
2000	0.053	0.091	0.101	0.557	0.764	0.725	0.725	0.693	0.446	0.117	0.066	0.052	
No. of backwashes/month													Total
1988	3.6	5.6	7.0	38.4	54.9	65.5	74.8	67.2	30.9	8.2	4.3	3.6	364
1989	3.8	5.9	7.3	40.2	57.4	68.4	78.2	70.3	32.3	8.6	4.6	3.7	381
1990	4.0	6.2	7.7	42.1	60.0	71.5	81.7	73.4	33.8	9.0	4.8	3.9	398
1991	4.2	6.5	8.1	44.0	62.7	74.7	85.3	76.7	35.4	9.4	5.0	4.1	416
1992	4.4	6.8	8.5	46.0	65.6	78.1	88.7	80.2	37.0	9.9	5.3	4.3	435
1993	4.6	7.2	8.9	48.2	68.6	81.6	88.7	83.8	38.7	10.4	5.6	4.6	451
1994	4.8	7.5	9.3	50.4	71.7	85.3	88.7	84.8	40.4	10.9	5.8	4.8	464
1995	5.1	7.9	9.8	52.7	74.9	85.8	88.7	84.8	42.3	11.4	6.1	5.0	474
1996	5.3	8.3	10.3	55.1	78.3	85.8	88.7	84.8	44.2	11.9	6.4	5.3	484
1997	5.6	8.7	10.8	57.6	81.8	85.8	88.7	84.8	46.2	12.5	6.8	5.6	495
1998	5.9	9.1	11.3	60.3	85.5	85.8	88.7	84.8	48.3	13.1	7.1	5.8	506
1999	6.2	9.6	11.8	63.0	89.4	85.8	88.7	84.8	50.5	13.7	7.4	6.1	517
2000	6.5	10.0	12.4	65.9	93.4	85.8	88.7	84.8	52.7	14.3	7.8	6.4	529

Table 4-3. (Continued)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOTAL
Total flow from wells, mg/month													
1988	31	28	31	30	31	30	31	31	30	31	30	31	365
1989	31	28	31	30	31	30	31	31	30	31	30	31	365
1990	31	28	31	30	31	30	31	31	30	31	30	31	365
1991	31	28	31	30	31	30	31	31	30	31	30	31	365
1992	31	28	31	30	31	30	34	31	30	31	30	31	368
1993	31	28	31	30	31	30	59	31	30	31	30	31	393
1994	31	28	31	30	31	30	85	49	30	31	30	31	437
1995	31	28	31	30	31	51	111	74	30	31	30	31	509
1996	31	28	31	30	31	75	138	101	30	31	30	31	587
1997	31	28	31	30	31	101	166	127	30	31	30	31	668
1998	31	28	31	30	31	127	195	155	30	31	30	31	750
1999	31	28	31	30	31	154	224	183	30	31	30	31	834
2000	31	28	31	30	31	181	255	212	30	31	30	31	921
Total flow from wells, mgd													
1988	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
1989	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
1990	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
1991	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	
1992	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.0	1.0	1.0	1.0	1.0	
1993	1.0	1.0	1.0	1.0	1.0	1.0	1.9	1.0	1.0	1.0	1.0	1.0	
1994	1.0	1.0	1.0	1.0	1.0	1.0	2.7	1.6	1.0	1.0	1.0	1.0	
1995	1.0	1.0	1.0	1.0	1.0	1.7	3.6	2.4	1.0	1.0	1.0	1.0	
1996	1.0	1.0	1.0	1.0	1.0	2.5	4.5	3.2	1.0	1.0	1.0	1.0	
1997	1.0	1.0	1.0	1.0	1.0	3.4	5.4	4.1	1.0	1.0	1.0	1.0	
1998	1.0	1.0	1.0	1.0	1.0	4.2	6.3	5.0	1.0	1.0	1.0	1.0	
1999	1.0	1.0	1.0	1.0	1.0	5.1	7.2	5.9	1.0	1.0	1.0	1.0	
2000	1.0	1.0	1.0	1.0	1.0	6.0	8.2	6.9	1.0	1.0	1.0	1.0	
Chemical costs													
1988	687	904	1162	2970	4199	6095	5045	4834	3066	1093	687	644	31385
1989	709	933	1199	3059	4323	6274	5193	4976	3156	1126	709	665	32321
1990	732	962	1236	3150	4451	6458	5346	5122	3249	1160	731	686	33284
1991	755	992	1275	3245	4583	6649	5504	5272	3344	1195	754	708	34275
1992	779	1023	1315	3341	4718	6844	5646	5427	3442	1231	778	731	35276
1993	803	1055	1356	3441	4858	7046	5646	5587	3542	1268	803	754	36159
1994	828	1087	1398	3544	5002	7254	5646	5633	3646	1306	828	778	36950
1995	854	1121	1442	3650	5150	7285	5646	5633	3752	1345	854	803	37534
1996	881	1155	1486	3758	5303	7285	5646	5633	3862	1385	880	829	38103
1997	908	1191	1532	3870	5460	7285	5646	5633	3975	1426	907	854	38688
1998	936	1227	1579	3986	5623	7285	5646	5633	4091	1469	935	880	39290
1999	965	1264	1627	4105	5790	7285	5646	5633	4210	1512	964	908	39909
2000	995	1303	1677	4227	5962	7285	5646	5633	4333	1557	994	936	40547

Table 4—3. (Continued)

Total Filtration Plant Cost/MG

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	TOTAL
1988	10.44	11.67	12.79	16.26	16.59	17.57	15.69	15.39	14.04	10.66	10.53	10.46	
1989	10.45	11.68	12.81	16.35	16.68	17.65	15.78	15.48	14.10	10.68	10.54	10.47	
1990	10.45	11.70	12.83	16.44	16.77	17.74	15.87	15.56	14.16	10.70	10.55	10.48	
1991	10.46	11.71	12.86	16.53	16.87	17.83	15.96	15.64	14.23	10.72	10.56	10.49	
1992	10.47	11.73	12.88	16.63	16.96	17.93	16.04	15.73	14.29	10.74	10.57	10.50	
1993	10.48	11.75	12.90	16.73	17.06	18.02	16.04	15.82	14.35	10.76	10.59	10.51	
1994	10.49	11.76	12.93	16.83	17.16	18.11	16.04	15.84	14.42	10.78	10.60	10.52	
1995	10.50	11.78	12.95	16.92	17.26	18.13	16.04	15.84	14.49	10.80	10.61	10.53	
1996	10.51	11.80	12.98	17.03	17.36	18.13	16.04	15.84	14.55	10.83	10.63	10.54	
1997	10.52	11.83	13.01	17.13	17.46	18.13	16.04	15.84	14.62	10.86	10.64	10.55	
1998	10.53	11.85	13.04	17.23	17.56	18.13	16.04	15.84	14.69	10.88	10.65	10.56	
1999	10.54	11.87	13.07	17.34	17.67	18.13	16.04	15.84	14.76	10.91	10.67	10.57	
2000	10.56	11.90	13.11	17.44	17.78	18.13	16.04	15.84	14.83	10.94	10.69	10.59	

Table 4-4. Summary — Base Case Conditions

COST SUMMARY BASE CASE - EXISTING FACILITIES 15-Dec

Turbidity mult.:	1.0		Filter area:	2112 sq ft
Run length mult.:	1.0		Growth rate:	2.70%
Interest rate:	8.00%		Max filter rate:	9.5 gpm/sf

	Total Demand	Influent Pumping	Backwash Pumping	Chemicals	Media Replac.	Filter Plant Total	Filter Cost per MG Treat.	Wellfield Costs	Total Filt+ Wellfield	Ave. Filter Rate gpm/sf
Units	MG	MG	MG	MG	SQ FT		--	MG		
Unit cost		$6.00	$70.00	--	$60		--	$16		
1988	5300	30,573	6,459	31,385	3,968	$72,384	$14.21	5,840	$78,224	4.59
1989	5443	31,488	6,754	32,321	4,290	74,853	14.26	5,840	80,693	4.73
1990	5590	32,430	7,063	33,284	4,630	77,408	14.32	5,840	83,248	4.87
1991	5741	33,400	7,385	34,275	4,989	80,050	14.38	5,840	85,890	5.01
1992	5896	34,377	7,714	35,276	5,358	82,724	14.44	5,888	88,613	5.16
1993	6055	35,237	7,994	36,159	5,675	85,066	14.48	6,290	91,355	5.29
1994	6219	36,004	8,239	36,950	5,954	87,147	14.52	6,987	94,134	5.41
1995	6387	36,608	8,418	37,534	6,158	88,717	14.54	8,148	96,865	5.50
1996	6559	37,205	8,594	38,103	6,360	90,261	14.56	9,397	99,659	5.59
1997	6736	37,819	8,778	38,688	6,571	91,857	14.57	10,680	102,537	5.68
1998	6918	38,451	8,971	39,290	6,793	93,505	14.59	11,998	105,503	5.77
1999	7105	39,102	9,171	39,909	7,025	95,207	14.61	13,352	108,559	5.87
2000	7297	39,771	9,381	40,547	7,268	96,967	14.63	14,741	111,709	5.97

Table 4—5. Summary — Effect of Adding Two Filters

COST SUMMARY   ADD TWO FILTERS   15-Dec

Turbidity mult.:	1.0		Filter area:	3168 sq ft
Run length mult.:	1.0		Growth rate:	2.70%
Interest rate:	8.00%		Max filter rate:	7.5 gpm/sf

	Total Demand	Influent Pumping	Backwash Pumping	Chemicals	Media Replac.	Filter Plant Total	Filter Cost per MG Treat.	Wellfield Costs	Total Filt+ Wellfield	Ave. Filter Rate gpm/sf
Units	MG	MG	MG	MG	SQ FT		--	MG		
Unit cost		$6.00	$70.00	--	$60		--	$16		
1988	5300	30,106	5,084	30,876	1,489	$67,557	$13.46	5,840	$73,397	3.01
1989	5443	30,994	5,317	31,782	1,674	69,767	13.51	5,840	75,607	3.10
1990	5590	31,906	5,560	32,713	1,874	72,053	13.55	5,840	77,893	3.19
1991	5741	32,844	5,814	33,670	2,090	74,418	13.59	5,840	80,258	3.29
1992	5896	33,809	6,079	34,654	2,322	76,865	13.64	5,840	82,705	3.38
1993	6055	34,801	6,356	35,667	2,572	79,395	13.69	5,840	85,235	3.48
1994	6219	35,820	6,645	36,708	2,839	82,012	13.74	5,840	87,852	3.59
1995	6387	36,869	6,946	37,778	3,126	84,719	13.79	5,840	90,559	3.69
1996	6559	37,948	7,261	38,879	3,431	87,519	13.84	5,840	93,359	3.80
1997	6736	39,057	7,590	40,012	3,757	90,415	13.89	5,840	96,255	3:91
1998	6918	40,197	7,933	41,176	4,103	93,410	13.94	5,840	99,250	4.02
1999	7105	41,370	8,292	42,374	4,470	96,507	14.00	5,840	102,347	4.14
2000	7297	42,577	8,666	43,607	4,860	99,710	14.05	5,840	105,550	4.26

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Total Filtration Plant Cost/MG												
1988	10.33	11.50	12.58	15.04	15.34	16.35	14.48	14.25	13.26	10.48	10.40	10.35
1989	10.33	11.50	12.59	15.11	15.41	16.42	14.54	14.32	13.30	10.48	10.40	10.35
1990	10.34	11.51	12.60	15.18	15.48	16.49	14.61	14.38	13.34	10.49	10.41	10.36
1991	10.35	11.52	12.61	15.25	15.55	16.56	14.68	14.45	13.39	10.50	10.42	10.37
1992	10.35	11.53	12.62	15.32	15.62	16.63	14.75	14.52	13.43	10.51	10.43	10.37
1993	10.36	11.54	12.64	15.39	15.70	16.70	14.83	14.58	13.48	10.52	10.44	10.38
1994	10.37	11.55	12.65	15.47	15.77	16.77	14.90	14.65	13.53	10.53	10.44	10.39
1995	10.37	11.56	12.66	15.55	15.85	16.84	14.97	14.72	13.57	10.54	10.45	10.39
1996	10.38	11.58	12.67	15.62	15.93	16.92	15.05	14.79	13.62	10.56	10.46	10.40
1997	10.39	11.59	12.69	15.70	16.01	16.99	15.13	14.87	13.68	10.57	10.47	10.41
1998	10.39	11.60	12.70	15.78	16.09	17.07	15.20	14.94	13.73	10.58	10.48	10.42
1999	10.40	11.61	12.72	15.86	16.17	17.15	15.28	15.01	13.78	10.59	10.49	10.42
2000	10.41	11.62	12.73	15.95	16.26	17.23	15.36	15.09	13.84	10.61	10.50	10.43

Effect of Constructing Additional Filters

Table 4-5 is the summary sheet for the case in which the filter area is increased by 50 percent to 3,168 sq ft (add two filters). The net effect of this change is to lower the filtration rate, resulting in greater run lengths. The cost savings are seen primarily in backwash pumping and in media replacement costs. The net operating cost savings are projected to be about $5,000 per year.

As discussed above, the annual debt service for the cost of constructing the new filters is estimated at about $89,000 per year. Obviously, the filters cannot be justified based on cost alone.

Effect of Operating at Higher Filtration Rates

Table 4-6 shows the effect on costs of raising the maximum allowable filtration rate from 10 gpm/sq ft to 12 gpm/sq ft. There is no effect at all until the year 1992, when the monthly demand from the filtration plant exceeds 30 mgd. No production from the wells in excess of the 1.0 mgd base flow is needed until 1999. The overall cost of supply goes up slightly, since the unit cost of production from the filtration plant are higher than the costs of production from the wellfield during the high demand months, when raw water turbidity is relatively high. Figure 6-1 (Section 6) shows how the costs are affected by filtration rate for the operating conditions encountered in fall/winter, spring, and summer.

The analysis indicates that if the process capability can be demonstrated, the filtration plant could be a cost-effective alternative to operation of the well field in case this supply becomes unavailable.

Table 4—6. Summary — Effect of Raising Maximum Filter Rate to 12 gpm/sf

COST SUMMARY INCREASE MAXIMUM FILTER RATE 15-Dec

Turbidity mult.:	1.0	Filter area:	2112 sq ft
Run length mult.:	1.0	Growth rate:	2.70%
Interest rate:	8.00%	Max filter rate:	11.5 gpm/sf

	Total Demand	Influent Pumping	Backwash Pumping	Chemicals	Media Replac.	Filter Plant Total	Filter Cost per MG Treat.	Wellfield Costs	Total Filt+ Wellfield	Ave. Filter Rate gpm/sf
Units	MG	MG	MG	MG	SQ FT		--	MG		
Unit cost		$6.00	$70.00	--	$60		--	$16		
1988	5300	30,573	6,459	31,385	3,968	$72,384	$14.21	5,840	$78,224	4.59
1989	5443	31,488	6,754	32,321	4,290	74,853	14.26	5,840	80,693	4.73
1990	5590	32,430	7,063	33,284	4,630	77,408	14.32	5,840	83,248	4.87
1991	5741	33,400	7,385	34,275	4,989	80,050	14.38	5,840	85,890	5.01
1992	5896	34,398	7,722	35,296	5,367	82,783	14.44	5,840	88,623	5.16
1993	6055	35,425	8,074	36,347	5,765	85,610	14.50	5,840	91,450	5.32
1994	6219	36,482	8,441	37,429	6,184	88,535	14.56	5,840	94,375	5.48
1995	6387	37,571	8,824	38,543	6,624	91,561	14.62	5,840	97,401	5.64
1996	6559	38,692	9,224	39,691	7,086	94,692	14.68	5,840	100,532	5.81
1997	6736	39,846	9,641	40,873	7,572	97,932	14.75	5,840	103,772	5.98
1998	6918	41,035	10,078	42,090	8,081	101,284	14.81	5,840	107,124	6.16
1999	7105	42,225	10,517	43,310	8,597	104,649	14.87	5,921	110,570	6.34
2000	7297	43,279	10,897	44,394	9,045	107,615	14.92	6,405	114,019	6.50

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Total Filtration Plant Cost/MG												
1988	10.44	11.67	12.79	16.26	16.59	17.57	15.69	15.39	14.04	10.66	10.53	10.46
1989	10.45	11.68	12.81	16.35	16.68	17.65	15.78	15.48	14.10	10.68	10.54	10.47
1990	10.45	11.70	12.83	16.44	16.77	17.74	15.87	15.56	14.16	10.70	10.55	10.48
1991	10.46	11.71	12.86	16.53	16.87	17.83	15.96	15.64	14.23	10.72	10.56	10.49
1992	10.47	11.73	12.88	16.63	16.96	17.93	16.05	15.73	14.29	10.74	10.57	10.50
1993	10.48	11.75	12.90	16.73	17.06	18.02	16.14	15.82	14.35	10.76	10.59	10.51
1994	10.49	11.76	12.93	16.83	17.16	18.11	16.24	15.91	14.42	10.78	10.60	10.52
1995	10.50	11.78	12.95	16.92	17.26	18.21	16.33	16.00	14.49	10.80	10.61	10.53
1996	10.51	11.80	12.98	17.03	17.36	18.31	16.43	16.09	14.55	10.83	10.63	10.54
1997	10.52	11.83	13.01	17.13	17.46	18.41	16.53	16.18	14.62	10.86	10.64	10.55
1998	10.53	11.85	13.04	17.23	17.56	18.51	16.63	16.28	14.69	10.88	10.65	10.56
1999	10.54	11.87	13.07	17.34	17.67	18.61	16.72	16.38	14.76	10.91	10.67	10.57
2000	10.56	11.90	13.11	17.44	17.78	18.72	16.72	16.47	14.83	10.94	10.69	10.59

SECTION 5
CHEMICAL FEED CONSIDERATIONS

It has been said that success in water treatment is 90 percent chemistry and 10 percent equipment. While the exact percentages may be debatable, there is no question that control of chemical feed is of prime importance in treatment plants. Not only does it affect effluent quality, but represents a significant portion of operating costs and operator attention. In this section several topics relating to chemical feed in the filtration plant are discussed.

COAGULATION EVALUATION

On November 12-13, 1987, a series of tests were run at the Richland filtration plant, with the purpose of evaluating the coagulation process. Specific goals were to:

- Characterize the alum coagulation process, with regard to dosage and pH relationships, and identification of the apparent mode of coagulation.
- Determine the applicability of cationic polymers for coagulation, both alone and in combination with alum.
- Explore the effect of flocculation on product water quality.

Procedures

The evaluations were all performed using a modified version of standard jar testing procedures. The laboratory's six-place Phipps and Bird jar stirrer was used. One liter beakers were filled with 500 ml of sample. Chemicals were added using multiple pipettes, so that they were added to all jars within a one to two second span, assuring nearly equal mixing times. All rapid mixing was done at the maximum speed. Flocculation was achieved at speeds of 10 or 20 rpm, with various durations.

After the flocculation period, the samples were subjected to a simulated filtration process. Circles of Whatman No. 2 paper were folded into glass laboratory funnels.

TABLE 5-1. SUMMARY OF JAR TESTS

TEST NO.	COAGULANT	DESCRIPTION	BEST RESULT DOSAGE	BEST RESULT NTU
1	Alum	Natural pH	16.0	0.11
2	Alum	Adjust pH to 7.4	0.5	0.32
3	Alum	Adjust pH to 7.0	2.0	0.23
4	Alum	Adjust pH to 6.6	16.0	0.13
5	Alum	Repeat #1 at higher sol'n	12.0	0.11
6	Alum	Flocculation time test	8.0	0.14 @ 5 min.
7	Nalco 8105	Dosage evaluation	0.2	0.36
8	Nalco 8102	" "	0.6	0.39
9	Nalco 8156	" "	0.2	0.23
10	CatFloc DL	" "	0.4	0.30
11	CatFloc TL	" "	0.2	0.32
12	Magnifloc 572C	" "	0.4	0.28
13	Magnifloc 573C	" "	0.2	0.24
14	Magnifloc 577C	" "	0.2	0.26
15	Magnifloc 591C	" "	0.3	0.27
16	Magnifloc 591C	Flocculation time test	0.3	0.24 @ 2 min.
17	Alum/591C	Dosage evaluation	4/0.3	0.22
18	Alum/8102	" "	4/0.6	0.17
19	Alum/CatFloc DL	" "	2/0.2	0.27

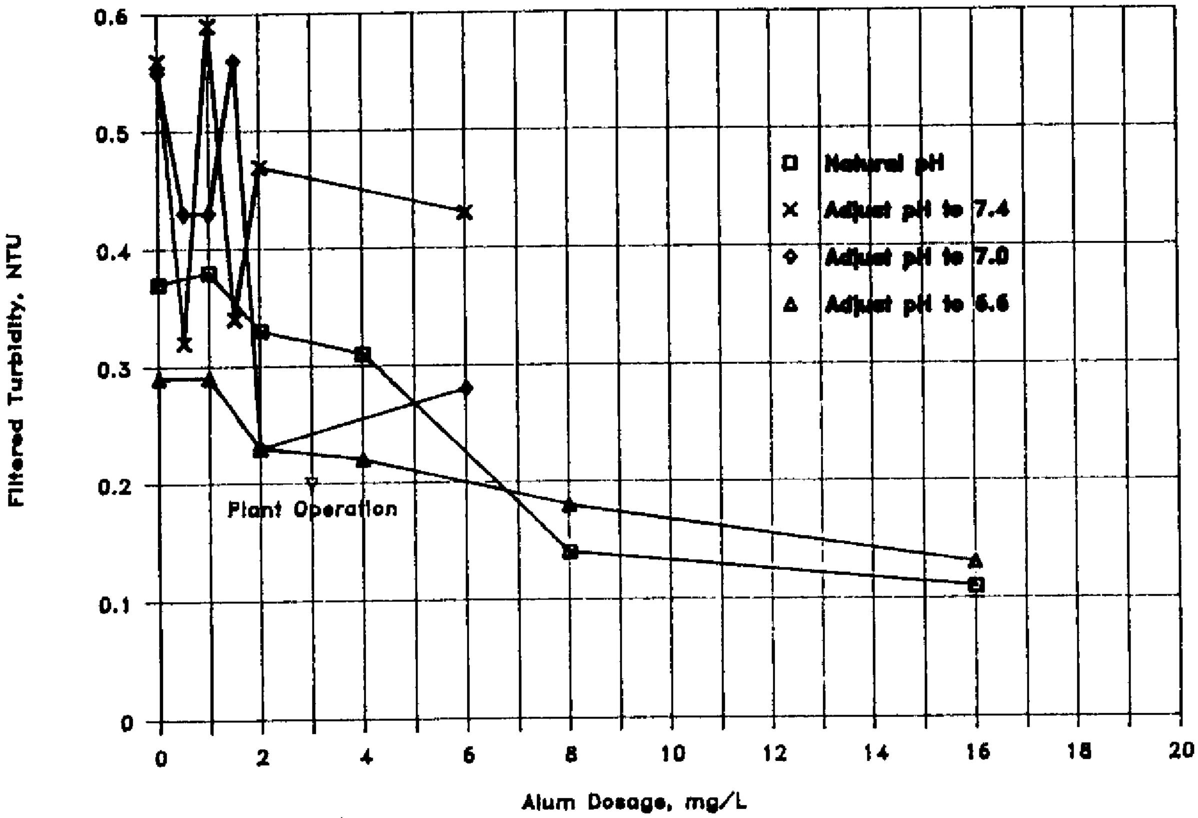

Figure 5—1. Results With Alum at Various pH Levels

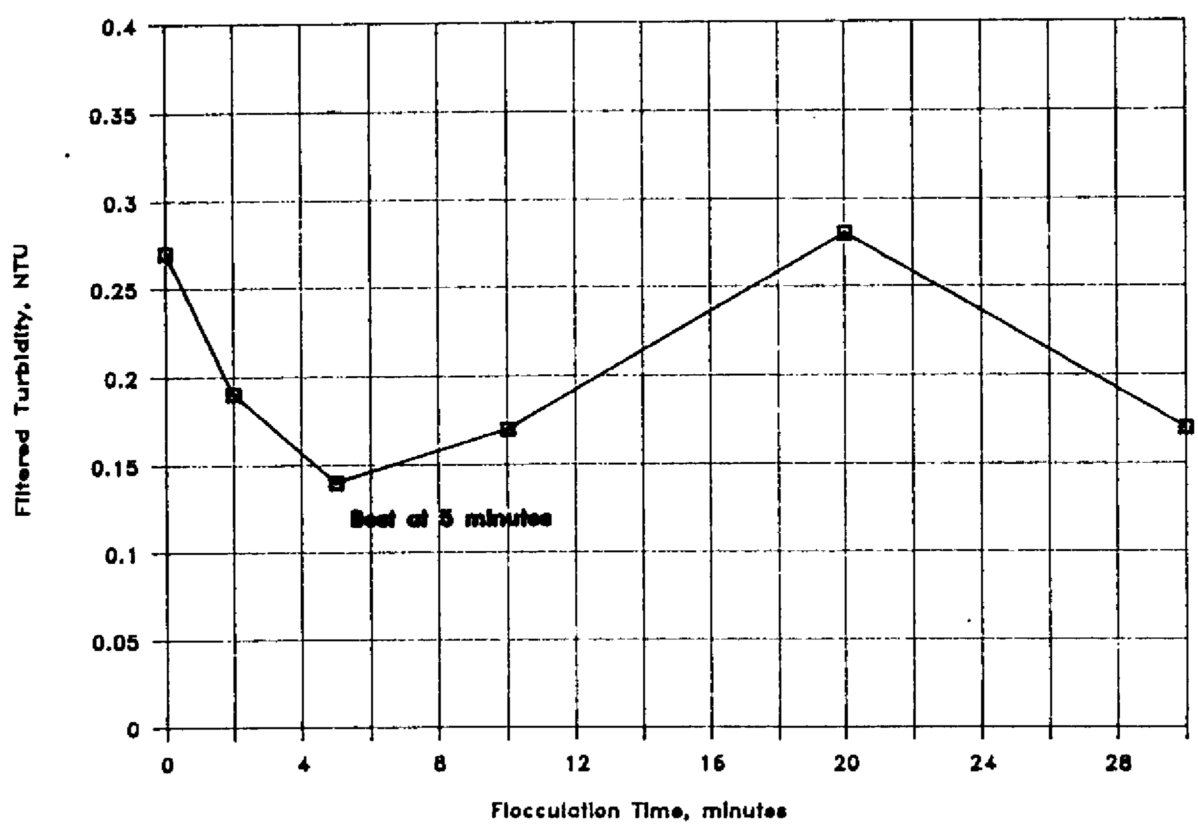

Figure 5—2. Effects of Flocculation Time With Alum at 8 mg/L

mixing intensity ("G") of about 10/sec. The best result was attained at a flocculation time of only 5 minutes. This represents a Gt value of about 3,000.

Cationic Polymers--

Figures 5-3, 5-4, and 5-5 show the results of test using polymers as supplied by three vendors. Samples were received from a fourth supplier (Allied Chemical) too late to use in the tests. None of the polymers were able to provide a filtered turbidity of less than 0.20 NTU, although two were able to produce turbidities less than 0.25 NTU.

Figure 5-6 shows the results of an evaluation of flocculation with Magnifloc 591C.

Combined Alum and Polymer--

Three polymers were selected for trials in combination with alum. Since about 8 mg/L of alum was needed in the jar test to produce filtered water with turbidity below 0.20 NTU, dosages of 2 and 4 mg/L were used with various dosages of polymer. Results are displayed in Figures 5-7, 5-8, and 5-9. The only favorable results were found with Nalco 8102, but high dosages of polymer and alum were required.

Discussion

Relevance of Jar Tests--

The jar test procedure was designed to simulate the operation of the full scale plant to the extent possible. As it turned out, the correlation was not ideal. While the plant produced water having turbidity of about 0.20 NTU with alum dosages of 1.5 to 3.0 mg/L, in the jar tests a dosage of about 8 mg/L was needed to produce the same results. There are two suspected reasons. First is that the filter paper was not providing the same level of treatment as the plant filters. The other is that the mixing conditions in the jar test did not duplicate the plant. This is a known limitation of the Phipps and Bird apparatus. At maximum speed, the stirrer achieves an estimated velocity gradient (G) of about 100/sec. This compares with typical values of 800/sec for hydraulic jump mixing, as used in the plant.

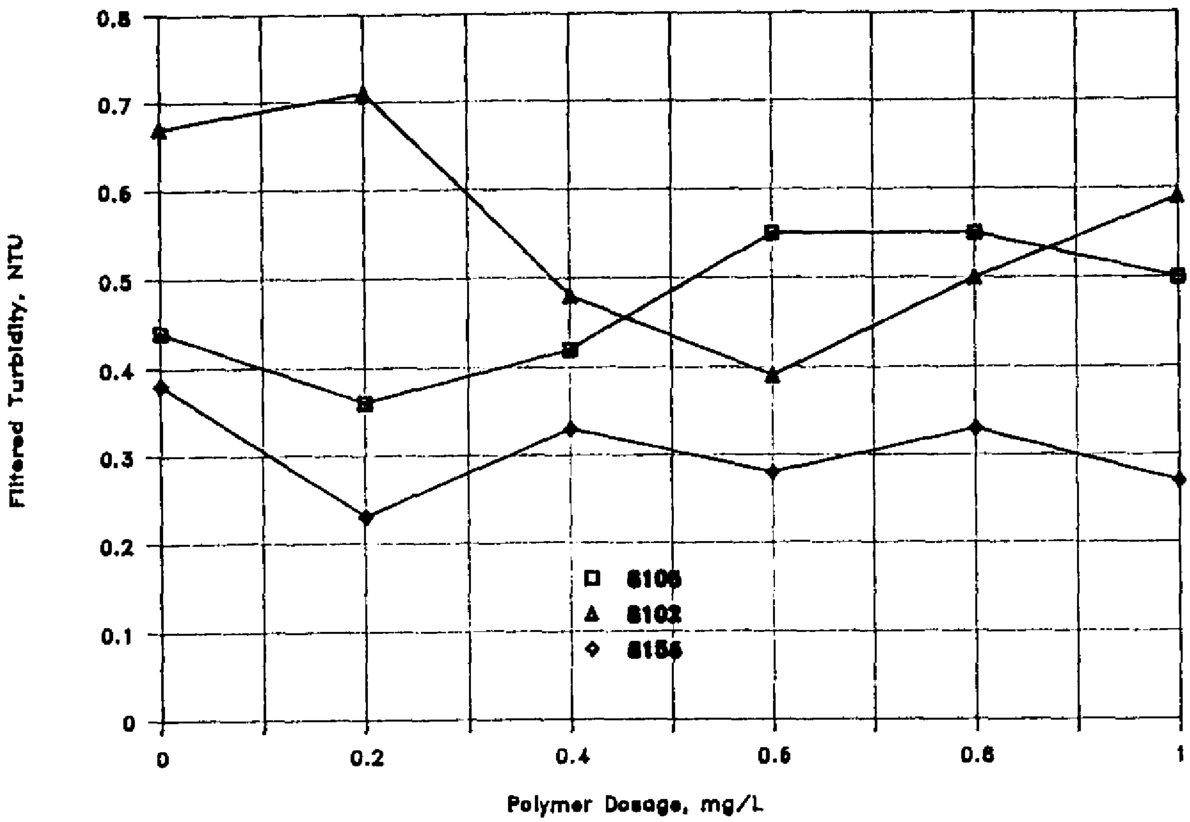

Figure 5—3. Results With Nalco Polymers

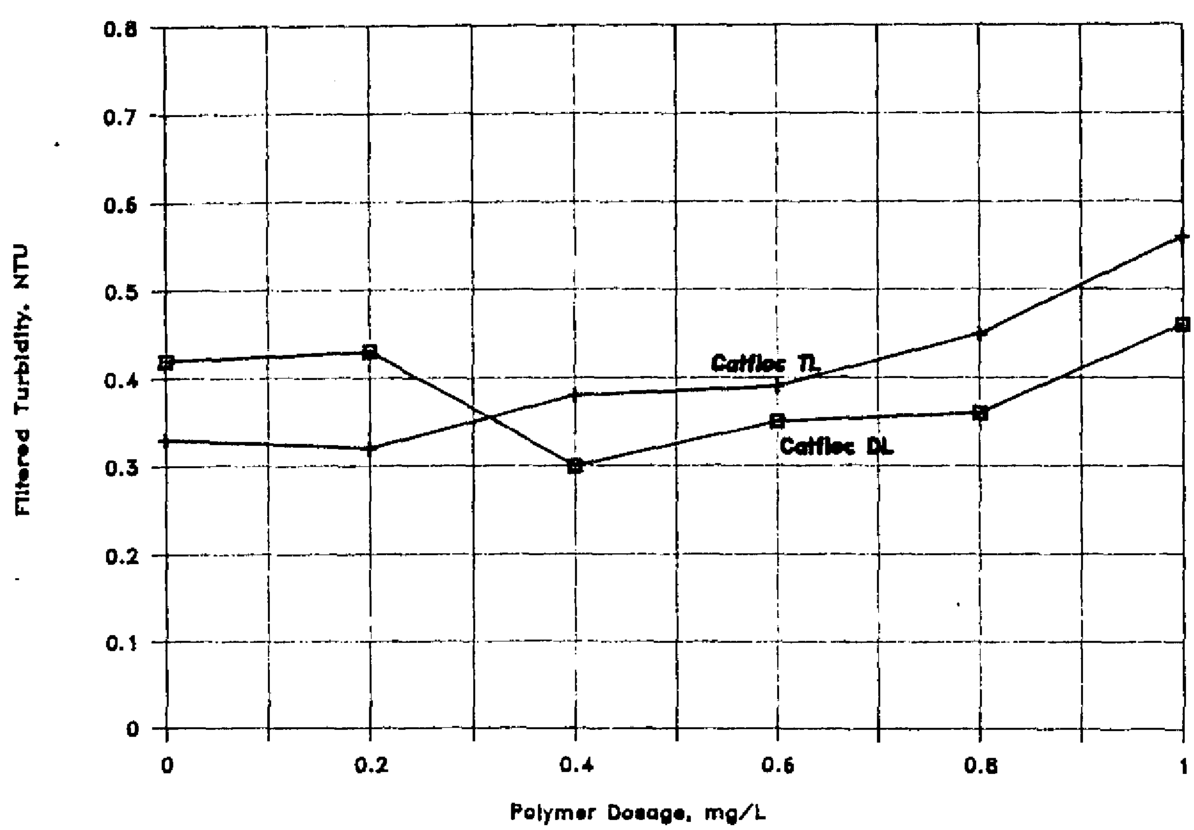

Figure 5—4. Results With Calgon Polymers

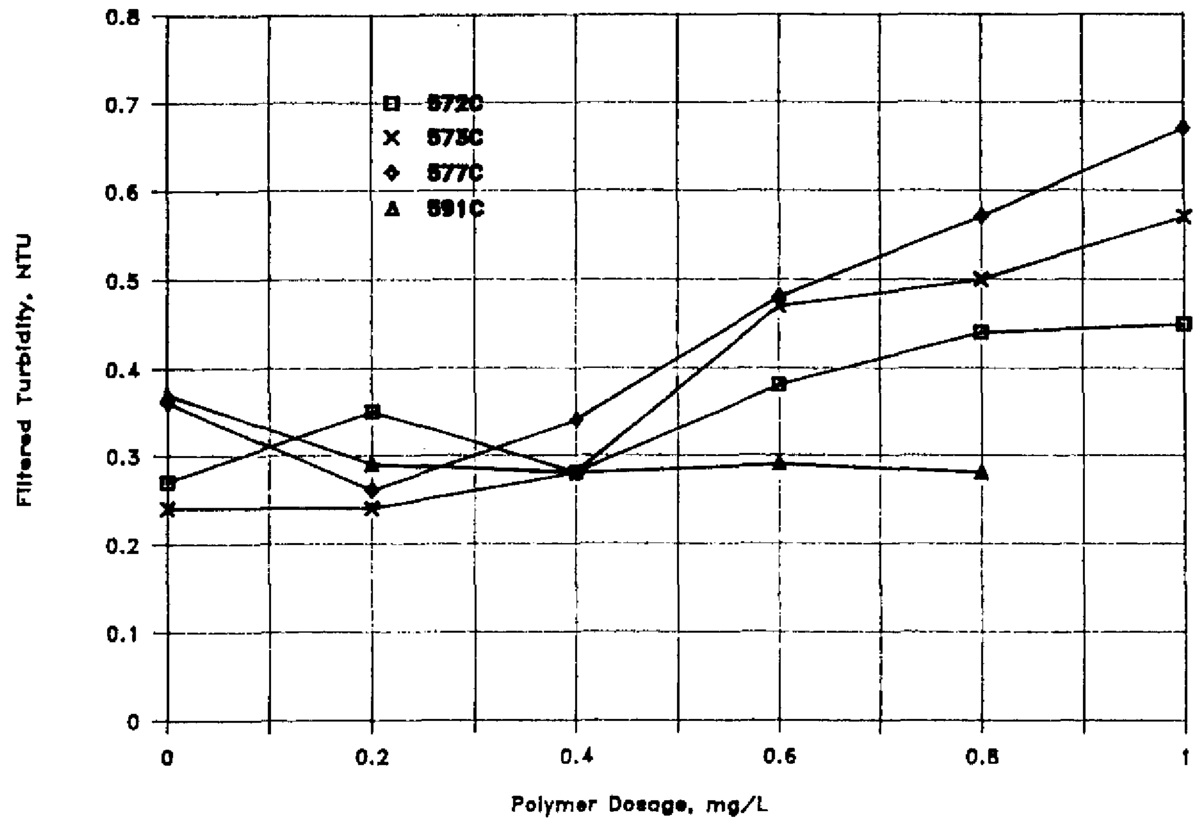

Figure 5—5. Results With American Cyanamid Polymers

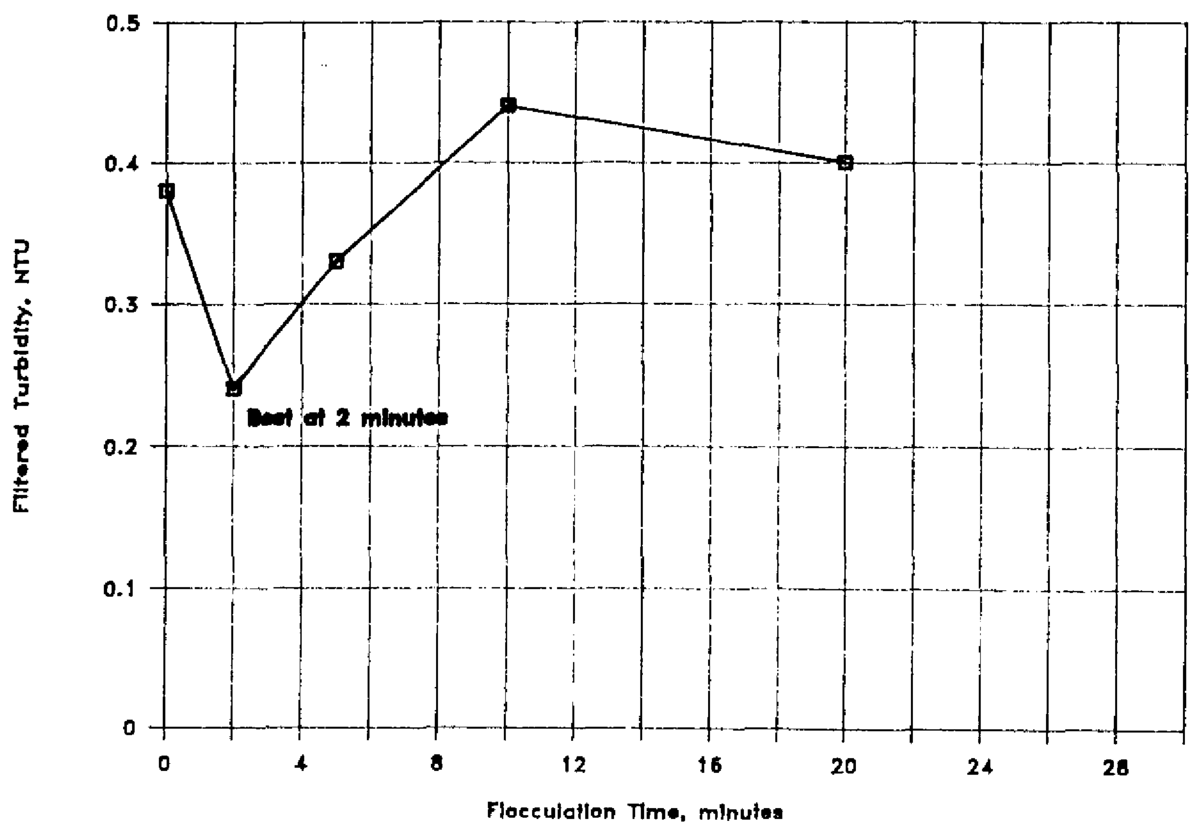

Figure 5—6. Effects of Flocculation Time With 591C

9 2 1 2 5 5 1 7 4 9

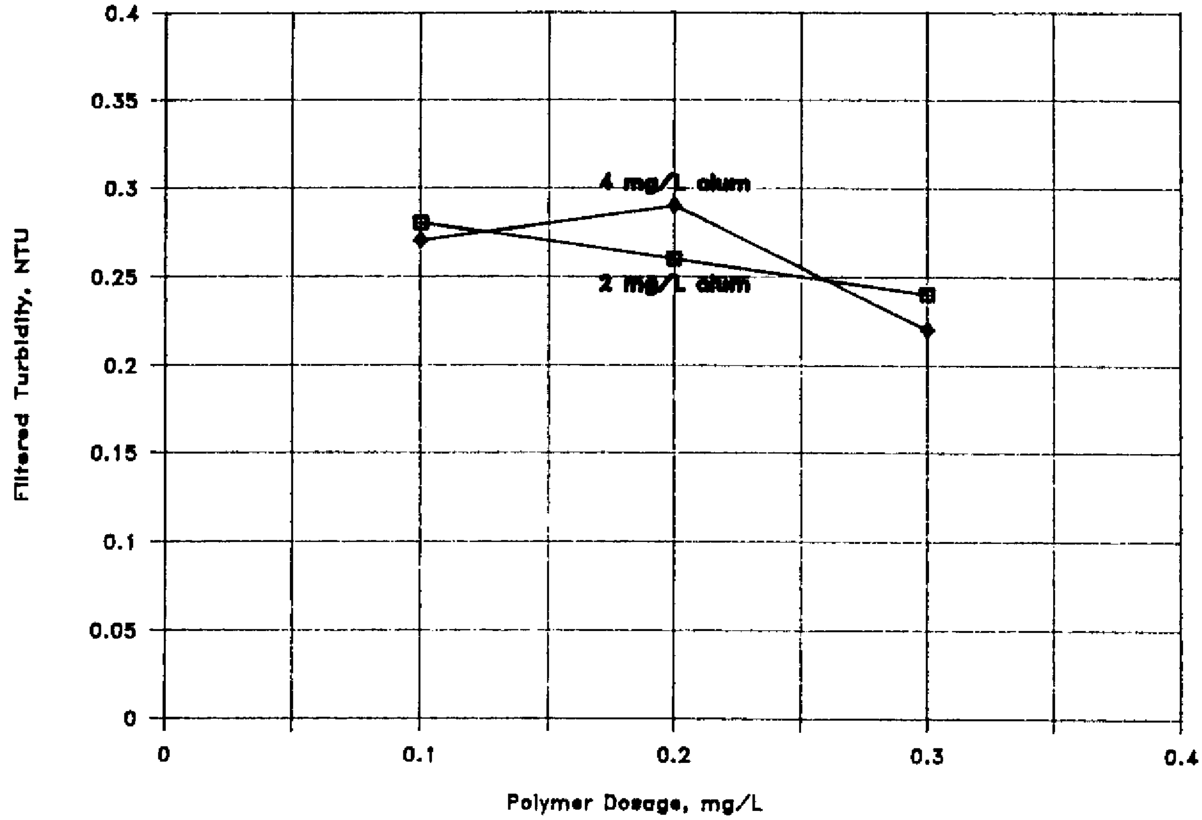

Figure 5—7. Results With Combined Alum/591C

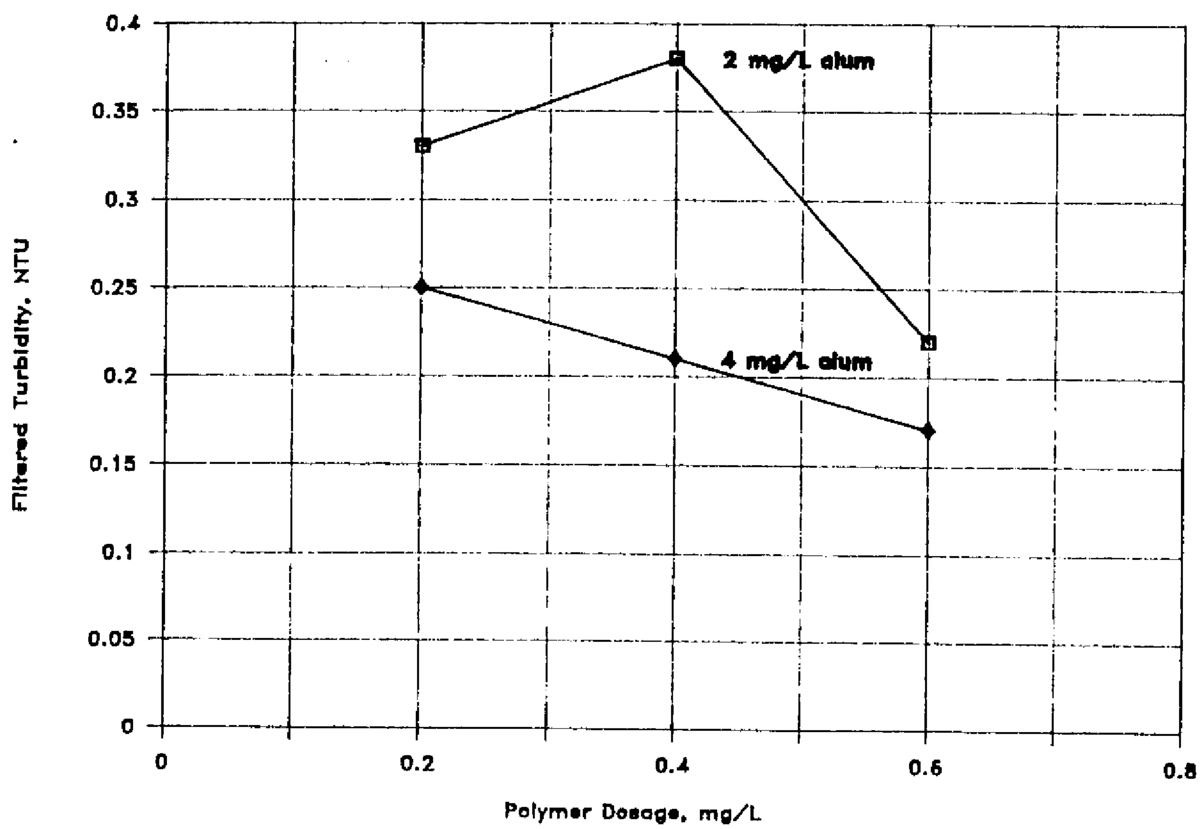

Figure 5—8. Results With Combined Alum/8102

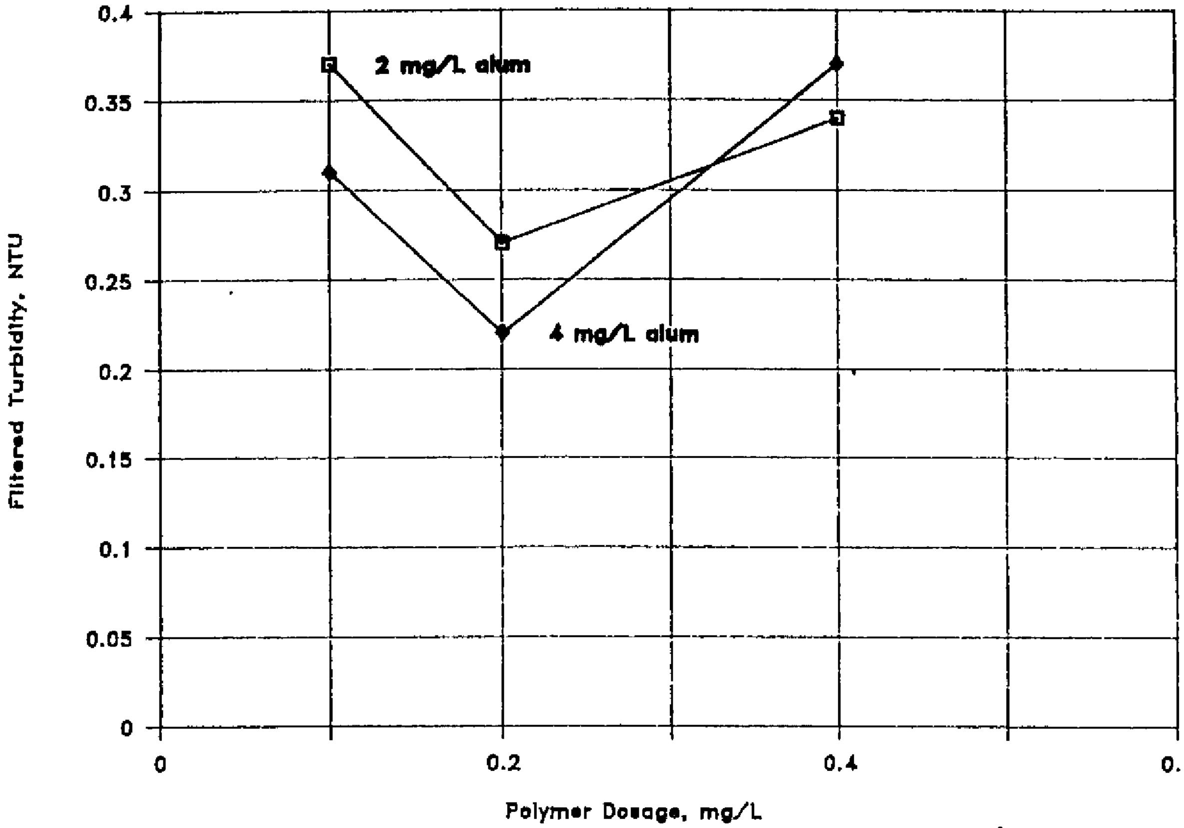

Figure 5—9. Results With Combined Alum/Catfloc DL

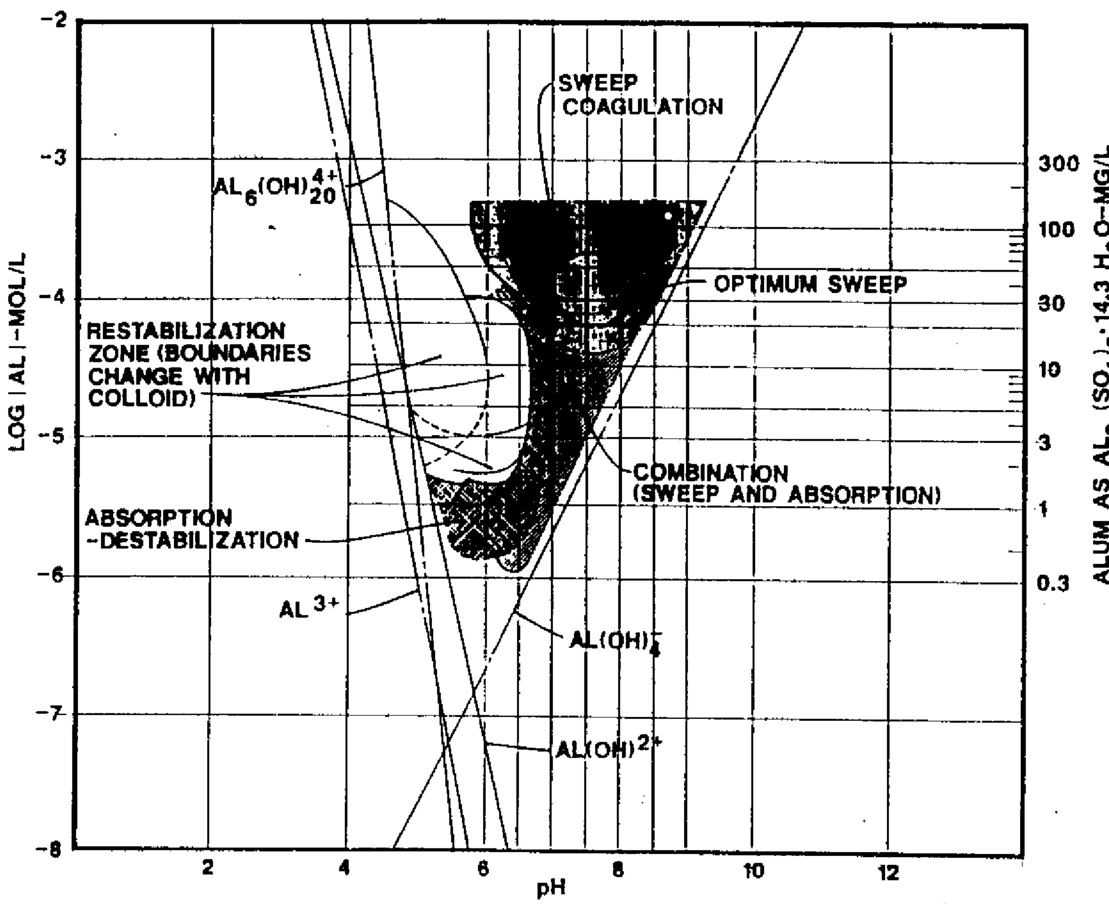

Figure 5—10. Alum—pH Coagulation Diagram

There are, of course, other differences. The plant has an extended contact time, which at 5 MGD, amounts to about 4 hours. This is not believed to be a major factor, however, since the experiment with various flocculation periods showed best results with only 5 minutes.

In spite of the lack of good correlation with the plant results, the tests still are valuable in terms of comparisons between various coagulants.

Alum Coagulation--

The following conclusions can be drawn from the tests involving alum:

Dosage Effect. In the jar tests, a dosage of 8 mg/L was sufficient to produce less than 0.2 NTU. A dosage of 12 to 16 mg/L produced 0.11 NTU. As mentioned earlier, these results are at variance with the plant results, which matched the quality at about one-third the dosage.

pH Control. No benefit was found in reducing the pH of the raw water before alum treatment, in spite of conventional theory that suggests that the natural pH is higher than an optimum level.

Flocculation. Extended flocculation was found to be detrimental to performance.

Polymer Coagulation--

In the tests, the only polymer which gave reasonably favorable results was Nalco 8156, which is an inorganic polymeric compound. It is comparable to treating with a combination of alum and regular organic polymers. It should be kept in mind that the raw water conditions at the time of the tests (very low turbidity) are not generally considered to be the best for application of polymers. They would be expected to fare better when the turbidity is higher.

Also, the jar tests do not provide the means with which to judge the effect on filter run length and on solids production. Both factors will favor the cationic polymers.

Combined Alum and Polymer--
These tests did not reveal any advantages for this combination. The only positive result was achieved with uneconomically high dosages.

Relative Costs

Although the jar test results have not given conclusive results, it is worthwhile to consider approximate relative costs of the alternative coagulants. Table 5-2 contains an analysis of three system. These are based on the water condition as they were at the time of the testing, i.e., with low turbidity.

TABLE 5-2

COMPARISON OF ALTERNATIVE COAGULANTS

	Alum[1] only Dose	Alum[1] only $/MG	Cationic Polymer[2] Dose	Cationic Polymer[2] $/MG	Combination[3] Dose	Combination[3] $/MG
Alum	3	1.50	0	0	1.5	0.75
Cationic Polymer	0	0	0.2	0.83	0.4	2.67
Filter aid	0.01	0.33	0	0	0	0
TOTAL		1.83		0.83		3.42

1 Based on cost of $0.06/lb.
2 Based on Nalco 8156 @ $0.50/lb.
3 Based on Nalco 8102 @ $0.80/lb.

An alum dosage of 3 mg/L was assumed. This is consistent with the plant operation on the second day of the testing, and reflects the historical record.

For the cationic polymer, the Nalco 8156 was assumed to be dosed at 0.2 mg/L, which produced the best result in the tests. This dosage, of course, would need to be confirmed with further testing.

For the combination, it was assumed that the alum dosage could be cut in half when aided by the cationic polymer at 0.4 mg/L.

For the alum system, a filter aid (nonionic polymer) dosage of 0.01 mg/L was used. Cationic polymers possess natural "stickiness", so filter aids are seldom required when they are used.

For the assumed conditions, the cationic polymer offers a reduction in chemical cost of over 50 percent. The alum-polymer combination is not cost effective.

Conclusions

1. For the alum-only system, there appears to be no advantage to adding flocculation, or to the use of acid for pH control.

2. Although not conclusive, the tests indicate that use of the inorganic polymer material in lieu of alum may offer significant cost savings. The chemical cost savings, estimated earlier at about $1.00 per million gallons, would total $5,300 per year at current consumption levels.

Recommendations

1. Run additional jar tests in the spring, when raw water turbidity is higher, to establish the performance of the cationic polymers at that condition.

2. Consider larger scale testing of the cationic polymer in order to verify performance. This could be done on the plant scale, or by using a pilot filtration apparatus. A pilot filter system can be rented for $250 per week; tests could be easily completed within one week.

LOCATION AND ORDER OF CHEMICAL FEED

In order to optimize the operation of any water treatment facility it is important to consider the potential positive and negative effects various process steps can have on one another. At the Richland plant, these include chlorination, coagulation, and the addition of lime, powdered activated carbon, and filter aid (polymer).

Chlorine--

Chlorine not only inactivates microorganisms present in the water but oxidizes other organic compounds including most taste and odor causing substances. However, chlorine can also combine with organic material to produce taste and odor causing substances and products that are known or suspected of being carcinogenic.

When chlorine gas is added to the water, it hydrolyzes to form hypochlorous acid ($HOCl$) which upon proteolysis yields hypochlorite ion (OCl). The relative concentration of these two species is a function primarily of pH but is also affected by temperature. Of the two, $HOCl$ is the strongest disinfectant and for that reason the most desirable. At a pH of about 7.5, the two chlorine species are present in equal amounts. As the pH decreases the hypochlorous acid portion starts to increase and becomes completely dominant at pH 6. Conversely, the hypochlorite ion dominates above pH 7.5 until practically no hypochlorous acid is present at a pH of about 9. The addition of chlorine will cause the pH of the water to decrease.

Chlorine is normally added to the head end of the plant or to the raw water line to aid in controlling microorganisms in the filters. In addition, many water treatment plants chlorinate after the filters. How and where the chlorine is added depends on the primary functions of the chlorine and what other processes are included in the treatment train. If as an example the pH of the water is adjusted to control corrosion, the chlorine should be added and allowed to react before the pH is increased to maximize its effectiveness. When using activated carbon to control taste and odor, it must be kept in mind that 20 parts of carbon will consume one part of chlorine.

Under normal operation at Richland, the water is prechlorinated at the entrance to the Parshall flume. Post chlorination is not common practice unless carbon is used for taste and odor control. Then prechlorination is eliminated and the disinfectant is added to the filter effluent. The practice of adding chlorine only to the influent has the advantage of providing maximum contact time before the water enters the distribution system. Potential disadvantages include increased opportunity for THM formation and less efficient use of the chlorine due to chlorine demand of the raw water. However, THM's are well below existing standards. If in the future it becomes desirable to increase the chlorine in the clearwell to provide additional disinfection, chlorine addition to the filter effluent could be incorporated into the normal operating procedure.

Coagulants--

When the coagulant is added to the water the primary concern is mixing. Because the reactions that follow are very rapid, it is essential that the coagulant becomes dispersed quickly to maximize its effectiveness. Another consideration is the pH of the water, particularly if the coagulant is alum. As illustrated in Figure 5-10 the optimum pH range is about 6.5 to 7.5. When the pH of the water increases, higher dosages of alum are usually required. Therefore, if an adjustment of the pH is desirable to minimize corrosion, any adjustments should be made after the alum reactions are completed. This becomes less important if a polymer is used as the primary coagulant since polymers are effective over a wide pH range. As suggested earlier, relocating the alum injection point to the effluent of the Parshall flume may increase the effectiveness of the coagulant.

Lime--

The main reason for adding lime is to reduce the corrosivity of the water. During most of the year the pH of the water remains in the alkaline range even after the addition of chlorine and alum, both of which undergo chemical reactions that cause the pH to decrease. Occasionally, however, the water can become slightly acidic, particularly during high raw water turbidity events when higher alum dosages are needed. By adding lime the pH can be controlled. Care should be taken when using lime so that it does not interfere with other treatment objectives. There are provisions to add lime at either the influent flume or at the filter influent. Generally, adding the lime to the filter influent will benefit the treatment process and the finished water the most.

Powdered Activated Carbon--

Powdered activated carbon is typically added in the fall when algae blooms in the river cause tastes and odors. Activated carbon can be added at any point in the treatment process prior to filtration. Regardless of the point of application, sufficient mixing must be provided together with adequate contact time. Good mixing assures that the carbon is evenly dispersed in as large a volume of water as possible. Adequate contact time provides the opportunity for the adsorbate to be adsorbed onto the carbon surface. When selecting the application point there are some important considerations to bear in mind. Activated carbon functions best at low pH values and it must be in suspension and circulation to provide adequate contact. Recommended contact times range from 15 minutes to one hour depending on the amount of mixing available. Care should be taken to prevent coagulants and other chemicals from sealing or coating the active surface of the carbon.

If the carbon is added to the rapid mix, it will provide a nucleus for floc formation which can be of value in low turbidity waters. However, the carbons adsorptive capabilities are reduced once it is incorporated into the floc. Addition of chlorine and carbon simultaneously can reduce the effectiveness of both.

The current practice of adding the carbon at the Parshall flume is sound and should be continued.

Polymers--

The user of polymers as filter aid has proven effective, particularly for filters operated at high rates. Filter aids strengthen the floc to avoid premature breakthrough. If the dosage is too high, however, it can prevent the floc from penetrating into the filter and thereby only the upper layers of the filter bed are used, resulting in high rates of headloss buildup and short filter runs. The nonionic or sometimes anionic polymer should be added after the coagulant is fed and at a location upstream from the filters that will allow uniform dispersion before the water reaches the filters.

At Richland, the polymer is added in the channel between the contact basins and the filters. A rectangular pipe-loop is situated in the channel perpendicular to the flow. Several nozzles are located on the pipe, facing inward towards the center of the loop and the passing water. This arrangement provides uniform dispersion of the polymer without intense mixing that may be detrimental to the alum floc.

MIXING CONDITIONS

The purpose of rapid mixing is to obtain complete and uniform dispersion of the coagulant throughout the entire flow of raw water as quickly as possible. With the current location of the coagulant injection point, mixing occurs in the 36 inch riser pipe, the Parshall flume, and in the stilling basin to which the water from the flume is discharged before entering the rectangular conduit leading to the contact basins.

The intensity of the mixing depends on the flow rate as well as the temperature of the water. A drop in each of these parameters will lower the amount of mixing available. Thus the lowest intensity occurs during the winter and early spring and the highest in July and August. The most common parameter used to express mixing intensity is the velocity gradient or G-value. Typical values suggested for mechanical mixers range from 700 to 10,000 and for in-line mixers from 3,000 to 5,000 sec^{-1}.

The initial mixing of the coagulant takes place in the 36 inch riser pipe. Assuming the minimum flow and temperature is 6 mgd and 5 ^{0}C, respectively, and the maximum 30 mgd and 21 ^{0}C, the G- value will range from 20 to 210 sec^{-1}. The detention time in the 10 foot section of pipe would vary from 1.5 to 7.7 sec.

The detention time on the Parshall flume is approximately 15 sec at 6 mgd and 8 sec at 30 mgd. Typically, a flume is designed to incorporate a hydraulic jump to provide mixing. The location of the jump will depend on the backwater depth immediately downstream from the flume. However, because the influent water at the treatment plant is discharged from the flume into the stilling basin, the opportunity for creating a hydraulic jump is lost. Consequently, the mixing that takes place in the Parshall flume is minimal.

As the water from the flume drops into the stilling basin, much of the water's energy is dissipated, resulting in localized vigorous mixing. A portion of this energy is transferred throughout the small basin where further mixing takes place but at lower energy levels. It is estimated that the area with the highest mixing intensity will have a G-value ranging from about 3,100 to 4,700 sec^{-1} depending on the flow rate and temperature. For comparison, the average for the entire stilling basin would vary from 600 to 1,900 sec^{-1}.

The rapid mixing available at the plant seems reasonable for most flow conditions. The seasonal variations in water demand, however, give rise to a wide range of mixing intensities which cannot be avoided as long as the plant relies on hydraulic mixing and its current configuration of only one influent stream. The analysis showed that most of the mixing takes place in the stilling basin. The mixing occurring in the riser pipe accounts for only 3 to 10 percent of the total energy available for dispersing the primary coagulant. By itself, this mixing intensity is inadequate. Although further mixing takes place in the stilling basin downstream, 10 to 23 seconds have elapsed from the time the coagulant was introduced into the influent stream and the point at which this mixing becomes available. When alum is used as coagulant the principal mechanisms are adsorption destabilization and sweep floc coagulation. The hydrolysis of the metal salt occurs in microseconds without the formation of polymers and less than one second if polymers are formed. Sweep floc coagulation is slower, occurring in the range of 1 to 7 seconds. Research has shown, however, that for treatment plants operating in the range of pH and alum dosages (See Figure 5-10) where the floc are formed due to a combination of adsorption-destabilization and sweep coagulation such as the case is at Richland, the mixing intensity or G-value is not as important as the overall mixing, Gt. Only when coagulation occurs by adsorption-destabilization is it essential to provide instantaneous dispersion of the coagulant. This accounts for the effectiveness experienced with alum despite the low initial mixing. Still it is conceivable that better utilization of the alum can be achieved by relocating the injection point to the discharge side of the Parshall flume. It is recommended that a dosing system similar to that for chlorine be installed and its operation alternate with the current system to evaluate its effect on the coagulation process. Instantaneous mixing is less essential when synthetic polymers of high molecular weight are used, since these coagulants species do not have to be formed within the system and their rates of adsorption are slower due to their larger size.

COAGULANT CONTROL SYSTEMS

As mentioned above, control of chemical feed is the most critical plant operation. Next to disinfection, the most important chemical fed is the coagulant. Its feed rate has a direct bearing on the plant effluent quality and on operating costs.
The plant now operates using manual control of coagulant feed rate. The effluent turbidimeter on the pilot filter system provides a quick reading to plant operators of the effect of a change in dosage. This system provides a good level of control. One of the characteristics of this system, however, is that one cannot distinguish between an overfeed and an underfeed situation; both result in high filtered water turbidity. In actual operation, this problem rarely if ever surfaces since an overfeed condition requires very high dosages that would not likely be reached.

Another weakness is that the pilot filter readings generally do not agree well with the actual plant effluent turbidity readings, although this has little practical significance, since they do have a general correlation. Lastly, while the pilot filter system provides quick readings, it is not instantaneous; one must wait several minutes to see the effect of a change.

One approach that is used for monitoring the coagulation involves the streaming current monitor (SCM). This device is based on the electrical charges that are involved in the coagulation process. A sample of coagulated water is passed continuously through the device, and it provides a readout that is indicative of the net surface charge on the particles in the water. This reading may be correlated with effluent turbidity readings. This provides the operator with another tool in making decisions regarding chemical dosage.

The advantage that could be gained with the SCM is that the operators would have more confidence in the adjustment of chemical feed. This would result in savings since there would be less tendency to be conservative in setting the dosage. It would also provide a quicker response to changes.

An SCM made by Chemtrac was installed in the filtration plant on a trial basis this past fall. It was found to work, but since the raw water conditions were very stable during that period, little use could be made of the information generated. The operators did note that it was useful to have the additional readings. The device will be brought back to the plant in the spring, when raw water turbidity will be higher and more variable. That will provide a greater challenge, and will also be of greater use to the operators. It is recommended that the decision on whether to purchase the machine be made after that trial period.

The SCM can also be used as a coagulant controller, in which it provides an output to the coagulant pump and controls it to maintain a preset voltage reading. In theory, this would eliminate the need for operator intervention in the control process. This type of control has its limitations, however. It is generally used in situations in which rapid changes in raw water turbidity are experienced and in which continuous supervision is not provided. These situations do not exist at Richland. Turbidity swings are gradual; there is ample time for operators to respond. It is therefore not recommended that the control option be considered at this time.

The SCM has a cost of $7,000 to $10,000, depending on the options chosen.

Another type of coagulant controller (Microfloc "Aquaritrol") uses a microprocessor to control the coagulant feed rate. The control is based on maintenance of a preset effluent turbidity. In effect, it automates the current practice. This has an advantage over the SCM in that effluent turbidity is a more direct indication of the desired operating conditions. The weakness of the system is that it requires that the turbidity signal be correct. There are several process and equipment problems that can lead to incorrect readings, and the controller cannot detect these problems. Again, for the situation at Richland, with stable raw water conditions, it is not felt to be justified to implement this type of system.

SECTION 6
WATER SUPPLY MANAGEMENT

INTRODUCTION

In this chapter, the various issues regarding future management of the City's water supply sources are examined.

IMPACT OF SDWA AMENDMENTS ON RICHLAND

The Safe Drinking Water Act Amendments were adopted in June of 1986. The new federal regulations apply to all systems serving 25 or more customers. Prior to the Amendments, allowable concentrations for 22 contaminants had been established by EPA. A list of 83 contaminants had been published by EPA for which it was felt that allowable concentrations should be considered. These 83 contaminants, 14 volatile organic compounds, 35 other organics, 23 inorganics, 6 related to microbiology or turbidity, and 5 radionuclides, will be regulated as a result of the new legislation. In addition, several unregulated organic compounds must be monitored. This means that operators are facing monitoring and reporting of ever increasing number of contaminants. (See Appendix E for an article discribing the amendments in further detail).

The filtration and disinfection requirements of the Safe Drinking Water Act Amendments, were proposed tó protect the public against the potential adverse health effects of exposure to Giardia lamblia, viruses, Legionella, and many other pathogenic bacteria that are removed by these treatment techniques. For systems that already practice filtration, the new rule specifies what parameters and how to report them to the State as well as the frequency of such reporting.

For systems that provide filtration, the proposed rules address plant performance as measured by the filtered water turbidity. The second barrier to microbial contaminants, disinfection, must meet certain criteria at the plant and also in the distribution system. The standards and monitoring requirements that apply to the distribution system are the same for water systems that provide filtration as for those that are exempt from filtration and rely solely on disinfection. A detailed outline of the proposed rules will not be given here, only a short summary to indicate what information is likely to be required for reporting to the State. (For additional information see: Federal Register/Vol. 52, No. 212/pp. 42178-42222/Tuesday, November 3, 1987.

Revised Turbidity Standard--

The new maximum contaminant level (MCL) for turbidity is 0.5 NTU for 95 percent of the time (down from 1.0). The Richland water treatment plant can easily meet that goal with proper chemical pretreatment. The operating records show that the filtered water turbidity has successfully been maintained below this level even at the most extreme raw water turbidity events. Rebuilding the filters will further assist the operators in meeting the effluent quality standards.

To monitor compliance with the MCL for turbidity, representative filtered water grab samples must be taken in four hour or shorter time intervals. Continuous turbidity monitoring can be substituted for grab sampling if the accuracy of the continuous records are validated by grab sample measurements on a regular basis as specified by the State.

Included in the monthly turbidity report to the State must be the total number of turbidity measurements, the date and values of any measurements which exceed 0.5 NTU, and the percent of the turbidity measurements that are less than 0.5 NTU. Of the total number of measurements taken every month 95 percent must be less than or equal to 0.5 NTU.

Disinfection Requirements--

All water systems will be required to provide, as a minimum, a 3 log (99.9 percent) removal/inactivation of <u>Giardia</u> cysts and a 4 log (99.99 percent) removal/inactivation of enteric viruses. For systems using filtration, it is assumed that 90 percent or 1 log of the cysts and viruses are removed by the filters, provided the treatment plant meets the turbidity performance requirements. The additional reduction must be achieved by disinfection before the water reaches the first customer. Based on the temperature and pH of the water at Richland, a CT value (free chlorine concentration in mg/L times the contact time in minutes) of about 140, according to the EPA regulations, would be required during the winter and early spring when the water is cold. The required CT value could be reduced to approximately 85 in the summer and fall.

An evaluation of the present disinfection practices is not part of this study and may be more beneficial once the new requirements have been finalized and adopted. However, if the well field can continue to operate as it has in the past, it appears that only minor changes/modifications will be needed to meet requirements for disinfection of the well water. The Federal guidelines for disinfection of ground water will not be developed until next year.

Adequate disinfection of the filtered water may require modifications to the clearwell, such as installation of baffles, to assure adequate contact time. To meet the CT requirement of 140 in the spring at a flow of 20 mgd would mean about 2.8 mg of contact at the normal residual chlorine level of 0.7 mg/L. The plant contact basin has about 0.8 mg, meaning that a full 2.0 mg would be needed in the 2.2 mg clearwell. This would require that any short circuiting be eliminated. It is anticipated that a tracer study will be needed to evaluate the current detention time in the clearwell before recommendations for modifications can be made.

Monitoring Requirements

Aside from the CT value, the City will be required to continuously monitor the chlorine residual of the water after filtration but before it enters the distribution system. A chlorine residual of at least 0.2 mg/L must be maintained at all times. Water samples for chlorine analysis must also be collected from representative sections of the distribution system, at the same frequency and location as required for total coliform measurements. For Richland, a minimum of 35 samples per month from a total of 105 locations in a 12 month period will be required. The chlorine residual cannot be less than 0.2 mg/L in more than five percent of the samples in a month, for any two consecutive months, on an ongoing basis.

Coliform Standards

For total coliforms in the distribution system there are two proposed MCL's that would neet to be met, a monthly MCL and long-term MCL. To comply with the monthly MCL, only one of the 35 samples can be positive. The long-term requirement permits up to five percent of the 60 most recent samples to be positive. Hence, the water distribution system could occasionally be in violation of the monthly requirement and still meet the long-term MCL, if the samples in the next 30 days are all negative. The draft requirements for coliform sampling also require that sampling locations be changed on a regular basis.

WELL FIELD STATUS UNDER SDWA

Background

A question that must be answered before finalizing the long range water management plan is the future status of the well field. Will it be classified as a surface water source, or can the City continue to operate it as a ground water source? The Surface Water Treatment Rule (SWTR) which is part of the Safe Drinking Water Act Amendments, defines a surface water as all waters that are open to the atmosphere (e.g., rivers, lakes, streams, impoundments, etc) and any subsurface sources such as springs, infiltration galleries, wells or other collectors which are at risk of being contaminated by surface water. Since a major purpose of the proposed rule is to control Giardia lamblia, being at risk of surface water contamination primarily refers to contamination by Giardia cysts.

If the well field is classified as surface water source it is highly unlikely that filtration of the water will be required. However, it would be necessary to conform to the surface water disinfection requirements of the SWTR. In short, the CT value would need to be about 100. Backup chlorination equipment with automatic switchover would be required along with continuous monitoring of the chlorine residual before the water enters the distribution system. The net effect is that the cost of using the well fields could increase if they are classed as a surface water.

It is the responsibility of the State to assess the risk and classify the water sources that have a potential for surface water contamination. According to the SDWA, the State has 18 months following the promulgation of the SWTR to finalize their own criteria for classifying subsurface sources and determining which systems must filter. The evaluation of the systems covered by the SWTR must be completed within 12 months. The SWTR will be promulgated in June, 1988.

At present, the State, in cooperation with the USEPA, is studying about one dozen subsurface systems to aid in determining what parameters are most reliable and practical in classifying these systems. In a recent conversation with the project manager of the study, it was made clear that insufficient data had been collected to date to have an indication of what the criteria might be. Until a draft of the State criteria is available in late spring, at the earliest, one can only speculate on what approach will be chosen. It seems safe to assume, however, that particle counting and size distribution without identification of the particles will not be used as a criteria by itself. This approach could erroneously classify a system as surface water just because the water contains particles in the Giardia cyst size range even though these particles could be of a completely different origin. Conversely, if all particles in certain size ranges had to be identified microscopically, it would place a heavy work load on the relatively few people who have the training and experience to make such identifications.

With the upcoming regulations still not formulated, only some general thoughts can be provided at this time. To assist the authors in the preliminary assessment of the well field, five water samples were collected on November 13, 1987, and analyzed for total hardness and particle size distribution. The hardness analysis were performed at the treatment plant by plant personnel and the particle analysis by Dr. J. Engeset using a Coulter Counter, model ZBI at the University of Washington. The five water samples were collected from: 1) Columbia River water entering the recharge basins, 2) Well 3000-H, 3) Well 3000-B, 4) Well 3000-D5, and 5) Well 1100-B.

Assuming the hardness of the water from Well 1100-B is representative of the level in the ground water at the well field, the water pumped from the three 3000-wells consisted of approximately 94 percent surface water and 6 percent ground water. By applying this information to the actual particle counts of the surface water and the water from Well 1100-B, the estimated particle concentrations in the other wells, assuming no particles in the surface water percolating downward through the soil would be retained or altered, could be calculated. These estimated numbers together with the actual particle counts in the various size ranges are shown in Table 6-1.

A casual look at the table reveals a significant reduction of particles in the surface water by the time it reaches the ground water table and is pumped out of the well. Upon closer examination of the data, it appears that the smaller sizes are retained more readily by the soil than some of the larger ones. Although this is possible since physical straining is only one of several mechanisms by which particles are removed in the soil, it could also be that some of the smaller particles eventually flocculate to form larger ones. This phenomenon not only would give the appearance of the smallest particles having the highest percentage removal, but also tend to underestimate the reduction of the naturally occurring larger particles.

Because the identity of the large particles and particularly those in the Giardia cyst size range, 8 to 12 microns is not known, it is not possible to have a firm opinion as to how the well field will be classified, ground water or surface water. The determination cannot be made until the State criteria for the evaluation have been promulgated. However, the soil is capable of retaining the majority of all particles present in the recharge water, regardless of size, which is encouraging information.

WATER SUPPLY NEEDS

Water supply requirements were studied in the City's Water System Plan, dated September, 1987. In the report it was stated that the annual average water consumption was about 410 gallons per capita per day. Peak day usage is about 950 gallons per capita per day. Population in 1982 was used as a base for projections, and annual growth of 2.7 percent was assumed. These values resulted in a projected 1994 population of 46,120. From the actual population data contained in the report, it appears that this growth rate is a conservatively high estimate; population actually declined almost 10 percent between 1982 and 1985, to 30,508. With the uncertain future of the service area, any type of projection is perilous. For the purposes of this analysis, the 1994 projections from the Water System Plan are used. Using the above factors, these result in an annual average demand of 18.9 mgd, and a peak day demand of 43.8 mgd.

Table 6—1. Particle Analysis of Recharge and Well Water

Water Source	PARTICLE SIZE RANGES - MICRONS: 2 - 4 — Particle Concentration: Expected(1) no/ml	2 - 4 — Actual(2) no/ml	2 - 4 — Reduction(3) %	4 - 8 — Particle Concentration: Expected(1) no/ml	4 - 8 — Actual(2) no/ml	4 - 8 — Reduction(3) %	8 - 14 — Particle Concentration: Expected(1) no/ml	8 - 14 — Actual(2) no/ml	8 - 14 — Reduction(3) %	> 14 — Particle Concentration: Expected(1) no/ml	> 14 — Actual(2) no/ml	> 14 — Reduction(3) %
Recharge Water Entering Recharge Basins	--	22,300	--	--	2,360	--	--	336	--	--	88	--
Well 3000-H Depth 57 ft Close to Basins	21,440	2,240	92	2,270	1,020	57	323	148	56	87	40	57
Well 3000-8 Depth 87 ft Close to Basins	21,300	2,690	90	2,250	450	83	321	60	84	87	28	73
Well 3000-05 Depth 137 ft About 250 ft North of Basins	21,560	3,290	86	2,280	536	78	325	44	88	87	28	73
Well 1100-8 Depth 77 ft Not Influenced by Recharge Basins	--	8,650	--	--	916	--	--	132	--	--	76	--

(1) Anticipated particle concentration in recharge and ground water mixture
(2) Actual particle concentration
(3) Particle reduction in recharge water portion in well

RELATIVE COSTS OF SUPPLY - FILTRATION PLANT AND WELL FIELD

Cost calculations made using the spreadsheet model described in Section 4 are summarized in Figure 6-1. The costs are shown for the filtration plant during the three "seasons" during which water conditions are significantly different, and as they vary with the plant flowrate. The flows are extended as high as 12 gpm/sf (36 mgd), even though the plant is not rated at that capacity. Also shown is the calculated cost of the well field operation, which is $16/mg (this is based on a recharge flow of only 1.5 times the produced flow; current rates are higher). Except for high flowrates during the spring, when water quality conditions are the most difficult, and therefore most costly to deal with, the filtration plant is less costly, and should therefore be the primary source. The well field is competitive, however, and can be used when needed without a great cost penalty. If the well field is classified as a surface water, it's economic position relative to use of the filtration plant would probably worsen.

RECOMMENDED PLAN FOR MEETING FUTURE NEEDS

Planning for future water supply is dependent primarily on flow requirements. As discussed above, conditions in Richland are not such that reliable projections can be made at this point. However, for even the most liberal flow projections, the existing system has the capacity to meet demand for at least the next ten years.

The other unknown at this point is the long-term status of the well field. If it is classified as a surface water source by the DSHS, it is possible (but not likely) that it will be required to be filtered. As a surface source, however, more stringent (presumably) disinfection requirements would need to be met which will include installation of additional disinfection equipment and provisions for increased contact time. No ruling by DSHS will be made until 1990.

With the capacity of the filtration plant restored to 30 mgd with media replacement, it is likely that the system demands could be met even without the well field, unless rapid growth is experienced. So, even if the well field were to be shut down immediately (also not likely), a water shortage would not occur. Options at that

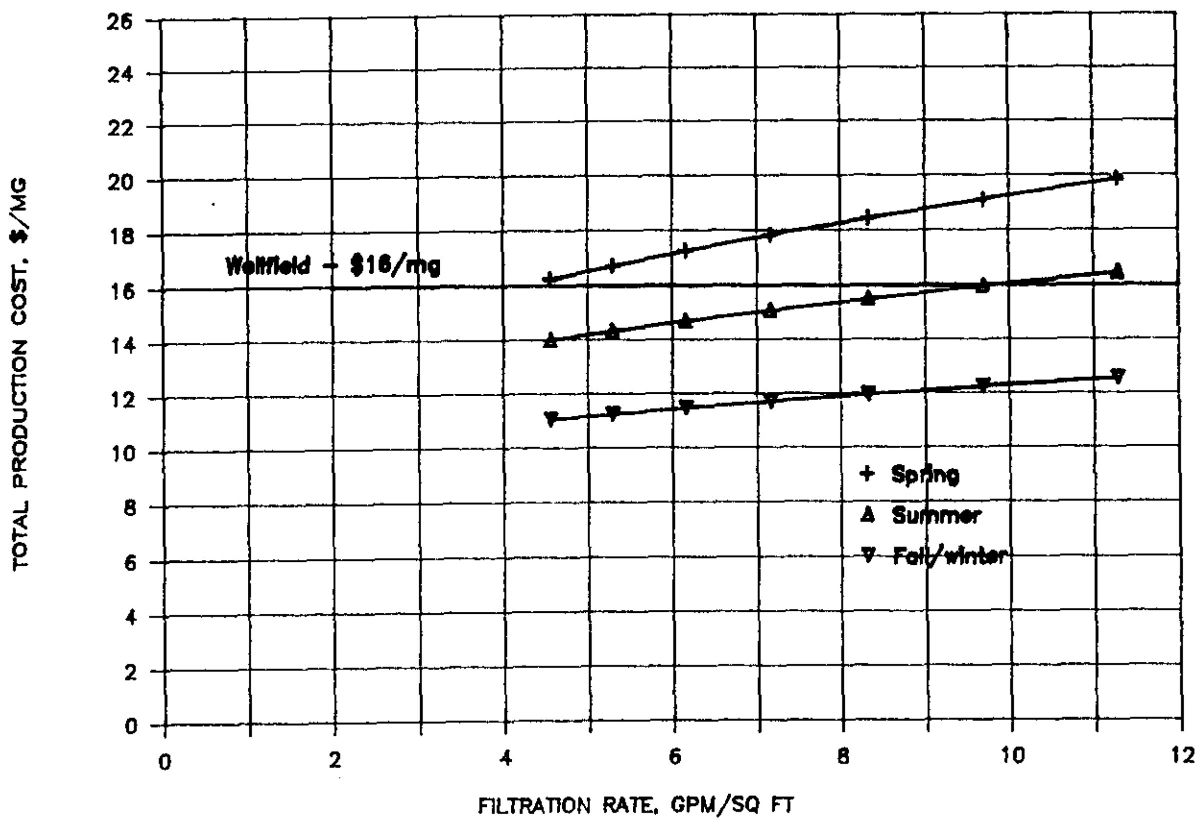

Figure 6-1. Filtration Plant Production Costs vs. Flow by Season

point would include conversion of the well field to a slow sand filter operation, or addition of filters to the filtration plant.

For the immediate future, it is recommended that the current practices be continued. The filtration plant should be used to satisfy the majority of system demands throughout the year, with the wells augmenting flows when necessary.

The expansion of the filtration plant is not cost-effective at this time, and will not be unless the well field becomes unavailable. Even then, plant expansion would be justifiable only if water demands increase.

APPENDIX A

HYDROGEOLOGIC STUDY OF

NORTH RICHLAND

WELL FIELD AND

GROUNDWATER

RECHARGE BASINS

Prepared By

ICF Northwest

November 1987

Principal Contributors:

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James G. Bush, Geologist

Frederick W. Bond, Hydrologist

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1.0 INTRODUCTION

ICF Northwest, under subcontract to HDR/CWC, Inc., has conducted a hydrogeological study of the City of Richland's North Richland Well Field and Groundwater Recharge Basin System. This study includes evaluation of current and historical operations of the system, on-site evaluation of the condition of the recharge basins, and aquifer evaluations through pumping tests using the pumps in place in the system.

The North Richland Well Field has been a significant historic source of drinking water for the City of Richland and continues to provide the largest portion of product water not processed through the city's filtration plant. In addition, the North Richland Well Field is the primary source of water during the annual winter shut-down of the filtration plant for maintenance.

Since the well field continues to be an important water source, the objectives of this study were two-fold:

1) evaluate the physical condition of the recharge basins and recommend maintenance procedures; and
2) evaluate the productive capacity of the native aquifer at the well field and recommend efficient pumping strategies accordingly.

The methods used to evaluate the condition of the recharge basins include the following:

1) observation of near-surface sediments in cores and hand-dug pits;
2) measurement of surface infiltration rates using a concentric ring infiltrometer at locations of observed extremes in surface conditions;
3) collection of samples in three-inch increments from the top foot of sediments in the basins and analysis of particle size distribution of the samples.

Evaluation of the aquifer at the well field was done through application of the following methods:

1) constant rate pumping tests of two wells using pumps in place and using nearby wells as monitoring wells;

2) calculation of coefficients of storage and transmissivity based on conditions observed during pumping;
3) evaluation of geologic strata as indicated in well logs of individual wells.

2.0 HISTORIC OPERATIONS

Since construction of the Richland Water Filtration Plant, the North Richland Well Field has been used to produce a daily average ranging from 0.5 to 7.8 million gallons of water per day. Water is pumped from the well field for 10 to 12 months of the year with the highest production occurring during the summer months of June through August and an additional peak in production during January and February when the filtration plant is shut down for maintenance.

The aquifer at the well field is recharged via a system of settling and recharge basins centrally located at the well field. Figure 1 indicates the location of the recharge basins and the production wells in the North Richland Well Field. Water from the Columbia River is pumped from the City's intake structure near the filtration plant to the settling basin through a 27 inch line. The recharge water enters the south end of the settling basin and flows to the extreme north end of the settling basin before discharging through a concrete weir and flow divider into the two recharge basins. Recharge flows into this system range from zero during low production periods to as high as 16.0 million gallons per day during July. Figure 2 illustrates the monthly totals for recharge and production for the years 1985 through 1987. The relationships between recharge and production are discussed in more detail in the section dealing with pumping strategies and recommendations.

The product water from the well field is treated with chlorine by a chlorinator system at the well field and then discharged directly into the city's supply system. No additional filtration or chemical treatment is applied.

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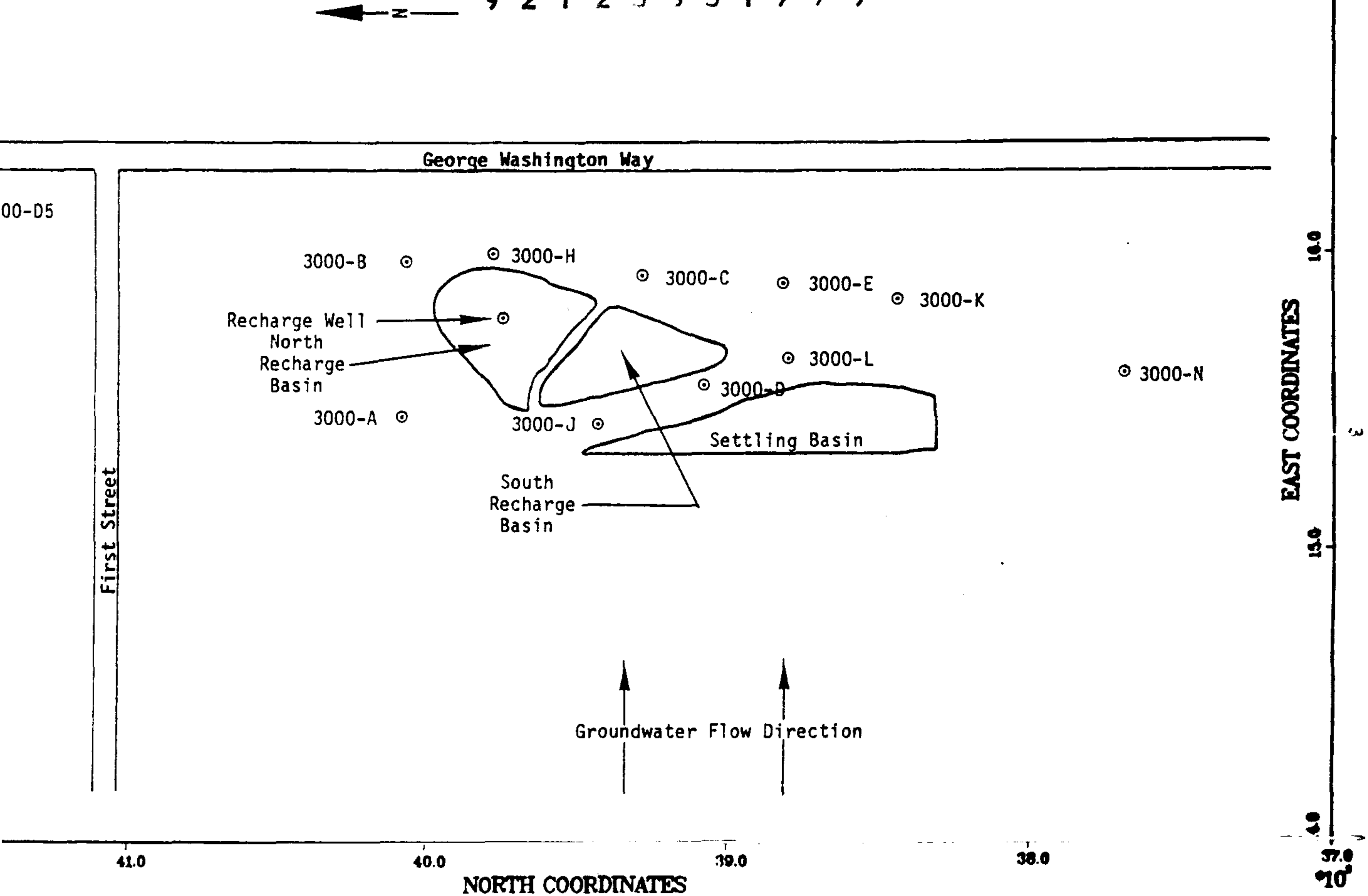

Figure 1. Location and Layout of North Richland Well Field and Recharge Basins.

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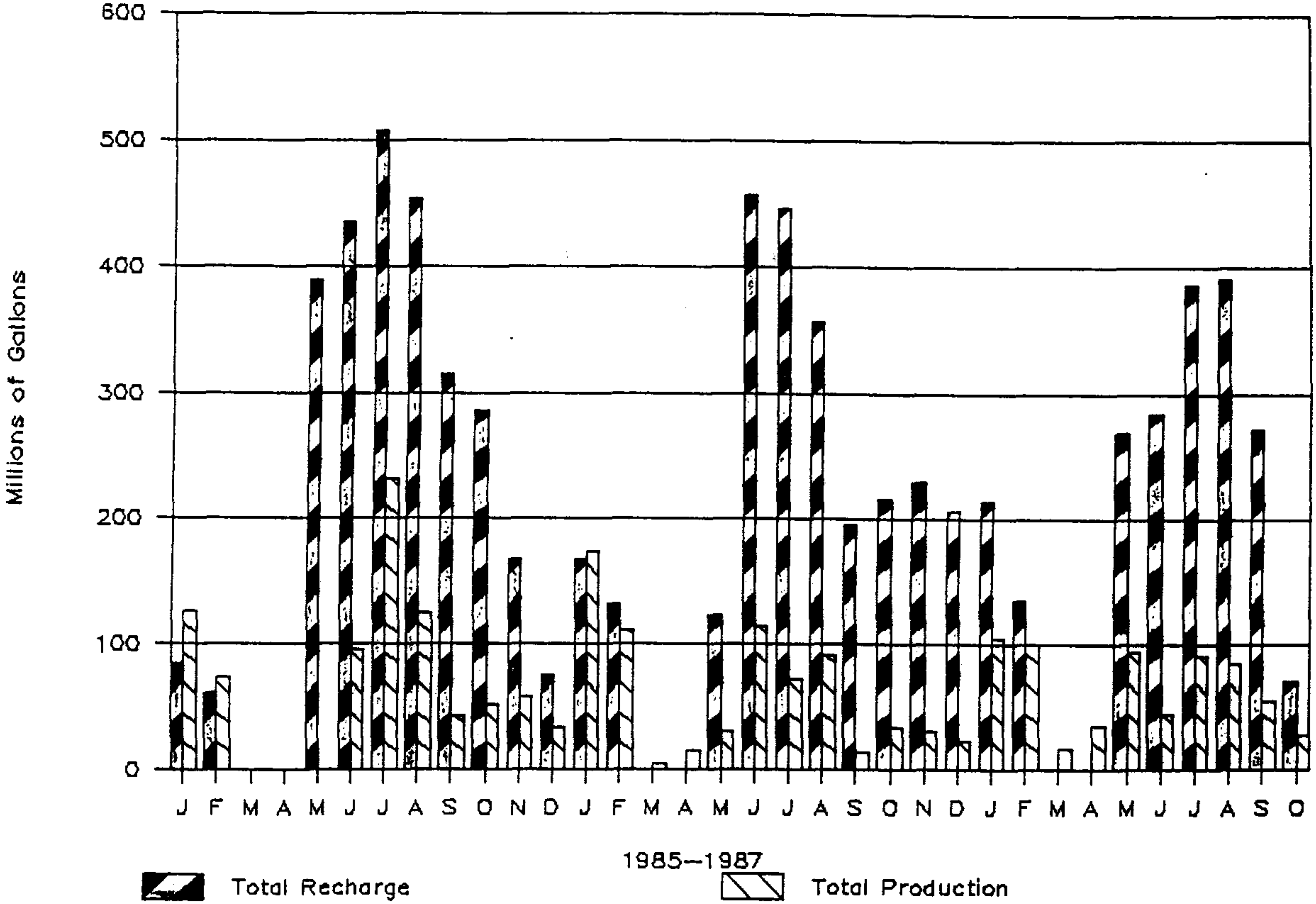

Figure 2. Total Recharge and Production for North Richland Well Field 1985 - 1987.

3.0 HYDROGEOLOGY

There are eleven production wells in the North Richland Well Field and the productive capacities of each varies widely from neighboring wells. A general description of the hydrogeology of the Richland area is given by Deju and Gephart (1976).

The surface layer of the North Richland Well Field area consists of approximately 25 feet of geologically young glaciofluvial deposits informally known as the Hanford Formation. This material consists of a heterogeneous mixture of boulders, rocks, gravels and sands. This layer is underlain by 100 to 150 feet of a much older alluvial deposit known as the Ringold Formation. The Ringold Formation is much finer textured than the overlying Hanford Formation and includes local deposits of fine silts and clays. The water table in the North Richland area occurs near the interface between the Ringold and Hanford deposits.

The groundwater in the North Richland area flows eastward from the recharge of the Yakima River in the west to discharge into the Columbia River. A groundwater contour map of the North Richland area compiled in 1985 is shown in Figure 3. This map indicates a notable depression in the aquifer in the vicinity of the North Richland Well Field, with two well levels measured at 340 feet above Mean Sea Level (MSL). This level was fourteen feet lower than levels observed during the current study where water levels near 354 feet MSL occurred in all wells in the field. During the two weeks of field work, the water level in all wells decreased approximately two feet. This trend is illustrated in Figures 4, 5, and 6, which show the observed water levels in upgradient, downgradient, and one distant well respectively. This trend most likely reflects some degree flattening of a groundwater mound beneath the recharge basins created by the recharge immediately prior to the field studies.

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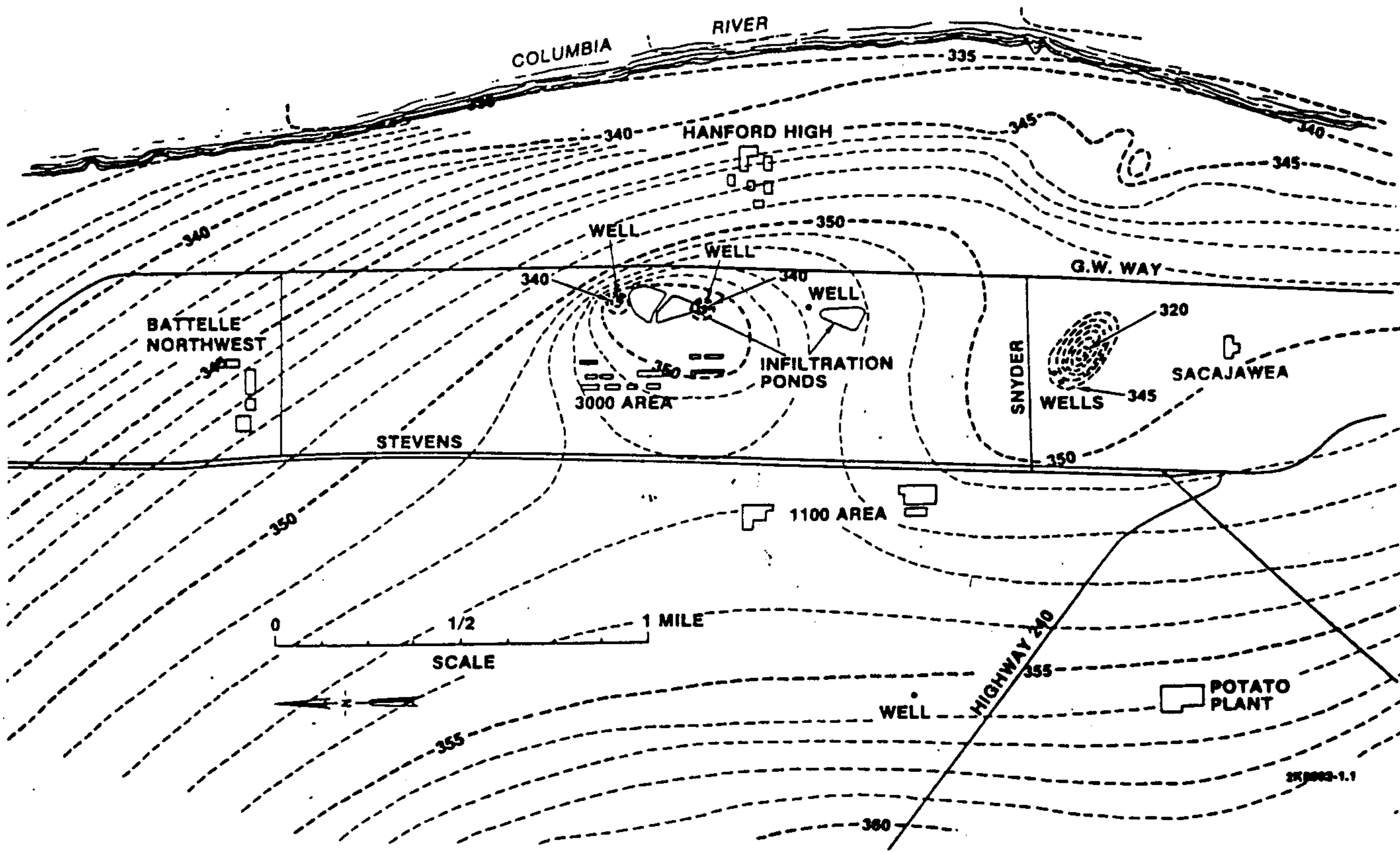

Figure 3. Ground Water Contour Map, North Richland Area, 1985.
(Source: Gerton, 1985)

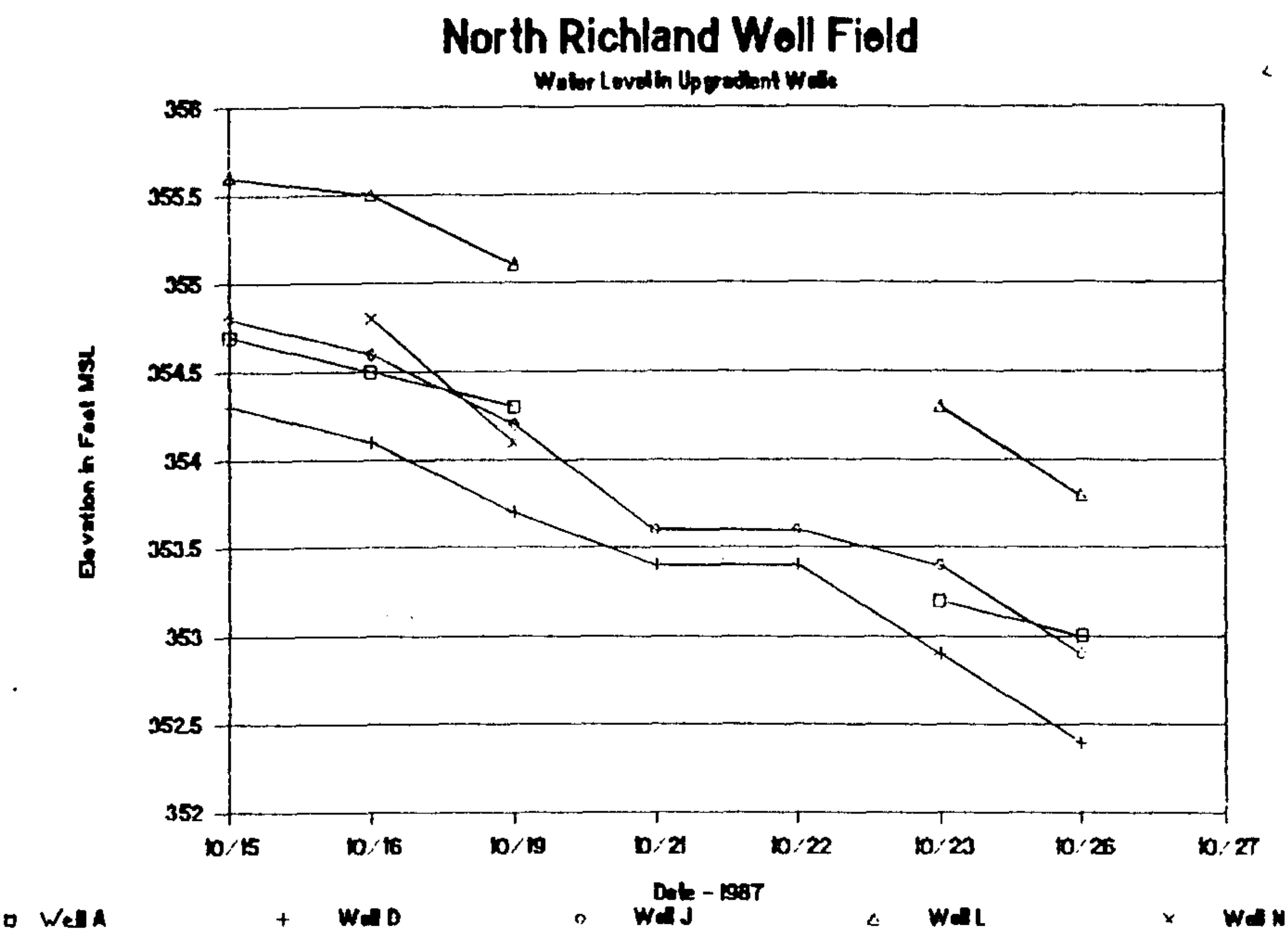

Figure 4. Water Levels Observed in Upgradient Wells, 1987.

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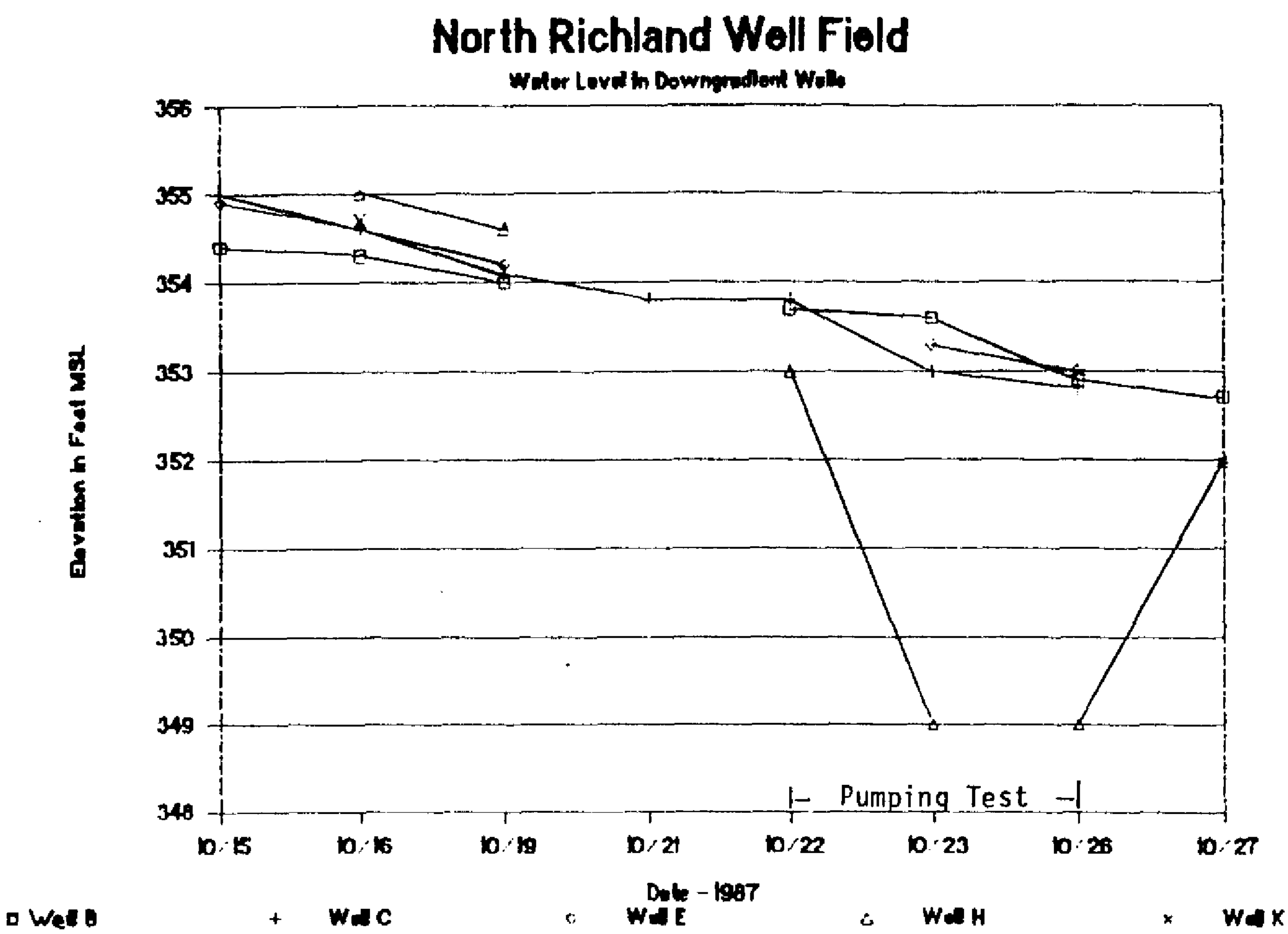

Figure 5. Water Levels Observed in Downgradient Wells, 1987.

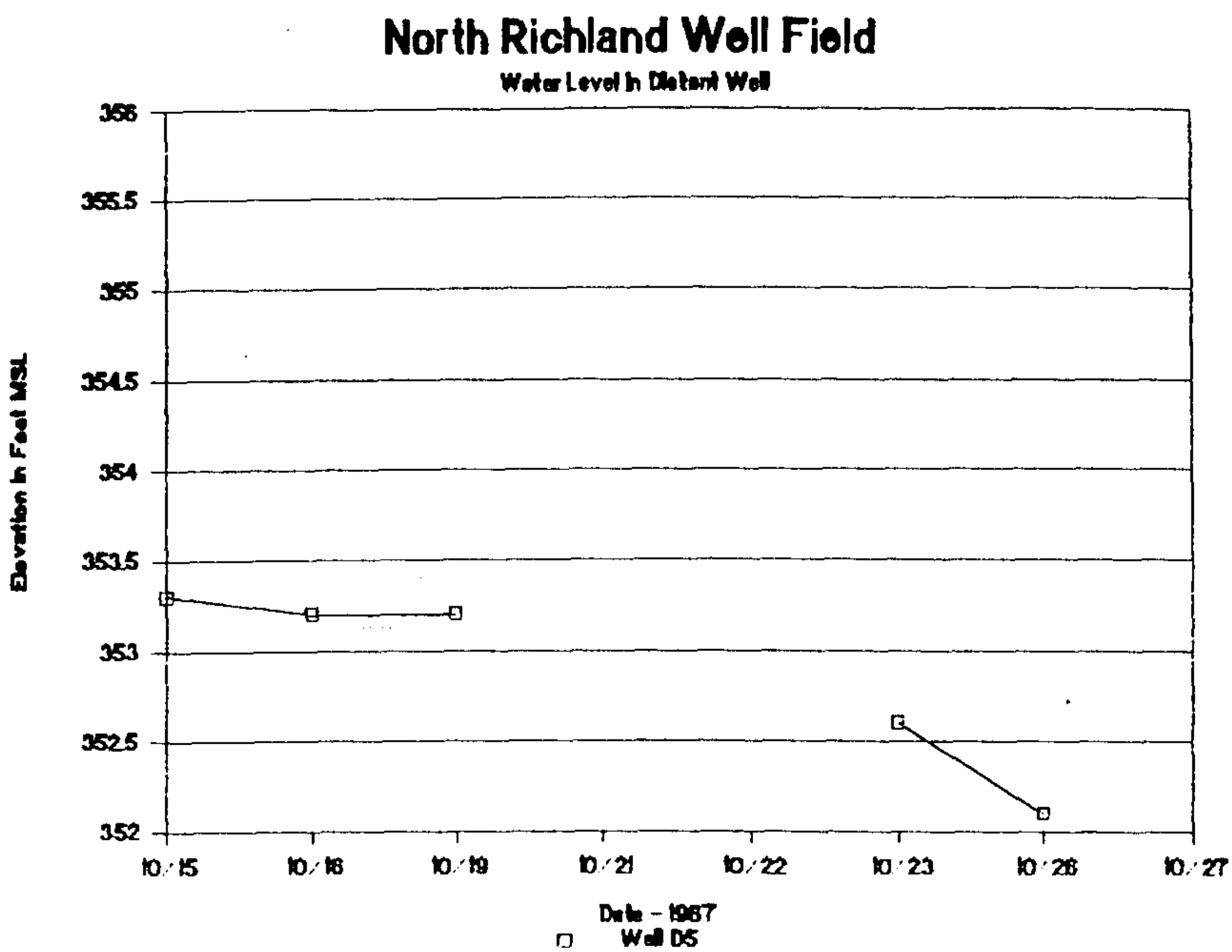

Figure 6. Water Levels Observed in Distant Well (3000-D5), 1987.

4.0 EVALUATION OF WELL LOGS

A study of the existing well logs of the North Richland Well Field was performed to evaluate the yield potential of the wells based on observed strata. Available well logs indicate that the aquifer is very complex. Subsurface strata differ substantially between neighboring wells. Geologic evaluation of the well logs indicates that individual well stratigraphy is primarily responsible for the different production characteristics of the wells.

For the purposes of this report, the subject wells have been divided into three major groupings, those with the best, moderate, and lowest yield potential, based on rock characteristics identified in the well logs and their positions relative to natural aquifer flow. The age of the well logs (most over 40 years) and lack of precise definition of some strata prevent detailed evaluation, however, the following general descriptions are consistent with the operational history of the well field.

Appendix A contains copies of the well logs for the North Richland Well Field. For the purposes of this interpretation, well log references to "clay", "silt", "rock", "cemented", or "tight" materials were assumed to be less permeable to water than those described as "gravel", "sand", "stones", and "boulders."

The wells of the highest yield potential, based on hydrogeologic interpretations, are wells 3000-J, D, B, and C. Wells 3000-J and 3000-D penetrate favorable rocks and probably receive water from the aquifer and from the south recharge basin and the settling pond. These wells should have high yields. They may benefit from installation of more casing perforations, particularly well 3000-D which indicates seventeen feet of native static water level head above the screen. The lower static water level in 3000-J may somewhat limit its yield during low recharge periods.

Wells 3000-B and C are completed in excellent rocks and have static water level fifteen feet or greater above the casing perforations. Upgradient wells A, J, and D may be extracting some aquifer water, however, B and C should receive ample recharge from both the north and south basins.

Wells 3000-K, D-5, and N show moderate yield potential. Well K terminates in a clayey horizon and is capped by a cemented gravel and sand. It has a thirty-five foot perforated interval in rocks with favorable permeability. Well K may recharge from the settling pond assuming the cemented gravel and sand cap do not extend beneath the pond, or the cap is permeable. The well has good potential and has no directly competing upgradient well.

Well D-5 penetrates rocks with favorable yield properties, however, its static water level is only three feet above the perforations and it is far removed from the recharge basins. It probably produces primarily from the aquifer through seventy feet of perforations.

Well 3000-N is similar to well K although located some distance from the recharge basins. It penetrates a slightly clayey layer from 351 to 346 feet MSL elevation, just below the static water level, but shows good potential.

Four wells, 3000-E, L, A, and H, have the lowest yield potential due to completion in poor quality rock units within the perforated interval. Logs of all four of these wells indicate less permeable sediments in 44% or more of the perforated interval and contain either overlying aquitards or low static water level.

Well A is completed in rocks with poor permeability characteristics. Most of A's production probably comes from an eleven foot confined sand and gravel interval overlain by two clayey units. It may produce from the aquifer more than from the recharge basin water.

Some data are missing from the log of well E. A sixteen-foot section of the perforated zone from elevation 311 to 327 feet MSL is not described in the log. It was assumed for this evaluation that this sixteen-foot zone is permeable to water. Well E has poor quality rocks in the upper part of the perforated interval and penetrates poor rocks higher in the well. We assume that "stone" means "cemented sediments" and therefore is less permeable. Well E is also constrained by an upgradient well, 3000-L.

Well L's poor yield may be improved by perforating the casing higher in the well. The perforated interval has no overlying clay beds so it should easily recharge from above. Its production without recharge will be limited, however, because static water level is only six feet above the perforations.

The perforated interval in well 3000-H includes some less permeable rocks. Only the upper fourteen feet are in excellent rocks and the top of the perforated interval is at the static water level. In addition, a cemented gravel layer occurs about five feet above the static water level. If the cemented gravel layer is extensive and indeed less permeable, it may inhibit recharge from above.

Figures 7 and 8 indicate the significant features of the well log interpretations. The positions of screened intervals in the wells relative to the currently observed water level is shown in Figure 9. The screened intervals of all wells except 3000-H are below the water level of 352 feet MSL. Figure 10, however, indicates that at water levels of 340 feet MSL, as observed in the 1985 study (see Figure 3), significant portions of the screened intervals of eight of the eleven wells would be above the water level.

The Recharge Well, located in the approximate center of the north recharge basin, is blocked, apparently filled in with silty material at a depth of approximately five feet below the surface of the basin floor. This well should not be used for any water level measurements unless the well is first cleaned out and rehabilitated.

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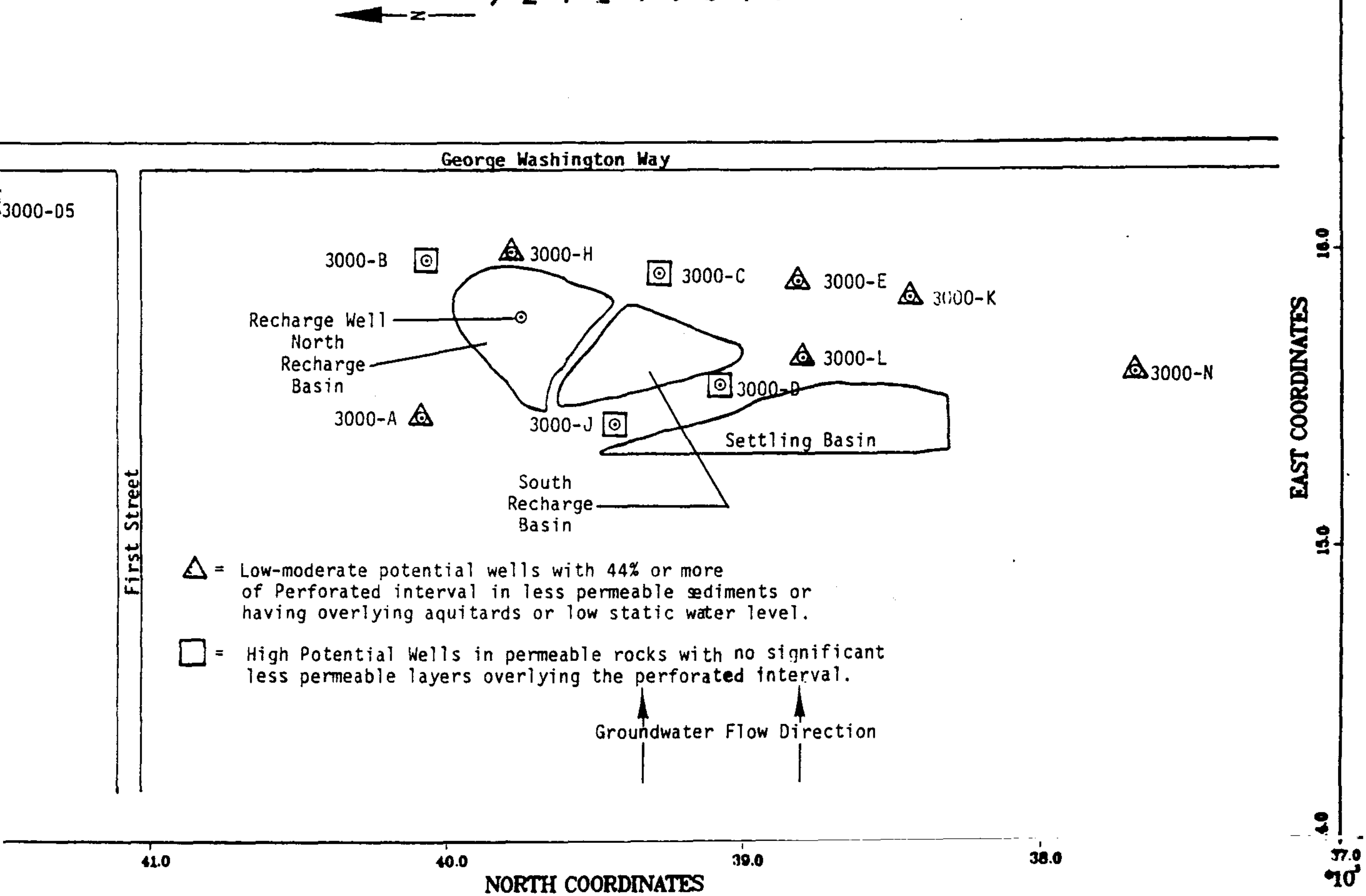

Figure 7. Relative Permeability of Strata Based on Well Log Data.

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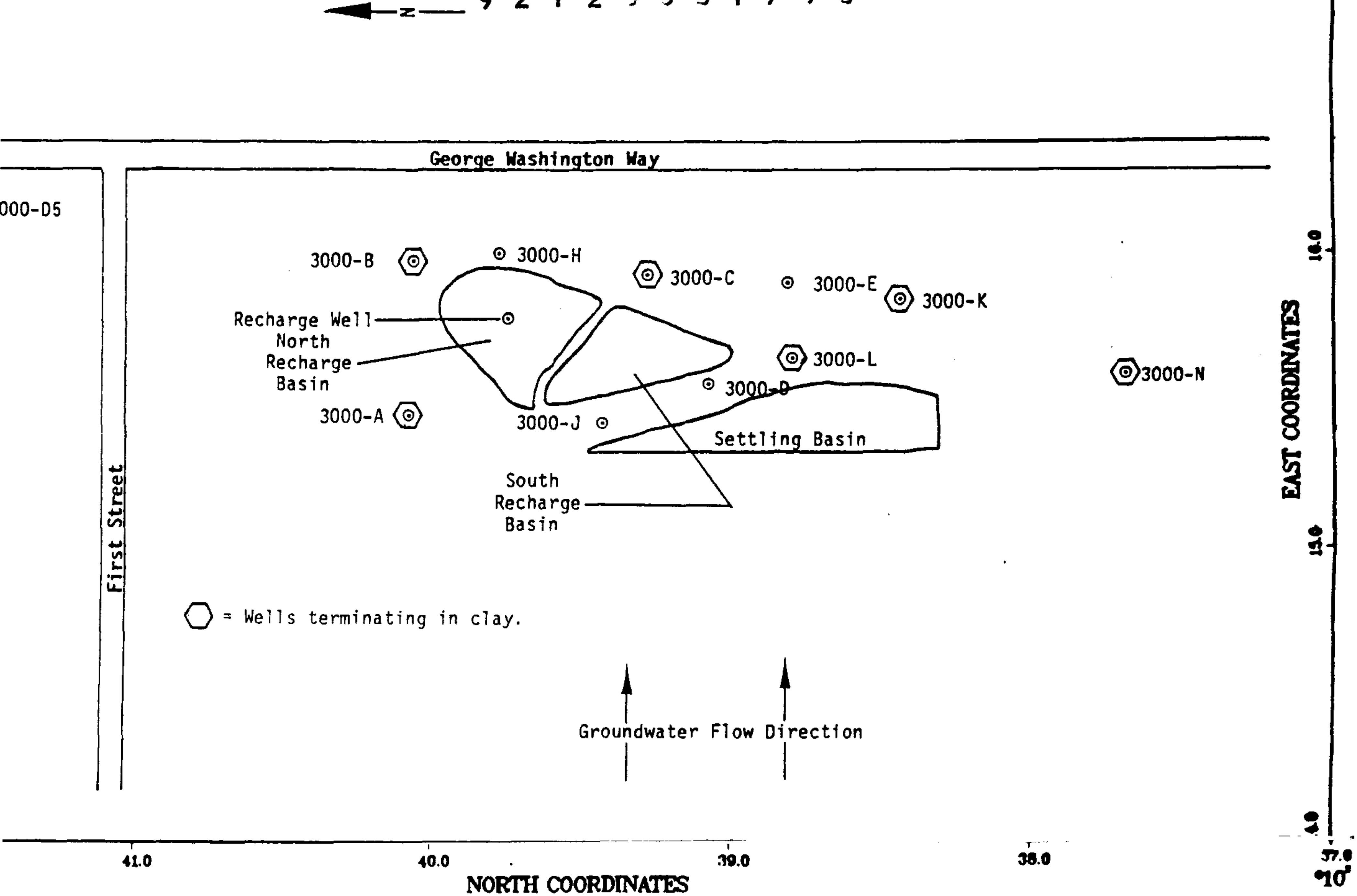


Figure 8. Wells Terminating in Clay Based on Well Log Data.

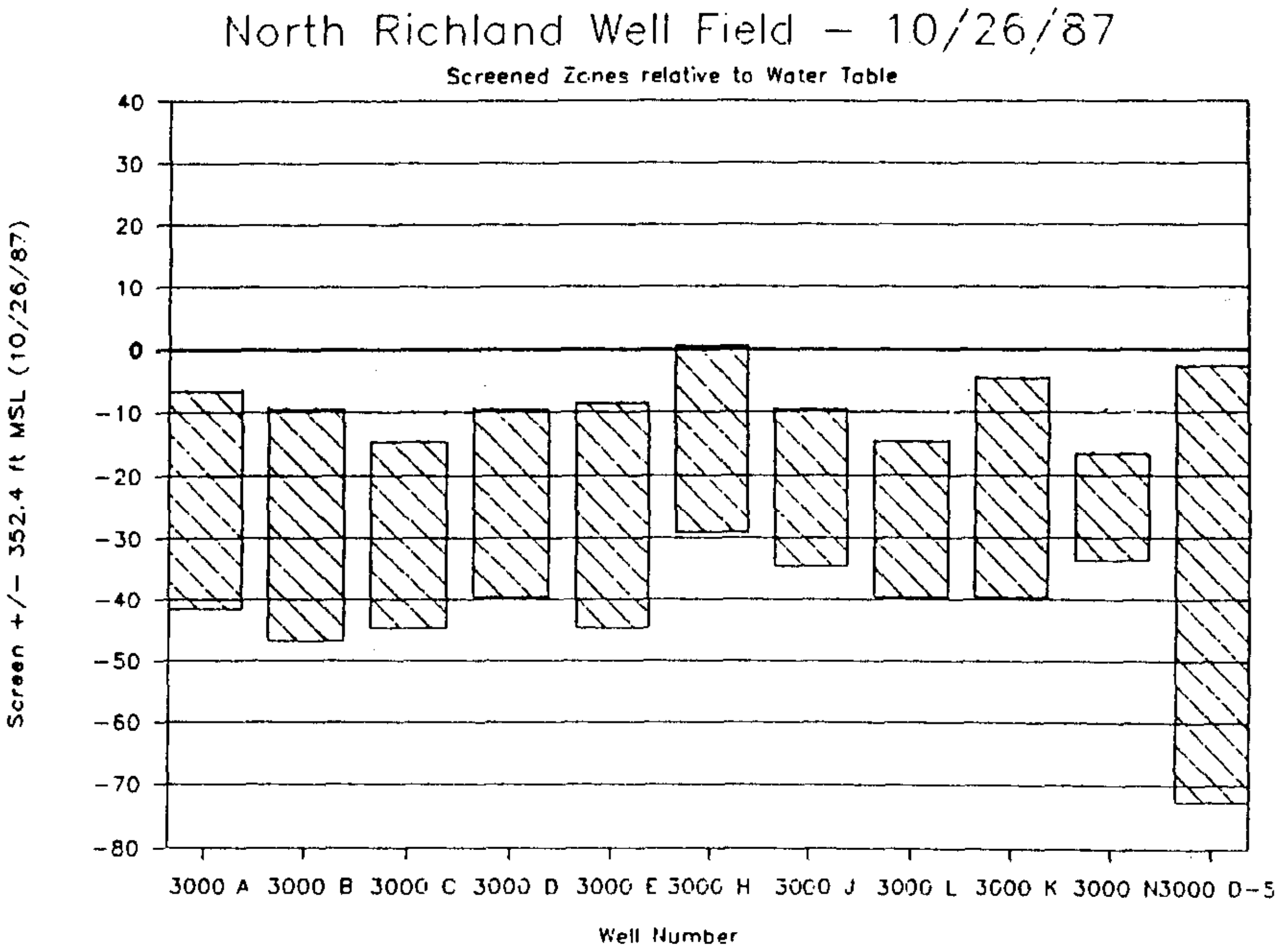


Figure 9. Location of Perforated Intervals Relative to Water Levels observed, 1987.

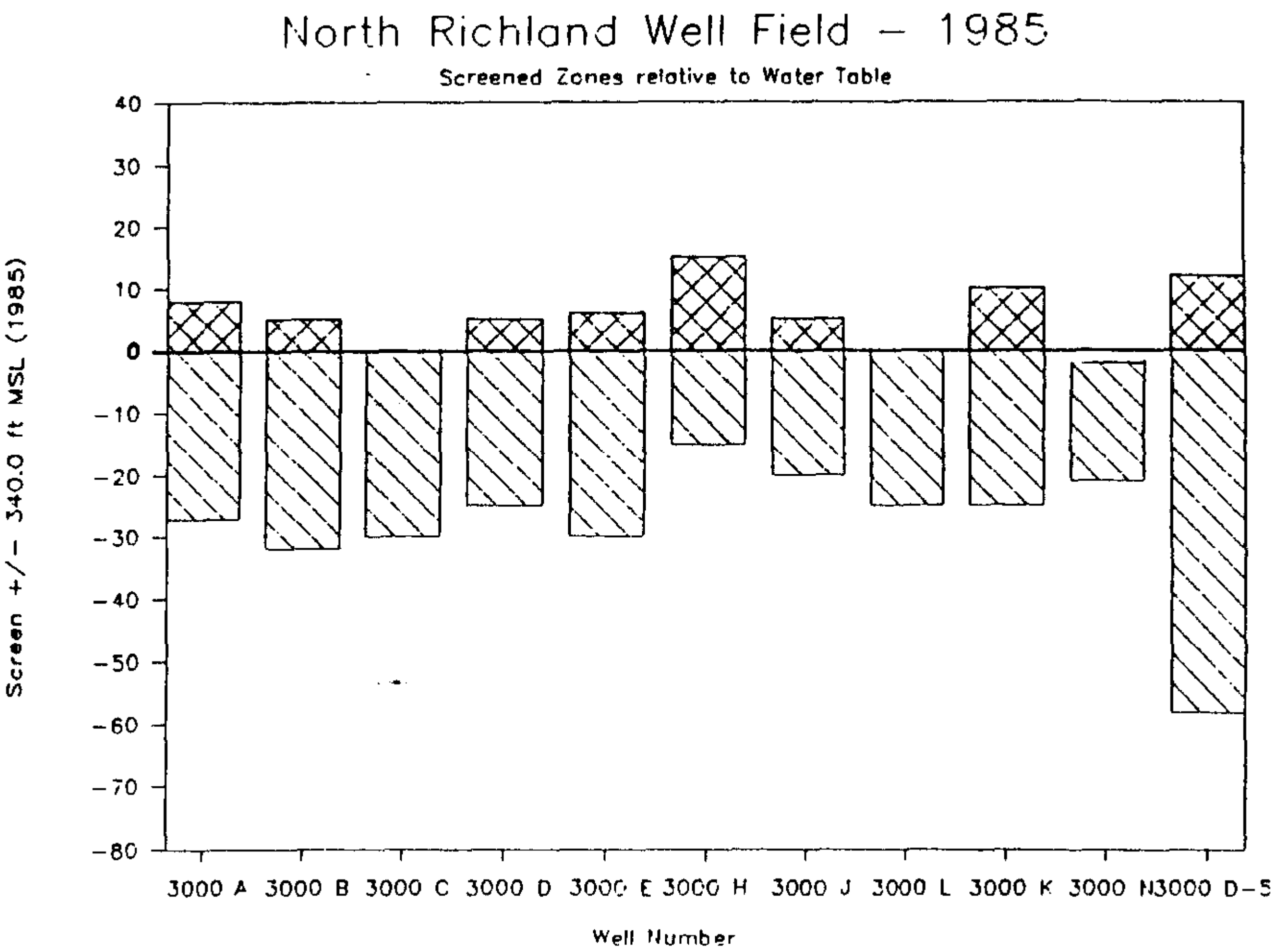
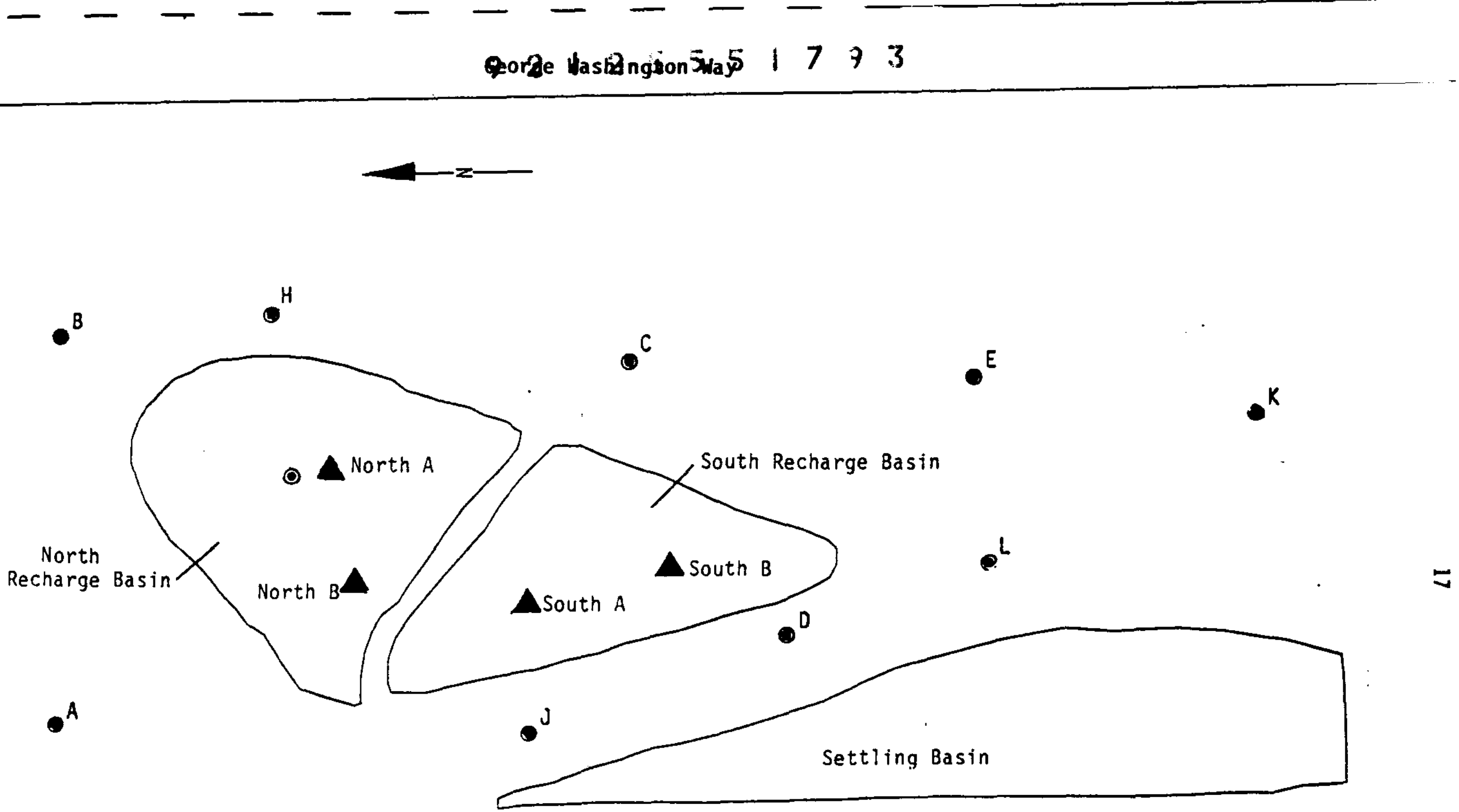


Figure 10. Location of Perforated Intervals Relative to Water Levels Observed, 1985.



▲ = Surface Testing Sites.

Figure 11. Location of Recharge Basin Sample Sites.

5.0 TEST RESULTS

5.1 North Richland Recharge Basins -- Particle Size Analysis

The recharge basins are centrally located within the North Richland Well Field. Evaluation of the basins was conducted after recharge waters had percolated and the basin floors were dry enough for vehicle access. Field evaluation of the north basin was performed on October 14, 1987 and in the south basin on October 22, 1987. The last recharge pumping prior to this study was completed October 11. Figure 11 indicates the approximate location of sample sites within the recharge basins.

Visual inspection of the north recharge basin floor indicates that approximately 60 % of the surface consists of a relatively deep (10 inches +) layer of coarse sand and small pebbles. Another 20 % of the area displays cobbles of 2 to 4-inch diameter at the surface. The remaining 20 % of the surface area, particularly near the basin inlet structure, exhibits a thin silt layer (less than 1.0 cm) at the surface. Approximately 60 % of the basin floor is host to a stand of aquatic plants, tentatively identified as Water Smartweed.

Two locations within the north basin were selected for detailed examination. Site A is located approximately 50 feet south east of the recharge well and is an area of coarse sand at the surface representative of the major portion of the basin area. Visual evaluation of the near-surface material at this site indicates a light brown, medium to very coarse sand from the surface to 6"; a black, medium to very coarse sand from 6" to 17" depth; and sandy gravel with cobbles from 17" down to 24" and beyond.

Site B in the north basin is located approximately 120 feet south west of the recharge well and 150 feet east of the basin inlet. The surface at Site B was covered with a uniform layer of silty material approximately 1.0 millimeter thick.

From the surface to a depth of 4", the profile is a black, medium to very coarse sand with some gravels; the next strata, from 4" to 10", is a similar black sand with a few gravels and cobbles; and the strata from 10" to beyond 24" in depth is primarily gravel and cobbles with some light brown, medium sand.

Samples were collected in three-inch increments from the top foot of material at each site for determination of particle size distribution by dry sieving. The results of the testing of individual samples is found in Appendix B. Since the top foot at all locations was generally homogeneous, a graphic presentation of the average distribution for each site is included here. Size fractions are based on particle diameters and are outlined in Table 1.

Table 1. Particle Size Diameters

Particle	Diameter (millimeters)
Gravel	>4.00
Pebble	2.00-4.00
Very Coarse Sand	1.00-2.00
Coarse Sand	0.50-1.00
Medium Sand	0.25-0.50
Fine Sand	0.106-0.25
Very Fine Sand	0.063-0.106
Silts and Clays	<.063

The particle size distribution for the top foot at Site A in the north basin is shown in Figure 12. The material is predominantly coarse sand to pebble-sized particles. Data for Site B indicate a less uniform material dominated by gravels as shown in Figure 13.

The surface of the south basin consists almost entirely of exposed cobbles and gravels with sands dominating the surface over only about 10% of the area. An area of aquatic plants coincides with the sandy surface area. The basin floor was covered almost entirely with an algae mat approximately 1-2 mm thick. Site A in the south basin was located near the center of the basin in an area of coarse sand with few gravels at the surface.

The profile at Site A consists of coarse sand with few gravels from the surface to 5"; coarse sand with some gravels and cobbles from 5" to 15"; and coarse sand with about 50% gravels and cobbles from 15" to beyond 24". Figure 14 shows the particle distribution for the top 12" at Site A in the south basin.

Site B in the south basin was located in the southern lobe of the basin and was dominated by gravels at the surface. The profile from the surface to 6" consisted of gravel and coarse sand; coarse sand with gravel from 6" to 11" and; coarse to very coarse sand from 11" to 48" and beyond. The particle distribution for Site B is shown in Figure 15.

5.2 North Richland Recharge Basins -- Surface Infiltration Rates

Surface infiltration rates were determined at each site using a concentric ring infiltrometer. The moisture content of surface sediments at all locations was at or near field capacity and was, therefore, favorable for rapid equilibration to a saturated flow condition.

The surface deposits in the recharge basins are generally highly permeable to water. The results of the infiltration tests are found in Figure 16. The results of infiltrometer testing provide a good basis for evaluation of the relative infiltration rates of various individual sites or surface conditions, but do not necessarily reflect the rate of percolation of the entire basin.

The infiltration rate of the entire basin is most likely less than the individual test sites due to the presence of restricting layers deeper within the profile that are not encountered during the infiltrometer testing.

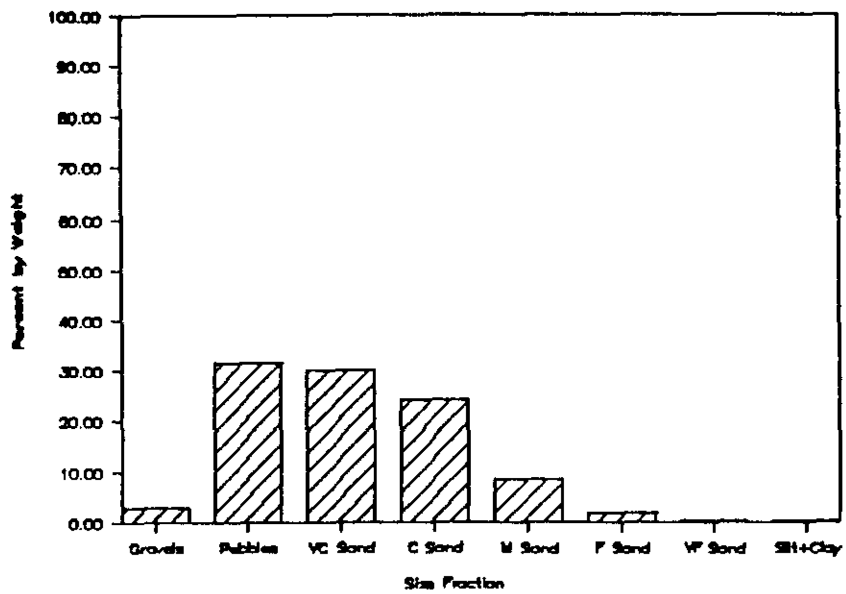


Figure 12. Particle Size Distribution, North Basin - Site A.

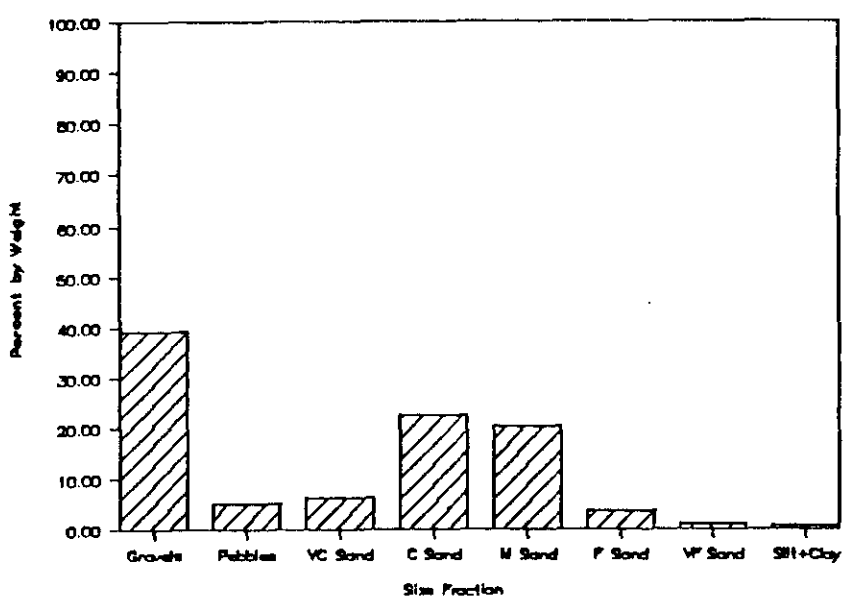


Figure 13. Particle Size Distribution, North Basin - Site B.

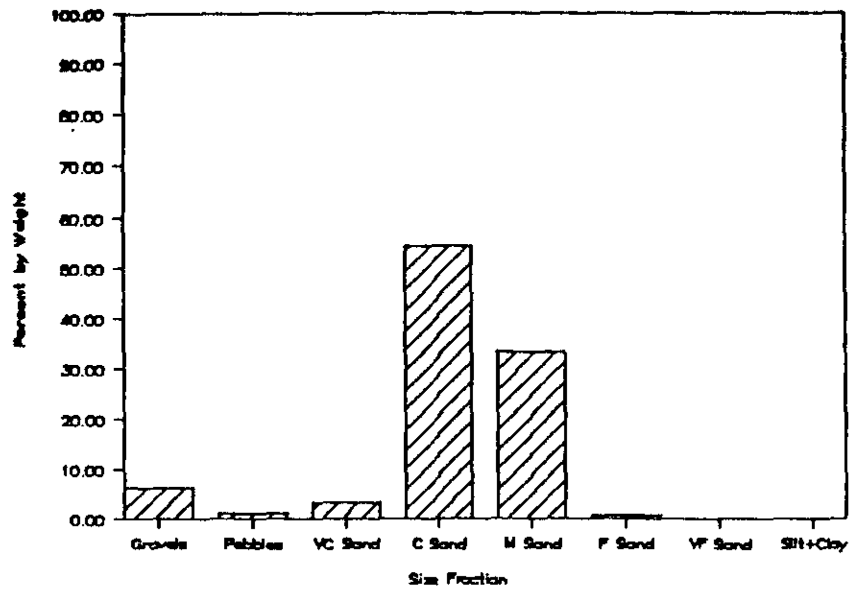


Figure 14. Particle Size Distribution, South Basin - Site A.

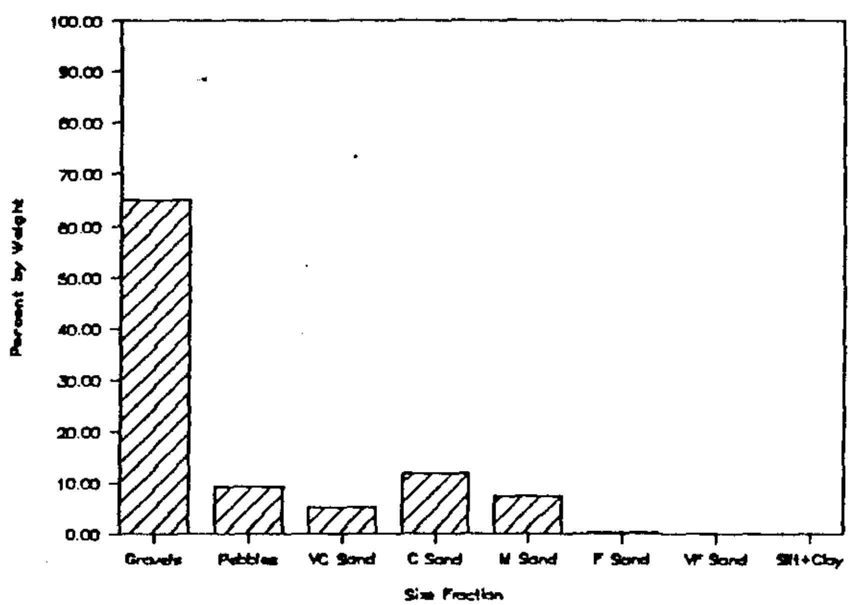


Figure 15. Particle Size Distribution, South Basin - Site B.

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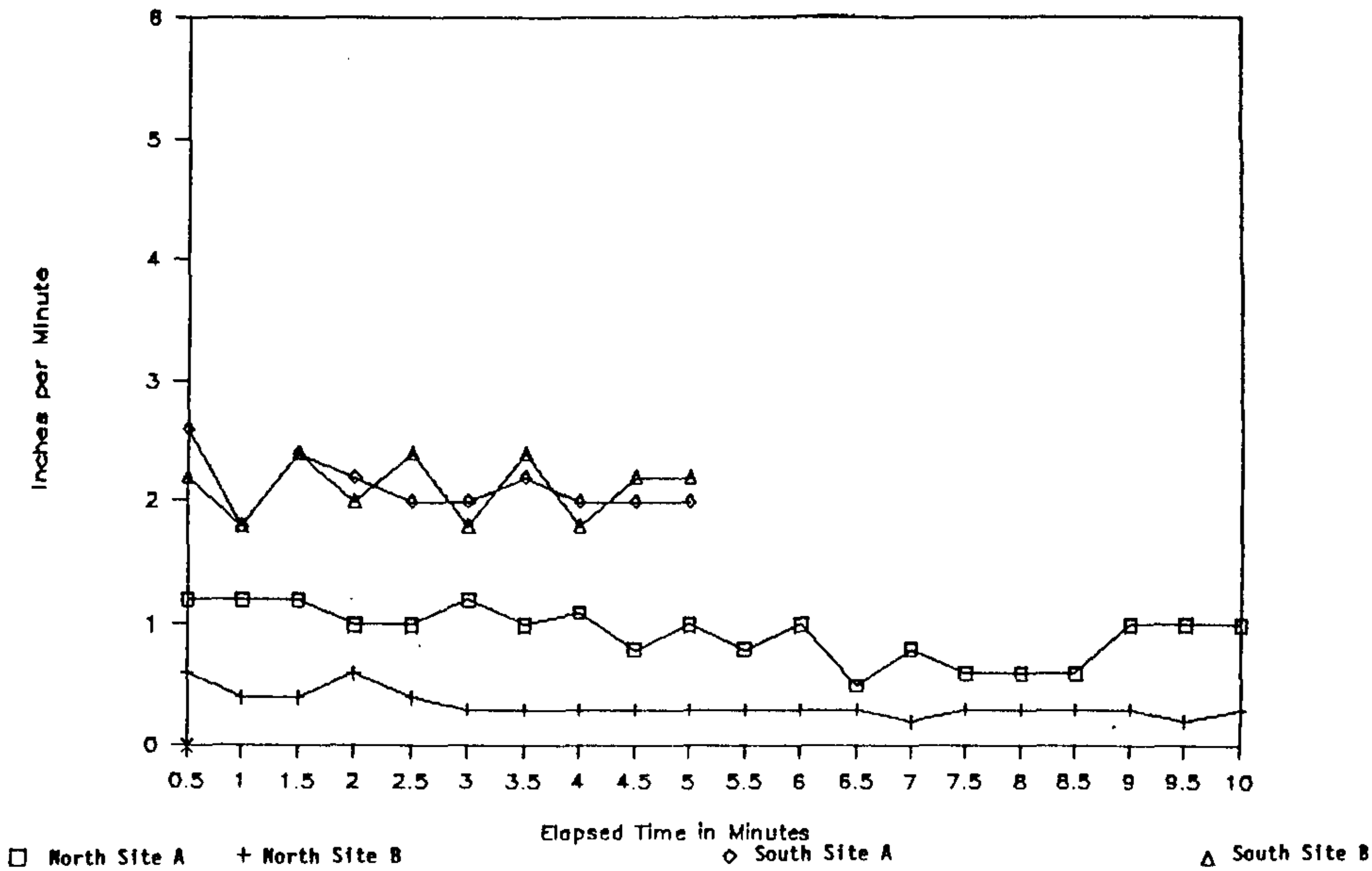


Figure 16. Surface Infiltration Rates Observed in North and South Basins.

As shown in Figure 16, the infiltration rate for Site A in the north basin was approximately 1.0 inch per minute over the period of the test. At Site B, where the thin silt layer was observed at the surface, the infiltration rate was still quite rapid (approximately 0.3 inches per minute) but was less than half that of Site A. This indicates that while siltation of the basins does not occur over large areas during the course of a season, small amounts of silt that could potentially cover the entire basin could have a dramatic effect on the rate at which recharge water ultimately enters the aquifer.

The infiltration rates observed at Sites A and B in the south basin are very similar (approximately 2.0 inches per minute) and about double the rates observed in the north basin. This reflects the generally coarser surface materials in the south basin.

5.3 <u>North Richland Well Field -- Aquifer Pumping Tests</u>

Constant rate pumping tests were performed on two wells in the North Richland Well Field. The first test was performed by pumping well 3000-J (a 125 hp pump) at a rate of 300 gallons per minute for 24 hours on October 21 and 22, 1987. Wells 3000-D and C were used as monitoring wells observe aquifer drawdown. After 24 hours, no drawdown was observed in either of the monitoring wells or in well J.

The second pumping test utilized well 3000-H with its 200 horsepower pump and well B as the monitoring well. Well H was pumped at a rate of 1340 gallons per minute for a 98 hour period from October 22 to 26, 1987. Total drawdown observed in well H was 4.0 feet. This level of drawdown was achieved within 60 minutes of the start of the test and the level in the well remained constant at a 4.0 foot drawdown throughout the remainder of the test. The maximum drawdown observed in the monitoring well, well 3000-B, was 0.66 feet which occurred after 24 hours of pumping and then remained constant at that level for the remainder of the test.

Twenty-four hours after completion of the pumping test, the water level in well H had recovered to within one foot of the pre-test level, and well B was unchanged.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 North Richland Recharge Basins -- Recommendations

Overall, no restrictions to infiltration were observed in the basins with the exception of the silted area near the inlet of the north basin. The generally rocky surface conditions of the basins, however, makes management of any silt deposits quite difficult. Tillage of the basin floors has minimal effect due to the implement's bouncing over rocks. For this reason, placement of a uniform layer of coarse sand approximately 10 to 12 inches deep over the floor areas of both north and south basins is recommended. The basins should be prepared for this application by removing remaining aquatic vegetation and mixing or removing existing silt layers by mechanical means such as use of a suction dredge. After installation of the sand layer, the basin floors may be easily maintained with periodic mechanical cultivation.

A possible source of sand for lining the basin floors is an excavation at the City of Richland's municipal landfill. A sample was collected from a horizon of black sand approximately eight feet thick and occurring 15 feet below the surface in a large excavation on the east side of the landfill. The results of dry sieving analysis of this material are shown in Figure 17. This material is dominated by coarse sand and has very few fines and no materials larger than very coarse sand. This sand is physically well-suited for use in the basins.

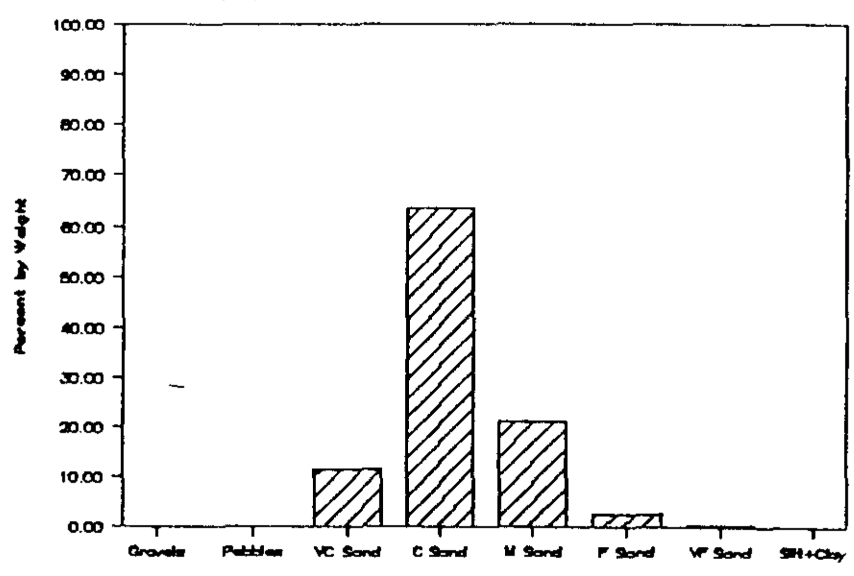


Figure 17. Particle Size Distribution, Landfill Sand Sample

Additional recommendations for maintenance of the North Richland Recharge Basins include repair of the dike separating the basins and repair of the basin perimeter fencing. Some erosion has occurred on both sides of the dike at the location of the two steel pipes that serve as overflow weirs between the basins and may eventually result in a breach of the dike. Repair of the existing perimeter fence will minimize unauthorized access to the basins both during recharge when a water hazard exists, and when the basins are dry.

6.2 Well Field Evaluation and Pumping Strategy Recommendations

Since there was no drawdown of the water level during pumping of Well J, no conclusions can be drawn from that test other than the capacity of the well to supply a sustained 300 gallons per minute with no measurable drawdown. The pumping test of Well H, however, supplied sufficient data to perform evaluation of aquifer storage and transmissivity. Total yield from this pumping test was 7.9 million gallons for the 98 hour period or approximately 1.9 million gallons per day (mgd). Utilizing the drawdown and pumping rate information, and the lateral distance between the wells H and B, coefficients of transmissivity and storage were calculated. The Coefficient of Transmissivity, T, was calculated using the following equation:

$$T = \frac{264\ Q}{s}$$

Where T = the Coefficient of Transmissivity
Q = the constant pumping rate
s = the slope of the observed drawdown curve

For this test, Q = 1343 gallons per minute
and s = 0.55 foot

For this pumping test, the Coefficient of Transmissivity, T, was calculated to be 644,600 gallons per day/foot, a very high level.

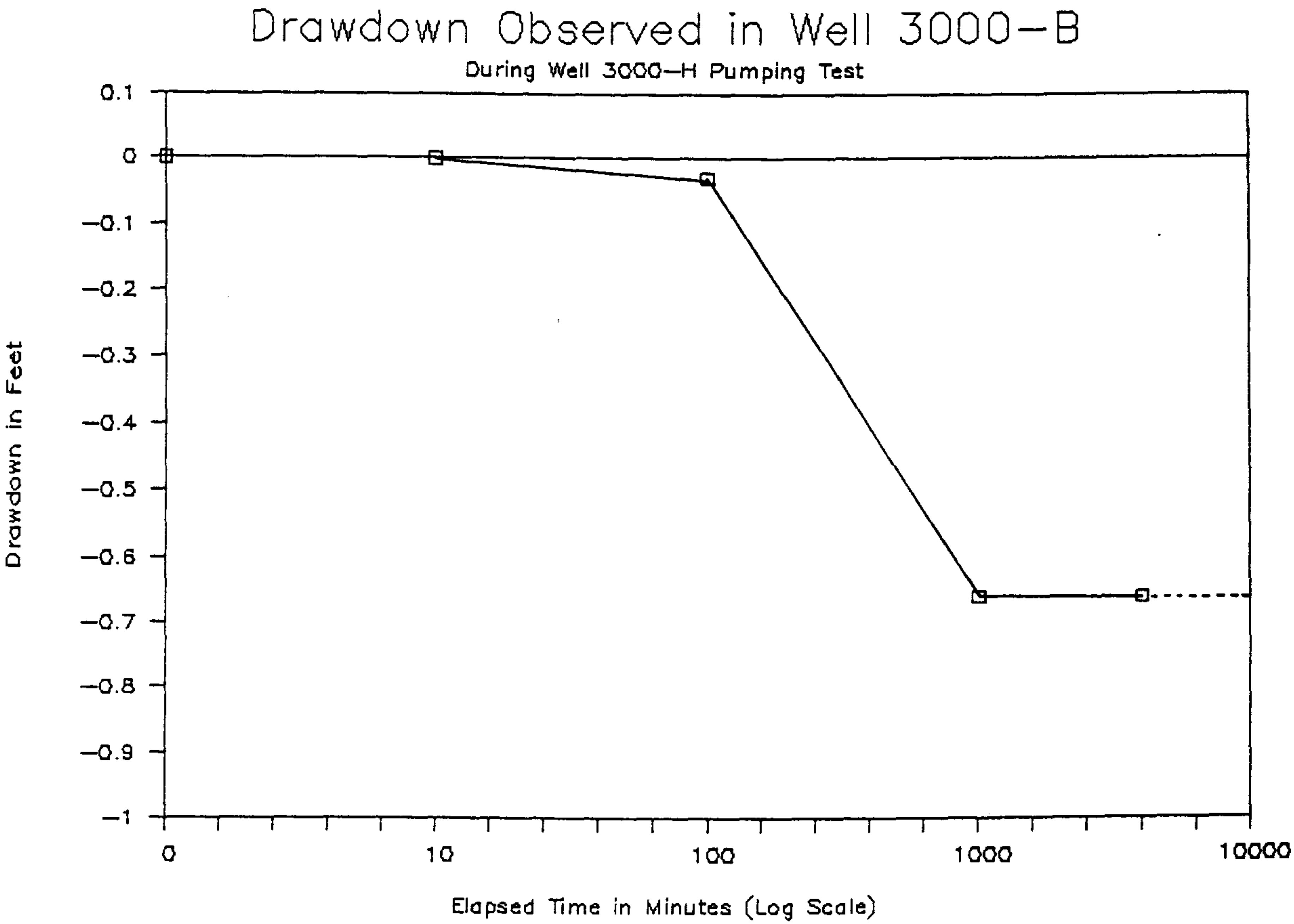


Figure 18. Drawdown Measured in Well 3000-B During Pumping Test.

The aquifer Storage Coefficient, S, is calculated by the following equation:

$$S = \frac{0.3\ T\ t_0}{r^2}$$

Where T = the Coefficient of Transmissivity
t_0 = the zero drawdown intercept of a straight line projected through the observed drawdown curve, in days
r = the distance in feet from the pumped well to the monitoring well

for this test, T = 644,600 gallons per day/foot
t_0 = .07 days
r = 350 feet

The Aquifer Storage Coefficient, S, calculated for this pumping test is 0.11, which is consistent with expected values for the types of sediments observed in the wells. Figure 18 is a semi-logarithmic graph of the water level drawdown measured in Well B during the pumping of Well H. Values of "s" and "t_0" used in the previous calculations were extrapolated from this curve.

We believe the aquifer at the North Richland Well Field to be capable of supplying a sustained 4.0 to 5.0 million gallons per day. This conclusion is based on the results of the pumping tests performed by ICF personnel and evaluation of previous pumping test results from Cornell, Howland, Hayes, and Merryfield (1961) (the previously mentioned 1961 report estimated the supply under unrecharged conditions to be 4.0 to 6.0 mgd).

Based on this information, four basic operating strategies for the system can be considered:

1. Continued operations of the well field using current pumping strategies.
   Advantages:
   - No additional costs or changes from normal maintenance and operations.

   Disadvantages:
   - Inefficient use of aquifer.
   - High cost of product water due to high volumes of recharge water pumped.

2. Use of the aquifer supply only, with no recharge operations.
   Advantages:
   - High efficiency of aquifer utilization.
   - Eliminates costs of recharge pumping.

   Disadvantages:
   - Reduces production capacity of the well field to about 4.0 mgd maximum.
   - May increase hardness of product water.*

3. Use of aquifer supply exclusively during periods when production demand is less than 4.0 mgd and supplying recharge water to meet the aquifer supply deficit during periods of high demand.
   Advantages:
   - Permits efficient aquifer utilization.
   - Reduces overall cost of product water while maintaining peak period productive capacity.

   Disadvantages:
   - May increase hardness of product water during low production periods.
   - Requires capital expenditure for placement of largest pumps in most productive wells.

4. Continued use of coinciding recharge and production, but reduce recharge volume to more closely match production.
   Advantages:
   - Reduces overall cost of product water while maintaining peak period productive capacity.
   - Maintain present water quality.

   Disadvantages:
   - Requires capital expenditure for placement of largest pumps in most productive wells.

Of these four options, the most practical appear to be options 3 and 4 because both strategies reduce the cost of product water associated with high levels of recharge, yet still maintain the high potential capacity of the well field through recharge.

An analysis of production records from the well field over the last three years, 1985 through October 1987, indicates that only four times during the last three years, and only once in the last two years, has average daily production (averaged over the month) exceeded 4.0 mgd.

* Information on the specific water quality of the aquifer in North Richland is beyond the scope of this study.

This analysis is illustrated in Figure 19, and indicates that the production requirements of the well field can be met in most instances by the conservative estimate of the natural aquifer capacity (4.0 mgd). This, of course, raises the question of quality (ie. hardness, possible chemical contamination from upgradient sources) of the natural aquifer water versus the recharge water from the Columbia River. The water quality question is beyond the scope of this report, but should be addressed in conjunction with consideration of minimum recharge operations.

The most efficient use of the North Richland Well Field involves use of the natural aquifer supply to the greatest extent possible and closely matching recharge flow to production during periods when production demand exceeds the aquifer capacity.

Applying this strategy and referring to the average daily production data in Figure 19, recharge of the aquifer would be needed during January and February (when the filter plant is down), and during the summer months of June, July, and August, when production typically exceeds 75 % of the estimated aquifer capacity. For the remainder of the year, recharge of the aquifer is probably not necessary. This strategy could result in saving the City the operational costs of pumping up to 1.6 billion gallons of recharge water per year.

Verbal information supplied by system operators indicates that wells 3000-K, L, N, and H display problems with drawing air when the system is operated at low recharge flows. This is consistent with the evaluation of the well logs that shows well K to have a moderate potential, yet it is equipped with one of the largest pumps in the well field (200 hp). Well N shows moderate production potential, but is quite distant from the primary recharge basins and thus would not be expected to show a significant response to low to moderate recharge of the north and south basins. Wells L and H both fall into the low yield potential category based on well log data. This is again consistent with operating experience. In addition, well H is equipped with a large, 200 hp, pump.

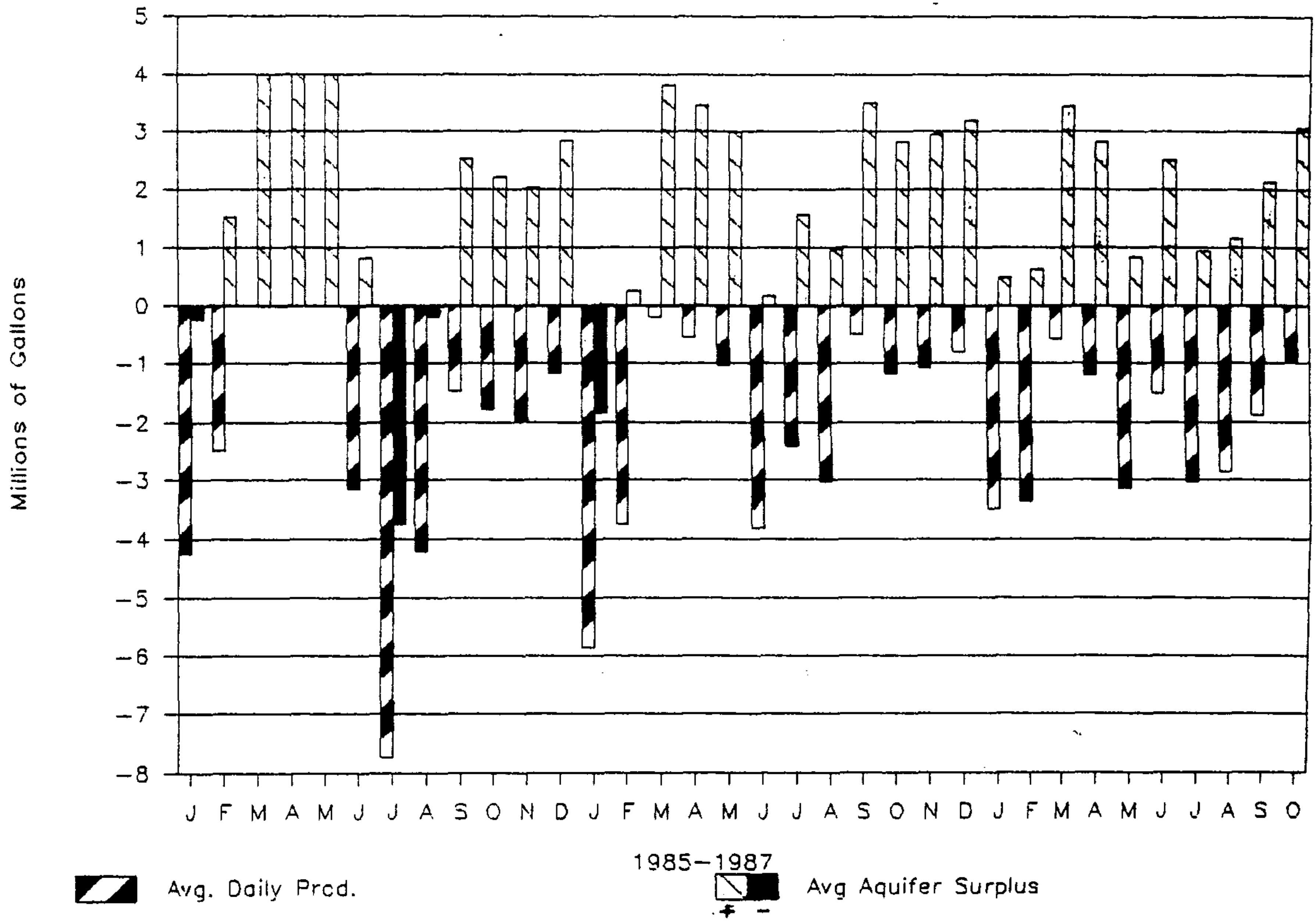


Figure 19. North Richland Well Field Production vs. Estimated Aquifer Supply Capacity of 4.0 mgd.

The pumps installed in the North Richland Well Field are outlined in Table 2. As previously stated, for optimum production under reduced recharge, the largest pumps should be located in good wells on the upgradient side of the field. As shown in Table 3, the situation is nearly reversed from the optimum.

Table 2. Pump Sizes and Locations.

Well	Pump Size (hp)
A	75
B	75
C	100
D	125
E	250
H	200
J	125
K	200
L	125
N	100
D-5	75

Table 3. Current Pump Distribution vs. Well Location.

Upgradient Wells	Downgradient Wells
A (75 hp)[3]	B (75 hp)[1]
J (125 hp)[1]	H (200 hp)[3]
D (125 hp)[1]	C (100 hp)[1]
L (125 hp)[3]	E (250 hp)[3]
	K (200 hp)[2]

Note: Wells N and D-5 appear to be too far from the central well field to be affected by upgradient wells.
1 = Wells identified as best yield potential.
2 = Wells identified as moderate potential.
3 = Wells identified as low potential.

A scheme that would bring pump placement more into line with optimum conditions is shown in Table 4, and would involve moving the two 200 horsepower pumps from wells 3000-K and 3000-H to wells 3000-J and 3000-D and replace them with the 125 horsepower pumps from J and D. An additional replacement would move the 125 hp pump from well 3000-L (which, while upgradient, is completed in low permeability rocks) to well 3000-B and replace it with B well's 75 hp pump.

Table 4. Recommended Pump Locations.

Well	Pump Size (hp)
A	75
B	125
C	100
D	200
E	250
H	125
J	200
K	125
L	75
N	100
D-5	75

6.3 Conclusions

An overview of the recommendations for the well field and recharge basins is outlined below:

A. Recharge Basins
   1. Line basins with 12 inches of coarse sand.
   2. Repair the dike separating the north and south basins.
   3. Repair the perimeter fence surrounding the basins.

The first two items, lining the basins with sand and reparing the dike, are maintenance items that will improve operation of the basins and prolong their useful life. The sand layer at the City's landfill is a possible source of material for the basin floors. While the sand was found to be physically suited for that use (ie. has desireable particle size distribution), the material should be chemically characterized to identify possible contamination from landfill operations prior to its use in the basins.

B. Well Field
   1. Move the 200 hp pumps from wells 3000-K and H to wells 3000-J and D.
   2. Move the 125 hp pumps from wells 3000-J and D to wells 3000-K and H.
   3. Move the 125 hp pump from well 3000-L to well 3000-B and replace it with the 75 hp pump from well 3000-B.
   4. Operate the well field based on a 4.0 mgd aquifer supply with recharge only during aquifer deficit periods, or;
   5. Supply recharge water during production at a rate very close to the production rate.
   6. After completion of the recommended pump changes (and given the high transmissivity of the aquifer), recharge should not have to exceed 150 percent of production during any production period.

Moving the large capacity pumps into the wells with the highest production potential should improve operation of the well field under conditions of low or no recharge or under high recharge. In order to maintain water quality at a level similar to current operations, particularly with respect to hardness, continuing the system of aquifer recharge during production is desireable. The greatest improvement in operational efficiency of the recharge basin/well field system is to match the recharge volume more closely to the production volume. The recommended changes should allow recharge to approach 150 % of production instead of the historic 300 to 400 %.

No technical problems were discovered in the course of this study that indicate the North Richland Well Field should not continue to supply a significant portion of Richland's municipal water needs. Based on the information available, we believe that the changes outlined above should permit a much more efficient operation of the North Richland Well Field than is now possible through more efficient capture of aquifer water and better utilization of recharge water.

7.0 REFERENCES

City of Richland. Daily Water Production and Consumption Reports, 1985-1987.

City of Richland. Water Maintenance Manual -- Wells, Pump Stations, and Misc.

City of Richland, 1987. Water System Plan.

Cornell, Howland, Hayes, and Merryfield, 1961. A Report on an Engineering Investigation of the Water Supply System. Prepared for the City of Richland, Washington.

Raul A. Deju and Roy E. Gephart, 1976. Evaluation of Current Water Problems of the City of Richland Using System Analysis.

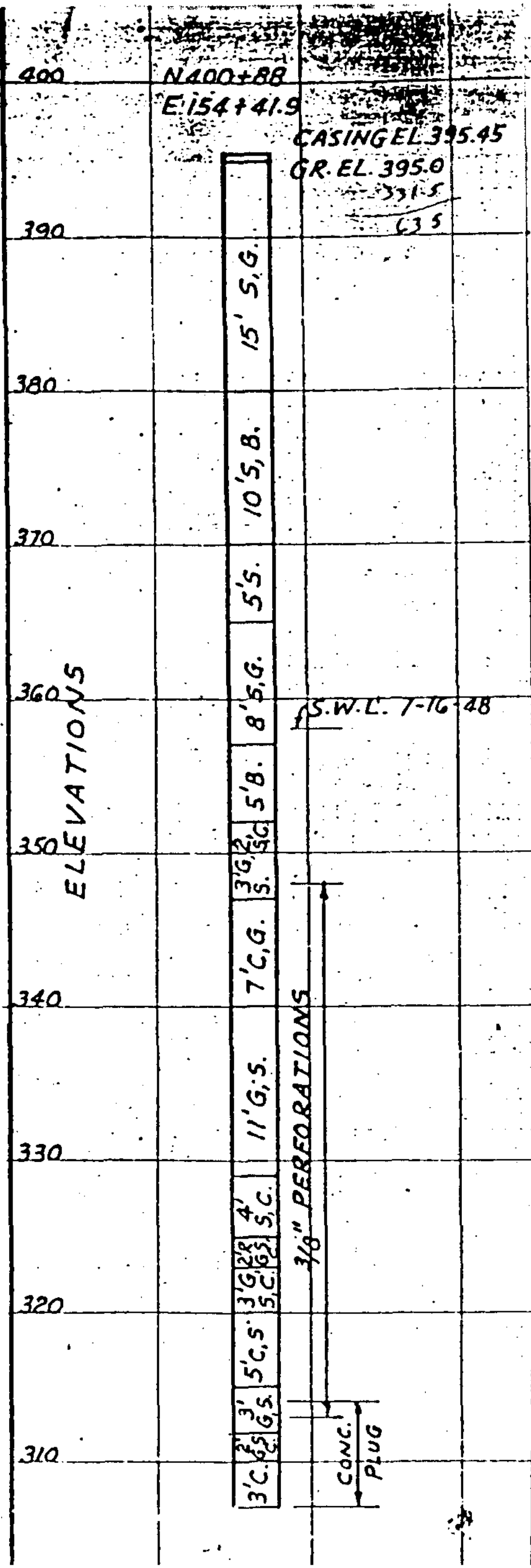
Fletcher G. Driscoll, 1986. <u>Groundwater and Wells, Second Edition.</u>

Ronald E. Gerton. Letter to William R. Gilbert, Public Works Director City of Richland, Washington, Feb. 12, 1985 (with enclosures).

U.S. Department of the Interior, 1971. <u>Ground-Water Hydraulics.</u> Geological Survey Professional Paper 708.

Appendix A. Well Logs of North Richland Well Field.

9212551813



WELL DATA

20" steel casing with 3/8" x 2" perforations from Elev. 313 to 343. Static water level after surging and before pumping Elev. 358. Sandfree in 15 minutes at first pumping. At 1,000 to 2,000 GPM for 12 hours. d.d. 3 to 5 ft. Specific capacity 333 to 500. 1,000 GPM 6 stage Pomona pump, 230 ft. head, set at Elev. 331.5.

KEY

- B Boulders
- C Clay
- G Gravel
- R Rocks
- S Sand

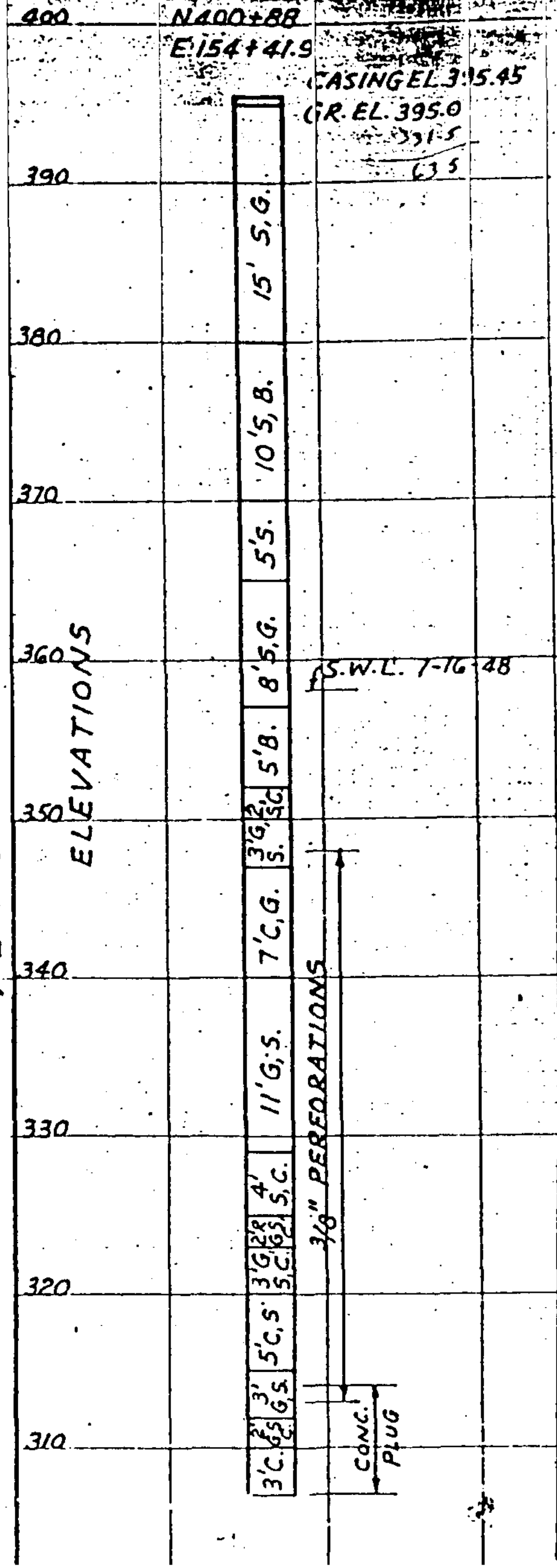
SCALE: 1"=10'	
APPROVED	DATE 8-20-48 DRAWN BY Quist CHECKED BY Quist

GENERAL ELECTRIC CO.
HANFORD WORKS

ALVORD, BURDICK & HOWSON
CONSULTING ENGINEERS CHICAGO

LOG OF WELL-3000-A

DWG.



WELL DATA

20" steel casing with 3/8" x 2" perforations from Elev. 313 to 348. Static water level after surging and before pumping Elev. 358. Sandfree in 15 minutes at first pumping. At 1,000 to 2,000 GPM for 12 hours. d.d. 3 to 5 ft. Specific capacity 333 to 500. 1,000 GPM 6 stage Pomona pump, 230 ft. head, set at Elev. 331.5.

KEY

B Boulders
C Clay
G Gravel
R Rocks
S Sand

SCALE: 1"=10'	
APPROVED	DATE 8-20-48 DRAWN BY Quist. CHECKED BY Quist.

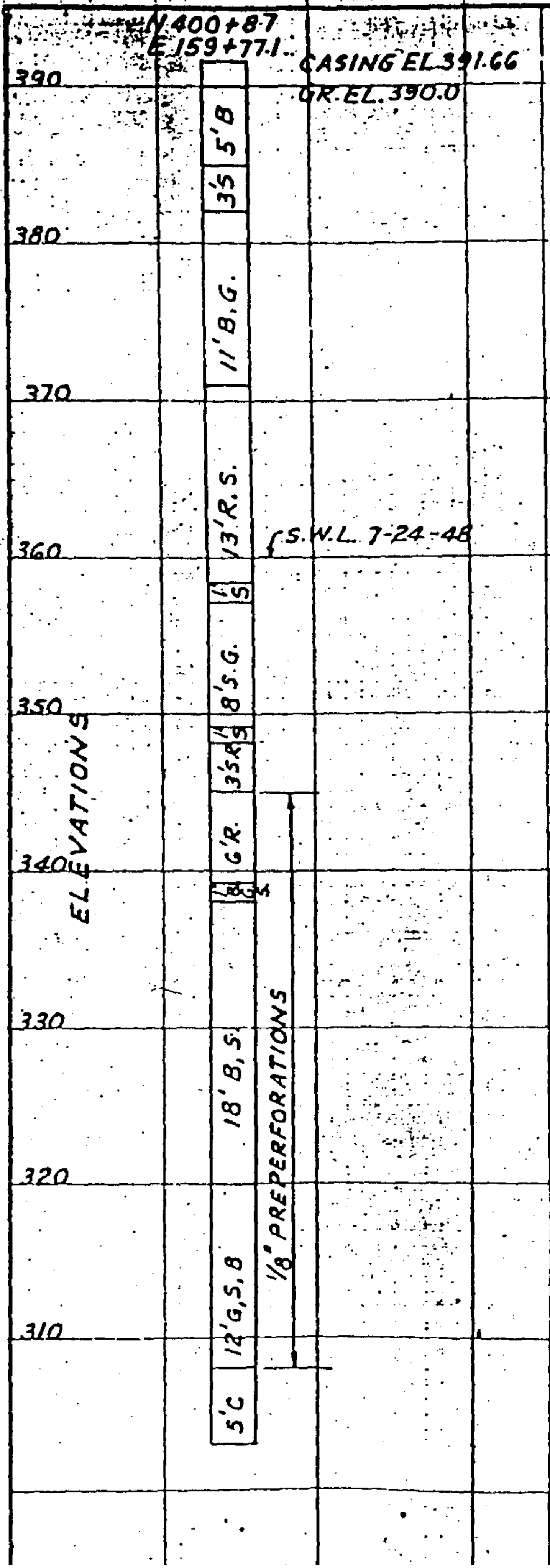
GENERAL ELECTRIC CO.
HANFORD WORKS

ALVORD, BURDICK & HOWSON
CONSULTING ENGINEERS CHICAGO

LOG OF WELL-3000-A

DWG.

9212515815



WELL DATA

20" steel casing, pre-perforated from Elev. 308 to 345. Static water level after surging and before pumping Elev. 360. Sandfree in 15 minutes at first pumping. Pumped at 1,000 to 2,000 GPM for 12 hours. Drawdown 4 to 14 ft. Specific capacity 143 to 250, 1,000 GPM, 6 stage Pomona pump, 230 ft. head, set at Elev. 312.5.

KEY

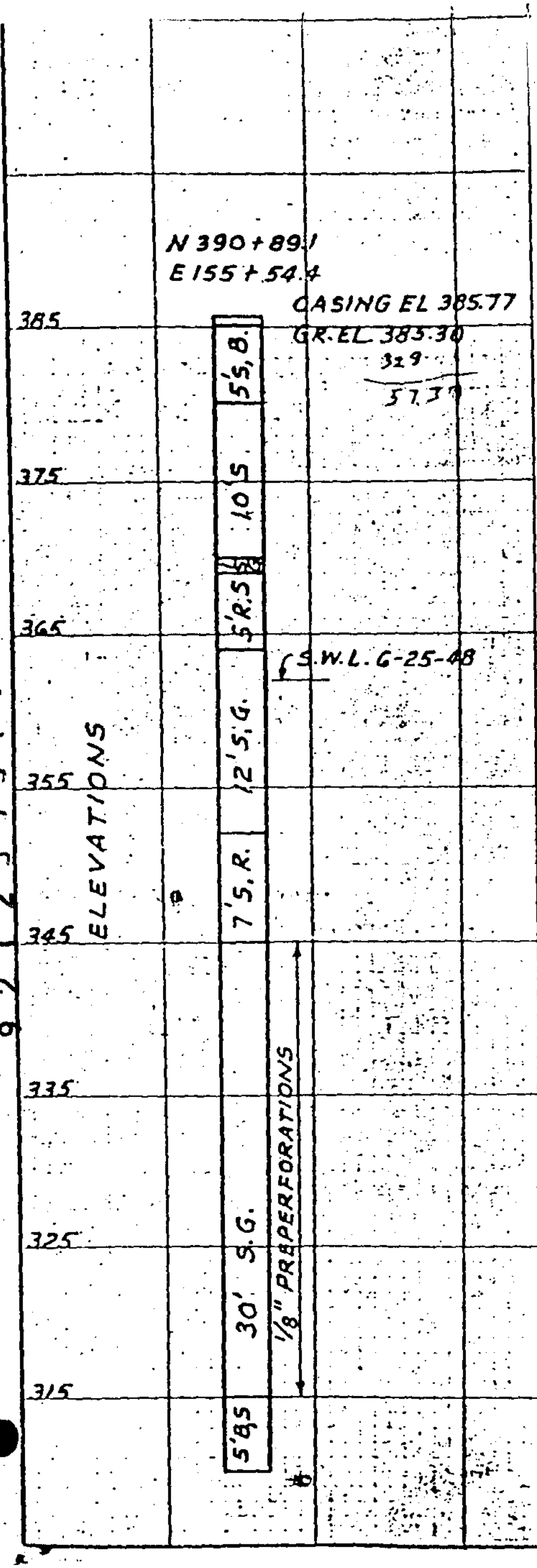
B Boulders
C Clay
G Gravel
R Rocks
S Sand

SCALE: 1"=10'	
APPROVED	DATE 8-20-48 DRAWN BY Quist. CHECKED BY Quist

GENERAL ELECTRIC CO.
HANFORD WORKS

ALVORD, BURDICK & HOWSON
CONSULTING ENGINEERS CHICAGO

LOG OF WELL-3000-B



9 2 1 2 5 5 1 8 1 6

WELL DATA

20" steel casing, pre-perforated from Elev. 315 to 345. Static water level after surging and before pumping Elev. 362. Sandfree in 10 minutes at first pumping. Pumped at 1000-2000 GPM for 12 hours. Drawdown 3 to 6 ft. Specific capacity 327 to 350. 1,000 GPM 6 stage Pomona pump, 230 ft. head, set at Elev. 328.

KEY

B Boulders
C Clay
G Gravel
R Rocks
S Sand

Pomona Turbine
Serial # P.J 3053

SCALE: 1"=10'	
APPROVED	DATE 8-20-48 DRAWN BY [illegible] CHECKED BY [illegible]

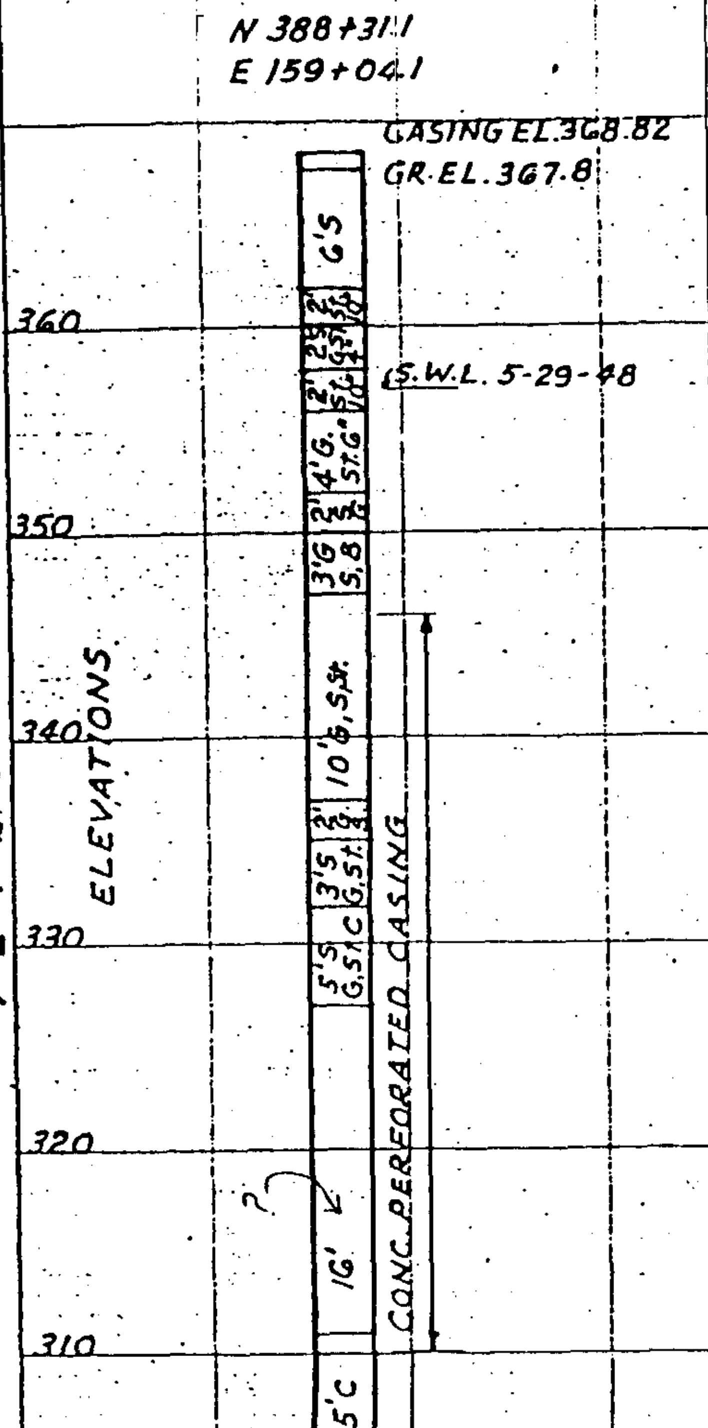
GENERAL ELECTRIC CO.
HANFORD WORKS

ALVORD, BURDICK & HOWSON
CONSULTING ENGINEERS CHICAGO

LOG OF WELL-3000-D

Abandoned 5/2/57 - New well Complete 4/7/58

DWG. NO. H-11-4123



WELL DATA

Kelly Well - 17" I.D. concrete casing, gravel packed. Top 20'-46", bottom 34". Concrete perforated casing Elev. 310 to 346. Static water level Elev. 357. Sandfree from beginning. 18 hours pumping at 1,385 GPM. Drawdown 2'4". Specific capacity 655. 2,000 GPM. Peerless pump 190 ft. head, set at Elev. 315.

KEY

B	Boulders
C	Clay
FS	Fine sand
G	Gravel
S	Sand
ST	Stone

SCALE: 1"=10'	
APPROVED	DATE 8-20-48 DRAWN BY Gus CHECKED BY Gus

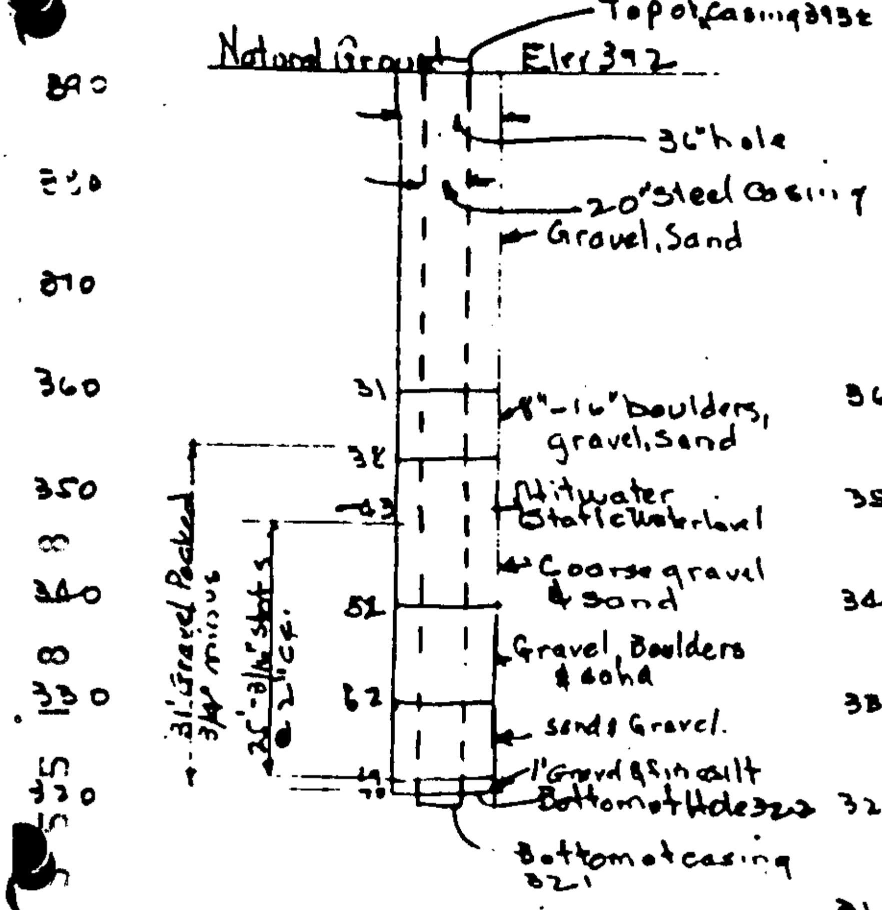
GENERAL ELECTRIC CO.
HANFORD WORKS

ALVORD, BURDICK & HOWSON
CONSULTING ENGINEERS CHICAGO

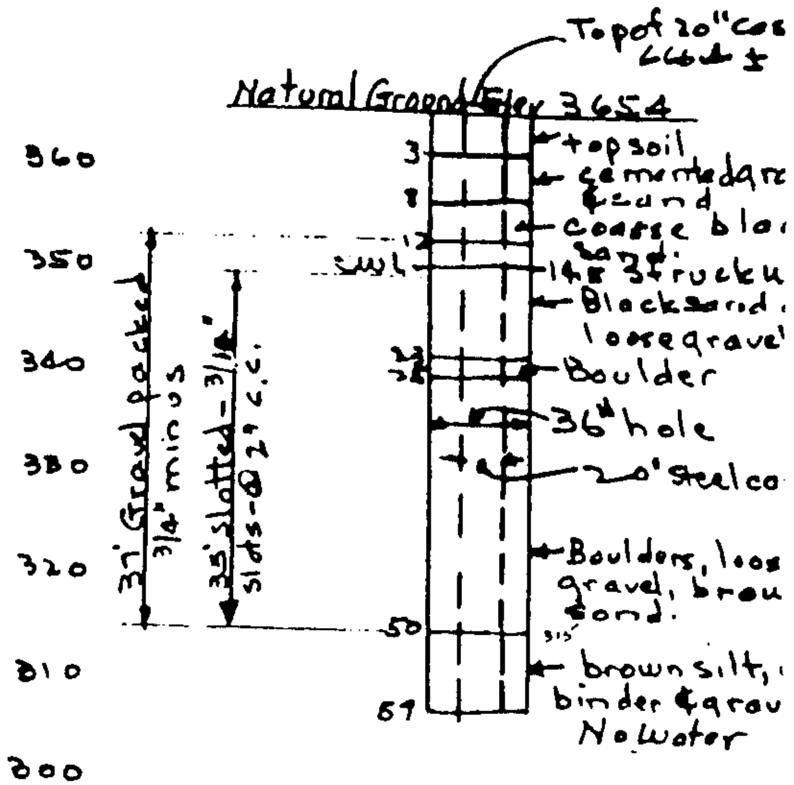
LOG OF WELL 3000-E

DWG. NO. H-11-4122

Well 3000 J



Well 3000 K



Test pumped - 1800 gpm - 18' d.d.
Spec Cap 100

Pump to be set with bottom of bowls @ 68' - 1400 gpm
TDH 230'

Test pumped - 2000 gpm, 25' dd, spec cap
Pump to be set with bottom of bowls @ 53' - 2000 gpm TDH 230'

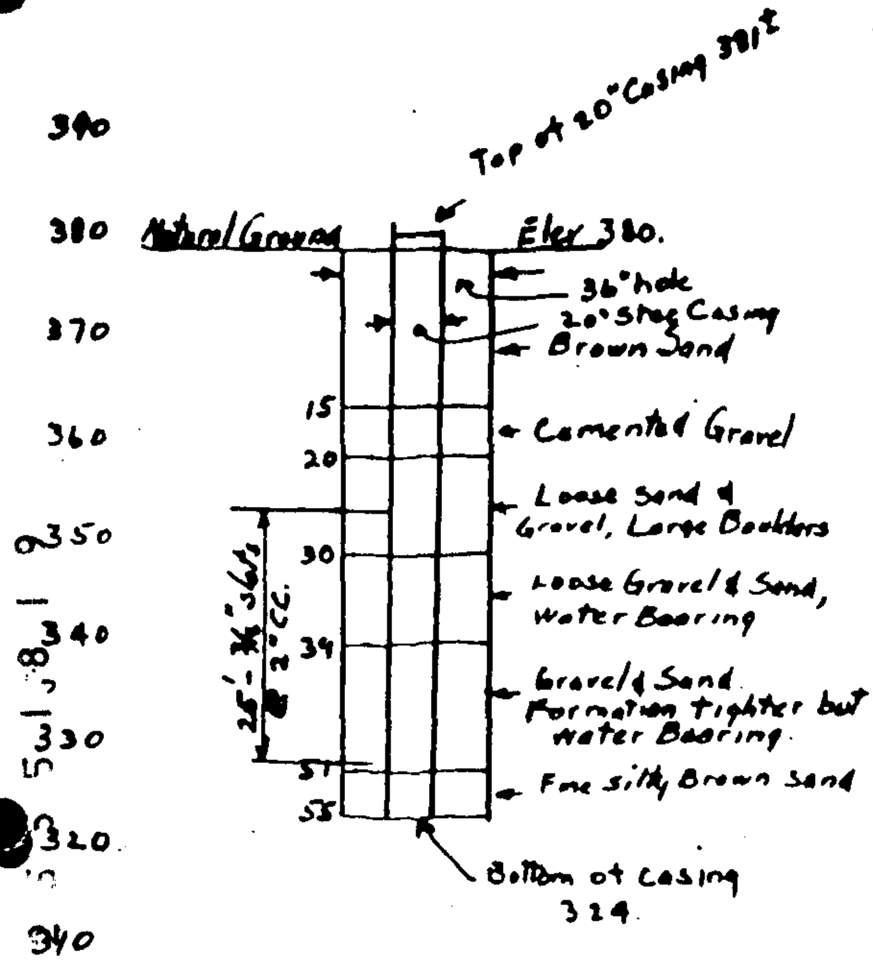
Comp ~~leted Well ... test pump~~ 11-10-52

(Base of pump to bottom of bowls 72'
Well D.)

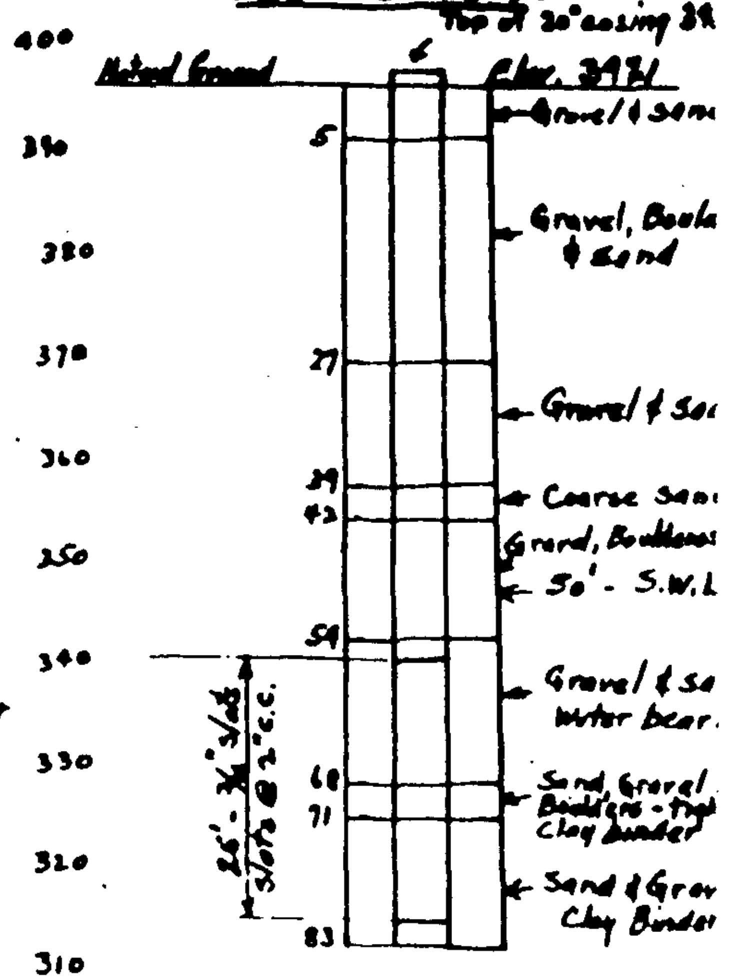
ALVORD, BURDICK & HOWSON
ENGINEERS - CHICAGO

BY HC SUBJECT Logs of Wells 3000H, - 3000L DATE 1- 57

WELL 3000-H



WELL 3000-L



Test pumped 2000 gpm, 15' DD.
Spec Cap. 133

Pump to be set with bottom of Bowls
@ 52' - 2000 gpm TDH 230'

JJ

R. J. Strasser Drilling Co.

8110 S. E. SUNSET LANE
PORTLAND 6, OREGON

Information concerning wells 3000 J, 3000 L, 3000 ~~M~~

Log of Formations; 3000 L

From	To	Formation
Surface	5	Gravel and sand
5	27	Gravel, boulders and sand
27	39	Gravel and sand
39	42	Coarse sand
42	54	Gravel, boulders, and sand, Hit water @ 50'
54	68	Gravel and sand (water bearing)
68	71	Sand, gravel, and boulders, tight clay binder
71	83	Sand and gravel, clay binder, After a depth of 68', sand pumping lowered elevation of water in hole. Open hole without caving was permissible due to tight clay binder in hole.

CASING: 3000L-Well cased to 83 ft. with ½" wall 20" O.D. pipe.
Casing perforated from 56' to 81'.

WELL 3000 ~~M~~: H

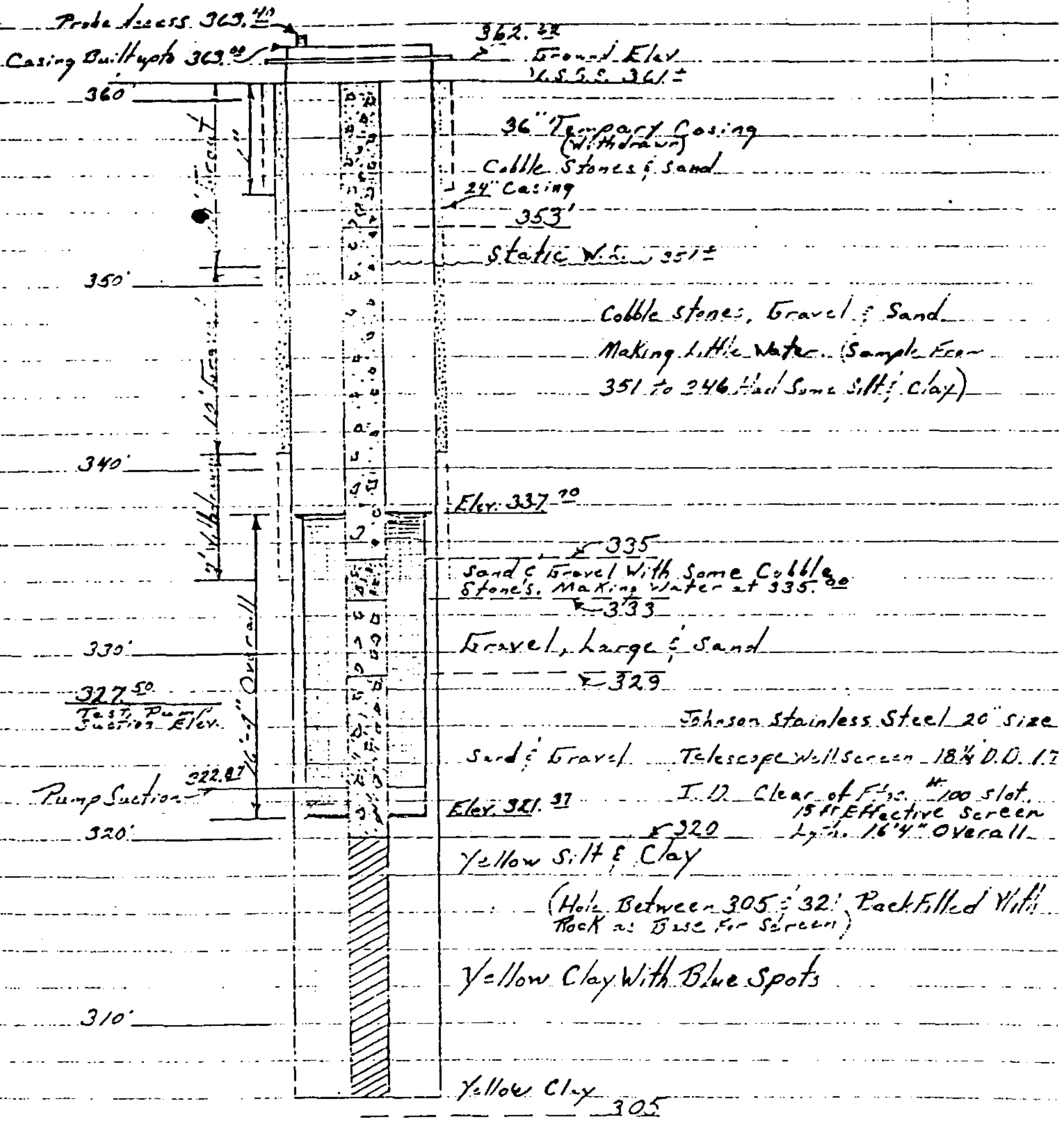
LOG OF FORMATIONS: 3000 ~~M~~ H

From	To	Formation
Surface	15	Brown sand
15	20	Cemented gravel
20	30	Loose sand and gravel, large boulders
30	39	Loose gravel and sand, water bearing
39	51	Gravel and sand, formation tighter but water bearing
51	55	Fine runny brown sand

CASING: 3000 M-Well cased to 55 ft. with ½" wall 20" O.D. pipe
Casing perforated from 28' to 50'.

Well 3000 J: Log of formation from 52 to 69'
From 50 ft. to 62 ft. Gravel, boulder, and sand
62 ft. to 69 ft. sand and gravel
69 ft. to 71 ft. gravel, sand and clay binder

3006-A Well Well Tested 11-10-[illegible]



3000 - D-5

~~3000 AREA DURAND #5 WELL~~

Started Drilling - 9-22-44
Finished Drilling - 10-13-44

Drilled by: Dishman & Morkert

12" Casing at 131'

Elev. Top of Pipe - 407.63
Ground Elevation - 407.50

Coordinates: N.415 ≠ 72.19
E.161 ≠ 80.37

Formation Record

0	2	2	Sand
2	6	4	Gravel and Boulders
6	16	10	Coarse Gravel
16	26	10	Medium Gravel
26	32	6	Coarse Gravel and Sand
32	40	8	Medium Gravel and some Sand
40	78	38	Coarse Gravel and some Sand
78	80	2	Medium Gravel and Sand
80	83	3	Boulders
83	94	11	Coarse Gravel and some Sand
94	98	4	Fine Sand and Medium Gravel
98	112	14	Coarse Gravel and Boulders
112	120	8	Medium Gravel and some Sand
120	124	4	Coarse Gravel and some Sand
124	128	4	Boulders and some Sand
128	134	6	Blue sandy Shale

Cement plug set at
130.8'

Elevation 276.83

Static - 10/13/44

52' Elevation 355.50

Perforated 11-6-44
8 Vertical Holes
Spaced at 12" Centers
From 55' to 125'

Tested 11-21-44

1125 G.P.M. with 4.55 drawdown
Specific Capacity - 250
Tested for 16 hrs.

Equiped with a Pomona 1000 gpm pump driven by a 75 H.P. Electric Motor. Serial # P.J. 3056

Appendix B. Particle Size Distribution of Individual Samples From North Richland Recharge Basins.

9 2 1 2 5 5 5 1 8 2 4

LOCATION	Gravels +5	Pebbles -5+10	VC Sand -10+18	C Sand -18+35	M Sand -35+60	F Sand -60+140	VF Sand -140+230	Silt+Clay -230
Landfill	0	0.05	11.7	63.4	21.2	2.8	0.5	0.3
NA 0-3	2.4	30.1	30.1	24.9	10.1	2.1	0.6	0.4
NA 3-6	3.7	31.7	30.3	23.9	8.1	1.7	0.4	0.4
NA 6-9	3	32	29.6	23.6	8.7	1.8	0.5	0.6
NA 9-12	3.3	31.9	30.3	23.6	8	1.7	0.5	0.5
Average NA	3.1	31.425	30.075	24	8.725	1.825	0.5	0.475
NB 0-3	49.9	9.3	9.4	15	12.4	2.5	0.7	0.5
NB 3-6	19.1	4.9	7.5	31.2	30.4	4.6	1.5	1.2
NB 6-9	46.4	2.2	3.8	22.4	20.8	2.6	0.9	0.8
NB 9-12	41.2	4.1	4.9	22.2	17.3	5.4	1.4	1.4
Average NB	39.15	5.125	6.4	22.7	20.225	3.775	1.125	0.975
SA 0-3	13	3.2	3.7	47.3	30.7	1.1	0.3	0.4
SA 3-6	4.3	0.5	2.5	59.3	32.9	1	0.3	0.2
SA 6-9	3.1	0.7	3.8	53	37.3	1.1	0.3	0.4
SA 9-12	4.9	1	3.3	57.8	32.3	0.8	0.2	0.2
Average SA	6.325	1.35	3.325	54.35	33.3	1	0.275	0.3
SB 0-3	63.6	13.9	6.5	10	5.6	0.7	0.2	0.2
SB 3-6	60.7	10.6	5.5	12.8	9.3	0.6	0.1	0.1
SB 6-9	63.5	8.2	4.5	13.9	9.4	0.3	0.1	0.1
SB 9-12	71.5	5.2	4.7	11.6	6.3	0.2	0.1	0.1
Average SB	64.825	9.475	5.3	12.075	7.65	0.45	0.125	0.125

Appendix C. Estimated Costs and Labor Requirements to Implement Recommended Actions.

Appendix C. Estimated Costs and Labor Requirements to Implement Recommended Actions.

The following cost estimates were developed through contacts with local (Tri-City) contractors only and reflect a probable range of costs for performing the specified recommended actions. These costs should not be construed as being firm quotes for performance of the work, but instead, should be used for planning purposes only.

1) Line Recharge Basins with Sand. If the sand located at the City landfill is deemed to be suitable for this purpose, the expense involved will be the cost of excavating, transporting, and spreading the sand. If a source other than the landfill is used, an additional expense for the sand itself will be included. The area of the north and south recharge basins is approximately 6.5 acres combined. To cover this area with sand to a depth of one foot will require approximately 10,000 cubic yards of material. Estimated costs for this action are as follows:

   a. Excavate sand, haul from landfill area, 10,000 yd^3 @ \$3.00 to \$5.00 per yd^3 ....\$30,000 to \$50,000

   b. Purchase sand from other source, delivered to site, 10,000 yd^3 @ \$8.50 to \$9.00 per yd^3 ....\$85,000 to \$90,000

2) Repair Dike Between North and South Basins. This job is a maintenance item that could be performed by City maintenance personnel. Estimated labor would be two man-days, and approximately two cubic yards of soil material are required.

3) Repair or Replacement of Fence Around North and South Recharge Basins and Settling Basin. The settling basin is currently unfenced, and the existing fence around the two recharge basins is in disrepair. Repair of the existing fence would be performed on an hourly fee basis and would require specific inspection for accurate costing. Replacement of the existing fence with new six-foot steel mesh and steel pole fence and installation of the same type of fence around the settling basin (for a total of 5800 feet of fence with three drive-through gates) is estimated to cost the following (depending on final specification):

   5800 linear feet @ \$6.75 to \$8.35 per foot.....\$37,000 to \$48,000

4) Relocate Existing Pumps Within Well Field. Relocation of the pumps per recommendations will involve lifting each motor and pump and resetting the motor and pump at the desired location. This type of work is performed on a hourly basis and the amounts estimated here do not include time or materials for disconnection or installation of electrical service, or disconnection and reconnection of the outlet manifold at each well. The estimated cost for relocation of pumps in the well field is a follows:

Total of 6 pumps @ $700 to $900 per pump...........$4200 to $5400

APPENDIX B

Please Print Plainly
USE HEAVY PENCIL
DO NOT WRITE IN SHADED AREAS

State of Washington
Department of Social and Health Services
Health Services Division
PUBLIC HEALTH LABORATORIES
1409 Smith Tower, B17-9, Seattle, Washington 98104

WATER SAMPLE INFORMATION FOR INORGANIC CHEMICAL ANALYSES

LAB. NUMBER	CO.	CITY	DATE RECEIVED	DATE COLLECTED	COLLECTED BY:
S 108147			03/29/85	03/26/85	MaryAnn Stone; Telephone: (509) 943-9161 Ext 269

Is this a follow up of a previous out of compliance sample? Yes ☐ No ☒

If yes, what was the laboratory number of the previous sample? — — — — — — —

SYSTEM I.D. NO.	SYSTEM NAME	SYSTEM CLASS (circle one)	COUNTY
72250W	City of Richland	(1) 2 3 4	Benton

SAMPLE LOCATION: 1

THIS SAMPLE TAKEN BEFORE ☐ AFTER ☒ TREATMENT (U / T) CHECK ONE OF THE ABOVE BOXES

IF TAKEN AFTER TREATMENT WAS IT ___FILTERED ___FLUORIDATED X CHLORINATED ___WATER SOFTENER: TYPE USED ___

SOURCE TYPE: X 1. SURFACE ___ 2. SPRING ___ 3. WELL ___ 4. PURCHASE

SOURCE NO.: — —

IF SOURCE IS LAKE OR STREAM, ENTER NAME: Columbia River

IF SAMPLE WAS DRAWN FROM DISTRIBUTION SYSTEM IT WAS COLLECTED FROM SYSTEM AT: (ADDRESS) 110 Saint (Combined)

DATE OF FINAL REPORT: 5/23/85

SEND REPORT TO: (PRINT FULL NAME & ADDRESS)
Name: Stan Aelt c/o City of Richland
Street: 505 Swift, P.O. Box 190
City: Richland WA. Zip Code: 99352
Telephone: (509) 943-9161 (Area Code)

REMARKS:
Chlorine Residual - 0.5ppm

LABORATORY REPORT
(DO NOT WRITE BELOW THIS LINE)

TESTS	*MCL	LESS THAN ≤	RESULTS	UNITS	Compliance YES	Compliance NO	Chemist Initials
Arsenic As	0.05[P]	≤	0.010	mg/l	✓		IGN
Barium Ba	1.0[P]	≤	0.25	mg/l	✓		MS
Cadmium Cd	0.01[P]	≤	0.002	mg/l	✓		IGN
Chromium Cr	0.05[P]	≤	0.010	mg/l	✓		IGN
Iron Fe	0.3	≤	0.05	mg/l	✓		SCB
Lead Pb	0.05[P]	≤	0.010	mg/l	✓		IGN
Manganese Mn	0.05	≤	0.010	mg/l	✓		IGN
Mercury Hg	0.002[P]	≤	.0005	mg/l	✓		MS
Selenium Se	0.01[P]	≤	0.003	mg/l	✓		IGN
Silver Ag	0.05[P]	≤	0.010	mg/l	✓		IGN
Sodium Na		≤	5	mg/l			MS
Hardness		—	70	mg/l AS CaCo3			SFP
Conductivity	700	—	160	Micromhos/cm 25° C	✓		PO
Turbidity	1.0[P]	—	0.2	NTU	✓		PO
Color	15.0	≤	5.	Color Units	✓		PO
Fluoride F	2.0[P]	≤	0.2	mg/l	✓		SFP
Nitrate as N	10.0[P]	≤	0.2	mg/l	✓		[illegible]

LABORATORY SUPERVISOR
(Name or Initials)
R. [illegible]

CHARGE: 8/40 [illegible]

REMARKS:

9212518299

CITY OF RICHLAND

WATER CHEMICAL ANALYSIS

Test	MCL*	RESULTS 1986	1985	1984	1983
Arsenic (As)	0.05 mg/l	< 0.010	< 0.010	< 0.010	0.001
Barium (Ba)	1.0 mg/l	< 0.25	< 0.25	< 0.25	0.03
Cadmium (Cd)	0.01 mg/l	< 0.002	< 0.002	< 0.002	0.001
Chromium (Cr)	0.05 mg/l	< 0.010	< 0.010	< 0.010	0.001
Iron (Fe)	0.3 mg/l	< 0.05	< 0.05	0.17	-----
Lead (Pb)	0.05 mg/l	< 0.010	< 0.010	< 0.010	0.017
Manganese (Mn)	0.05 mg/l	< 0.010	< 0.010	0.051	-----
Mercury (Hg)	0.002 mg/l	< 0.0005	< 0.0005	0.0005	0.0004
Selenium (Se)	0.01 mg/l	< 0.003	< 0.003	< 0.003	0.001
Silver (Ag)	0.05 mg/l	< 0.010	< 0.010	< 0.010	0.006
Sodium (Na)	20 mg/l	< 5	< 5	< 5	-----
Hardness	mg/l as Ca CO3	80	70	70	-----
Conductivity	700 micromhos/cm 25 degrees C	160	160	150	-----
Turbidity	1.0 NTV	0.4	0.2	0.4	-----
Color	15.0 mg/l	< 5.0	< 5.0	< 5.0	-----
Fluoride (F)	2.0 mg/l	< 0.2	< 0.2	< 0.2	0.2
Nitrate (as N)	10.1 mg/l	< 0.2	< 0.2	< 0.2	0.1
Chloride (Cl)	250 mg/l	5	5	< 5	-----

*Maximum Contaminant Level allowed

CWC-HDR, Inc.
An HDR Infrastructure Company

Suite 204
300 Admiral Way
Edmonds, Washington
98020-4127

Telephone
206 774-1947

Water Resources
Wastewater
Industrial
Hazardous Waste

CWC-HDR, Inc.

CITY OF RICHLAND

TECHNICAL MEMORANDUM

DATE: November 6, 1987

PREPARED BY: Brian Hemphill, P.E.

SUBJECT: Report on Inspection of Filters at Water Treatment Plant

BACKGROUND

This report is written as part of a study being conducted by CWC-HDR, Inc. for the City of Richland on the water treatment plant and North Richland well field. One of the tasks in the contract calls for an evaluation of the filters at the treatment plant. They have shown signs of deterioration over the past few years. These signs include a noticeable drop in filtered water quality (although still well within federal and state guidelines); the finding of filter media materials in the plant clearwell; shortened filter runs, which result in increased operating expense and lowered net production; and increased chemical costs.

Mechanism of Filter Disruption

A preliminary evaluation of the available information pointed to a significant disruption of the underlying support gravel layer in the filters. The several layers of specially graded gravel are intended to act as a barrier to leakage of filter coal and sand and to help distribute backwash water throughout the media. It is normal for the gravel layer to become gradually disturbed as a result of jetting action during backwash. The action is depicted in Figure 1. This results in localized high-velocity areas that lift the smaller top-layer gravel into the filter. As the top layer is disrupted, the action becomes more aggravated. The result is that the top protective layer is destroyed, allowing leakage of media into the underdrain system. Backwashes become more difficult to perform since the distributive ability of the gravel is impaired. Because some areas do not get cleaned as well as others, the effective filtration area is reduced, resulting in poorer filtrate quality and shorter runs.

As mentioned, this type of gradual disruption is natural and occurs in virtually all gravel-supported granular media filters. Typical lives of such filters are on the order of five to ten years. The filters at Richland have been in operation for 24 years. This is well beyond the normal "life expectancy" of such systems, especially those that are operated at high filtration rates and high terminal headloss levels as are the Richland filters.

Although the preliminary conclusion called for replacement of the filter media, an inspection was made of the filters in order to confirm the conclusion.

INSPECTION

Procedures

The filters were inspected on October 22 by Jo Engeset and Brian Hemphill of CWC-HDR, Inc., with the assistance of Scott Meyer. The filters had been drained to expose the media; they had not been recently backwashed and had been in service for various durations prior to the inspection.

Two types of examination were conducted. First, test excavations were made in all four filter basins. Media was removed by shovel to expose an area of gravel roughly three feet by five feet. The filter media was examined for evidence of mudballs or other signs of contamination or incomplete cleaning. Depths of the media layers were measured. The integrity of the gravel layer was checked by visual inspection.

Figure 2 shows the locations of the test holes. These were selected based on observations by the operators that these were areas that are most often affected by air releases during backwash.

The second examination consisted of probing of the gravel surface in order to get a more thorough measurement of the extent of disruption. This involved placing the filter into backwash mode in order to fluidize the media. A probe, consisting of a garden hoe with an extension handle and measuring tape, was lowered down through the media until it rested on the gravel surface. This was done while walking up and down the washwater troughs. The hoe head was moved about on the surface while the distance to the water surface was measured. It is easy to detect mounds and depressions in this manner. The location and depths of the disturbances were recorded. Probing was performed in filters 1 and 2.

Results

Test Holes--

Condition of Media. In all test holes, the media was found to be in excellent condition. There were no mudballs found of any size. Virtually no foreign materials were found, except for a very slight layer of what appeared to be plant materials (stems, etc.) on the surface. Generally, the gradation and mixing of the coal and sand layers appeared to be proper. In two of the holes, however, there were areas in which pockets of coal were found in the sand layer, in a sort of marble effect. This demonstrates abnormal distribution of backwash water that results in downward currents.

Media Depth. Measurements of the depths of media in the test holes are summarized in Table 1. These agree reasonably well with measurements made by plant personnel in January, 1987. The typical total depth of about 20 inches represents a loss of one third of the original depth of 30 inches.

Condition of Gravel. In all test holes, there was evidence of disruption of the gravel surface. In four of the six holes, one to three significant craters were found in the gravel. The typical crater had a diameter at the surface of about 12 inches, and was 5-6 inches deep at the center. At the bottom of the craters, the gravel was of the size range of 3/8 in. to 3/4 in., which would be expected at this depth. This compares to the surface layer which consists of pea gravel about 1/8 in. to 3/16 in. in diameter, which is designed to retain the sand. The coarser, lower layers, of course, do not provide an effective barrier, and are the paths of media leakage.

Significant mounding of gravel was found along the walls. These were evidenced by disturbances in the otherwise flat media surface along the walls.

Probing--

The probing permitted a more extensive examination of the gravel surface, although still over only a fraction of the total area, due to access limitations. The results are shown in Figure 3.

In the three passes over filter no. 1, a total of five noticeable craters of 3-5 inches in depth were detected, along with a number of smaller (1-2 inch) mounds and depressions. In filter no. 2, the disruption was much more extensive. A total of 18 craters were detected, ranging in depth from 4-7 inches. The area probed represented about 30 to 40 percent of the total area.

DISCUSSION

The examination confirmed the suspicion regarding the filters. The support gravel is significantly disrupted, with the integrity of the upper barrier having been lost. The craters in the surface are obviously the paths that the media that has been found in the clearwell has travelled.

The filters have been kept in a very clean condition, which probably has helped keep them in shape to provide reasonably good service. However, the filtering capability is becoming increasingly compromised by the lack of full media depth.

The filters have reached the end of their useful life. If they are continued to be operated in their present condition, rapid deterioration in performance can be expected as the disruption process continues. The deterioration will manifest itself in the following ways:

- lessening of the quality of the product water

- higher chemical usage

- higher operating costs resulting from more frequent and more lengthy backwashes

- perhaps most significant, is that the effective capacity of the filters will continue to drop as they become more prone to premature floc breakthrough

RECOMMENDATION

The media and support gravel should be replaced in all four filter basins as soon as possible. In our opinion, an attempt to operate them for another year in their present condition would be very risky, and could result in total loss of the use of one or more of the basins. Replacement of the media will improve plant operation in every way, and will insure the production of high quality water at the lowest practical cost.

TABLE 1 - MEDIA DEPTH MEASUREMENTS

FILTER NO.

	1	2	3	4
SAND	4"	5"	6"	6"
COAL	15"	12"	15"	13"
TOTAL	19"	17"	21"	19"
January, 1987				
Measurments:	21.5"	18.0"	20.3"	19.6"

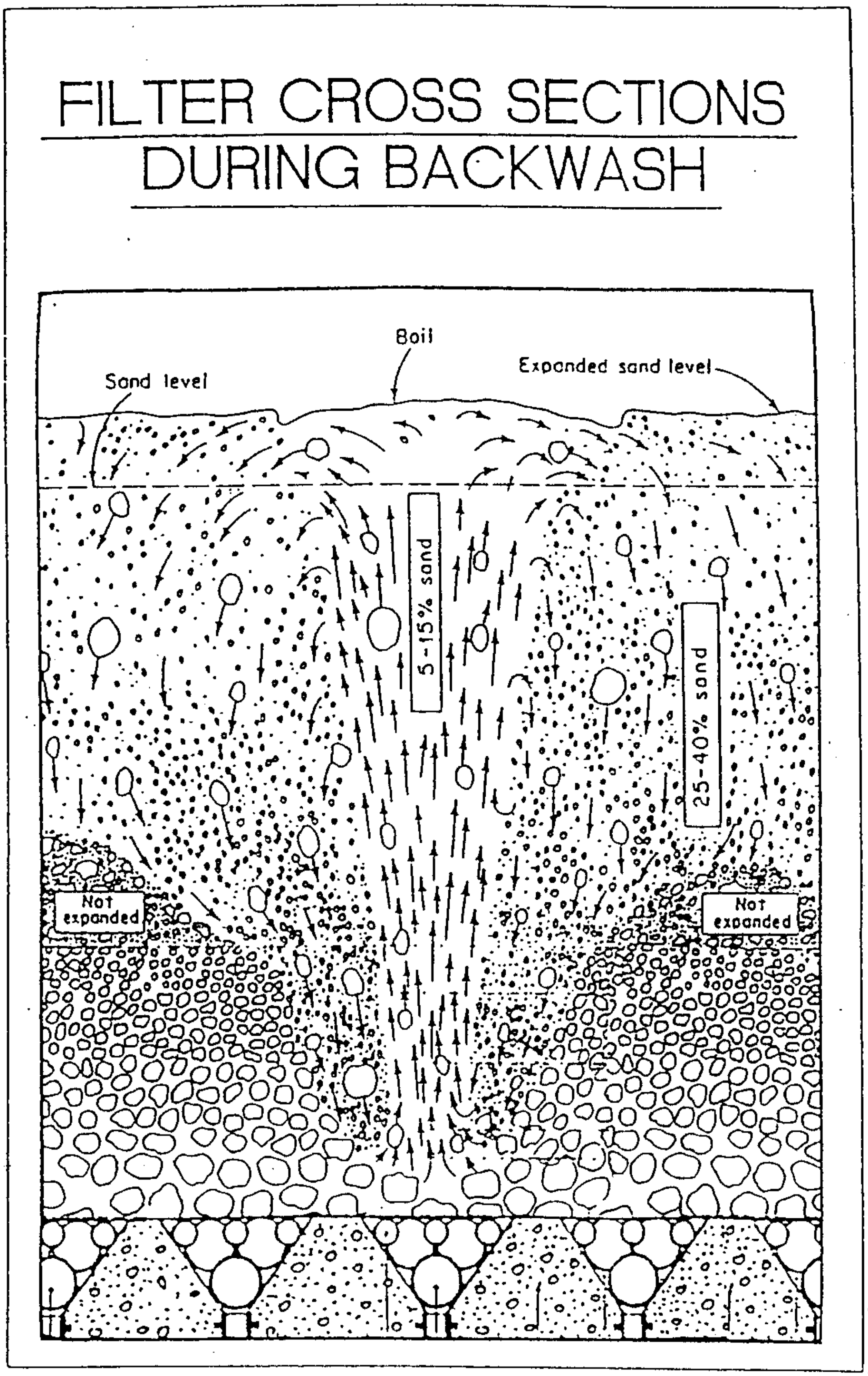


FIGURE 1. MECHANISM OF GRAVEL DISRUPTION

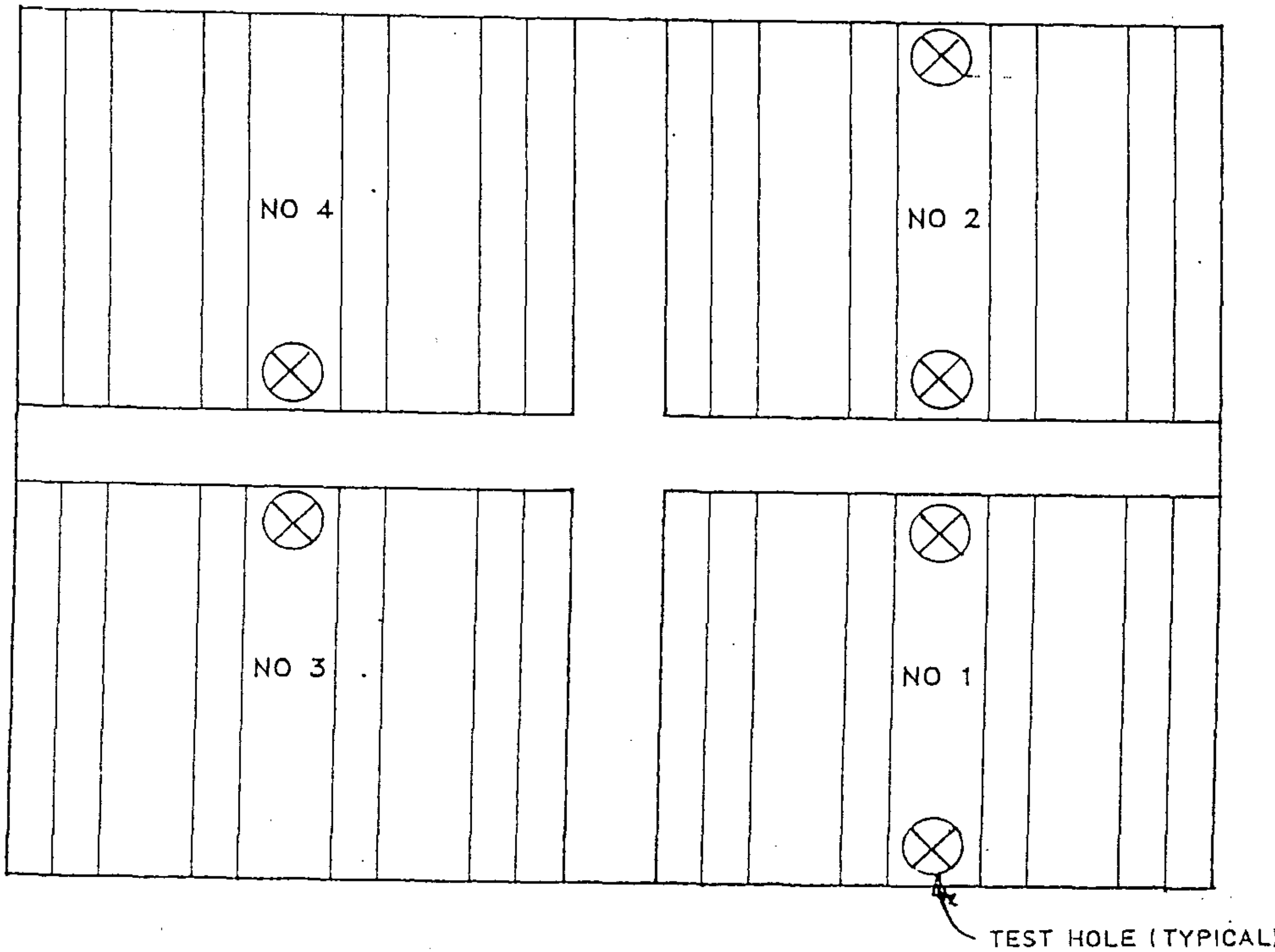
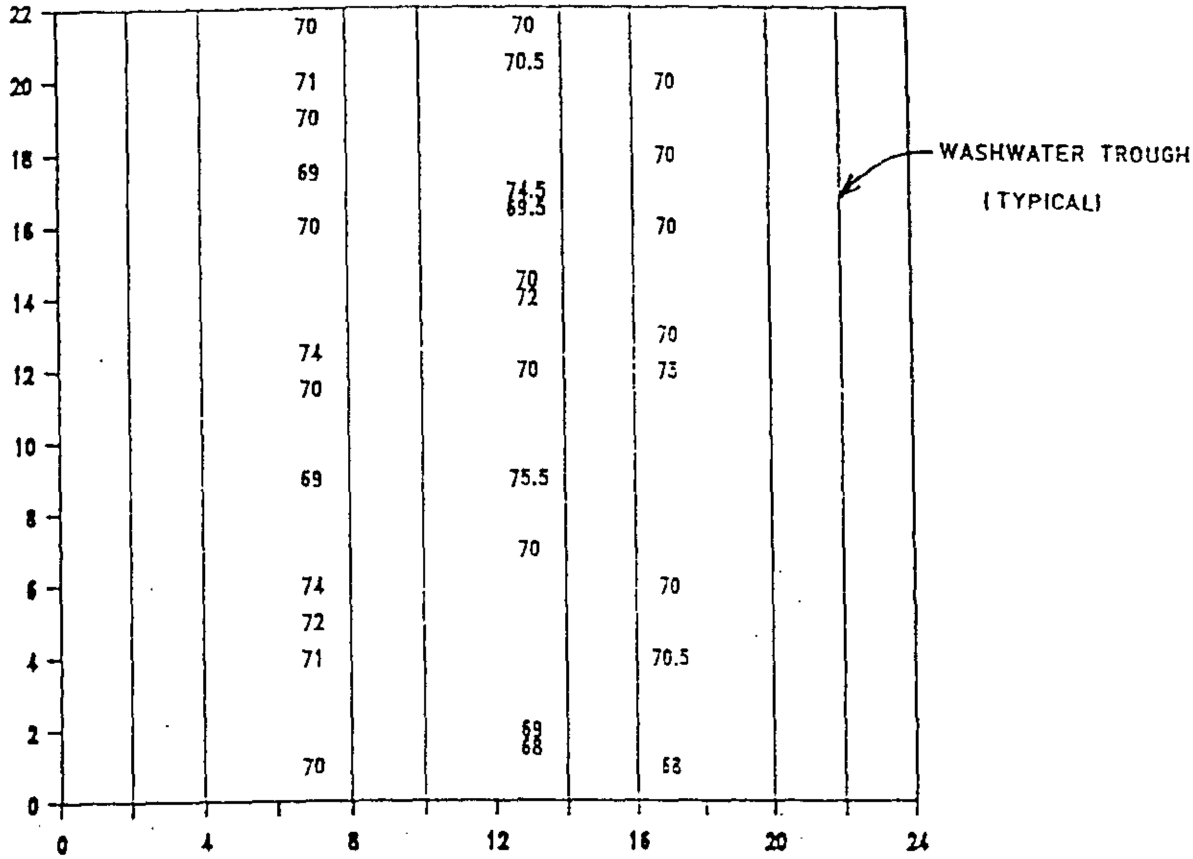
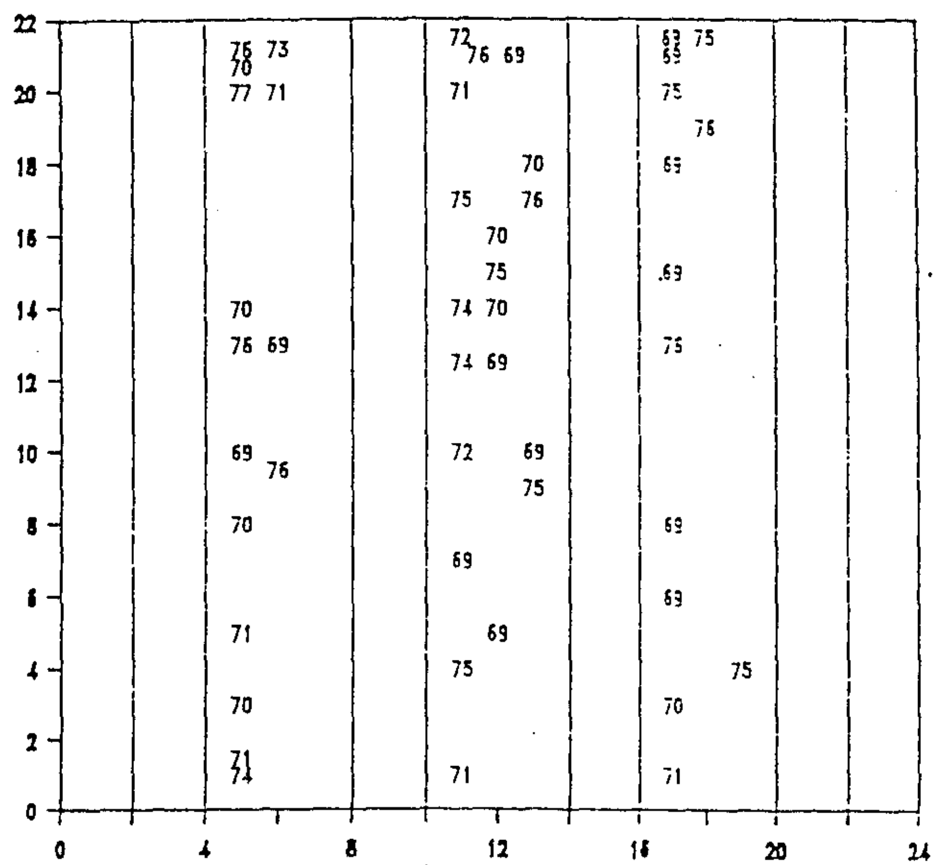


FIGURE 2. LOCATION OF TEST HOLES



FILTER NO. 1

READINGS IN INCHES MEASURED DOWN FROM WATER SURFACE



FILTER NO.2

FIGURE 3. RESULTS OF PROBING OF GRAVEL SURFACE

9212551839

9212551840

APPENDIX D

SECTION 11329

FILTER MEDIA

PART 1 - GENERAL

1.01 DESCRIPTION

A. SCOPE
   1. Remove media and gravel from 4 existing gravity filters
   2. Dispose of material on City property as instructed by City
   3. Furnish and install replacement media and gravel

1.02 QUALITY ASSURANCE

A. Preliminary Acceptance Tests
   1. All materials to comply with AWWA Specification B-100-80 for filter materials and the following specific requirements
      a. Filter sand
         1) Sieve analysis
            a) Coarse high density sand - ASTM C136
            b) Fine sand - ASTM C136, including calculation of effective size and uniformity coefficient
         2) Loss of ignition - ASTM C25
         3) Acid solubility - AWWA, B-100-80
      b. Anthracite
         1) Sieve analysis - ASTM C136, including calculation of effective size and uniformity coefficient
      c. Gravel
         1) Sieve analysis - ASTM C136

1.03 SUBMITTALS

A. Shop Drawings and Product Data
   1. Sectional view of filter showing specific bed design
   2. Materials descriptions and certified test analyses of filter materials proposed
   3. Certification that actual materials to be supplied have been tested according to preliminary acceptance tests and meet specified quality
B. Operating and Maintenance Manuals
   1. Description of materials
   2. Recommended backwash rates
   3. Maintenance requirements
C. Certificate of proper installation

1.04 JOB CONDITIONS

A. Existing Filters
   1. Type: Dual medial (anthracite/silica sand)
   2. Media depth: approximately 20"
   3. Number: 4

4. Dimension of each filter
   a. Length: 24'
   b. Width: 22'
5. Bed area
   a. Each filter: 528 sf
   b. Total plant: 2112 sf
6. Underdrain gravel
   a. Depth: estimated 10" (verify)

PART 2 - PRODUCTS

2.01 MATERIALS

A. Filter Materials Quality
   1. Filter Gravel: AWWA 3-100-80
   2. High density san and gravel
      a. Garnet or ilmenite
      b. Specific gravity: greater than 3.8
      c. Acid solubility less than 5% when tested in accordance with AWWA 3-100-80
      d. Free of shale, mica, clay, silt, sand, shells, dirt, organic impurities, or other foreign matter.
   3. Filter sand: AWWA B-100-80 except as modified herein
      a. Loss on ignition: 4% maximum
   4. Filter Anthracite: AWWA B-100-80 except as modified herein
      a. Acid solubility: 1% maximum
      b. Caustic solubility: 2% maximum in 1% sodium hydroxide solution at 190 F

B. Inspection and Sampling
   1. Filter media and gravel must be tested at source prior to shipment
   2. Notify City 10 calendar days prior to shipment
   3. Samples to be taken and analyzed to verify compliance with previously submitted test reports
   4. All materials to be approved prior to shipment
   5. Representative samples will be taken of filter media at the rail siding and before placement in each filter to verify compliance with certified test results

C. Media Composition and Depths
   1. Filter gravel

Layer Above Filter Bottom	Depth of Layer in Inches	100% Retained On ASTM Sieve Size	100% Passing ASTM Sieve Size
1st	2	3/4"	1 1/2"
2nd	4	3/8"	3/4"
3rd	3	3/8"	3/16"

   2. High density gravel
      a. Place over top layer of filter gravel
      b. Depth: 3"
      c. Size: 100% passing No. 4 sieve; 100% retained on No. 14 sieve

3. High density sand
   a. In 3" layer directly over high density gravel and with the following characteristics based on ASTM Standard square hole sievers
      1) Effective size: 0.18 to 0.28 mm
      2) Uniformly coefficient: Less than 2.2
      3) Specific gravity: 4.0 ± 0.2
4. Silica sand
   a. In 9" thick layer directly over high density sand with the following characteristics based on ASTM standard square hole sieves
      1) Effective size: 0.45 to 0.55 mm
      2) Uniformity coefficient: Less than 1.5
      3) Specific gravity: 2.6 ± 0.05
5. Anthracite
   a. In 18" layer over silica sand and with the following characteristics based on ASTM standard square hole sieves
      1) Specific gravity: Greater than 1.55
      2) Effective size: 1.0 to 1.1 mm
      3) Uniformity coefficient: Less than 1.7

D. Extra Filter Media
   1. Furnish not less than 5% of total amount of fine high density san, silica sand, and anthracite coal as extra media
   2. Furnish not less than 2% of total amount of each layer of gravel as extra media
E. Packaging
   1. Provide all materials in woven polyethylene bags; or in multiwall plastic-lined paper bags

PART 3 - EXECUTION

3.01 Storing and Protecting Media

A. ?
   1. At the plant site, filter media is to be stored at locations approved by the City
   2. The media is to be protected from rain, wind, and damage from other sources until placed in filter boxes. Protection against moisture is essential. Beneath the bags of media pallets or lumber may be used. The pallets of bags are to be covered with plastic, canvas, or other water-repellent materials

3.02 INSTALLATION

A. Removal of Existing Media
   1. Remove existing media and gravel and dispose of as directed by City
   2. Sand and gravel may be removed by hydraulic means as approved by Engineer
   3. Repair any damage to underdrain system caused during media removal at Contractor's sole expense
   4. Notify City 48 hours in advance of media removal

5. Coordinate timing of work with City personnel to minimize disruption of plant operation
6. Porcelain spheres in filter bottom may be retained if sound and undamaged

B. New Media and Gravel
1. Take care to avoid contamination of materials in transport and placement
2. Replace material that becomes contaminated with organic matter
3. Place all material under supervision of media supplier's field representative
4. Gravel
   a. Place material in accordance with AWWA B-100-80 as modified herein
   b. Prior to placement, thoroughly clean filter compartment; remove any foreign material and check to ensure that spheres in underdrain are properly placed and clean of obstruction
   c. Place first layer by hand to ensure that underdrain spheres are not disturbed
   d. Complete entire layer in filter before beginning next layer
5. Filter Media
   a. Place in accordance with AWWA B-100-80 except as modified herein
   b. Place high density filter sand to a depth 1/2 greater than finished depth of layer
   c. Backwash filter for a minimum of 15 minutes at a rate of 20 gpm/sf
   d. Skim top 1/2" layer of high density san and discard
   e. Place silica sand filter media to a depth 1/2" greater than finished depth of san layer
   f. Backwash filter for a minimum of 15 minutes at a rate of 20 gpm/sf
   g. Skim top 1/2" layer of silica sand and discard
   h. Place anthracite coal to a depth 1" greater than finished depth of coal layer
   i. Backwash filter for a minimum of 15 minutes at a rate of 20 gpm/sf
   j. Skim top 1/2" layer of anthracite coal and discard
   k. Repeat step j

PART 3 - EXECUTION

3.01 FIELD QUALITY CONTROL

A. Filter Media Testing: To be provided by City
1. Filter gravel and coarse sand: One sieve analysis per carload or truckload of each size of gravel or coarse sand or more at City's option
2. Fine sand: One sieve analysis per carload or truckload or more at City's option
3. Anthracite: At least 1 sieve analysis per carload or truckload

B. Manufacturer's Field Service
1. Provide services of Supplier's factory trained field representative

during placement of filter media in all filters
2. Supplier's representative to check out completed filter operation and submit a certification of proper media installation

END OF SECTION

The cover of the September, 1986 *CALIFORNIA* magazine features actors Rosanna Arquette dumping a glass of water to highlight the story "Should Tap Water Be for Drinking." The opening paragraph sets the tone:

> "A Sacramento teenager reaches for a glass of tap water on a hot summer's day, sips, grimaces and throws the water back into the sink. Then he reaches for a bottle of mineral water. He is part of a growing revolution that may replace the tax rebellion as the centerpiece of California politics - a rebellion for safe drinking water."

Professionals in the water supply field have seen many other similar "scare" stories over the years. However, there was soon proof that this article did reflect public concerns. In the general election, two months later, a new State law (Proposition 65) reviewed in the above article was overwhelmingly (a 2:1 margin) approved by the California electorate. The campaign slogan was "Get Tough on Toxics". Based on the spread to other states of the property tax limitation established in the famous Proposition 13 in California, other states may soon follow suit with similar regulations.

The author of the *CALIFORNIA* magazine article was not alone in sensing public unrest. The United States Congress also perceived public concerns about drinking water quality and in June, 1986 sent a sweeping set of amendments to the 1974 Safe Drinking Water Act to President Reagan, who promptly signed them into law. The new federal regulations apply to all systems serving 25 or more customers and will effect dramatic changes in the drinking water industry over the next few years. As evidenced by Proposition 65 in California, new state regulations may go even further. While we can only speculate about future state regulations, we can assess the 1986 amendments to the Federal Safe Drinking Water Act. There is no question that these amendments will have significant impacts on everyone involved in the drinking water industry - designers, owners, and operators. It is the purpose of this paper to review some of the more widespread effects.

<u>Number of Regulated Contaminants Have Increased (and will continue to!)</u>

Prior to the 1986 Amendments, allowable concentration for 22 contaminants had been established by EPA. EPA had published a list of 83 contaminants (14 volatile organic compounds, 35 other organics, 23 inorganics, 6 related to microbiology or turbidity, and 5 radionuclides) for which they felt allowable concentrations should be considered. Congress brushed aside any further deliberation on the list and simply dictated by law that all 83 will be regulated by 1989 - nine by June, 1987; 40 in the next 12 months; and the rest in the third year. The first nine regulated in June, 1987 include eight volatile organic compounds (VOC) and fluoride (see Tables 1 and 2). EPA estimates that 1,300 systems will be in violation of the VOC MCLs. Congress also directed EPA to add 25 additional contaminants to the list every 3 years with no limit on number of additions! This means that operators are facing monitoring and reporting an ever increasing number of contaminants. Designers are faced with the task of designing facilities with a 10 to 20 year life without knowing which contaminants will be regulated only 3 years from now. EPA published in June 1987, 45 candidate compounds for the next 25 to be regulated. MCLs for at least 25 will be published by January, 1990 and MCLs established one year later.

<u>The Procedures for Setting Allowable Concentrations Has Changed</u>

Previously, allowable concentrations were established in a two-step process. First, a recommended maximum contaminant level (RMCL) was published for comment. The RMCL was that determined by EPA to have no known adverse health effect. EPA would then publish a maximum contaminant level (MCL) that EPA considered to be technically and economically feasible and as close as possible to the RMCL. The concentration with no known health effect is now called the maximum contaminant level goal (MCLG) rather than the RMCL. The MCL nomenclature hasn't changed. Now, both the MCLG and the RMCL are promulgated simultaneously.

The MCLG and MCL procedure raises a potential liability dilemma for designers and owners. A system which meets the MCL but not the MCLG, (which is all that is required by EPA), is by definition distributing water which contains a concentration of a contaminant that has a known adverse effect on health. Joe Karaganis, a prominent environmental attorney[1] thinks that this approach has opened Pandora's Box for litigation. He believes there will be public lawsuits for toxic torts and, in some states, for mental anguish caused by knowing that known carcinogens in "harmful" (as defined by EPA) concentrations are present in their water

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1 Karaganis, Joseph, "And Not a Drop to Drink. The Toxic Crusade Takes On the American Drinking Water Industry: The 1986 Safe Drinking Water Act Amendments and Toxic Torts". Presented at CWC-HDR Lake Tahoe Water Treatment Seminar (Oct. 1, 1986).

supplies. In case you think the legal profession hasn't noticed this potential new market, the American Bar Association held a conference dedicated solely to these amendments in February, 1987. Karaganis believes that water designers and owners of drinking water systems face potential exposure to liability.

Some Limitations Will Change

Some limitations may change. For example, the existing MCL for lead is 50 ppb. In early November 1986, EPA proposed lowering the concentration to 20 ppb. The Environmental Defense Fund claimed the proposed new limit was too high to end a "lead epidemic" and that it should be lowered to 10 ppb. In December, EPA announced it was revising an earlier estimate of the number of customers using water with concentrations in excess of 20 ppb upward to 42 million and was reassessing the 20 ppb proposal. The manufacturers of PVC pipe (Chemical Week, Nov. 26, 1986) gleefully noted that the market for PVC pipe would be expanded greatly by the proposed new lead limits and cautioned that new limits on copper may well also restrict the use of copper pipe. Since the primary source of lead at the tap is the lead in distribution system piping, the utility cannot rely on control by removing lead at the treatment plant but must attempt to control the corrosivity of the treated water. Compounding the potential difficulty in meeting the lower limits is a requirement in the amendments that utilities must notify their customers if there is lead in the distribution system, there is the potential that there could be a violation of the lead MCL. This notice must be given even if lead concentrations in the water are less than the MCL.

Triholomethane (THM) concentrations have been limited to a maximum of 0.1 mg/L; however, EPA is considering a reduction. Many utilities are anxiously awaiting the EPA proposal because rumors have placed the new limit as anywhere from 5 to 75 ppb. Some would have difficulty meeting the higher limit, most would have major difficulty in meeting the limits in the lower end of this range.

Beaver Fever Will Take Its Toll

Giardiasis, is a disease with most unpleasant symptoms such as severe - and often prolonged- diarrhea. It is caused by Giardia, an organism carried by a wide range of warm blooded animals such as man and beavers - hence its synonym "beaver fever". The Giardia organism has been isolated from many "controlled" raw water reservoirs - not surprising since absolute control of man is not possible and beavers and other animal carriers haven't yet been trained to avoid controlled watersheds. The Giardia organism is very difficult to kill by disinfection alone and is much more effectively removed if the water is filtered. Of course, the same is true for viruses. Publicity on outbreaks of Giardiasis may have spurred Congress to consider requiring that all surface waters be filtered. EPA has estimated that there are 1,000 unfiltered surface supplies. Some large utilities - such as New York City, Seattle, Portland, Tacoma, Boston, San Francisco, Reno - which filter none or only part of their water let their Congressmen know that the costs for them would be significant. In the final amendments, Congress has given EPA 18 months to establish criteria to be used to determine who must filter and who need not filter.

EPA has issued several drafts of the filtration criteria. Although the drafts have no official status, they give some insight into what it will take to avoid filtration. According to current drafts, it will take a low turbidity raw water (less than 1 NTU monthly average turbidity), low fecal coliform concentrations (less than 20/100 ml 90 percent of the time), redundant disinfection systems with automatic startup and alarms, a watershed control program which will have to be evaluated annually, and demonstrated compliance with finished water limits on THM's and coliforms. The final rules are still uncertain but the end result will be that many - especially the smaller - systems using unfiltered surface supplies will be building filters. The potential economic effects are significant. If the 15 unfiltered systems serving over 100,000 population were to all add filters, the costs would approach one billion dollars. EPA has estimated that this requirement will have an overall cost of about $2.4 billion.

Even those with filters are not immune to impact by the amendments. The regulations will establish performance criteria for filters which many existing systems may not meet. The current draft proposes that the filtered water turbidity must be less than 0.5 NTU for 95 percent of the time. The draft version allows the regulatory agency some flexibility in establishing criteria where raw water turbidities are less than 1 NTU. Some states currently define satisfactory filter performance as a monthly average of 1.0 NTU - a much more liberal requirement than 0.5 NTU for 95 percent of the time. Depending on the final criteria, many utilities may be faced with upgrading their existing filtration systems. EPA estimates that the current draft will require expenditure of about $330 million to upgrade existing systems.

Disinfection

Similarly to the filtration requirement, Congress has dictated that all supplies - including groundwaters - be disinfected with the potential for some groundwaters to be exempted. EPA has until June, 1989 to adopt the final rules, including definition of the criteria by which an exemption can be granted for some groundwaters. There are thousands of public groundwater supplies, many of which do not disinfect. In those systems where the wells are scattered on all sides of town, multiple disinfecting systems will have to be built. For those cases where the groundwater contains iron or manganese, disinfection may create another water quality problem - oxidizing these materials will turn the water red, black, or a little of both. A filter will have to be added to remove the iron or manganese. If the wells are scattered, the City is faced with the unpleasant alternatives of building a filter at every well or pumping all the raw water to a common point for disinfection and filtration and then returning the treated water to the distribution system. The distribution system hydraulics may or may not be compatible with returning all the treated water at a single point. If they are not, a major repiping project may be in order.

While the rules for an exemption for disinfection of a groundwater supply are still two years away, it is reasonable to expect that a well documented case that the supply and distribution system are microbiologically safe will be essential. It is none to soon to make sure that the data you are collecting on your groundwater supply and distribution system presents a thorough, accurate picture of its microbiological quality.

There is no exemption from disinfection for surface supplies - and don't be to quick to decide that a supply which is never exposed to the atmosphere is automatically a groundwater. If a "ground" supply shows substantial variations in quality - as it might in an infiltration gallery system - it can be classified as a surface supply. Evidence of direct influence of a surface water on a ground water (i.e., significant fluctuations in turbidity) may result in reclassification of a groundwater as a surface water.

All systems must have disinfection systems which can provide 99.9 percent inactivation of Giardia cysts and 99.99 percent of enteric viruses. EPA will be publishing guidance for calculating the combinations of disinfectant dosages and contact times which will meet these requirements under various raw water conditions. Poorer quality raw waters will, of course, require both filtration and disinfection to meet these requirements.

Treatment Techniques to be Identified

For each regulated contaminant, EPA must identify the treatment technique that it finds to be feasible for meeting the MCL for the contaminant. This identification does not mean that the identified technique must be used, but one of comparable efficiency must be. For synthetic organic compounds (SOC), Congress specified in the amendments that granular activated carbon (GAC) is feasible. Cincinnati, Ohio has already designed a 150 mgd GAC treatment facility for its water supply. This means that any proposed techniques to meet SOC requirements must be at least as effective as GAC. For VOC's, EPA has designated GAC and packed tower aeration (PTA) has BAT (except for vinyl chloride for which PTA is specified). If EPA finds that it is not practical to measure the level of a given contaminant, it can require the use of a specific treatment process. Under the original law, a system could apply for a variance by presenting data which showed it likely that a required concentration could not be met by applying available technology. Under the amendments, the best technology or its equivalent must be installed and operated before a variance can be obtained.

EPA must also evaluate treatment technology every three years to see if advances would ". . . provide for greater protection of the health of persons . . ." and must explain why it finds any such advances not feasible.

Monitoring Requirements Will Increase

Obviously, with 83 regulated contaminants, many communities will be monitoring more parameters. In addition, all community water systems and non-community, non-transient (serving same 25 people at least six months) systems must monitor and report VOC concentrations. Table 8 shows the schedule which phases in monitoring as a function of community size. Quarterly samples must be taken from each source - surface or ground. If the average of the four samples exceeds an MCL, there is a violation. It the first sample is negative for VOC's, the State may waive the requirement for the other quarterly samples.

EPA has also proposed a coliform rule which will impact sampling. The rule will be finalized in December, 1987 but the current draft indicates major changes. First, the allowable coliforms will no longer be defined in terms of density, but rather the absence or presence of coliforms - a single positive sample is a violation. The minimum number of samples per month is five. If a single sample is positive, five repeat samples must be collected in one day at the location of the positive sample. If any of the repeat samples are positive, five more repeat samples must be collected and cultured for fecal coliforms. The locations of coliform sampling must be changed on a regular basis.

Enforcement

Previously, EPA could assess fines up to $5,000 per day for willful violations. If the State did not act on such violations, EPA had the authority but was not required to initiate action against the violator. Under the amendments, the maximum fines are increased to $25,000 per day whether the violations are willful <u>or not</u>. If the State does not act within 30 days of notification by EPA, EPA <u>must</u> initiate action. Congress appears to have sent a message that they were not pleased with past enforcement and have removed EPA's latitude.

Exemptions

The compliance dates can be extended once for a period of up to 3 years if EPA finds that (1) the system cannot complete the capital improvements in time; (2) the system needs financial assistance and has made appropriate arrangements; or (3) the system has entered into an enforceable agreement to become part of a regional system. If the system serves less than 500 connections, the exemption can be reviewed for one or more additional 2-year periods.

Variances

A variance can be obtained if (1) the source characteristics prevent compliance with an MCL, (2) BAT has been applied, (3) there are no adverse health effects; and (4) a compliance schedule has been agreed to.

Lead Prohibition

The amendments forbid the use of pipe, solder, or flux that is not "lead-free" after June, 1988 in public water systems or any plumbing connected to a public water system. Lead-free is defined as 8 percent lead for pipes and fittings and 0.2 percent lead for solder and fluxes. The requirement is not retroactive! However, those systems with lead materials in their distribution systems must provide notice to "persons that may be affected by lead contamination of their drinking water". This notice <u>must</u> be provided even if lead concentrations in the water do <u>not</u> violate allowable lead concentrations. No VHA or HUD mortgages or mortgage guarantees can be given to new homes which do not meet the lead free requirement. Although the amendments do not require retroactive replacement of existing lead pipes or joints, Washington D.C. officials have announced (*ENR*, February 19, 1987) that they are developing plans to replace <u>all</u> lead pipes in the City, including the pipes in houses. The program is expected to cost "at least tens of millions of dollars". The combination of the amendments and the precedent being set by Washington D.C. may affect the plans of other cities.

<u>Protection of Groundwater Sources</u>

Monitoring methods must be identified by the EPA under the regulations for Class I injection wells within 18 months of enactment of the amendments. These new monitoring methods are to provide the earliest possible detection of fluid migration from injection wells toward underground sources of drinking water. The monitoring responsibility lies with the states that have primacy.

In addition to regulations for protection against groundwater contamination from injection wells, the amendments require establishment of wellhead protection areas by the states. Within three years from enactment of the amendments, states must adopt a program for wellhead protection. The wellhead protection area includes the surface and subsurface surrounding a well or wellfield through which contaminants are reasonably likely to move toward a well.

<u>Public Notification of Violations</u>

The form and frequency requirements for public notices have been deleted and EPA has been given 15 months to develop regulations which will provide for different types and frequencies of notice based on the frequency and severity of the violation. However, the amendments do establish some minimum provisions: (1) notice of a violation of a MCL or a serious potential health threat within 14 days; (2) notice of continuous violations every three months; (3) notice of any violation at least once per year. Notice methods must include the general circulation newspaper, and, when appropriate, electronic media and individual mailings. Failure to provide the required notice can result in a civil penalty of up to $25,000.

The Costs

Various groups have claimed the 1986 amendments should be renamed as the 1986 Full Employment Act for Engineers (or lawyers - depending on who is talking.) There is no question that a bureaucratic maze will be created by the amendments and required regulations. It will cause many utilities and municipalities to call upon their attorneys and engineers to- or at least attempt to - lead them through the maze. Trying to ignore the maze may be even more costly in the long run.

It is certain that much money will be spent to build treatment facilities dictated by the amendments. For example, even if an unfiltered supply meets the raw water quality criteria requirements to qualify for an exemption, the utility may still be faced with added costs to:

- Upgrade the disinfection system to provide redundant capacity, automatic startup, and contact times adequate to meet Giardia and virus removal requirements.

- Add continuous turbidity monitoring of raw water.

- Add monitoring of pH, temperature, and chlorine residual following disinfection.

- Conduct added bacteriological monitoring in both the raw and finished waters.

- Make annual sanitary surveys of the watershed and have a continuing watershed monitoring program.

A lot of laboratories will be very busy conducting tests needed to meet monitoring requirements.

The American Water Works Association has estimated the average cost of drinking water in the United States is $1.40/1,000 gallons - or about $14 per month at a consumption rate of about 350 gallons per day per typical residence. Smaller systems (46,000 utilities serve 1,000 users or less) experience higher costs per thousand gallons than this average value. Estimates of the potential effects of the amendments on costs vary. EPA (Civil Engineering, March, 1987) has estimated that the impacts may range from $0.25 to $7.00/1,000 gallons ($2.50 to $70 per month), depending upon the size of the system. The impacts will be most dramatic for the small utilities, some of whom may face rate payer rebellions if rate increases in the vicinity of $70/month become necessary.

There are several areas where litigation can be expected which will also likely cost utilities and their engineers grief and money. The potential for toxic tort liability exists because the MCLs for some contaminants are higher than the zero health effect concentrations - time will tell how significant this will be.

There will - and already have been - litigation over proposed standards. A group of chemical manufacturers has challenged EPA's recommendation that zero concentrations be established as MCLG's for some contaminants. Others have challenged EPA for not setting more zero limitations. How much time and money will be spent in the courtroom is anybody's guess. Battling EPA in court prompted one industry official to note (*Chemical Week*, February 4, 1987) that "It's a lot easier to move to another country altogether. . ." Assuming that most readers will find this to be an impractical alternative, the following immediate steps may be prudent:

- Assess monitoring and data gathering programs - although the rules are not yet final, many of the data needs can be accurately projected now. If one anticipates seeking a variance or exemption, now is the time to begin.

- Educate policy makers and the public - there may be some potentially significant rate increases for many utilities. Beginning now to explain the changes in the Federal law and the projected costs may lessen the shock later and may generate some added input to the development of regulations.

- Identify system improvements - the potential cost of system improvements is an important element in the education process and in fiscal planning. By identifying the potential range of system improvements, alternative courses of action can be evaluated and long range planning begun.

Whether the costs of the programs dictated by Congress are justified by the benefits can be debated. What cannot is the fact that the drinking water industry will never be the same. Planning to cope with this reality now may reduce costs and minimize the eventual impacts on the public.

TABLE 1

MCL'S FOR VOC'S REGULATED IN JUNE, 1987

Compound	MCL (ppb)
TCE	5
Carbon tetrachloride	5
Vinyl chloride	2
1,2-Dichloroethane	5
Benzene	5
Para-dichlorobenzene	75
1-1, Dichloroethylene	7
1,1,1-Trichloroethane	200

TABLE 2

FLUORIDE MCL

RMCL = 4 mg/L

Secondary MCL = 2 mg/L

Quarterly notices required if violate 2 mg/L

TABLE 3

CONTAMINANTS FOR WHICH STANDARDS MUST BE ESTABLISHED BY EPA

Volatile Organic Chemicals

Trichloroethylene	Benzene
Tetrachloroethylene	Chlorobenzene
Carbon tetrachloride	Dichlorobenzene(s)
1,1,1-Trichloroethane	Trichlorobenzene(s)
1,2-Dichloroethane	trans-1,2-Dichloroethylene
Vinyl chloride	1,1-Dichloroethylene
Methylene chloride	cis-1,2-Dichloroethylene

Microbiology and Turbidity

Total coliforms	Viruses
Turbidity	Standard plate count
Giardia lamblia	Legionella

Inorganics

Arsenic	Molybdenum*
Barium	Asbestos
Cadmium	Sulfate
Chromium	Copper
Lead	Vanadium*
Mercury	Sodium*
Nitrate	Nickel
Selenium	Zinc*
Silver*	Thallium
Fluoride	Beryllium
Aluminum*	Cyanide
Antimony	

Organics

Endrin	1,1,2-Trichloroethane
Lindane	Vydate
Methoxychlor	Simazine
Toxaphene	PAH's
2,4-D	PCB's
2,4,5-TP	Atrazine
Aldicarb	Phthalates
Chlordane	Acrylamide
Dalapon	Dibromochloropropane (DBCP)
Diquat	1,2-Dichloropropane

TABLE 3 (Continued)

Endothall	Pentachlorophenol
Glyphosate	Picloram
Carbofuran	Dinoseb
Alachlor	Ethylene dibromide
Epichlorohydrin	Dibromomethane*
Toluene	Xylene
Adipates	Hexachlorocyclopentadiene
2,3,7,8-TCDD (Dioxin)	

Radionuclides

Radium 226 and 228	Gross alpha particle activity
Beta particle and photon radioactivity	Radon
Uranium	

* Substitute contaminants adopted by EPA in June, 1987 - see Table 4.

TABLE 4

EPA SUBSTITUTIONS - JUNE, 1987

Out	In
Zinc	Aldicarb Sulfoxide
Silver	Aldicarb Sulfone
Sodium	Ethylbenzene
Aluminum	Heptachlor
Molybdenum	Heptachlor epoxide
Vanadium	Styrene
Dibromomethane	Nitrite

TABLE 5

1986 NATIONAL PRIMARY DRINKING WATER REGULATIONS
U.S. ENVIRONMENTAL PROTECTION AGENCY

Constituent	Maximum Contaminant Level
Arsenic	0.05 mg/L
Barium	1 mg/L
Cadmium	0.010 mg/L
Chromium	0.05 mg/L
Fluoride	Varies with temperature
Lead	0.05 mg/L
Mercury	0.002 mg/L
Nitrate as N	10 mg/L
Selenium	0.01 mg/L
Silver	0.05 mg/L
Sodium	Analyze 1 sample per year per plant at entry to distribution system for surface waters and once every 3 years for groundwater systems
Radium^{226} and ^{228}	5 pCi/L
Gross alpha activity (Including radium^{226} but excluding radon and uranium)	15 pCi/L
Beta and photon radioactivity (Detailed studies must be made if the gross beta activity exceeds 50 pCi/L)	4 mrem/yr
Corrosivity	Measure pH, calcium hardness, alkalinity, temperature, total dissolved solids, and calculate the Langlier Index on a mid-winter and mid-summer sample each year for surface sources and 1 per year for groundwater systems
Total Coliforms	a) 1 per 100 mL, average of all samples in a month measured by the membrane filter technique b) 4 per 100 mL in more than 5 percent of the samples measured by the membrane filter technique
Turbidity	a) 1 ntu*, average of all samples in a month as measured at entry points to the distribution system b) 5 ntu, average of two consecutive days as measured at entry points to the distribution system
Endrin	0.0002 mg/L
Lindane	0.004 mg/L
Methoxychlor	0.1 mg/L
Toxaphene	0.005 mg/L
2,4-D	0.1 mg/L
2,4,5-TP (Silvex)	0.01 mg/L
Total Trihalomethanes	0.1 mg/L, 12 month running average of quarterly samples, each quarterly sample consisting of at least four individual samples

* May be 5 ntu under certain conditions

TABLE 6

PROPOSED RMCLs (MCLGs) FOR ORGANIC COMPOUNDS

Constituent	Proposed RMCL (MCLG) mg/L
Acrylamide	Zero
Alachlor	Zero
Aldicarb, aldicarb sulfoxide, aldicarb sulfone	0.009
Carbofuran	0.036
Chlordane	Zero
cis-1,2-Dichloroethylene	0.07
Dibromochloropropane (DBCP)	Zero
1,2-Dichloropropane	0.006
o-Dichlorobenzene	0.62
2,4-D	0.07
Ethylene Dibromide (EDB)	Zero
Epichlorohydrin	Zero
Ethylbenzene	0.68
Heptachlor	Zero
Heptachlor epoxide	Zero
Lindane	0.0002
Methoxychlor	0.34
Monochlorobenzene	0.06
Polychlorinated Biphenals (PCBs)	Zero
Pentachlorophenol	0.22
Styrene	0.14
Toluene	2.0
2,4,5-TP	0.052
Toxaphene	Zero
trans-1,2-Dichloroethylene	0.07
Xylene	0.44

TABLE 7

PROPOSED RMCLs (MCLGs) FOR INORGANIC COMPOUNDS

Constituent	Proposed RMCL (MCLG) mg/L
Arsenic	0.050
Barium	1.5
Cadmium	0.005
Chromium	0.12
Copper	1.3
Lead	0.020
Mercury	0.003
Nitrate-N	10.0
Nitrite-N	1.0
Selenium	0.045

TABLE 8

VOC MONITORING SCHEDULE

Population	Monitoring Deadline
>10,000	December 31, 1988
3,300 - 10,000	December 31, 1989
<3,300	December 31, 1991