CDM

DAVID R. WARREN

Camp Dresser & McKee

CITY OF WICHITA, KANSAS

WATER TREATMENT PLANT EVALUATION

STATUS REPORT

PLANT EXPANSION ALTERNATIVE EVALUATION

JANUARY 1991

CDM
environmental engineers, scientists, planners, & management consultants

CAMP DRESSER & McKEE INC.

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May 28, 1991

Mr. David Warren
Director
Wichita Water & Sewer Department
455 N. Main Street
Wichita, KS 67202

Re: Wichita WTP Expansion
Intermediate Filter Improvements

Dear Mr. Warren:

Camp Dresser & McKee Inc. (CDM) is in the process of completing the design report for the Wichita Water Treatment Plant (WTP) Expansion. The report is basically complete, except for the section relating to filter improvements. A one-month pilot plant study will be conducted in June to evaluate filtration rates and media. We feel this study will prove that the existing fourteen (14) filters can be upgraded from their existing capacity of 120 mgd (approximately 4.1 gpm/sf) to 160 mgd (approximately 5.5 gpm/sf). The study is necessary in order to receive KDH&E approval for higher filtration rates. If the existing filters can be upgraded to the higher capacity, a construction cost savings of about $6 million will be realized due to the elimination of the need for six new filters and associated building and appurtenances.

If the project were to proceed in a normal design and construction sequence, the following schedule for expansion of the Wichita WTP from 120 mgd to 160 mgd is anticipated.

Item	Duration	Estimated Completion Date
Pilot Plant Study	1 month	June 30, 1991
Finalize Design Report	1 month	July 31, 1991
Final Design	11 months	June 30, 1992
Advertise/Bid/Award	3 months	September 30, 1992
Construction	24 months	September 30, 1994
	(21 months)*	(June 30, 1994)*

* Compressed Construction Schedule

If the Construction contract was divided into two phases, with the improvements to the fourteen (14) filters being performed first, the plant's capacity could be increased to approximately 140 mgd after Phase 1. Although the filters would have a capacity of 160 mgd, the hydraulics through the remainder of the plant would restrict the flow to the filters. A compressed schedule for design and construction would allow completion of the Phase I filter improvements by March 31, 1993.

In the summer of 1990, the City's WTP operated at its full capacity of 120 mgd for extended hot periods, and this was with some water use restrictions in place. A new 10 million gallons clearwell is now nearing completion which hopefully will help meet the August and September demands of 1991; however, no additional WTP production capacity is available. Looking forward, the schedule to provide increased WTP capacity by the spring of 1993 may not be soon enough to meet the City's growing demand.

It would be possible to increase the filtration capacity by approximately 15 mgd by the summer of 1992 by modifying the original six existing filters and expediting the design and construction schedule as follows.

1. Begin design of the filter modifications in June 1991. A conceptual design of the filter modifications has been completed pending results of pilot plant and KDH&E approval. The actual media depth, size, and type would be specified upon completion of pilot plant study.

2. If the pilot plant results do not show that the filter capacity can be increased or if KDH&E does not give approval for the increased rates, the design effort will not be wasted. The filters, because of their age and condition, will be modified to include new underdrains, media, controls, etc., whether the existing filters are designed for higher capacity or if additional new filters are required. Overall plant capacity could then be increased from 120 to 135± mgd.

3. A compressed design schedule of 4 months would allow for design documents to be completed, reviewed by KDH&E, and ready for bidding by September or early October, 1991.

4. Normally, a bidding/award period of three months is desirable. However, the project could be advertised for 3 weeks and contract awarded (with notice to proceed) within 3 weeks depending upon the City requirements. Therefore construction could begin by November 1, 1991.

5. A compressed construction schedule of six months will allow completion of filter modifications by May 1, 1992. Construction during March and April will reduce the existing plant capacity. This must be considered during construction staging to assure sufficient water supply during those months.

In addition to the filter modifications, minor hydraulic improvements will be required to assure that the increased flow can be delivered to the treatment plant. It would be prudent to proceed with intermediate filter improvements to the six older filters to assure increased WTP capacity for the City by the summer of 1992.

CDM will submit copies of the draft report to you and your staff by June 6, 1991 for your review. It will not include the filter pilot plant results. Please contact me if you have any questions or need any additional information.

Sincerely,

CAMP DRESSER & McKEE INC.

Dan Shannon

J. Dan Shannon, P.E.

cc: Mike Withrow
Carl Houck
Ashok Varma

file: CO-1.0

CITY OF WICHITA, KANSAS

WATER TREATMENT PLANT EVALUATION

STATUS REPORT: PLANT EXPANSION ALTERNATIVE EVALUATION

Prepared For:

WICHITA WATER AND SEWER DEPARTMENT
City Hall-Eighth Floor
455 North Main Street
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Prepared By:

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January 1991

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1.0 INTRODUCTION

The purpose of this status report is to present the work completed as part of Task 4 of the study -- the evaluation of alternatives for expansion and upgrade of the existing Wichita Water Treatment Plant to a capacity of 160 mgd. The primary objectives of this task are:

- Establish treatment goals.
- Evaluate process alternatives to meet the established goals.
- Conduct a tracer study through the existing Central Plant.
- Conduct bench scale studies to evaluate treatment.
- Develop a hydraulic model to evaluate improvements necessary to increase the hydraulic capacity of both plants.
- Develop and evaluate feasible expansion alternatives to meet the treatment goals.

This report is not intended to be all inclusive. It is intended to be a vehicle for review and discussion between the City and the Project Team. Following discussion and reviews, the report will be edited and incorporated as part of the study's final report. The recommended alternative presented in Section 4.0, with consensus from the City, will be developed in the Final Report in greater detail. A cost estimate will be presented in the final report which encompasses all improvements for the project. An implementation schedule for design and construction will be developed at that time.

2.0 PROCESS ALTERNATIVE DISCUSSIONS

2.1 GENERAL

The Status Report: Data Base discussed raw water quality as well as drinking water standards which must be met by the Wichita Water Treatment Plant (WTP) in future years. Some of the major treatment goals which must be met include:

- Turbidity reduction - the SDWA ammendments have established 0.5 NTU as the criteria for filtered water. The goal for the expanded plant facilities will be to achieve a filtered water turbidity of 0.3 NTU or less.

- Hardness - the raw water hardness averages about 180 mg/l for the Cheney Reservoir supply and about 235 mg/l for the Equus Bed wellfield supply. Hardness levels have reached considerably higher levels from both supplies. A finished water goal of 100 - 105 mg/l hardness will be targeted.

- Disinfection - the new SDWA ammendments will require primary disinfection to meet specific CT values dependent upon the disinfectant used and the temperature and pH of the raw water.

- Disinfectant By-products (DBPs) - the DBPs of major concern for the Wichita Water Treatment Plant will be total trihalomethanes (THM) and total organic halogens (TOX). The latest information indicates that a maximum THM level of 50 ug/l will be set by EPA, and thus we will use this as our goal. A goal of 100 ug/l for TOX will be set, although no current regulations have been established.

- Lead - the proposed lead and copper rule set an MCL of 0.005 mg/l for lead. The final rule may be centered around a treatment technique instead of an MCL. The treatment technique will likely include a target lead level of 0.015 mg/l at the tap in 90% of the samples. Therefore, a goal of 0.01 mg/l lead will be used. The raw water has an average lead concentration of 0.007 mg/l.

The existing lime softening treatment process scheme of aeration, rapid mixing, flocculation, sedimentation, recarbonation, filtration, and disinfection will be utilized for the expanded plant. The following subsections will discuss alternatives for the various unit processes. The processes recommended in this Section will be incorporated into a series of treatment plant expansion alternatives to be developed and evaluated in Section 3.0.

2.2 AERATION

2.2.1 GENERAL

Aeration, in water treatment, is the process of bringing water and air into close contact in order to: 1) remove or reduce objectionable dissolved gases such as carbon dioxide, hydrogen sulfide, and methane, and 2) oxidize dissolved metals such as iron and manganese. The use of aeration can also be beneficial in removing substances that interfere with or add to the cost of subsequent water treatment. Examples of this would be the removal of hydrogen sulfide prior to chlorination and the reduction of carbon dioxide before lime softening.

2.2.2 EXISTING AERATORS

The existing aerators at both the Central plant and East plant are the multiple-tray type, consisting of four trays each, with coke media placed on each of the trays. Raw water flows to the top of each aerator and passes through a distribution plate, which provides relatively equal distribution of water across the plan area of each aerator. The water then flows, by gravity, across each of the four coke trays and is collected in a flume below the aerator building. The aerators at Central plant have a surface loading rate of 6.5 gpm/sf for a plant flow of 80 mgd. The East plant aerators have a surface loading rate of 6.2 gpm/sf for a plant flow of 30 mgd.

Aeration is utilized, primarily, for the reduction of carbon dioxide (CO_2) and the oxidation of both iron and manganese. A history of surface water, well water, and blended raw water CO_2 contents is not available; however, blended raw water CO_2 contents measured during the jar testing ranged from 10 to 20 mg/l with an average of about 15 mg/l. Generally speaking, when the raw water CO_2 content is 10 mg/l or above, it is recommended to aerate the water prior to lime softening to prevent excess lime consumption and consequentially higher chemical costs.

A summary of average raw water quality data provided in Status Report - Data Base reveals that the surface water, groundwater, and blended raw water iron contents are significantly higher than the water quality standard of 0.3 mg/l. The manganese content in both the groundwater and blended raw water is also significantly above the water quality standard of 0.05 mg/l. Aeration of the raw water is required to oxidize both iron and manganese so that insoluble forms of each will form and later be removed by subsequent processes (i.e., sedimentation and filtration).

The aerators at Central plant currently experience clogging of the orifice holes in the distribution plate due to Asian clam shells that originate from Cheney Reservoir. This clogging both increases headloss through the aerators and reduces the efficiency of the aeration process. The aerators at East plant experience considerable flooding due to high headloss. For this reason, no more than 30 mgd is typically treated by the East plant aerators.

2.2.3 TYPES OF AERATORS

Aerators are basically classified into two categories: the water-into-air type and the air-into-water type. The water-into-air type include spray aerators, multiple-tray aerators, and cascade aerators. The air-into-water type include the diffuser aerator and the draft-tube aerator.

2.2.4 RECOMMENDATION

Because the existing aerators adequately reduce CO_2 contents and oxidize iron and manganese and the aerators at both Central plant and East plant are in good condition, it is recommended that the multiple-tray aerators continue to be utilized. Modifications to or additional aerators will be required in the overall plant upgrade. Because of this, several aerator alternatives are presented in Section 3.0 along with their respective costs.

2.3 RAPID MIXING

2.3.1 GENERAL

Rapid mixing for lime softening plants serves many important purposes, including dissolving the relatively insoluble calcium hydroxide; dispersing chemicals used for coagulation into the raw water; and mixing of recycled sludge with both the raw water and chemical (i.e., calcium hydroxide and coagulant) feed.

Coagulation in water treatment occurs predominantly by two mechanisms: 1) adsorption of the soluble hydrolysis species on the colloid and destabilization, or 2) sweep coagulation where the colloid is entrapped within the precipitate. For lime softening, the latter form of coagulation takes place, with the colloids becoming entrapped in the precipitating calcium carbonate.

Recycling of previously formed calcium carbonate crystals, that primarily make-up the sludge, is an important element with regard to the speed and efficiency of the softening process since sweep coagulation is predominant. The recycled crystals serve as nuclei, or "seed", for the precipitation of newly formed calcium carbonate. This reaction also furthers the growth of larger calcium carbonate particles which will settle more readily, thicken to a greater extent, and dewater more easily and with a lower final moisture content.

Traditionally, rapid mix basins for lime softening plants have been designed to provide a detention time anywhere from 30 seconds to 5 minutes and velocity gradients (G) from 300 to 1,000 sec^{-1}.

2.3.2 EXISTING RAPID MIX BASINS

The existing rapid mixers at the Central plant are the traditional "rotating impeller in mix chamber type". This plant has one rapid mix basin with four mixers in series. The entire rapid mix chamber has a theoretical detention time of about 12 seconds at a plant flow of 80 mgd.

Slaked lime is fed by gravity about 10 feet upstream of the first mixer and/or at the second mixer. Cationic polymer is fed about 125 feet upstream of the rapid mix basin (immediately outside the aerator building) and one to two miles ahead of the plant in the 66-inch raw water pipeline.

The rapid mixers at Central plant experience overloading due to lime build-up. Presently, the temperature of the gear box grease in each mixer is monitored and when it exceeds 300 °F, the mixer is turned off so that it can cool down. A portable fan is also located near the rapid mixers to continuously blow air across them.

The theoretical velocity gradient (G) for the rapid mixers at Central plant was calculated as being about 190 sec^{-1}. This value is considerably below the range of "G" values, previously mentioned, for rapid mixing. This lack of enough input mixing energy is more than likely causing the overloading/overheating condition on the rapid mixers.

Results from the jar tests, presented in the Discussion Paper: Treatment Plant Studies, indicate that at least 30 seconds mixing time is required - longer detention times did not significantly improve settled water quality. The maximum "G" value, however, capable of simulation during these tests was only 90 sec^{-1}. As discussed in the previous section, lime coagulation resembles sweep floc coagulation. During sweep floc coagulation, mixing time is not as critical, just so long as enough mixing energy is applied to adequately dissolve the calcium hydroxide. From the jar tests, the duration of the flocculation process proved more critical than the duration of rapid mixing with regard to both turbidity and hardness removal. For these reasons, it is believed that the detention times of the rapid mix basin could be shortened to below 30 seconds so long as adequate mixing energy is applied.

The most significant disadvantage with the existing Central plant rapid mix basin is its lack of operational flexibility. Because there is only one basin, when it is taken out of service for maintenance purposes, all water must be diverted to the flocculation basins, thereby foregoing any lime addition.

The East plant has one rapid mix basin with a single paddle wheel type mixer, consisting of seven reels, running the length of the basin. This arrangement creates an axial flow pattern. The theoretical detention time through the rapid mix basin is 64 seconds for a plant flow of 30 mgd. Slaked lime is fed by gravity at the entrance of the basin, and cationic polymer is fed just prior to the lime application point.

When the East plant is operational, the existing rapid mixer does not experience any major maintenance problems. Although there is only one rapid mix basin and mixer, the lack of operational flexibility for this plant is not as critical as the Central plant since the East plant is normally brought on-line only when one of Central plant's process trains is taken out of service for cleaning and maintenance.

Based on the desire to improve the rapid mixing process to incorporate a mixing arrangement which would provide short-duration, high energy dispersion, it is recommended that various mixing arrangements be evaluated which would be applicable in achieving this design criteria for the Wichita WTP. These various alternatives are discussed below.

2.3.3 RAPID MIXER TYPES

There are a number of rapid mixer types which may be appropriate for use at Central plant and East plant. These are listed below with a discussion of their respective advantages/disadvantages.

<u>Injection of Chemical into a Pipe with High-Velocity Flow</u>

In situations where the raw water pipe carries high-velocity flow, it is possible to simply inject the chemical(s) into the pipeline and allow the turbulent flow to perform the mixing. The main advantage of this type of mixing is the elimination of mechanical equipment required other than the chemical feed equipment. The disadvantages of this type of mixer are: 1) possible clogging of injection nozzles - especially when feeding slaked lime; 2) the intensity of mixing varies with the flow rate; and 3) poor control over the mixing process. Because the Wichita WTP has varying flow rates due to seasonal demand and both plants feed slaked lime, this type of mixing arrangement is not appropriate.

Static In-Line Mixer

These mixers consist of a series of helical vanes installed in the raw water pipe in such a way that the incoming water and chemicals are mixed by a combination of the turbulence generated by the vanes and the way the vanes repeatedly divide and join the flow. Mixing occurs radially, not longitudinally (i.e., plug flow). Fluids pass through the mixer in the exact proportions in which they enter; there is no backmixing. The advantages of a static mixer include: 1) no mechanical parts, 2) short mixing time, and 3) a potential savings in the amount of chemicals required. The disadvantages are: 1) it has significant head loss, 2) the chemicals are injected into a closed pipe limiting access to the nozzles for inspection and maintenance, and 3) the intensity of the mixing varies with the flow rate. Due to the fact that both Central plant and East plant feed slaked lime, which could possibly clog the injection nozzles, and flows to each plant are variable, a static in-line mixer is not recommended.

Pump Injection Diffusion

Similar to chemical injection into a pipe discussed previously, pump injection involves the rapid dispersion of chemicals into the raw water flow. In this case, a pump takes water from the raw water flow and pumps this water through an injection nozzle directly against the flow of the balance of the raw water confined in the inlet pipe or channel. The coagulant chemical can either be injected into the pumped flow and dispersed through the nozzle or added immediately in front of the nozzle and dispersed in a similar fashion. Figure 2-1 shows the general arrangement of pump injection diffusion. Because the mixing energy is a function of the velocity head through the blending nozzle rather than the velocity in the raw water pipe or channel, pump injection provides a constant mixing energy over a wide range of flows. The advantages of pump

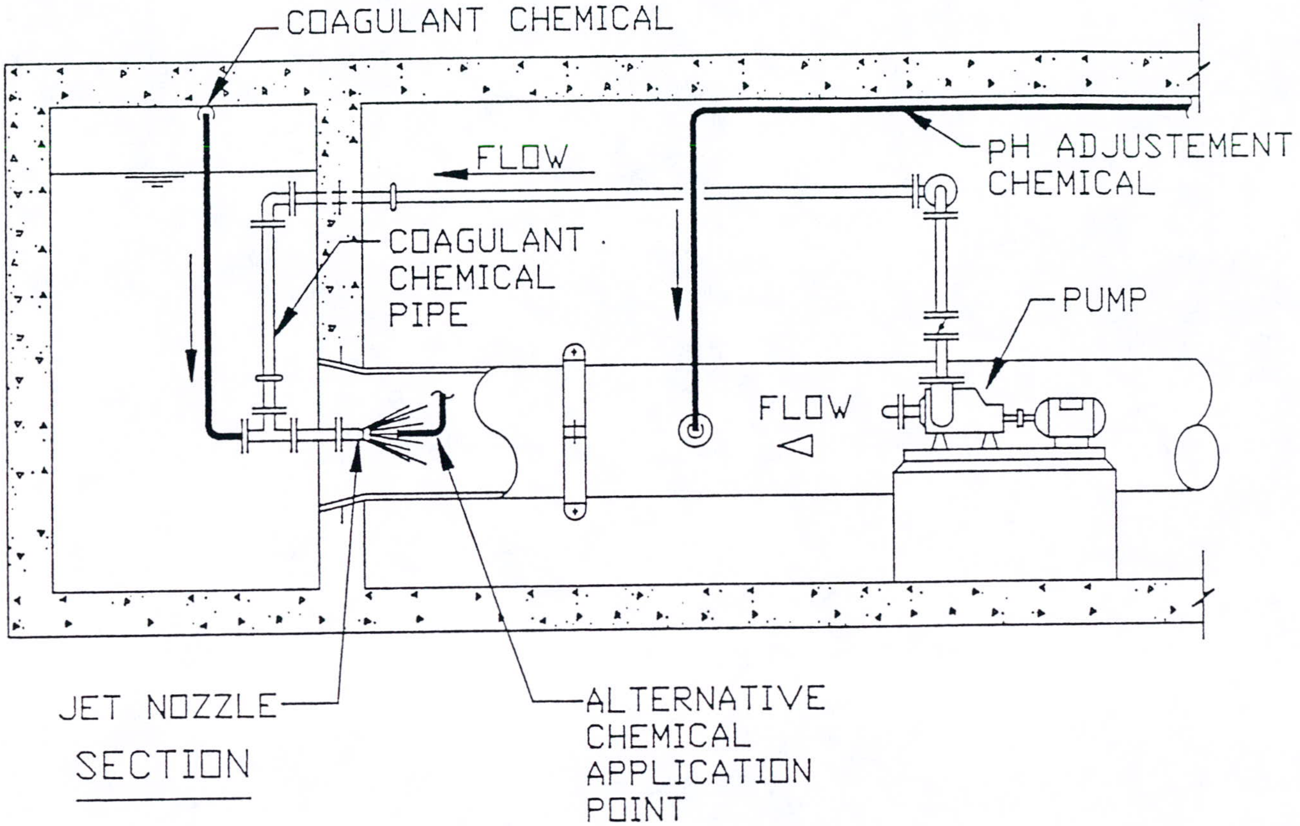


FIGURE 2-1

PUMP INJECTION RAPID MIXER

injection are that it provides high-energy, rapid dispersion of chemical into the flow. If the nozzle is located in the raw water channel, it can be easily removed for inspection and cleaning. The disadvantage of pump injection is the clogging of the injection nozzle due to scaling. If the chemical is added in front of the injection nozzle, as mentioned above, the nozzle might still clog due to scale build-up, especially if slaked lime is fed. Because slaked lime will continue to be fed at both Central and East plants, a nozzle scaling problem is almost assured. For this reason, pump injection diffusion is not recommended for the Wichita WTP.

Rotating Impeller in Mix Chamber - Vertical Shaft

This is the most common type of mixer used in the water treatment industry and is what currently exists at the Central plant. It has little head loss and is suitable for widely varying flow conditions. The impeller may be a paddle type, turbine type, or propeller type. As described earlier, the mixer design is typically based on a "G" value of 300 to 1,000 sec^{-1}. The rotating impeller type rapid mixers used in the water treatment industry today normally have no underwater bearings and the motor is mounted above the mix chamber providing easy access for maintenance. The potential disadvantage of this type of mixer is that backmixing occurs.

When the predominant mechanism of coagulation is sweep coagulation, as is the case here, backmixing is not a disadvantage because it actually enhances the enmeshment of colloidal particles by increasing the theoretical detention time in the basin. It is considered that this type of mixer is a viable option for use at Central plant.

In-Line Mechanical Mixer

For this type of mixing arrangement, two or more impellers are located inside a section of pipe. The mixer motors are mounted outside the pipe and the connecting shafts pass through stuffing box seals in the pipe wall. The chemicals are fed into the flow upstream of the impellers. The impellers are arranged in pairs with opposed axial flow. This type of mixer provides rapid dispersion, high energy mixing that is independent of flow rate, and low headloss. The disadvantages of the in-line mechanical mixer are the pipeline must be taken out of service for access to the impellars and injection tubes or to replace the stuffing box seals. Also, due to the fact that slaked lime is being fed, the probability of the chemical injection lines scaling or clogging is most assured. For these reasons, the in-line mechanical mixer is not recommended.

Side-Entering Mixer

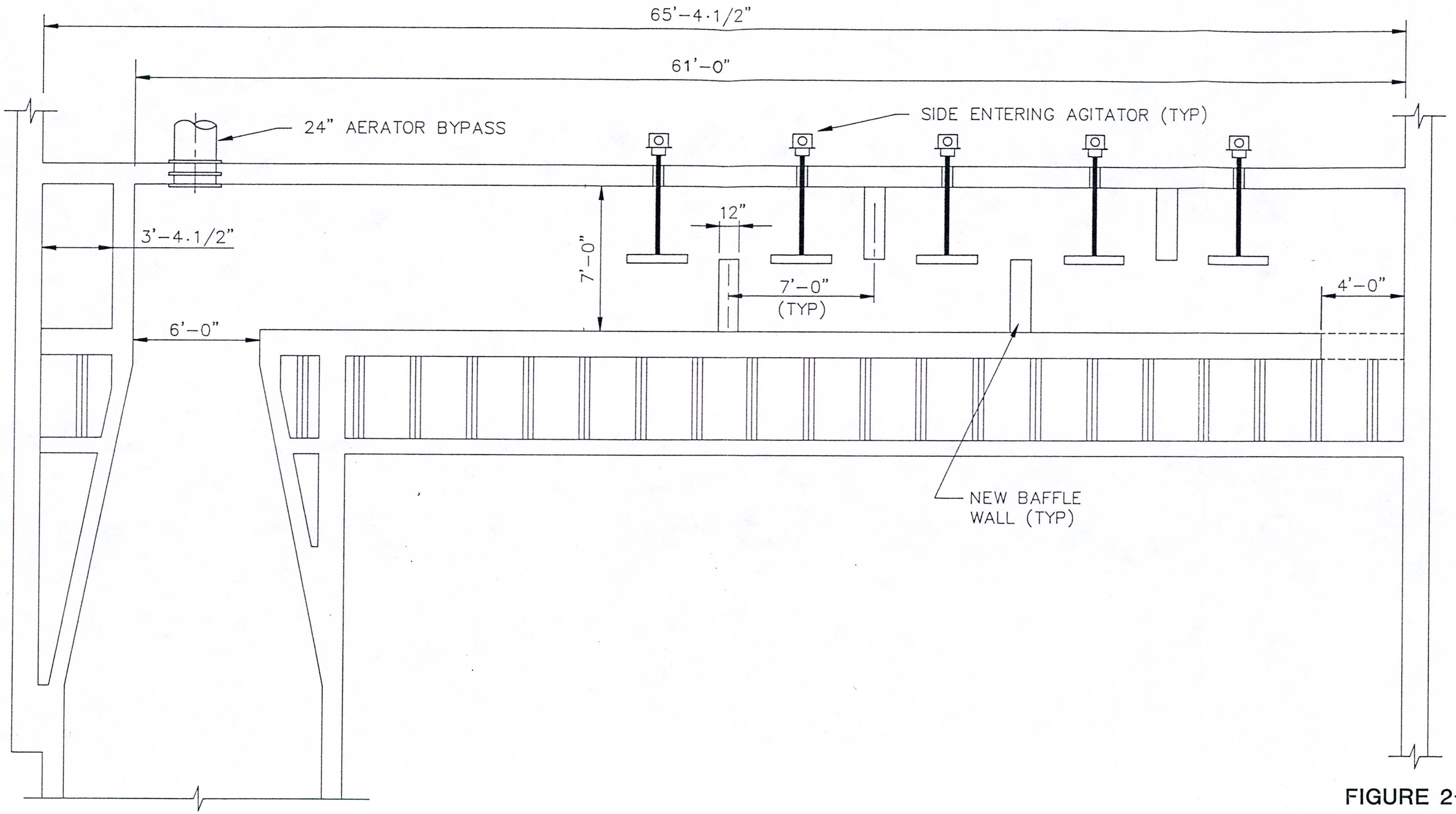
A variation on the in-line mechanical mixer is the side-entering mixer. Instead of the mixer being located outside a section of pipe, it is located outside the rapid mix basin wall, if room permits. Figure 2-2 shows a typical arrangement for this type of mixer as it would apply to the East plant. The side-entering mixer provides rapid dispersion, high energy mixing that is independent of flow rate, and low headloss. The disadvantage of this type of mixer is that it is not generally recommended for municipal applications as stated, and therefore its performance is unproven. For this reason, the side-entering mixer is not recommended.

Paddle Wheel Mixer

This type of mixer is very similar to the paddle wheel type mixer commonly used for flocculation, however, it typically has more blades and operates at a higher tip speed in order to achieve higher "G" values. The paddle wheel mixer is considered "older technology". Its advantages are that it has low headloss and is suitable for a wide variety of flow conditions. This type of mixer is presently being used at the East plant and due to the rapid mix basin configuration, it is considered that this type of mixer is a viable option for use at East plant (should the future use of this plant be recommended).

2.3.4 RECOMMENDATION

From the above discussion, two types of rapid mixers are suitable for use at the Wichita WTP. For Central plant the rotating impellar type is recommended, and for East plant the paddle wheel type mixer is recommended.



EAST PLANT RAPID MIX BASIN

SCALE: 3/16" = 1'-0"

FIGURE 2-2

SIDE ENTERING MIXER

"FLRPLN2"

"FLRPLT2"

Various alternatives for each of these recommendations are presented in Section 3.0 along with their respective costs.

2.4 FLOCCULATION

2.4.1 GENERAL

Adding coagulant to the raw water in a rapid mixer destabilizes particulate matter, which results in the small colloidal particles forming into larger coagulated particles called floc. Flocculation is the process of slowly agitating the water and coagulated particles to encourage collision and therefore agglomeration of the particles into floc particles heavy enough to settle by gravity or large enough to be removed during filtration.

Flocculation basins for lime softening plants are generally designed to allow sufficient time for completion of the softening reaction while providing enough energy input to keep the solids suspended and in contact. The degree of agitation provided in the flocculation basin is usually measured in terms of the velocity gradient G. Typical values of "G" vary from 10 to 80 sec^{-1}. Flocculation basins are normally sized for a detention time of 20 to 30 minutes at the rated plant flow.

Best results are usually obtained using tapered flocculation. Baffle walls are used to divide the flocculation basin into three or more chambers in series. The flocculator mechanism in each chamber provides a lower energy input than in the preceding chamber. As the water flows through the flocculation chambers, it is subjected to decreasing agitation while the floc size increases. In this way, initial particulate collisions and agglomerations are encouraged in the early stages, while low-shear mixing forms larger, heavier floc in the latter stages.

Variable speed flocculation is desirable to respond to seasonal mixing requirements and to achieve optimum "G" values for development of particles which will settle rapidly. The baffle walls between chambers separate the flocculation basins into defined zones of tapered energy input and reduce short-circuiting between the chambers. The baffle walls are generally sized with an open area of about five percent or a velocity of 1 fps through the open area at maximum flow conditions.

2.4.2 EXISTING FLOCCULATION BASINS

The existing flocculators at both Central and East plants are the horizontal paddle wheel type. The Central plant flocculation basins consist of three rows, all operating at the same tip speed. No baffle walls are present between the rows. The theoretical detention time for the Central plant basins is 32 minutes at 80 mgd. The East plant flocculation basins consist of five rows of flocculators. As with Central plant, all East plant flocculators are operated at the same tip speed and no baffle walls exist between the rows of flocculators. The East plant flocculation basins have a detension time of 30 minutes at 30 mgd.

The flocculation basins for both plants experience considerable floc settling within the basins, primarily due to low input "G" values. The Central plant flocculation basin also experiences considerable backmixing due to the size of the slots in the outlet baffle wall. As was stated above, baffle walls are typically designed for approximately a five percent open area. The outlet baffle wall at Central plant has 32 percent open area.

2.4.3 FLOCCULATOR TYPES

Horizontal Axle Agitator ("Paddle Wheel" Type)

This has been the most common type of flocculator used in the water treatment industry and is what is currently used at the Wichita WTP. The paddle wheel flocculator consists of a horizontal axle with protruding arms on which are mounted wood, metal, or plastic blades. The axle slowly rotates (approximately 60 to 100 revolutions per hour) imparting gentle agitation to the water. The degree of agitation can be varied either by varying the speed of rotation or by changing the size and number of blades. The axle may be mounted transverse to or parallel with the direction of flow. One disadvantage with this type of flocculator is that it has underwater bearings. The horizontal axle agitator, however, is a tried and proven method of agitation for flocculation and is recommended as a viable alternative for the Wichita WTP.

Reciprocating Agitator ("Walking Beam" and "Flocsillator" Types)

In the walking beam type of flocculator, a crank arm and connecting rods convert the rotary motion of the motor unit into a reciprocating motion of the driveshaft. A number of walking beams are attached along the length of the driveshaft and from the ends of these beams hang the paddles. As the driveshaft rotates backwards and forwards in a reciprocating motion, the paddles move up and down, causing agitation in the water. There are no bearings, sprockets, drive chains, or other moving parts under the water, simplifying maintenance. However, the driveshafts and walking beams take up significant space above the flocculation basins. This is both unsightly and restricts physical access to the basins. In the "Flocsillator" type of flocculators, instead of walking beams and hanging paddles, rigid frames with paddles are attached directly to the driveshaft. These frames oscillate backwards and forwards in the water as the driveshaft rotates in a reciprocating motion. The reciprocating agitator arrangement has been used successfully in many plants.

The aeration facilities at both East plant and Central plant are located above the flocculation basins, thus eliminating the space requirements for the reciprocating agitators. Therefore, this is not a feasible alternative and is not recommended.

Vertical Shaft Turbine Agitator

In recent times, these have become a common type of agitator for flocculation in the water treatment industry. The impeller may have paddles, straight blades, or pitched blades. The most effective type of impeller is usually a hydrofoil (or "High-efficiency") type with low pitch blades, as these provide a moderate amount of mixing action with low shear. Vertical shaft turbine flocculators normally have no underwater bearings. The motor is mounted above the flocculation basin providing easy access for maintenance and it takes up much less space than the walking beam type flocculator. The motor is often equipped with a speed control so that the "G" value can be easily varied.

Again, because the aeration facilities are located above the flocculation basins, the vertical shaft turbine agitator is not a viable option.

2.4.4 RECOMMENDATION

Due to the physical constraints of the existing flocculation basins, the paddle wheel type mixer is the only feasible alternative for use at Central plant and East plant. Various modifications to and/or additional flocculators are recommended for the different alternatives presented in Section 3.0.

2.5 SEDIMENTATION

2.5.1 GENERAL

Sedimentation (or clarification) is the process of removing floc particles from suspension by keeping the water in a relatively quiescent state for a sufficient time so that a desired fraction of the particles settle out by gravity to the bottom of the basin.

The most important parameter in the design of sedimentation basins is the surface loading rate (sometimes called the surface upflow rate). The surface loading is defined as the flow rate divided by the basin surface area. As important as it is, the surface loading rate is not widely understood. It may be conceptualized as a "design particle settling velocity." The surface loading rate is usually quoted in units of gallons per day per square foot of basin surface area (gpd/sf). The surface loading rate has dimensions of "volume per time per area", or simplifying, "length per time". Thus, it would be equally valid to quote surface loading rates in dimensions of "length per time", for example, cm/min. A surface loading rate of 1,000 gpd/sf is the same as 2.8 cm/min. That is, a sedimentation basin designed for a surface loading rate of 1,000 gpd/sf at the design flow can be thought of from a conceptual viewpoint as being just capable of settling out a particle with a settling velocity of 2.8 cm/min. (A typical heavy lime floc would have a settling velocity of about 9 cm/min.) Note that the "design particle settling velocity for a horizontal sedimentation basin is related to the basin surface area and flow only; the basins settling performance is measured succinctly by the surface loading rate without any reference to basin length, depth, detention time, or horizontal velocity.

The Kansas Department of Health and Environmental (KDHE) "Policies Governing the Design of Public Water Supply Systems in Kansas" specifies a maximum surface loading rate of 600 gpd/sf for horizontal flow sedimentation basins.

The State Policies also specify a minimum detention time of 3 hours. As indicated above, the detention time and water depth are not as important to settling performance as the surface loading rate. This point can be illustrated by considering the path of a discrete particle as it settles in a sedimentation basin. The particle falls at a constant velocity, depending on the size and weight of the particle. At the same time, the water is moving through the sedimentation basin with constant horizontal velocity. The net result of these two velocity components is that the particle moves in a straight line at a downward slope.

For a given basin size and flow, a particle entering the sedimentation basin at a point near the water surface will travel horizontally and vertically in these velocity components until it reaches the floor, at some point along the length (assuming the basin is long enough for the particle to settle out). If the depth of the basin is increased (which increases the detention time by the same proportion), then in theory, sedimentation will be unaffected because the particle will settle to the same point on the floor. Even though the particle would have a greater depth to settle, the horizontal velocity would be less so there is more time for settling to occur. The combined effect of the greater depth of fall and the slower horizontal velocity is that the particle will settle to exactly the same location on the floor regardless of the depth. Because the surface area and thus surface loading rate is unchanged, the settling performance of a horizontal flow sedimentation basin is unaffected by changes in water depth.

In practice, the depth and detention time do have some impact on performance, but the important point is that the surface loading rate is a much better indication of sedimentation basin performance than basin depth or detention time. Thus, a low detention time is not necessarily a sign of poor sedimentation basin performance.

An important aspect of sedimentation basin performance is achieving a steady uniform flow pattern in the basin to allow the floc particles to settle out. The flow pattern in traditional sedimentation basins often is not a horizontal uniform pattern. Typically, the flow enters the basin from an inlet channel or over an inlet weir and passes through an inlet zone where a uniform flow pattern is established. At the outlet end, the settled water is often collected in launders. The upward flow pattern in the region of the launders forms an outlet zone. In between the inlet zone and outlet zone is a region of uniform horizontal flow with relatively quiescent conditions called the settling zone. The presence of the inlet and outlet zones reduces the area of the basin that is effective for settling. Also, the vertical flow pattern up to the launders tends to promote circulating currents in the basin, particularly if the basin is deep and if there are density differences or wind effects.

CDM prefers to design sedimentation basins with inlet and outlet perforated walls to minimize the size of the inlet and outlet zones. The inlet perforated wall is designed to provide equal flow through each of the uniformly-spaced holes. This generates a uniform flow pattern in the basin and reduces or eliminates the inlet zone. Similarly the outlet perforated wall provides for uniform horizontal flow right up to the perforated wall and thus eliminates the outlet zone. Because the settling zone comprises a greater fraction of the basin, sedimentation basins utilizing inlet and outlet perforated walls are more efficient. The perforated walls are designed so that there is adequate head loss through the ports to achieve uniform flow distribution, but at the same time, the velocity through the ports should not be so high that it causes floc break-up. With near-parallel plug flow in the basin, density currents caused by turbidity or temperature differences will be less likely to occur or less severe if they do occur.

CDM also prefers to design sedimentation basins with a relatively high length to width ratio, typically in the range 4:1 to 6:1. A long length reduces the tendency for short-circuiting to occur.

2.5.2 EXISTING SEDIMENTATION BASINS

The Central plant has two primary sedimentation basins and two secondary sedimentation basins. The primary and secondary basins are only separated by a slotted wall, and in effect they act as one long sedimentation basin.

The primary basins are equipped with circular sludge collection mechanisms. The secondary basins have no sludge collection equipment. There are long (100 foot) effluent launders in the secondary basins. These launders result in a long outlet zone. At the inlet end, there is a slotted baffle wall which is intended to provide a uniform flow pattern into the sedimentation basins. The existing slotted baffle wall has a relatively large open area (approximately 32 percent open area), so it is doubtful that it is effective in generating a unifrom flow distribution into the sedimentation basin. The tracer study found that there is backmixing into the flocculation basins.

The existing basins have a total length to width ratio of about 2.5. Consequently, there is a tendency for short-circuiting, as was found in the tracer study. The only practical means of increasing the length to width ratio of the existing basins is to add a center wall to each basin. This would mean removing the existing circular sludge mechanisms and replacing them with an alternative sludge removal system. One advantage of installing a center wall is that if it were a hydrostatic wall, then in effect there would be four sedimentation basins instead of two. One basin could be taken out of service for maintenance with less impact on the plant's capacity.

The combined primary and secondary sedimentation basins have a surface loading rate of 610 gpd/sf and a theoretical detention time of 4.6 hours at a 80-mgd flow. The surface loading rate is equal to the State guideline of 600 gpd/sf at 80 mgd. The detention time is more than the 3 hour State guideline. As discussed in the previous section, the surface loading rate is the better indicator of settling performance than detention time.

The East plant has two primary sedimentation basins and one small secondary basin. The primary basins are square, center-feed radial-flow type basins with permeter V-notch units and circular sludge collection mechanisms. The two primary sedimentation basins have a combined surface loading rate of 870 gpd/sf and combined detention time of 3.4 hours for a 30-mgd plant flow.

2.5.3 TUBE OR PLATE SETTLERS

Tube or plate type settlers may be installed in existing sedimentation basins to increase the settling capacity. By installing numerous inclined tubes or plates, the settling area per unit of gross tank area is greatly increased. They are normally only cost effective when available land is limited or very expensive. When adequate land is available for conventional sedimentation basins, they are usually not used.

Plate settlers were investigated as a possible option for increasing the settling capacity of the Central plant sedimentation basins. The estimated cost of installing plate settlers in the existing basins to handle a 160-mgd flow is $8,000,000. Given the high cost and the potential maintenance problems of using plate settlers in a lime softening plant, it was decided that plate settlers are not a viable option in this case.

2.5.4 SLUDGE REMOVAL EQUIPMENT TYPES

If the existing two sedimentation basins at the Central plant were each divided into two to yield four basins, it would be necessary to remove the circular sludge mechanisms and install new sludge removal equipment. Also, sludge removal equipment should be added to the Central plant secondary sedimentation basins. The possible sludge removal mechanisms which could be used at the Wichita WTP are briefly described below.

Circular Collector Mechanism

It is possible to install multiple circular mechanisms in rectangular sedimentation basins. However, the existing rectangular basins would require significant structural modifications to permit the use of multiple circular collector mechanisms. The bottoms of the existing basins would have to be extensively grouted to fit both the scraper arms and cornersweeps and allow for a sludge hopper. Extensive piping would be required to remove the sludge from the sludge hopper.

The advantages of the circular collector mechanisms system include the following:

1) Proven and successful technology
2) Operating machinery above the water surface
3) Low operation and maintenance cost

The disadvantages include:

1) Extensive structural modifications required to basins.
2) Grouting required for installation.

Traveling Bridge Collector Mechanism

The traveling bridge type of sludge collector travels up and down the sedimentation basin at speeds of about 10 feet per minute. The bridge has wheels at each end which ride along the top of the basin walls and provide the traction. The wheels may be either the pneumatic type riding on concrete, or steel wheels riding on a steel rail with gear and pinion drive.

There are two basic types of traveling bridge sludge collectors: the scraper type and the siphon type. The former has a scraper blade hanging from the bridge which scrapes the sludge to hoppers or a screw conveyer at one end of the basin. On the return trip, the scraper is raised off the floor. The siphon type of collector has a suction header suspended from the bridge. The sludge is "vacuumed" off the basin floor and discharged into a sludge trough on one side of the basin.

The sedimentation basin floor has to be level if the traveling bridge siphon type collector is used, whereas with the traveling bridge scraper collector, the floor can slope to one end to provide better drainage for when the basin is emptied.

The advantages of the traveling bridge collection system include the following:

1) Proven and successful technology
2) All operating machinery is above the water surface.

The disadvantages include:

1) Basins must be drained to maintain header assemblies.
2) Grouting of floor bottoms is necessary to provide a straight operating floor.
3) Significant structural modifications required to the existing basins.
4) Places restrictions on location of equipment near the top of the sedimentation basins, such as walkways, light standards, etc.
5) Bridge has a limited basin width over which it can span.

<u>Floating Bridge Collector Mechanism</u>

The floating bridge type sludge collector is similar to the traveling bridge siphon type sludge collector, except that the bridge is supported by floats rather than wheels. The bridge floats on the water surface and is towed up and down the basin by a cable. Small guide wheels at the ends of the bridge run against the face of the basin side walls. A suction header supported from the bridge a minimum of 1 inch above the basin floor vacuums the sludge into a sludge trough. Floating bridge sludge collectors are normally used in sedimentation basins with effluent launders. With a floating bridge unit, it is important to have both a flat floor and a relatively constant water level to maintain the suction header at the correct clearance from the floor. The use of effluent launders provides good control of the water level.

The advantages of the floating bridge collection system include the following:

1) Proven and successful technology
2) All operating machinery is above the water surface.

The disadvantages include:

1) Basins must be drained to maintain header assemblies.
2) Grouting of floor bottoms is necessary to provide a level operating floor.
3) Not suitable for use with perforated outlet walls, since requires a constant water surface.

Underwater Suction Collector

This type of sludge collector is relatively new. It consists of a suction header mounted on a tractor unit. The suction header is typically 20 feet long and has 3/4-inch holes along the underside. The tractor unit rides along a single rail and has either a pneumatic drive system or is pulled along by a cable. Each tractor unit typically can travel a distance of 160 feet. A flexible umbilical hose attached to the center of the suction header connects to a fixed suction pipe located at the mid-point of the 160 foot travel. Sludge is "vacuumed" off the basin floor and passes through the umbilical hose. Differential head hydraulically forces sludge from the bottom of the basin through the orifices in the header pipe into the umbilical hose and collection header for disposal. The tractor carries the suction header up and down the basin floor much the same way as a suction type traveling bridge carries a suction header up and down the basin.

The advantages of a floor mounted underwater suction collectors include the followinwg:

1. Less structural modifications required for existing basins, since units can tolerate a moderate floor slope in either direction.

2. Tractor units can be pulled from the basins for maintenance without taking the whole system out of service.

The disadvantages of a floor mounted suction collector system include the following:

1. Most all of the mechanical operation is below water.

2. The sludge withdrawn from these systems tends to be of lower concentration and thus results in increased loading on the sludge handling facility.

3. Suction header is prone to clogging problems in a lime softening plant.

Chain and Flight Scraper Collectors

These are a simple and common method of sludge removal. Typically one to three collectors are installed side by side in each sedimentation basin which scrape the sludge into hoppers or cross collectors at one end of the basin. A single motor per basin can often be used to drive all the collectors in that basin. A disadvantage of chain and flight scrapers is that there are underwater bearings and cogs which require maintenance and replacement from time to time. In a retro-fit situation, the floor of the sedimentation basin would have to be grouted flat. Also, a short ramp would be formed in the floor at one end so that the flights would pull the collected sludge up the ramp and drop it into a narrow channel with a cross collector to transfer the sludge to a single hopper. Using a ramp in this manner saves having to break out the existing floor to construct a below-floor trench for the cross collector.

The advantages of chain and flight collector mechanism system include the following:

1) Proven and successful technology
2) Operating machinery above the water surface
3) Low operating cost

The disadvantages include:

1) Moderate maintenance cost
2) Structural modifications required to basins
3) Grouting required for installation

2.5.5 RECOMMENDATIONS

Various modifications to the sedimentation basins are recommended for the different plant expansion alternatives presented in Section 3.0. The following general recommendation can be made at this point with respect to the existing sedimentation basins.

It is recommended that the existing sedimentation basin outlet launders at the Central plant be removed and a perforated outlet wall be installed. A perforated outlet wall will eliminate the large outlet zone under the launders and improve the flow pattern and efficiency of the basins.

It is recommended that the baffle wall between the flocculation and the sedimentation basins be modified to improve flow distribution into the sedimentation basins and to reduce backmixing into the flocculation basins.

It is recommended that chain and flight sludge collectors be used in the Central plant secondary sedimentation basins and to replace the circular mechanisms in those expansion alternatives that propose removing the existing circular mechanisms.

2.6 FILTRATION

2.6.1 GENERAL

Filtration, as it applies to water treatment, is the process of passing water through a porous medium for the removal of suspended solids. Rapid gravity filters are the standard for municipal water treatment plants.

The existing facilities at the Wichita WTP include fourteen mono-medium sand, constant rate filters. These filters are currently rated for a maximum operating capacity of 120 mgd at a filtration rate of 4.3 gpm/sf.

In order to increase the filtration capacity of the plant, there are three basic alternatives:

- o operate the existing filters at a higher rate with only minor modifications (with State approval),
- o upgrade the existing filters including replacement of media and underdrain system so that the filters can operate at a higher rate (with State approval), or
- o add new filters to provide additional filtering capacity.

For the Wichita WTP, it is not considered that the first alternative is a viable option because:

1) the filtration rate of 4.3 gpm/sf at 120 mgd is already significantly higher than the KDH&E guideline of 3 gpm/sf for rapid sand filters and it is doubtful that the State would approve a higher rate for the existing filters with no major improvements,

2) the head loss through the existing sand medium and filter outlet piping would be so high at increased filtration rates that there would be insufficient driving head remaining to provide a reasonable filter run time, and

3) higher filtration rates would drive the particles deeper into the filter bed, resulting in a longer backwash duration using the existing water-only backwash system and thus an increased loading on the sludge handling facility.

The two alternatives that will be considered for increasing the filtration capacity of the Wichita plant from 120 mgd to 160 mgd are to upgrade the fourteen existing filters to handle the higher rate or to rehabilitate the existing filters (to operate at the same rate) and add new filters similar to the existing ones.

The basic criteria to be considered for filter design include:

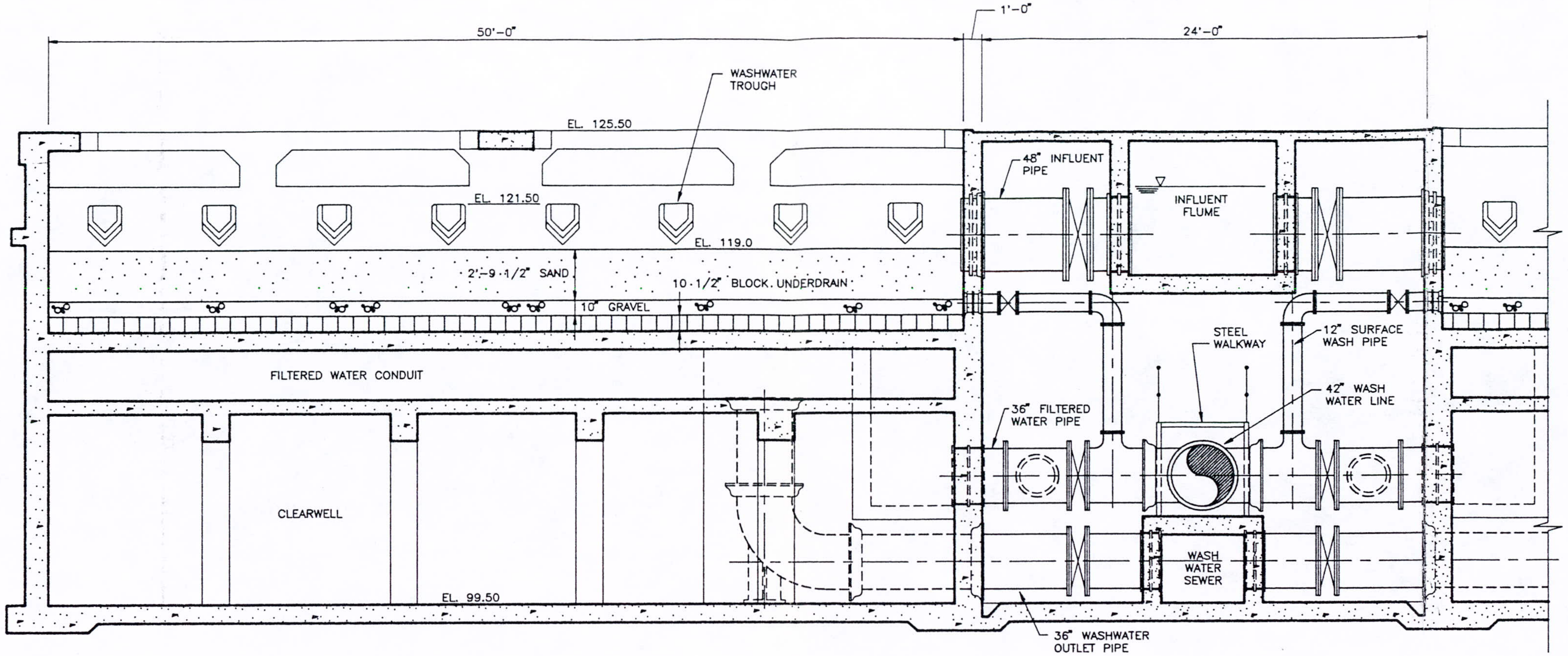
- Filter loading rates
- Filter media
- Filter backwash method
- Filter underdrain system
- Filter control
- Filter conditioning
- Filter aids
- Filter monitoring

The existing filters at the Wichita WTP will be briefly described and then the design criteria listed above will be discussed as they relate to the Wichita filters.

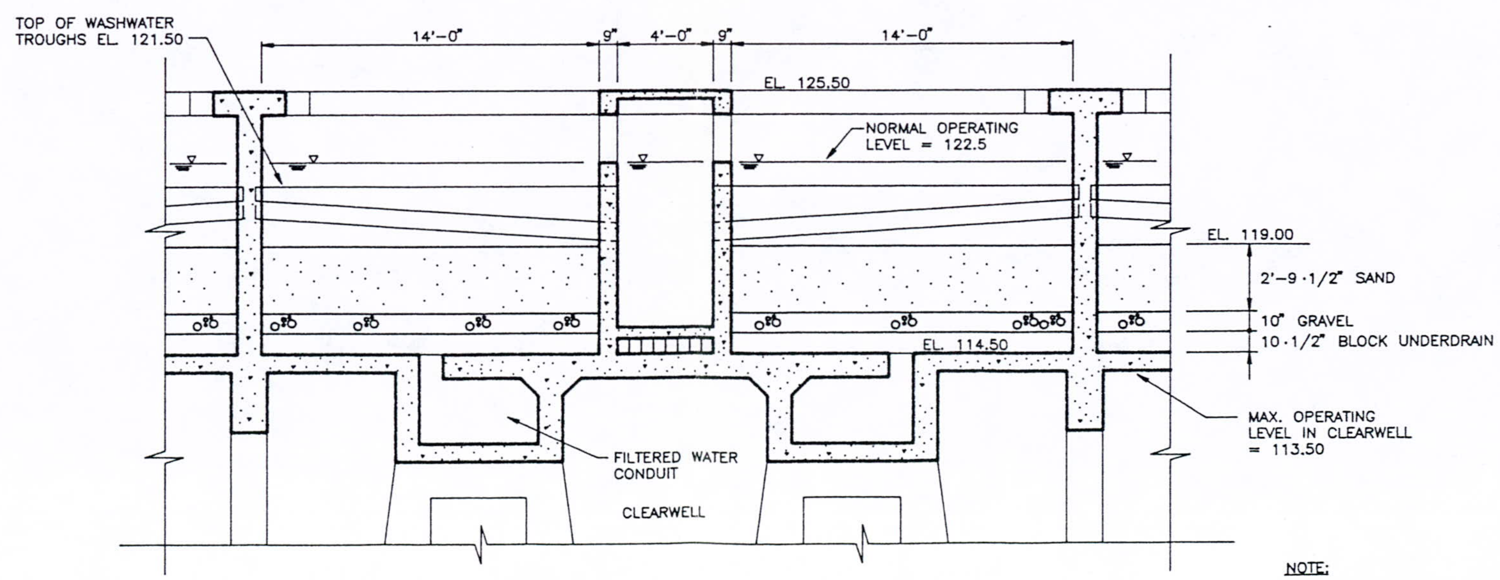
2.6.2 EXISTING FILTERS

There are fourteen sand mono-medium filters at the Wichita WTP. They were constructed in 3 stages: filters 1 to 4 were constructed in 1939 when the East Plant was constructed; filters 5 and 6 were added in 1947; and filters 7 to 14 were constructed in 1955 when the Central Plant was constructed. A new filter control system was installed in 1974.

Figure 2-3 shows the general arrangement of the existing filters. Typically, the filters are 50'-0" long and 28'-0" wide (two halves each 14'-0" wide) providing a total area of 1,400 square feet per filter. For filters 5 and 6, one half is 13'-3" wide, yielding an area of 1,363 square feet per filter.



SECTION
SCALE: 1/8" = 1'-0"



SECTION
SCALE: 1/8" = 1'-0"

NOTE:
THIS SECTION IS TYPICAL OF FILTERS 7 TO 14. FILTERS 1 TO 6 ARE SIMILAR EXCEPT THE UNDERDRAIN IS THE PIPE LATERAL TYPE AND THE SAND AND GRAVEL ARE 36" AND 18" DEEP RESPECTIVELY.

FIGURE 2-3
SECTIONS THROUGH EXISTING FILTERS

Settled water enters the filters via an enclosed influent flume and 48-inch influent pipe. Filtered water is collected in the underdrain system and conveyed through a filtered water conduit under the filter to the outlet piping in the pipe gallery and then into the clearwell underneath the filters.

There are two types of filter underdrains at the Wichita WTP. Filters 1 to 6 have a pipe lateral type underdrain. The pipe lateral system consists of 3-inch pipes located at the bottom of the gravel layer as shown in Figure 2-4. There are 132 pipe laterals in each filter. Each lateral has 78 3/8-inch holes drilled in the bottom. A tee located at the center of each pipe lateral extends through the floor slab and feeds directly into the filtered water conduit.

Filters 7 to 14 are the newest filters and utilize a ceramic block underdrain as shown in Figure 2-5. The blocks are 10 1/2" high and are overlain by 10" gravel and 2'-9 1/2" sand mono-medium. In contrast, filters 1 to 6 have 1'-6" gravel and 3'-0" sand.

The filters are all constant-rate, outlet-control type. Each filter has a venturi flow meter and butterfly control valve in the outlet piping. Filters 1 to 6 have 16-inch venturis and 14-inch control valves. Filters 7 to 14 have 20-inch venturis and 18-inch control valves.

Each of the filters is equipped with a stationary surface wash system and is backwashed using water only. The washwater is supplied using two backwash pumps which draw from the clearwell and feed into a 42-inch washwater line at the center of the pipe gallery. Dirty washwater is collected in a washwater sewer and taken to the sludge handling facility located to the west of the Central Plant.

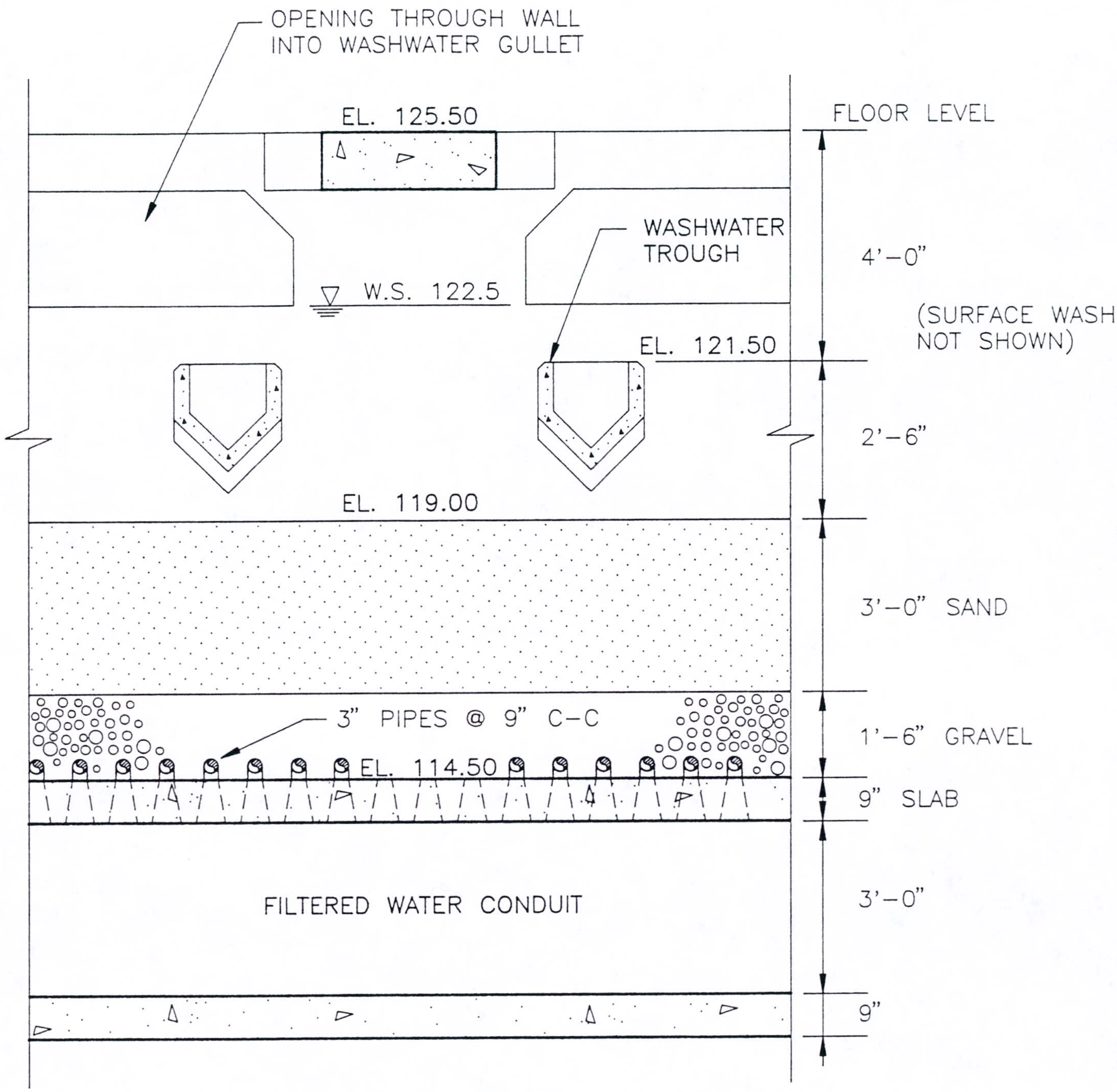


FIGURE 2-4

SECTION THROUGH FILTER BOX
EXISTING PIPE LATERAL UNDERDRAIN

SCALE: 3/8" = 1'–0"

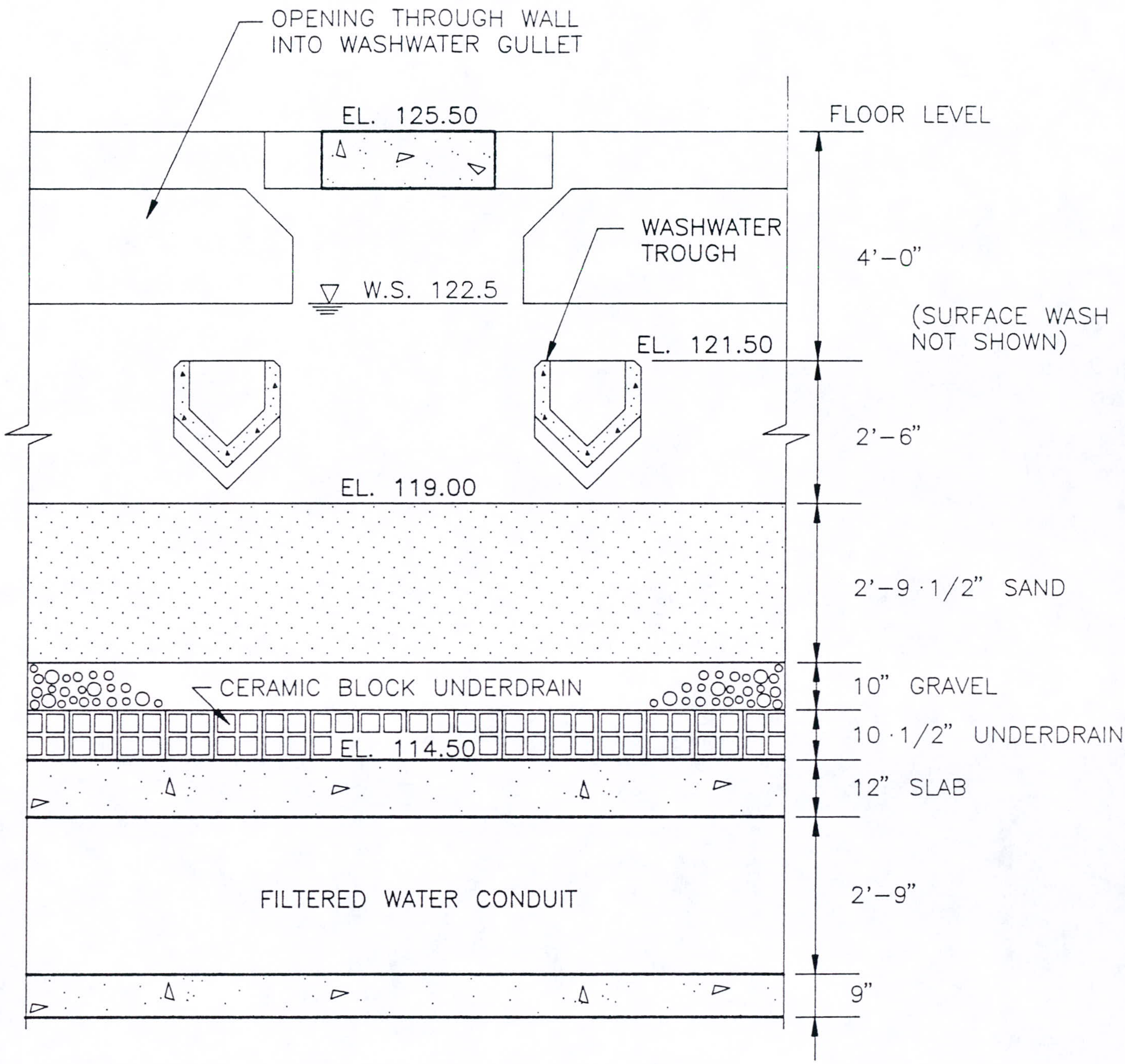


FIGURE 2-5

SECTION THROUGH FILTER BOX
EXISTING CERAMIC BLOCK UNDERDRAIN

SCALE: 3/8" = 1'-0"

There are a number of aspects of the existing filters which would need to be addressed in order to increase the filtration rate. These are:

- The existing sand mono-medium has a small effective size which, combined with the large depth and small porosity of sand, results in a high clean-bed head loss. The estimated clean-bed loss at 120-mgd plant flow is approximately 3'-0". At 160 mgd, the clean-bed loss would be about 4'-0", which severely limits the driving head available for clogging of the medium.

- The existing underdrain system does not permit air scour during backwashing. At high rates, the particulates will be driven deeper into the bed, making it more difficult to clean the medium. The existing water-only backwash and surface wash system may prove inadequate at higher filtration rates, particularly if a filter aid is used.

- There is minimal clearance between the existing washwater troughs and the sand. The location of the washwater troughs limits the type of replacement media which could be used, unless the troughs are either raised or removed.

- The existing outlet piping is relatively small and results in high velocities and high head losses. For filters 1 to 6, the head losses on the outlet piping amounts to approximately 4'-6" at a 120-mgd plant flow. For a 160-mgd flow, this increases to over 8'-0". Such losses are intolerable since there would be no driving head remaining for clogging of the bed.

- There is a limited available driving head for the Wichita filters. When the reservoirs are full (and accordingly the clearwell is full) the difference in elevation between the water surface in the filter box and the water surface in the clearwell is only 8'-9". Given the high losses in the clean sand mono-medium and in the outlet piping as discussed earlier, the limited driving head when the clearwell is near full means that 1) the filter runs will be short for filtration rates approaching 4.3 gpm/sf (corresponding to a 120-mgd plant flow) and 2) the existing filters have insufficient driving head to operate at rates much higher than 4.3 gpm/sf.

These issues will be discussed further in the following sections.

2.6.3 FILTER LOADING RATES

The Kansas Department of Health and Environment (KDH&E) "Policies Governing the Design of Public Water Supply Systems in Kansas" specify a maximum design filtration rate of 3 gpm/sf for single media filters. For dual-media filters, the maximum design filtration rate specified by the State is 4 gpm/sf.

The City's water quality reports show that the filtered water turbidity at the Wichita WTP averages around 0.3 NTU, but varies from less than 0.1 NTU to 1.5 NTU or more on occasion. Filter run lengths are 80 to 100 hours in the winter and 40 to 80 hours in the summer (terminated by head loss or turbidity breakthrough). For a 120-mgd plant flow, the existing fourteen filters would be operating at a filtration rate of 4.3 gpm/sf. Considering the variable filtered water turbidity presently achieved and the head loss problems described earlier, it is unlikely that the existing sand filters would operate satisfactorily at rates above 4.3 gpm/sf without major filter rehabilitation.

2.6.4 FILTER MEDIA

One of the key factors for proper performance of a filter is selection of an appropriate filter media. Dual-media filter beds are common for gravity filters with filtration rates over 2.0 gpm/sf. These beds normally use 20 to 27 inches of anthracite coal with a specific gravity of about 1.4 to 1.7 as the top layer, overlaying a 6 to 16-inch sand layer with a specific gravity of about 2.65 and an effective size of one-half that of the anthracite. Sometimes a multi-media bed is used, in which case a third layer, usually 3 to 4 inches of garnet or ilmenite with a specific gravity of around 4.2 and an effective size smaller than the sand, is placed below the sand. The reasoning for the dual-media and multi-media systems is to simulate the coarse through fine filter concept, thereby allowing greater penetration of particulate matter into the bed and providing longer filter runs for the same head loss. A properly designed dual-media filter will also provide filtered water quality equal to a mono-medium sand filter but at a higher filtration rate.

A recent trend for high rate filters is to use deep mono-medium filter beds. These have only one type of filter material, uniformly graded, and usually with an effective size as large or larger than the upper layer of a dual-media bed. Mono-medium beds are normally much deeper than dual-media beds. The main advantage of mono-medium filter beds is that if an air scour system is used for backwashing, the medium need not be fluidized for restratification. This reduces the amount of washwater needed. It also reduces the height needed between the top of the medium and the washwater troughs.

The City has indicated that the existing sand filter medium is performing satisfactorily even though it has never been replaced. However, we do not consider that the existing sand medium would be suitable at higher filtration rates. We recommend that the sand be replaced with either an anthracite mono-medium or a dual-media bed. The existing 36-inch sand bed could be replaced with a standard 30-inch dual media. It may be feasible to skim about 26 inches of sand off the existing filter beds and add 20 inches of new anthracite rather than purchase new sand. Alternatively, on anthracite medium of about 40-inches depth could be used. The final decision on the new media type, depth, and grain size would be made based on both the type of underdrain used and the results of pilot plant testing.

2.6.5 FILTER BACKWASHING

Water-Only Backwashing of Filters

During the operational cycle, filters must be periodically backwashed to remove particulates (flocculated material) collected on and in the filter media. In a water-only backwashing system, water is passed upward through the media at sufficient velocity to fluidize the bed and achieve an expansion of 10 to 15 percent. Higher backwash rates will not improve the cleaning of the media and only add to backwashing costs. Ideally, the percent of expansion should be constant, requiring adjustments to the backwash rate to accommodate the wide seasonal variations in water temperature and viscosity. Dual-media filters must be washed at rates between 18 and 24 gpm/sf (depending on the effective size of the media and water temperature) to ensure that the media is fluidized during the backwash and restratifies when the backwash rate is reduced and stopped at the end of the backwash sequence.

Auxiliary Scour

For some filters, particularly high-rate, deep bed filters, the use of water only backwash has proven insufficient for effective cleaning of the grains in the medium. To overcome this problem, mechanical rakes, fixed surface jet washers, rotating surface jet washers, and air scouring systems were developed. Mechanical rakes are rarely used in filters today. Fixed or rotating surface jet washers have been the traditional auxiliary scour methods used in the United States. A concern with surface jet wash systems is whether the sand-anthracite interface in a dual-media bed will be adequately cleaned. Some rotary washers are designed with a second rotating head for sub-surface washing at the interface, but in a number of locations there have been maintenance problems with these units. Air scour is being used more frequently.

Air-Water Backwashing of Filters

Air-water systems provide a better degree of cleansing than surface wash systems. Air in the backwash sequence creates the necessary agitation in the filter to clean the adhered particulate matter off the grains of the filter medium. The shearing forces produced by air scour are about twice those of a water-only fluidized bed backwash. A further advantage is that this cleaning action is effective throughout the full depth of the filter. The air scouring action does the cleaning; water is used as a conveyance mechanism for the dislodged floc particles. Fluidizing the media by using water is not needed for cleaning when air is used. However, if air scour is used in a dual-media filter, fluidization of the media at the end of the backwash cycle is necessary. Fluidization allows the restratification of the grains from the two media that have been intermixed at the interface because of the violent action of the air scour.

CDM's experience in the design and operation of filter backwash systems has shown that concurrent air-water backwash is the most effective way to clean a filter media when the filtered solids have high adhesive forces, particularly when polymers have been used for coagulation or as filter aids. Concurrent air-water backwash requires designing the filter underdrains specifically for this mode of backwashing. Not all filter underdrain systems which can provide air scour are capable of concurrent air/water backwashing. The additional cleaning provided by the concurrent use of air and water as compared to separate air and water is difficult to quantify. For the filters at the Wichita WTP, both concurrent air-water and separate air and water backwash systems will be considered.

Recommendation

We recommend that an air-scour backwash system be incorporated in the filters at the Wichita WTP. It is CDM's opinion that an air scour backwash system (concurrent air/water or separate air and water) would provide more positive full depth cleansing than the existing water-only surface wash system. This modification would ensure effective cleaning of the media and reduce the quantity of washwater required. The filter underdrain system would have to be replaced or modified to allow the air scour.

2.6.6 FILTER UNDERDRAIN SYSTEMS

An underdrain system has two purposes: to collect the water that passes through the media, and to distribute backwash air and water uniformly across the filter. Since the support gravel does not contribute to separation of particulate matter but aids in the distribution of the backwash water, it is technically a part of the underdrain system.

There are a number of underdrain systems available. These include:

- o Ceramic or plastic block laterals, with holes, nozzles, or porous plates
- o Plenum (precast or monolithic concrete false floor) with holes, nozzles, or porous plates.
- o Other systems, including pipe laterals and precast concrete T-Pees.

Some of these systems allow the use of water only, some water and air but not both together, while others allow the concurrent use of water and air.

The existing filters have both the pipe lateral underdrain and the ceramic block lateral type. Neither of the two existing underdrains can handle air scour. The following paragraphs discuss prominent filter underdrain systems currently available which CDM considers suitable for air/water backwashing.

Roberts Ceramic Block and Air Grid Underdrain

Roberts Filter Manufacturing Co. has for many years marketed a ceramic dual-lateral underdrain system. Figure 2-6 shows the general arrangement of this type of underdrain. The ceramic block underdrain provides good backwash distribution and little head loss, but suffers from the disadvantage that an integral air-scour cannot be used. Roberts Filter has recently introduced an air-grid system which provides concurrent air-water backwash capabilities for their ceramic block underdrain.

The arrangement of the Roberts ceramic block and air grid underdrain is shown in Figure 2-7. The ceramic blocks are placed on a grout bed as normal, except stainless steel support rods are installed between the blocks at regular intervals to support the air header and pipe lateral system. The air grid system is constructed such that there is an array of 3/4-inch pipe laterals covering the entire filter area at the level of the top of the gravel layer. These laterals have 1/8-inch air nozzles screwed into the bottom side at 8-inch centers.

The fact that the Roberts system utilizes a block underdrain and air grid that are independent of each other provides a couple of advantages over systems utilizing integral air-water backwashing. Firstly, it avoids the design complications associated with feeding air and water concurrently through a dual-lateral or nozzle system. For underdrains with integral air-water, it is critical that pressures and head losses for both air and water be properly accounted for throughout the underdrain system, otherwise the underdrain may not perform as expected. Secondly, if there is a problem with the air grid system, it will not affect the performance of the block underdrain for filtration or for water-only backwashing. If, for example, the recarbonation system malfunctioned, resulting in

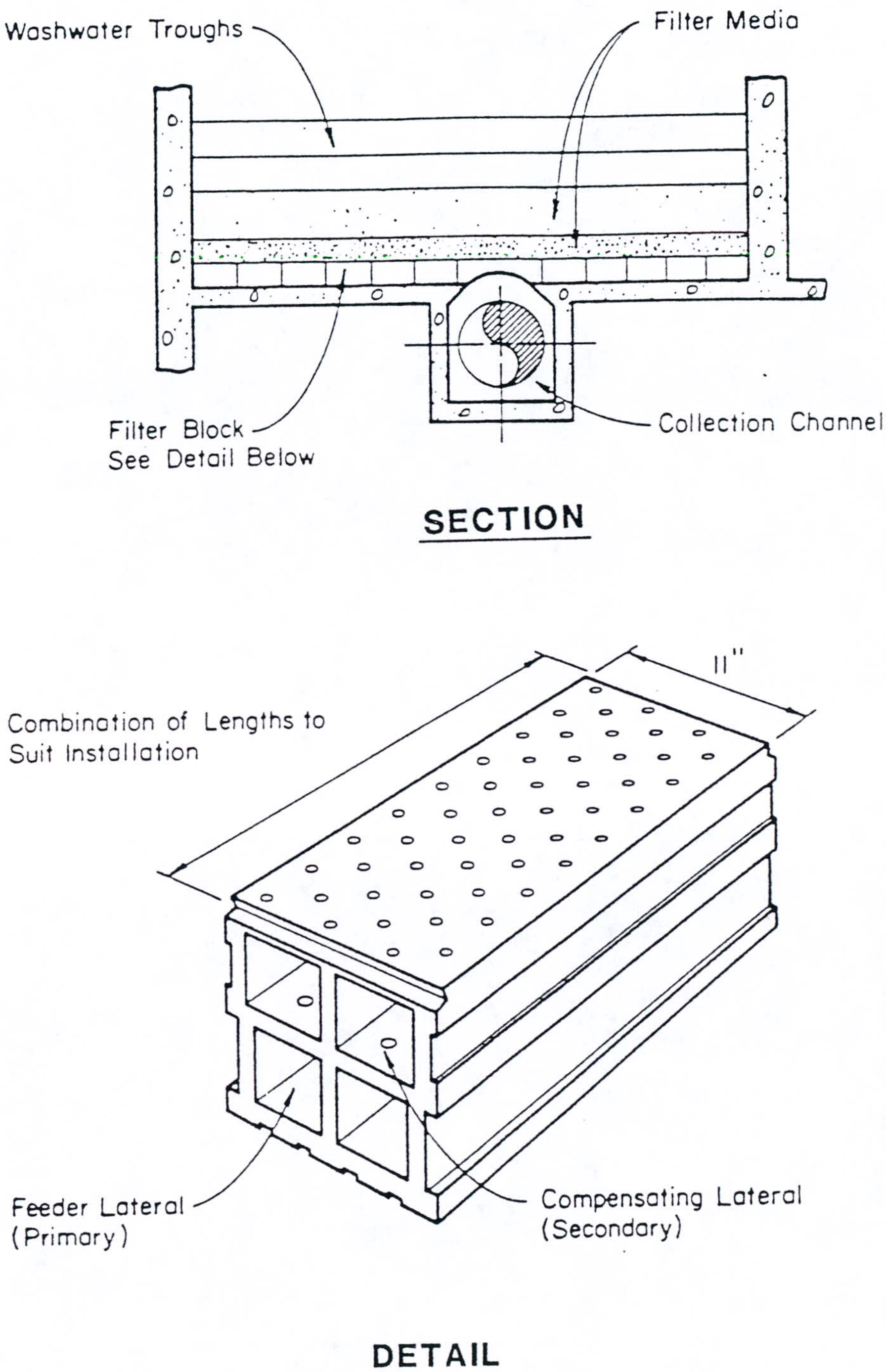


FIGURE 2-6

DUAL-LATERAL TILE
FILTER UNDERDRAIN SYSTEM

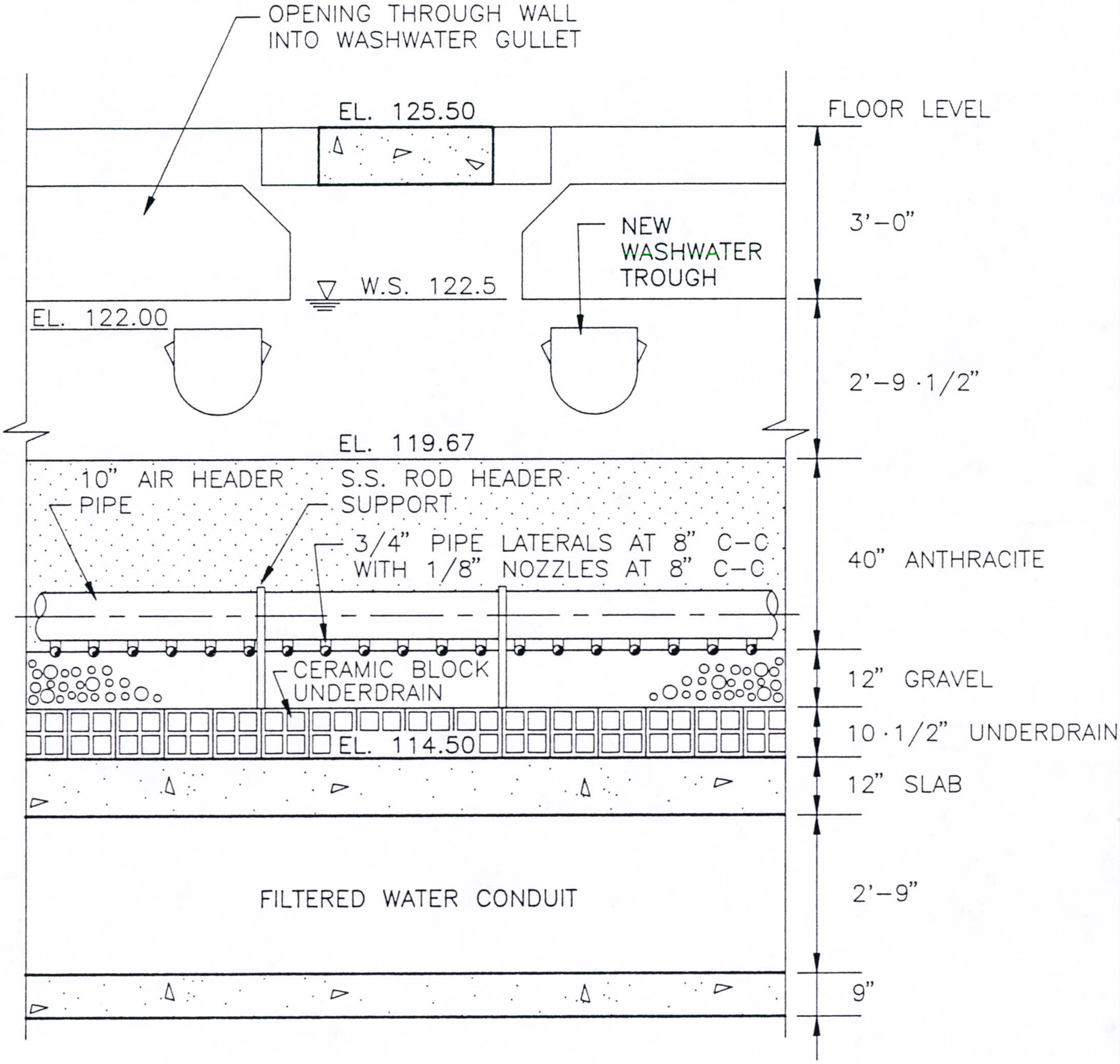


FIGURE 2–7

SECTION THROUGH FILTER BOX
ALTERNATIVE ROBERTS UNDERDRAIN

SCALE: 3/8" = 1'–0"

calcification in the filters, the resultant clogging of the Roberts air nozzles would not have as severe an impact as clogging of the PCI nozzles. Furthermore, it would be a relatively simple matter to feed a small quantity of acid solution back through the air grid system to unclog the Roberts air nozzles, compared with the task of clearing the PCI nozzles.

The Wichita filters could be easily fitted with the Roberts ceramic block and air grid underdrain system. Eight of the filters already are fitted with ceramic block underdrains and depending on the condition of these filters, it may be possible to use the existing block underdrains. If the existing blocks are suitable, the aid grid system could be added to the filters and the gravel and media replaced, saving the cost of removing and replacing the block underdrain in eight of the fourteen filters.

Because the Roberts underdrain requires a 12-inch gravel layer on top of the block, it would not be possible to install either a standard dual media or anthracite mono-medium without modification to the washwater troughs. Using a 30-inch dual media would put the top of the bed at elevation 118.84, only 2 inches below the existing top of sand. A dual media has to be fluidized at the end of the backwash cycle to restratify the sand and anthracite. With the existing washwater troughs, there would be a problem with loss of anthracite during backwashing. It would be necessary to either raise the existing troughs, replace the existing troughs with a wider and shallower trough, or remove the washwater troughs altogether and use a side weir.

If an anthracite mono-medium is used, it would not be necessary to fluidize the bed during backwashing (provided air-scour is used to achieve cleaning), eliminating the media loss problem. However, the anthracite mono-medium would be at least 10 inches deeper than a dual media. The top of the anthracite would be above the bottom of the existing troughs, so the washwater troughs would have to be raised or removed regardless of which media is used.

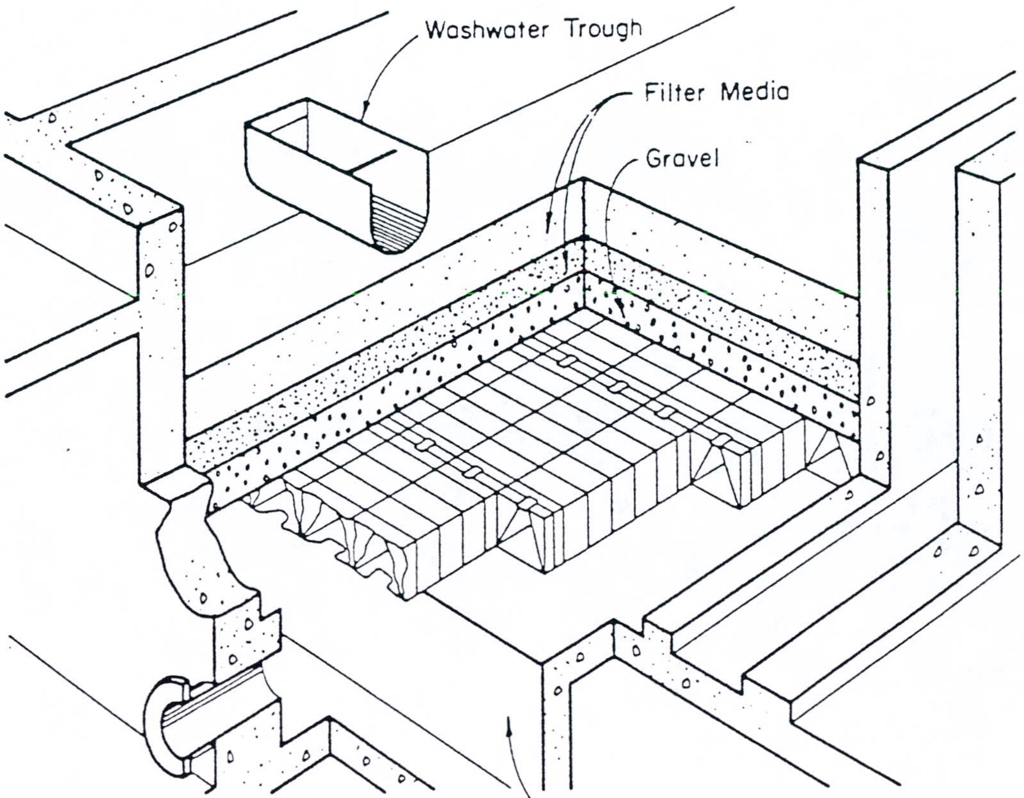
Figure 2-7 shows the arrangement of the Roberts underdrain system assuming an anthracite mono-medium is used and assuming the washwater troughs are replaced with new fiberglass troughs installed at a higher elevation. The specific recommendations for media and washwater trough modifications will be made during the design phase following pilot plant testing.

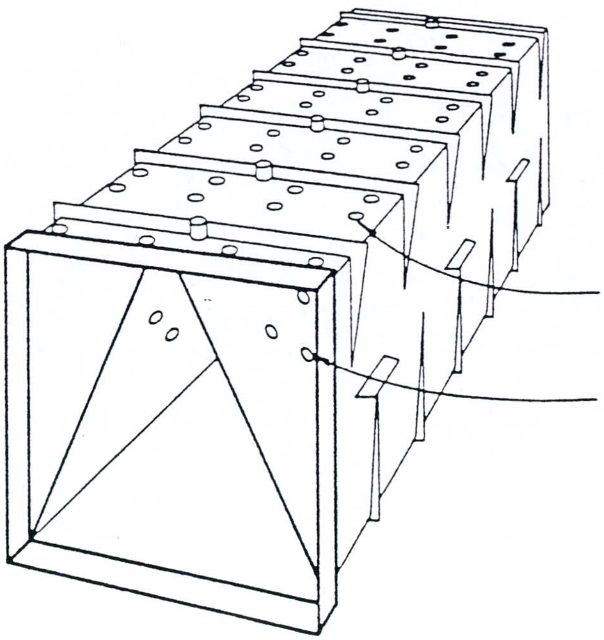
Leopold Dual-Lateral Underdrain

F.B. Leopold Company manufactures a dual-lateral air-water Universal underdrain system which consists of polyethylene blocks each about 1 foot square in section by 3 feet long. The blocks are mechanically joined to form a continuous lateral run equal to the length or width of the filter. This type of underdrain system is shown in Figure 2-8.

The blocks contain a central feeder lateral with 3/4-inch diameter water orifices and 3/16-inch diameter air orifices connecting this lateral to the compensatory laterals. The arrangement and location of the orifices allows for an even distribution of air during backwashing, at delivery rates of up to 5 scfm/sf. A 12-inch layer of gravel is normally required between the underdrain blocks and filter media. The Leopold Universal underdrain is suitable for either separate or concurrent air/water backwashing.

The existing filters could be relatively easily retrofitted with Leopold Universal underdrains, as shown in Figure 2-9. The Universal plastic dual-lateral block is about 2 inches taller than the existing ceramic underdrain blocks. The major work involved with installing the Leopold underdrains would be 1) replacing the existing filter underdrains with the Universal dual-lateral type and 2) installing air piping to feed compressed air into the individual underdrain blocks. As for the Roberts underdrain alternative, it would be necessary to either raise or remove the existing washwater troughs.





Polyethylene Dual Lateral Block

FIGURE 2-8

LEOPOLD DUAL-LATERAL FILTER UNDERDRAIN SYSTEM

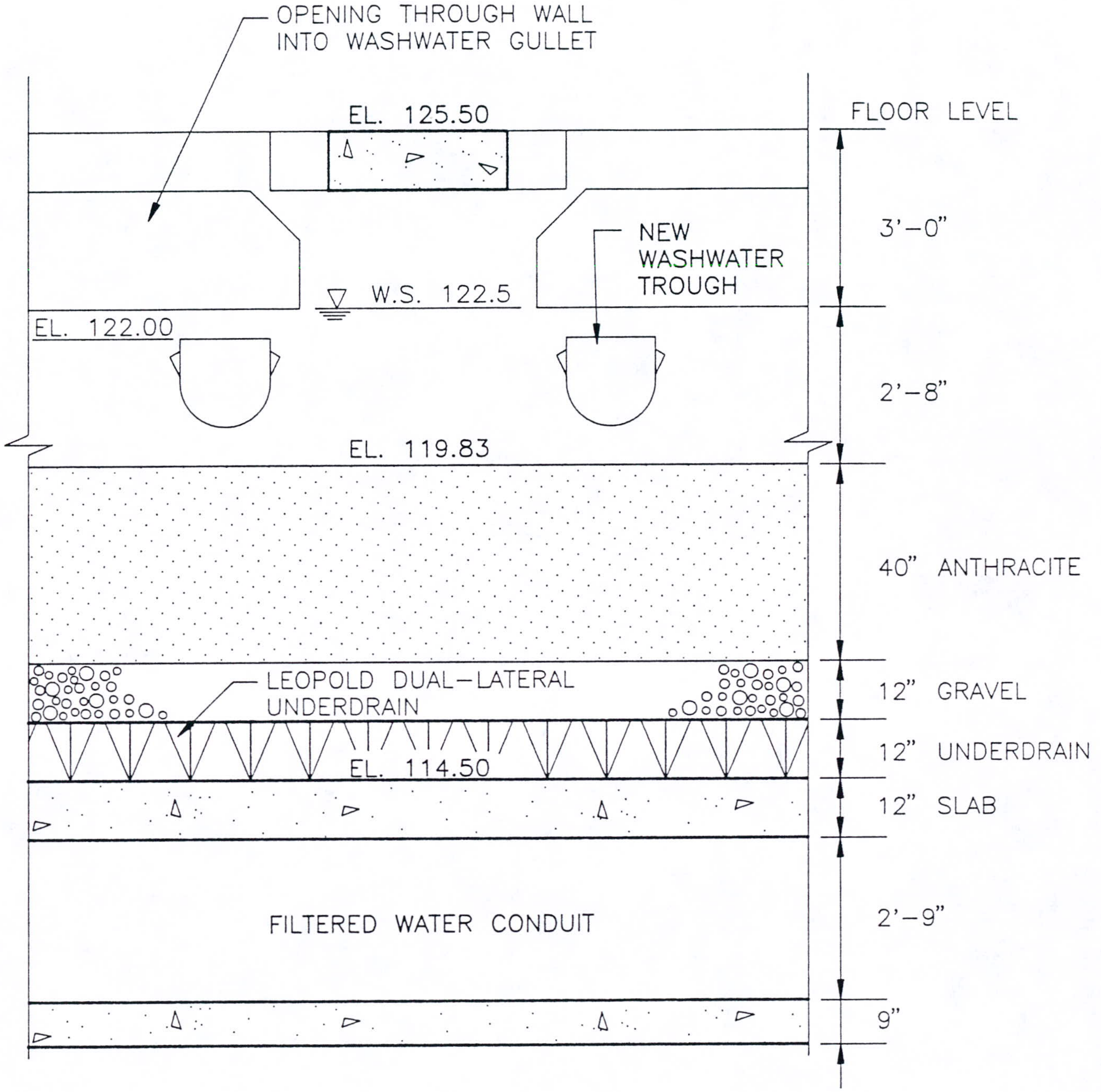


FIGURE 2–9

SECTION THROUGH FILTER BOX
ALTERNATIVE LEOPOLD DUAL–LATERAL UNDERDRAIN

SCALE: 3/8" = 1'–0"

PCI Monolithic False Floor with Nozzles

Patterson Candy International, Ltd., (PCI) markets an underdrain system which consists of a monolithic false bottom with polypropylene plastic nozzles. Figure 2-10 illustrates the monolithic floor type of PCI nozzle system.

The nozzles consist of a slotted dome and stem. The domes have vertical convergent slots, generally 0.015-inch wide on the outer surface and 0.0625-inch wide on the inner surface. The stems contain two sets of orifices, one for passage of water and one for air. Gravel is not required to support the media. However, a 3-inch layer of gravel is often used to aid in distributing the backwash water.

The system functions as follows:

- During filtration, water passes through the media, nozzles, plenum, and into the collection duct.
- During backwash, water passes through the duct, plenum, nozzles, and filter media.
- During air scour, air is forced into the collection duct and then through air distribution holes into the plenum. The water level in the plenum is depressed by the compressed air until it reaches the air orifice of the nozzle stems. At that time, air enters the bottom of the filter media via the nozzles.
- During concurrent air-water backwashing, both air and water are introduced into the duct. Air distribution holes and water distribution openings in the duct walls feed air and water into the plenum. A steady air-water interface is maintained in the plenum, with air passing through the air orifice of the nozzle stems and water passing up the stem from its bottom.

The PCI monolithic floor underdrain is ideal for backwashing with air and water simultaneously. However, there is insufficient depth in the Wichita filters to add a monolithic floor. If a monolithic false floor underdrain were installed in the Wichita filters, the filter media would be raised by

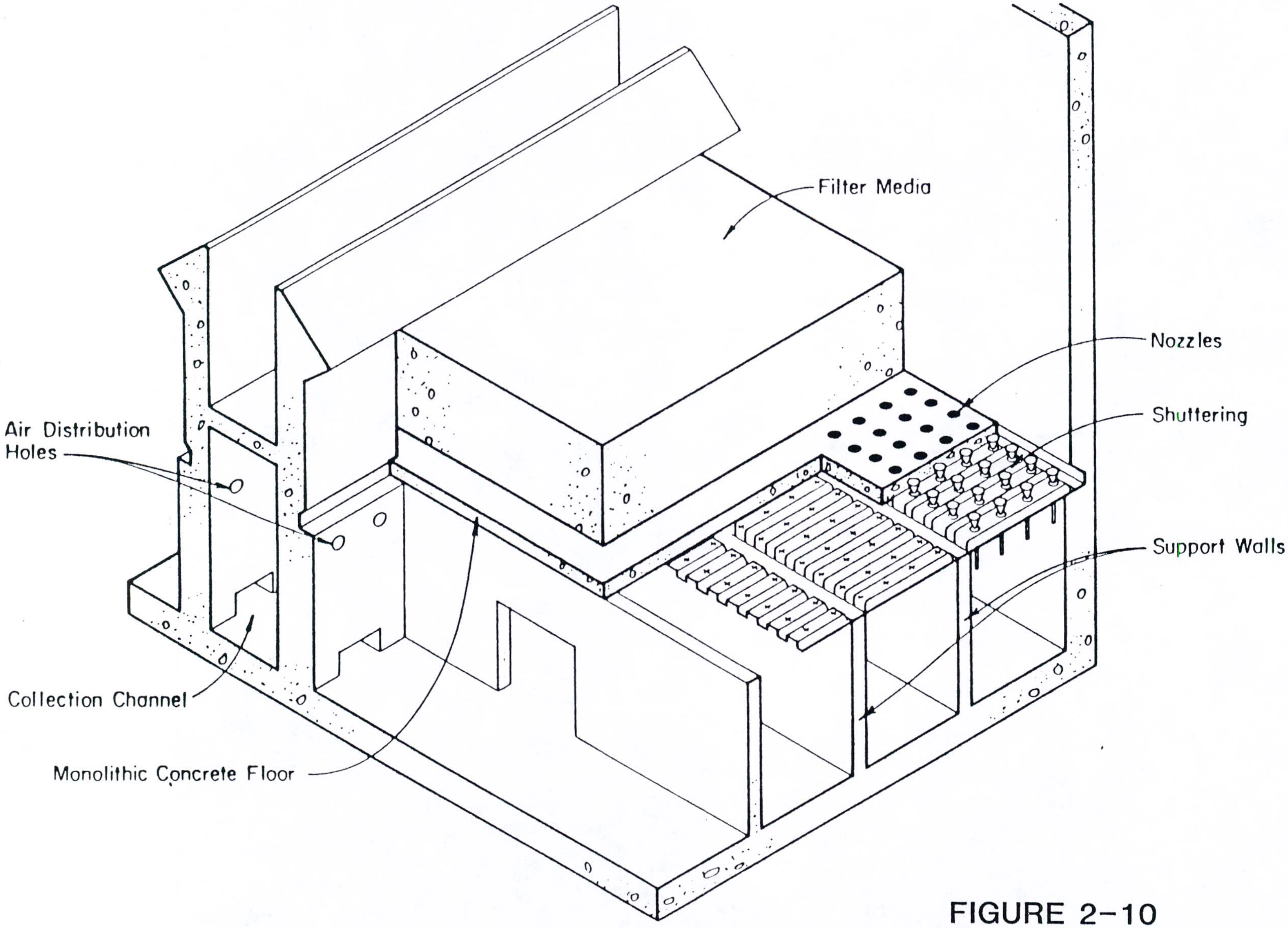


FIGURE 2-10
PCI MONOLITHIC FALSE FLOOR WITH NOZZLES
FILTER UNDERDRAIN SYSTEM

about two feet due to the height of the plenum. An alternative underdrain arrangement utilizing the PCI nozzles is described in the following section.

PCI Pipe Lateral System with Nozzles

The Patterson Candy nozzles can also be used with pipes, as shown in Figure 2-11. The advantage of this arrangement is that it eliminates the height needed for the plenum and false floor. Since a gravel layer is not required under the media, the PCI pipe lateral arrangement would actually lower the bottom of the media, allowing a deeper filter bed to be installed without removing the washwater troughs. Figure 2-12 shows how the PCI pipe lateral system could be used in the Wichita filters.

The disadvantage of the pipe lateral arrangement is that it is only suitable for separate air scour and water backwashing, and cannot be used for concurrent air-water backwashing. As mentioned earlier, air scour alone does not provide quite as good cleaning as concurrent air/water backwashing, however, the addition of air scour would certainly provide much better media cleaning than water-only backwashing.

PCI has recently introduced a modified version of the pipe lateral system which allows concurrent air/water backwashing. The pipe laterals have an internal wall which divides the pipe into two halves, one for water and one for air. CDM is presently evaluating this system and depending on our findings, we may include the new PCI concurrent air/water pipe lateral system as a feasible underdrain alternative for the Wichita filters.

Recommendation

Of the filter underdrain systems discussed above, the Roberts ceramic block and air grid underdrain, Leopold Universal underdrain, and PCI pipe lateral underdrain would all be suitable for the Wichita filters. In order to secure the lowest price, it is recommended that designs be provided for more than one underdrain type and the Contractor be allowed to select the underdrain to be used. This will promote competition between the

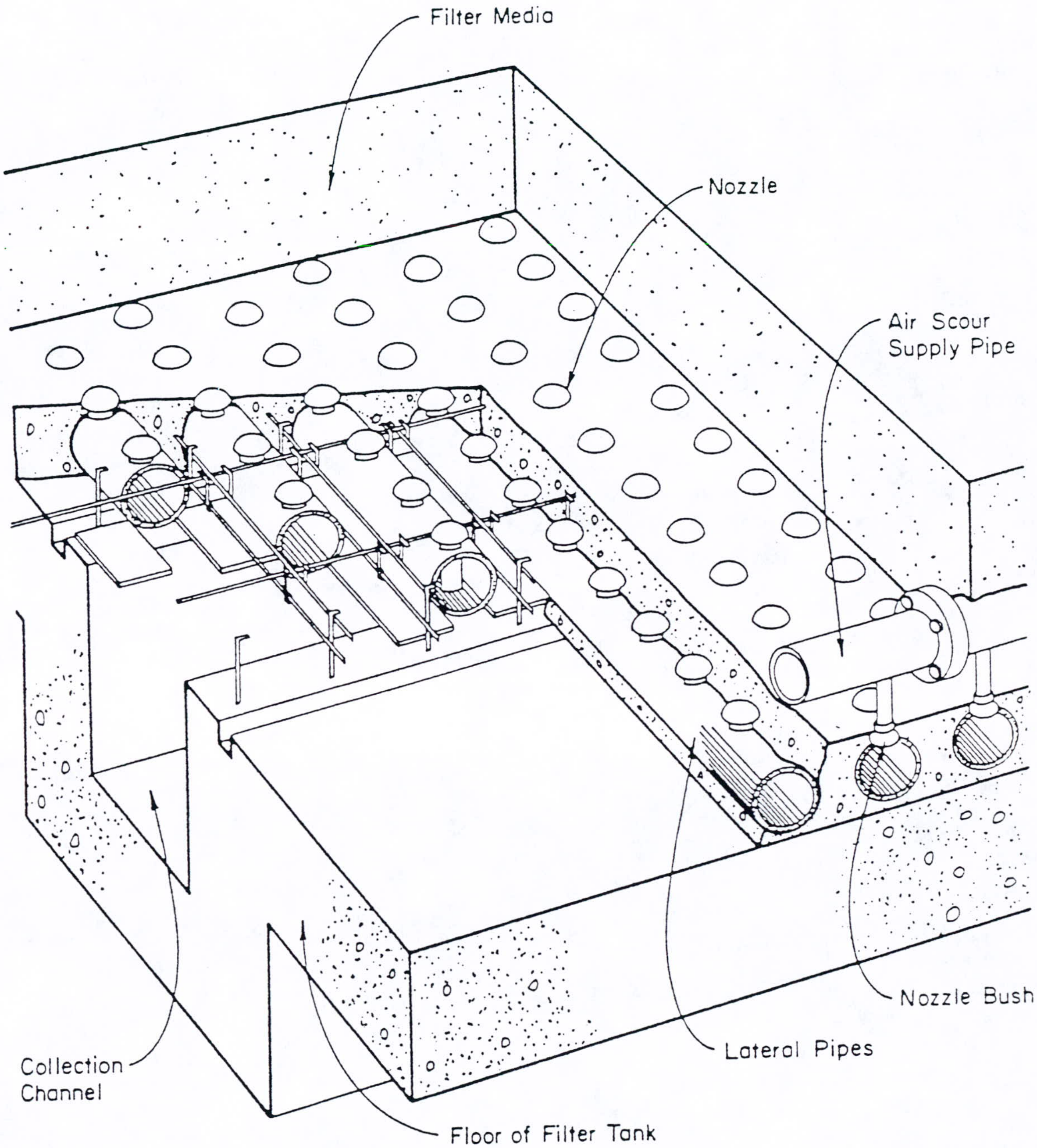


FIGURE 2-11

PCI PIPE LATERAL WITH NOZZLES
FILTER UNDERDRAIN SYSTEM

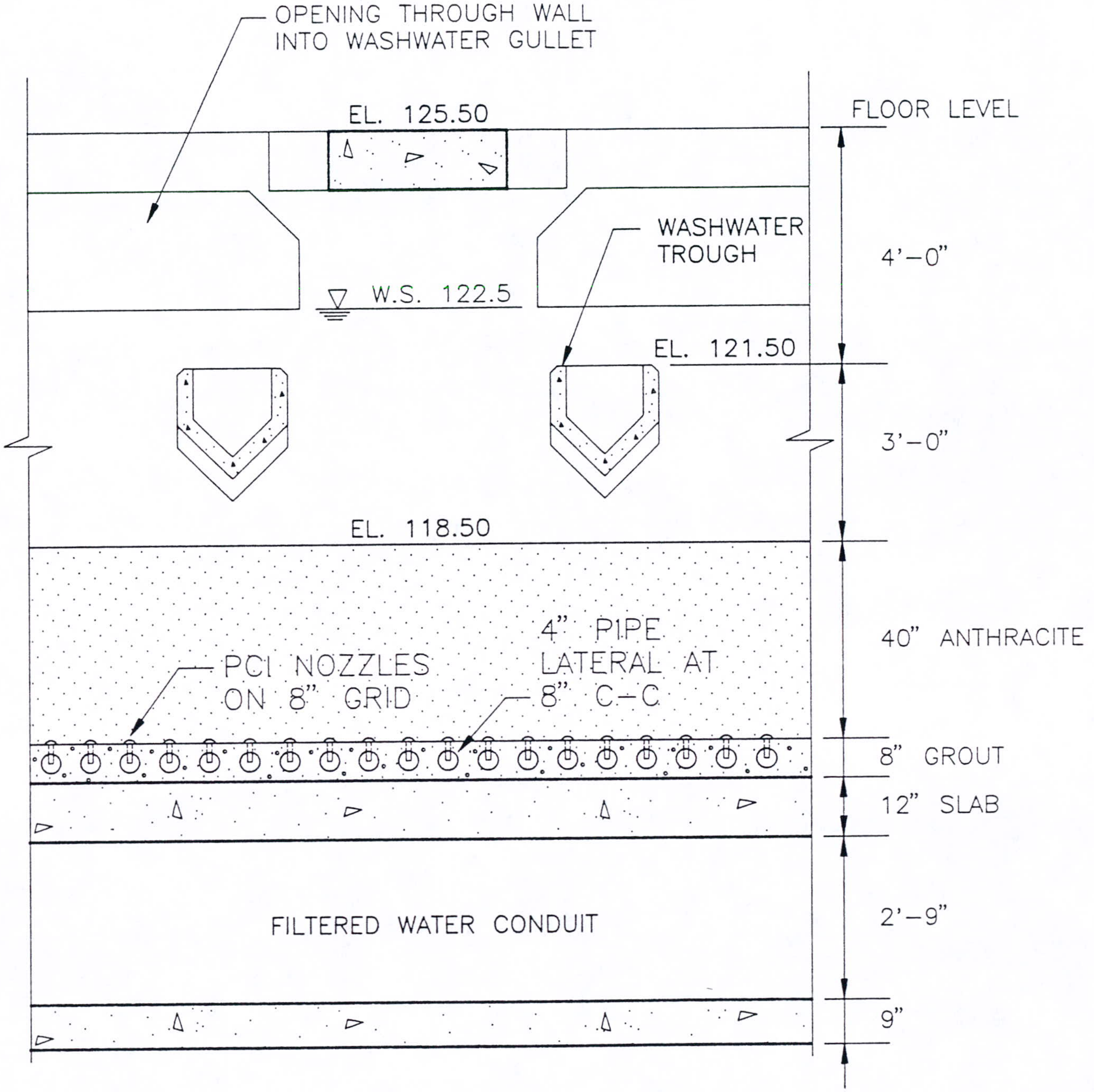


FIGURE 2-12

SECTION THROUGH FILTER BOX
ALTERNATIVE PCI PIPE-LATERAL UNDERDRAIN

SCALE: 3/8" = 1'–0"

underdrain suppliers. The Contractors will evaluate the underdrain material costs and installation costs and then bid the underdrain system with the lowest installed cost, ensuring that the City gets the most economic filter underdrain system.

2.6.7 FILTER CONTROL

The existing filters operate in the constant-rate outlet-control mode. All fourteen filters are equipped with a venturi flow meter and modulating control valve in the outlet piping. The flow through each filter can be set manually or can be set automatically by a master level controller in the inlet flume. In the automatic mode, an increase in plant flow will cause an increase in water level, which will open all rate controller modulating valves to adjust to the new flow. A decrease in flow will cause a reverse adjustment. All operating filters will automatically equally share the plant flow.

Outlet flow controllers offer excellent flexibility for operating filters in both manual and automatic modes, and at varying rates to evaluate performance. There is no reason to change the type of filter control system. However, the existing filter control system instrumentation is unreliable and should be replaced.

2.6.8 FILTER GALLERY PIPING

The existing piping from the filtered water collection conduit under the filters to the tee in the filter pipe gallery is 36 inches in size. After the tee, the outlet piping reduces to 16-inch or 20-inch diameter for the flow controller. Filters 1 to 6 have a 16-inch venturi flow meter and 14-inch control valve. Filters 7 to 14 have a 20-inch venturi flow meter and 18-inch control valve.

The velocity in the 14-inch piping at a 160-mgd plant flow (assuming all filters in operation) is 16.5 fps. This is excessive. The total head loss in the existing filter outlet piping would be about 8'-0" for a 160-mgd flow. There is only 8'-9" of total driving head when the clearwell is full, so obviously the filter outlet piping will have to be replaced with larger piping if no extra filters are to be constructed.

The available driving head for the filters is affected by the water level in the clearwells. When the clearwells reach elevation 108.00 (60 percent full), the water level in the clearwell submerges the filter outlet weir and reduces the available driving head. With the clearwells full, the available driving head for the filters is reduced by 5'-6" compared with the clearwell level below 108.00. There is no simple way to overcome this situation, because the problem is caused by the relative elevations of the filters and the clearwells. It is an operational issue; at times when both the plant flow and clearwell levels are high, the filter runs may be terminated earlier than normal because of insufficient available driving head.

2.6.9 FILTER CONDITIONING

When a filter is brought back into operation following backwashing, the initial turbidity of the filtered water is normally high compared with the turbidity objective. As the filter settles down, the turbidity level drops to an acceptable level from the initial high peak. The cleaner the media, the larger the media, or the higher the filtration rate, the longer the filter will take to ripen or become efficient. These small short spikes have little effect on the overall turbidity levels of the plant. However, these spikes may reduce the microbiological quality of the treated water. Giardia cysts, viruses and bacteria can pass through the filters with these turbidity spikes.

There are three alternative methods of filter conditioning to deal with this initial quality problem.

- o Filter to waste
- o Slow initial filtration rate ("slow-start")
- o Polymer injection into backwash water

We recommend that as part of the filter improvements, measures be taken to control turbidity spikes following backwashing. The specific details will be determined in the detailed design phase. It should be easy to program slow initial start into the existing filter control system and so this method is likely to be favored.

2.6.10 FILTER AIDS

A polyelectrolyte filter aid may be added to the water just before passage into the filters to help control the performance of the filters. Filter aids tend to lengthen the filter run in terms of turbidity but reduce the run with respect to head loss. Using a filter aid adds one more variable that an operator can use to control filter behavior and maximize filter performance. It would be a relatively simple matter to install a polymer feed system so that filter aid could be fed upstream of the filters. It is recommended that filter pilot testing include an evaluation of filter aids.

2.6.11 FILTER MONITORING

Filter performance can be monitored by such parameters as:

- Head loss
- Turbidity
- Particle counting
- Filter run time

Head loss across the media has been a standard means of monitoring the filter performance. When the total available filtering head has been used, the filter run must be terminated. Head loss monitoring will be maintained.

All water treatment plants should continuously monitor the turbidity of the filtered water from each filter. The Surface Water Treatment Rule (SWTR) promulgated in 1989 by the Environmental Protection Agency (EPA) specifies certain turbidity monitoring requirements. These requirements are designed to confirm that filtration plants are well-operated and achieve maximum removal of Giardia cysts, viruses, bacteria, and turbidity. The SWTR requires that grab samples be taken at least every four hours from a representative location of the combined effluent of the filters, such as the clearwell influent or effluent. Alternatively, continuous turbidity monitoring may be used as long as the monitors are validated for accuracy on a regular basis.

Although not mandatory, the EPA's Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources recommends that "all filtration plants should provide continuous turbidity monitoring from each individual filter". It is possible that one or more filters may have a high effluent turbidity due to such problems as bed upsets, failure of media support or underdrain system, or excessive filtration rate due to a malfunction of the flow controller. Although the combined effluent from all filters may meet the SWTR requirements, the turbidity level from an individual filter may exceed the limits. This could result in the passage of Giardia cysts or other parthogens. Another reason the EPA recommends providing continuous turbidity monitors on all individual filters is to detect excessive turbidity spikes after backwashing.

CDM recommends the use of continuous turbidity monitors on all filters, in accordance with EPA recommendations and sound engineering practice. The estimated cost of installing fourteen continuous turbidimeters, one on each filter, is $42,000.

The number of particles in a unit of filtered water is monitored in some plants. Operators claim the particle count gives them an earlier warning of potential filter turbidity breakthrough than the conventional turbidimeter approach. Monitoring of particle counts is not recommended for the Wichita WTP.

Filter performance is often judged by the longevity of the filter run. However, too long a filter run may not be good for filter operation. Long filter runs make cleaning the filter that much more difficult because of the compaction of particulate matter. In addition, long filter runs indicate that the filter is not working at its most cost-effective capacity.

2.6.12 EVALUATION OF FILTER ALTERNATIVES

The two options for increasing the filtration capacity of the Wichita WTP are 1) upgrade the existing 14 filters or 2) add 6 new filters.

Whether or not the 6 new filters are added, it is recommended that the underdrain and media in the existing filters be replaced to provide the capability of air scour backwash and to reduce head loss. Air blowers and piping will also have to be added to supply the air for the air scour backwash. We also recommend that the filter control system instrumentation be replaced whether or not the additional filters are constructed. If the six new filters are added, it will not be necessary to replace the filter outlet piping.

Table 2-1 shows the cost comparison between the two filter expansion alternatives. The estimated costs for filter expansion alternatives 1 and 2 shown in Table 2-1 include an allowance for reusing the existing washwater troughs (which may not be necessary if the PCI pipe lateral underdrain is used), for minor rehabilitation of the backwash pumps, for installation of individual continuous turbidity monitors for a filter aid feed system, and for installing additional access walkways in the existing filter gallery. The specific details of these improvements will be worked out in the detailed design phase.

As shown in Table 2-1 upgrading the existing 14 filters is clearly the most economical alternative. The estimated cost of modifying the existing filters to provide a filtering capacity of 160 mgd is $6,144,000. Alternative 2 (adding six more filters) is more than twice this cost. It is therefore recommended that the existing 14 filters be upgraded to handle the 160-mgd design flow and no new filters be constructed.

TABLE 2-1
COST COMPARISON OF FILTER EXPANSION ALTERNATIVES

ALTERNATIVE 1 - UPGRADE 14 EXISTING FILTERS

Replace underdrains & media	$3,206,000
Raise washwater troughs	492,000
Replace filter outlet piping & fittings	504,000
Install air scour blowers and piping	110,000
Rehabilitate backwash pumps	40,000
Install access walkway in filter gallery	50,000
Replace filter control system	650,000
Install continuous turbidity monitors	42,000
Install filter aid feed system	25,000
Contingencies (20%)	1,025,000
ESTIMATED COST OF ALTERNATIVE "1"	$6,144,000

ALTERNATIVE 2 - REHABILITATE 14 EXISTING FILTERS & ADD 6 NEW FILTERS

Replace existing underdrains & media	$3,206,000
Raise washwater troughs	492,000
Construct 6 new filters, including structure, underdrains, media, washwater troughs, piping & fittings	5,880,000
Install air scour blowers and piping	110,000
Rehabilitate backwash pumps	40,000
Install access walkway in filter gallery	50,000
New filter control system	650,000
Install continuous turbidity monitors	60,000
Install filter aid feed system	25,000
Contingencies (20%)	2,103,000
ESTIMATED COST OF ALTERNATIVE "2"	$12,616,000

2.7 DISINFECTION

2.7.1 GENERAL

In terms of public health, disinfection is the most important water treatment process. Disinfection destroys or inactivates bacteria, viruses, and protozoa responsible for acute diseases such as gastroenteritis, hepatitis, and giardiasis. Poor disinfection practices could seriously compromise public health. The disinfection process, on the other hand, may also create by-products with possible long-term health effects. Of special concern are the disinfection by-products (DBPs) produced by free chlorination, particularly trihalomethanes (THMs) which are formed when free chlorine reacts with naturally occurring organic matter in surface waters.

The current EPA regulations include a maximum contaminant level for total trihalomethanes (TTHMs) at 100 ug/l. It is anticipated that future DBP regulations will limit TTHMs to 25 - 50 ug/l, with 50 ug/l being the probable limit. THMs are not the only DBPs which have come under scrutiny. Future regulations of DBPs will most likely include by-products of other disinfectants such as ozone and chlorine dioxide as well.

The available means for disinfection acceptable to the EPA are greatly affected by both the current TTHM MCL and more recently the "CT" values published in the Surface Water Treatment Rule (SWTR). The SWTR has specific requirements for removal/inactivation of Giardia and enteric viruses. These requirements include filtration and disinfection for surface waters and groundwaters which are influenced by surface water, which generally are most groundwaters.

The disinfection requirements are established based upon the CT value for individual disinfectants. The CT value is the product of the specific disinfectant concentration, C (in mg/l), and the contact time, T (in minutes), required for inactivation of a certain percentage of the microorganism under specified pH and temperature conditions. A disinfection scheme must be used which will meet the CT value requirements, minimize DBP (specifically THM) formation, and provide a disinfectant residual in the distribution system.

2.7.2 TRIHALOMETHANE/DBP CONTROL

THMs can be controlled by three general methods:

- Removal of organic precursors before contact with chlorine and subsequent THM formation.
- Removal of THMs after formation.
- Disinfection of the water with an alternative means which does not cause THM formation.

Removal of THM Precursors

The reduction in concentration of naturally occurring organic substances can be accomplished with the following processes:

- Conventional settling and filtration. With a coagulant and polymer aid, sedimentation and filtration will reduce the levels to some extent.
- Adsorption onto activated carbon. If this process is used, either powdered activated carbon (PAC) must be injected prior to prechlorination, or granular activated carbon (GAC) filter beds must be used. If GAC filter beds are used, prechlorination should not be practiced. Without very specific raw water quality information, the expected life of a GAC bed used for precursor removal cannot be accurately determined.
- Oxidation of precursors. The concentration of precursors may be reduced by adding non-halogen oxidants, such as ozone or potassium permanganate. Ozonation systems, while usually highly effective, are expensive to install and operate. Ozone will, however, alleviate the need for prechlorination in addition to oxidizing precursors. Both ozone and permanganate are used to oxidize organics which cause taste and odor problems, and they aid in color removal. In some cases, however, ozonation has resulted in THM formation following chlorination reaching higher levels than without the use of ozone.

Removal of THMs After Formation

THMs usually take time to completely form. Thus, even if conditions are suitable for their formation, they may not be completely formed in a treatment plant, but may instead complete their formation in the distribution system. Where THMs have developed, several removal methods are effective:

- Adsorption onto activated carbon. THMs do not absorb onto carbon as well as precursors, hence greater dosages of PAC are required, or if GAC beds are used, they must be replaced more frequently, as THMs will begin to pass through the beds in a few weeks of operation. Regeneration periods for the carbon beds may be three months or less, compared to up to two years for typical precursor removal.

- Aeration of the THMs. This process transfers the THMs into the surrounding air. Only the volatile compounds are removed, which include THMs but not precursors. Thus, THM formation may occur again later if sufficient time has not elapsed since the water was chlorinated for THM formation to be complete.

- Adsorbant Resins. Certain adsorbant resins are being developed, some of which show an affinity for THMs and other low molecular weight chlorinated organics. Unreacted precursors will, however, generally not adsorb onto these resins, and as above THM formation could occur again later upon subsequent chlorination.

Alternative Disinfectants

Certain disinfectants, such as ozone and chlorine dioxide, do not cause formation of THMs when contacting the precursors, while others such as chloramines produce only very low levels of THMs. The precursors are not necessarily removed, although in some cases they may be reduced. Therefore, the danger of high THM formation remains if free chlorine is introduced at a downstream point. The use of these alternative disinfectants in place of free chlorination is discussed below.

- Ozonation. Ozone is a powerful disinfectant which is considerably more effective than chlorine, particularly with regard to viruses. The oxidizing power of ozone is far greater than that of chlorine; and therefore, it is usually much more effective for the reduction of color and most types of tastes and odors. Generally, ozone reacts with the precursors of THMs making them less susceptible to reactions with chlorine and more susceptible to removal by coagulation and filtration. The main disadvantages of using ozone are that it must be generated on site using relatively expensive equipment, and it does not persist for very long, so no long-term residual can be maintained. If a disinfecting residual is required, post disinfection, preferably with chloramines, is necessary.

  If hydrogen peroxide is used with ozone, hydroxyl radicals are formed, which react more rapidly with organic compounds than ozone alone. However, the EPA allows a CT credit for disinfection for free ozone only. The benefits of perozonation, as it is sometimes called, applies to reducing taste-and-odor-producing organic compounds.

- Chlorine Dioxide. Like ozone, chlorine dioxide is usually more effective than chlorine for taste and odor removal, has an oxidizing capacity about two and one half times greater than chlorine, and does not produce trihalomethanes. Also, like ozone, chlorine dioxide--a gas--must be generated on site.

  Generating chlorine dioxide is not difficult. It can be produced by adding chlorinated water from a chlorinator to a solution of sodium chlorite. While it is theoretically possible to add just enough chlorine to react with the sodium chlorite, in practice an excess of chlorine is normally used.

  Although this excess chlorine may be sufficient to allow for a free chlorine residual in the water being treated, it produces fewer THMs than would be obtained with chlorine alone. Capital costs for chlorine dioxide generating equipment are low, but operating costs are high, due to the high cost of sodium chlorite. A major concern about the use of chlorine dioxide are the by-products of chlorites and chlorates which may pose health problems and are likely to be regulated in the future.

- Chloramines. When chlorine and ammonia are both added to water, they react to form products collectively known as chloramines. Chloramines are often referred to as "combined chlorine", as opposed to "free chlorine". The process of adding chlorine and ammonia to form chloramines is called chloramination. Chloramines are far less effective as a disinfectant than free chlorine or chlorine dioxide. However, chloramines produce significantly lower levels of THMs than does chlorine. Chloramines are adequate when disinfection requirements are not substantial. The high CT requirements for chloramines limits their use as a primary disinfectant. Chloramines also provide a longer lasting residual than chlorine, but they are not very effective for the control of tastes and odors.

2.7.3 CURRENT DISINFECTION PRACTICE

Disinfection at the Wichita Water Treatment Plant is currently performed by adding free chlorine to the surface water at the Cheney Pump Station and to the groundwater at the wellfield. Combined chlorine residual is provided in the distribution system by adding chlorine and ammonia prior to filtration to form chloramines. TTHMS in the distribution system fall below the maximum level of 100 ug/l. However, in some instances the TTHMs have exceeded the goal of 50 ug/l. Therefore, modifications to the existing disinfection practice must be made for the future plant improvements.

2.7.4 DISINFECTION SCHEME ALTERNATIVES

If the current use of free chlorine through the plant (up to the filters) is to be continued, the THM precursors must be removed prior to chlorine addition or the THMs must be removed after formation. Removing the THM precursors is not feasible since it is desirable to add chlorine at the Cheney Pump Station for oxidation and disinfection purposes. Removal of THMs after formation would require the addition of carbon absorbers following treatment or the use of GAC media in the filters. The carbon adsorbers, as well as being cost prohibitive, would also require significant hydraulic modifications to route water from the filters through the adsorbers and back to the clearwell. The addition of GAC media to the filters would require periodic regeneration of the media, an operational problem, but more importantly, the existing filter boxes are not deep enough to be modified to provide sufficient empty bed contact time. Aeration, currently practiced, most likely removes some THMs, but does not remove sufficient amounts to meet the TTHM goal.

Therefore, alternative disinfectants must be used. Based on historical data and bench scale studies, it is evident that the use of free chlorine can be continued for disinfection, but not for the length of time it is currently used. To maintain our goal of 50 ug/l TTHM, ammonia should be added in the Cheney pipeline downstream of the free chlorine addition to form chloramines and reduce the formation of THMs. The ammonia should be added in the pipeline a distance downstream of the chlorine addition to assure that the appropriate CT value is met for disinfection with free chlorine. This is approximately one mile from the Cheney Pump Station. The THM formation potential for the groundwater is minimal, and therefore ammonia addition is not required prior to treatment.

The CT value required for chloramine disinfection is on the order of 15 to 20 times that of free chlorine, thereby eliminating chloramines as a possible means for primary disinfection. The use of ozone or chlorine dioxide are considerably more costly than chlorine/chloramines and can be eliminated from an economic standpoint. It should also be noted that the by-products from chlorine dioxide (chlorates and chlorites) are highly undesirable.

2.7.5 RECOMMENDATION

Based upon the information provided above, it is recommended that free chlorine be used as the primary disinfectant for both the surface water from Cheney Reservoir and the groundwater from the Equus Bed wellfields.

Free chlorine should be added as currently practiced at both locations. Further, it is recommended that ammonia be added to the 60-inch Cheney pipeline approximately one mile east of the pump station to allow time to meet the disinfection CT values and to cease the formation of THMs. The current practice of chloramination prior to filtration should also be continued to provide and maintain sufficient chlorine residual in the distribution system.

2.8 CHEMICAL FEED SYSTEMS

2.8.1 GENERAL

The existing East and Central plants each currently have operational feed systems for lime (calcium oxide), cationic polymer, and sodium hexametaphosphate. Common storage facilities and feed equipment for ammonia, chlorine, and carbon dioxide are located at the Central plant for service to both the Central and East plants. In addition, Central plant contains unused storage facilities for both lime and granular ferric, or aluminum, sulfate.

2.8.2 LIME FEED SYSTEM

The City of Wichita currently practices, and will continue to practice, lime softening at the Wichita WTP. The raw water entering the Wichita WTP contains almost all carbonate hardness (i.e., calcium bicarbonate) with very little, if any, non-carbonate hardness (i.e., calcium or magnesium sulfate and calcium chloride). The treatment goal is to reduce the raw water hardness from between 190 to 250 mg/l (as $CaCO_3$) to a finished water hardness between 100 to 105 mg/l. A reduction in raw water alkalinity from between 180 to 230 mg/l to a finished water alkalinity between 100 to 105 mg/l is also desired.

Water can be softened by using one of two types of commercial lime. The first is quicklime, or calcium oxide (CaO), which is the product resulting from the calcination of limestone in kilns at temperatures of 2,000 to 2,400 °F. The second is hydrated lime, or calcium hydroxide [$Ca(OH)_2$], which is a very finely divided powder resulting from the hydration of quicklime with enough water to satisfy its chemical affinity. Quicklime has advantages over hydrated lime in that it: 1) is less costly since the hydration process is not accomplished by the supplier, 2) requires less storage volume for equivalent available calcium, and 3) is pebble form, has better flowability, less compressibility, and less dust relative to storage and feeding. The advantages of hydrated lime over quicklime are: 1) hydrated lime is purer, 2) on-site slaking is not necessary, and 3) hydrated lime will not air slake, which can cause pebble quicklime to glaze and bind in long-term storage.

The advantages of quicklime far outweigh the advantages of hydrated lime for a facility that uses large quantities of lime. Therefore, we recommend that softening continue to be accomplished by slaking quicklime on-site using plant service water to form a "milk of lime" (hydrated lime) slurry which is added to the raw water at the rapid mix basin(s).

Storage curves for the existing lime facilities at both East plant and Central plant are shown in Figure's 2-13 and 2-14 respectively. The current lime storage facilities are slightly under capacity for average lime dosages at average design plant flows of 15 mgd for East plant and 100 mgd for Central plant. The State presently requires a minimum 30 day chemical storage for water treatment plants. If it can be shown that lime suppliers can replenish this supply on a frequent enough basis as required by the plant, the State may allow a variance to the 30 day guideline. In addition, an added 775,500 pounds unused storage capacity is available at Central plant, so only the East plant would require such a variance.

The lime feed equipment at each plant has experienced rapid belt wear and routine shear pin breakage on the grit conveyors. It is recommended that the lime storage facilities and feed equipment at each plant be upgraded and/or replaced as part of the overall plant improvements.

2.8.3 COAGULANT FEED SYSTEM

As reported in the Discussion Paper: Treatment Plant Studies, the jar tests determined that both a cationic polymer and ferric sulfate proved effective as a coagulant for the Wichita WTP. Further analysis of the two coagulants reveal that the cationic polymer (Calgon Cat-Floc T) is significantly less expensive with regard to chemical costs, as shown in Figure 2-15. Assuming an average Cat-Floc T dose equal to 0.5 mg/l and an average ferric sulfate dose equal to 10 mg/l, the annual savings in chemical costs for using polymer instead of ferric sulfate is $189,000 (based on 1990 chemical costs) for an average total plant flow of 100 mgd.

FIGURE 2-13
Lime On-Site Storage Time
East Plant

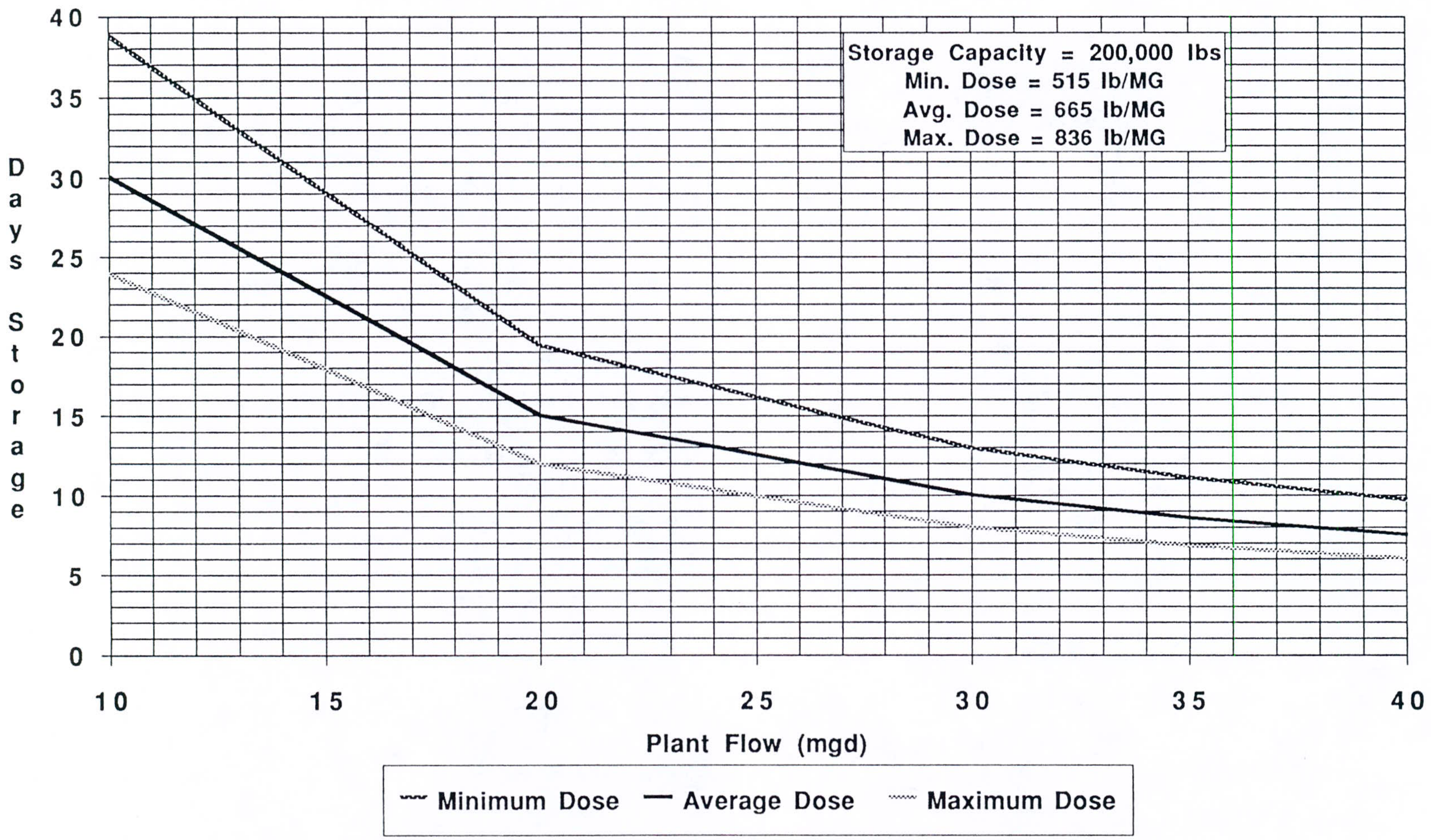


FIGURE 2-14
Lime On-Site Storage Time
Central Plant

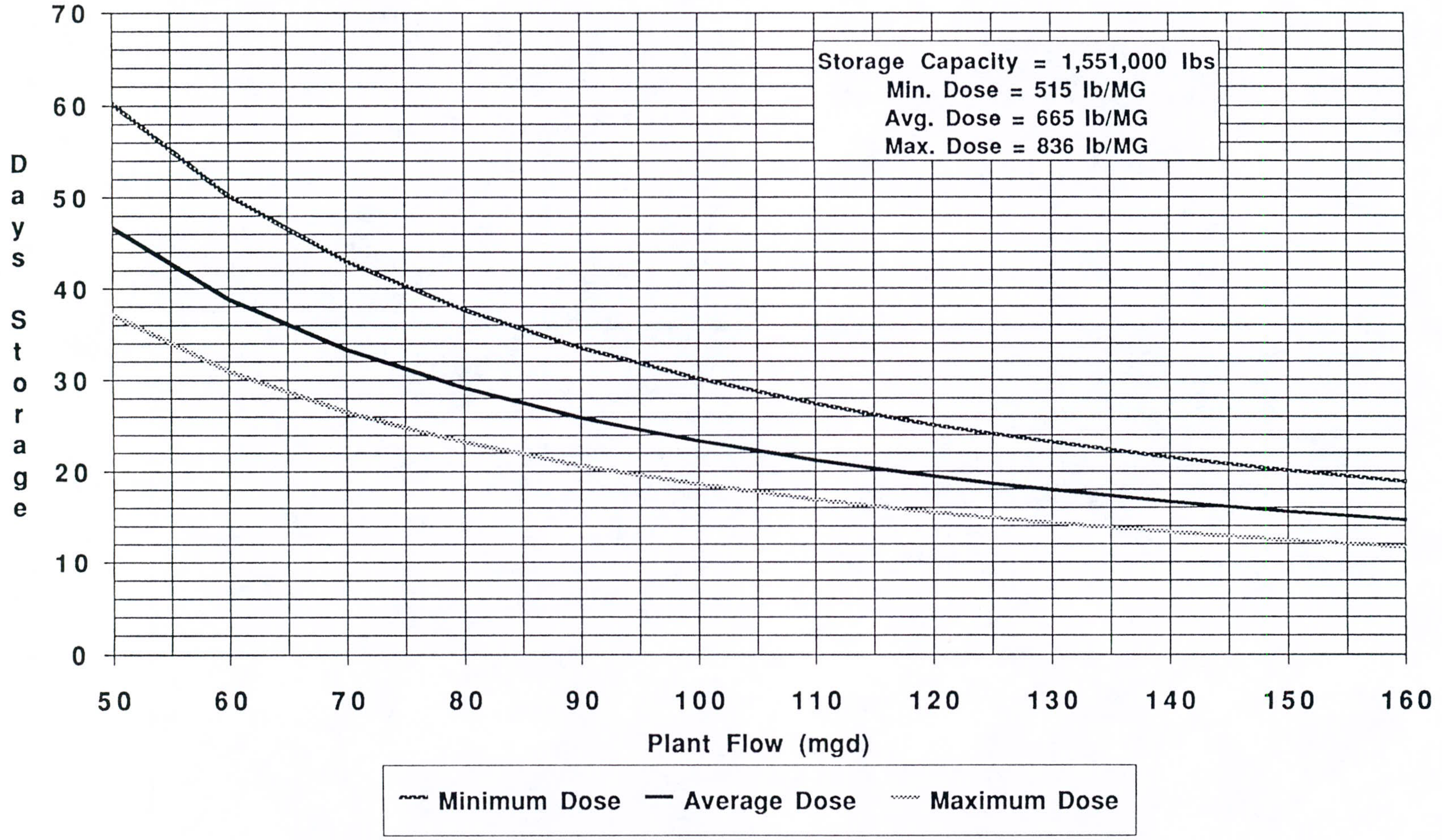
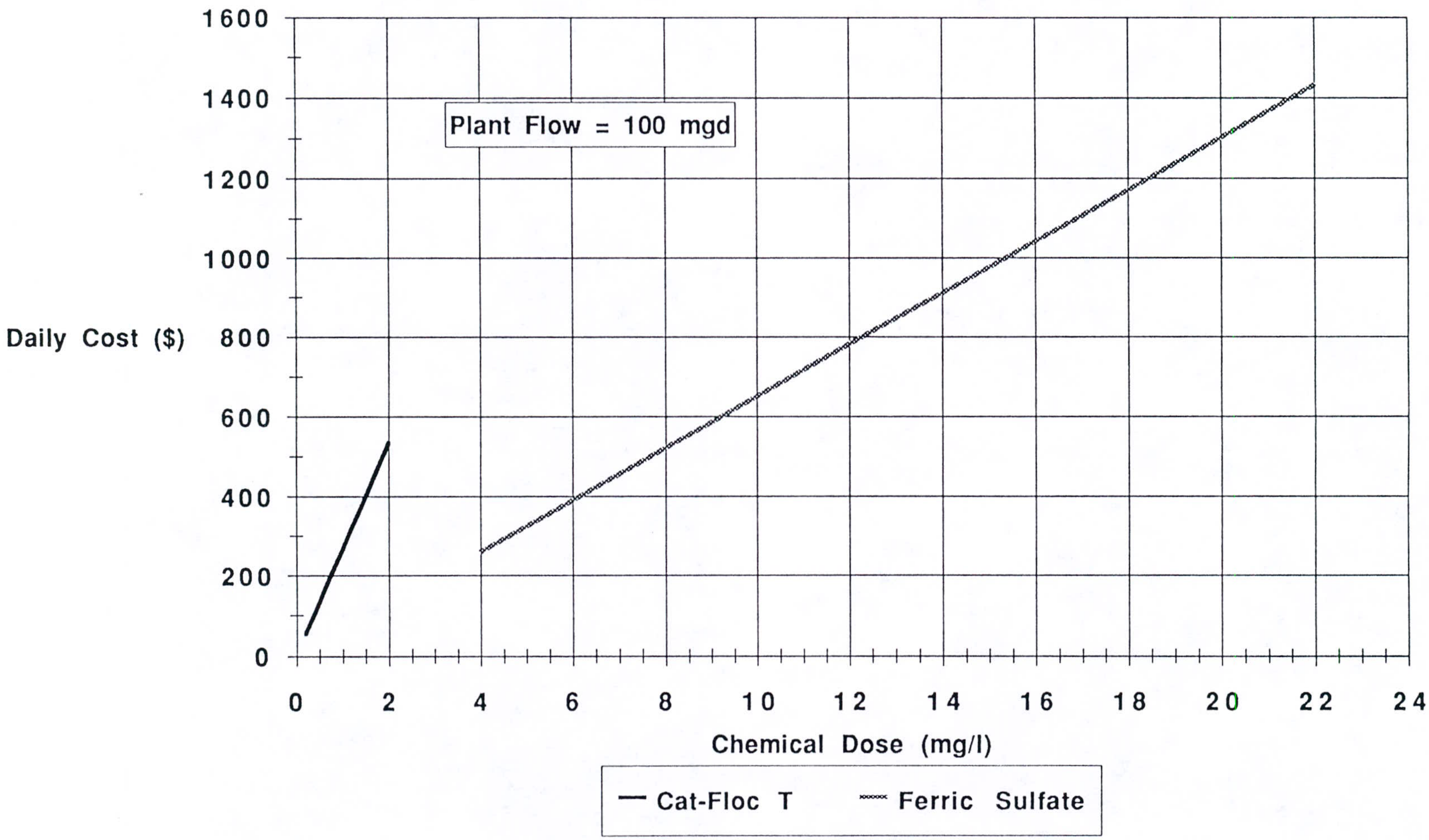


FIGURE 2-15
Coagulant Evaluation
Daily Costs of Cat-Floc T and Ferric Sulfate



In addition to the chemical cost advantage of using a cationic polymer, this coagulant has the benefit of more than adequate chemical storage at both East plant and Central plant, as shown in Figures 2-16 and 2-17. Based on the above information, we recommend the continued use of a cationic polymer as the coagulant for the Wichita WTP. It is further recommended that the cationic polymer feed systems at each plant be upgraded and replaced as part of the overall plant improvements.

2.8.4 DISINFECTION FEED SYSTEM

Chlorine and ammonia will continue to be used at the Wichita WTP as a method of maintaining a minimum combined chlorine residual of 1.0 mg/l at the farthest points in the distribution system. The existing chlorine and ammonia storage capacities are shown in Figure's 2-18 and 2-19, respectively. The existing ammonia storage tank is more than adequate in meeting the State's minimum 30 day storage requirement. The existing chlorine storage facilities do not meet the minimum 30 day storage criteria; however, since the chlorine is supplied in 1-ton cylinders, the State may allow a variance to this guideline if it can be shown that chlorine suppliers can replenish this supply on a frequent enough basis as required by the plant.

The existing chlorine and ammonia feed equipment is 20 to 30 years old. Replacement part's availability is a concern as well as the high costs in doing so. It is recommended that the chlorine and ammonia feed systems be replaced in the overall plant improvements.

2.8.5 STABILIZATION FEED SYSTEMS

Presently, carbon dioxide is used at the Wichita WTP for conversion of insoluble calcium carbonate to soluble calcium bicarbonate (i.e., recarbonation). A storage curve for the existing carbon dioxide storage tank is shown in Figure 2-20. For an average dose at an average combined plant flow, the existing storage tank does not meet the minimum 30 day

FIGURE 2-16
Polymer On-Site Storage Time
East Plant

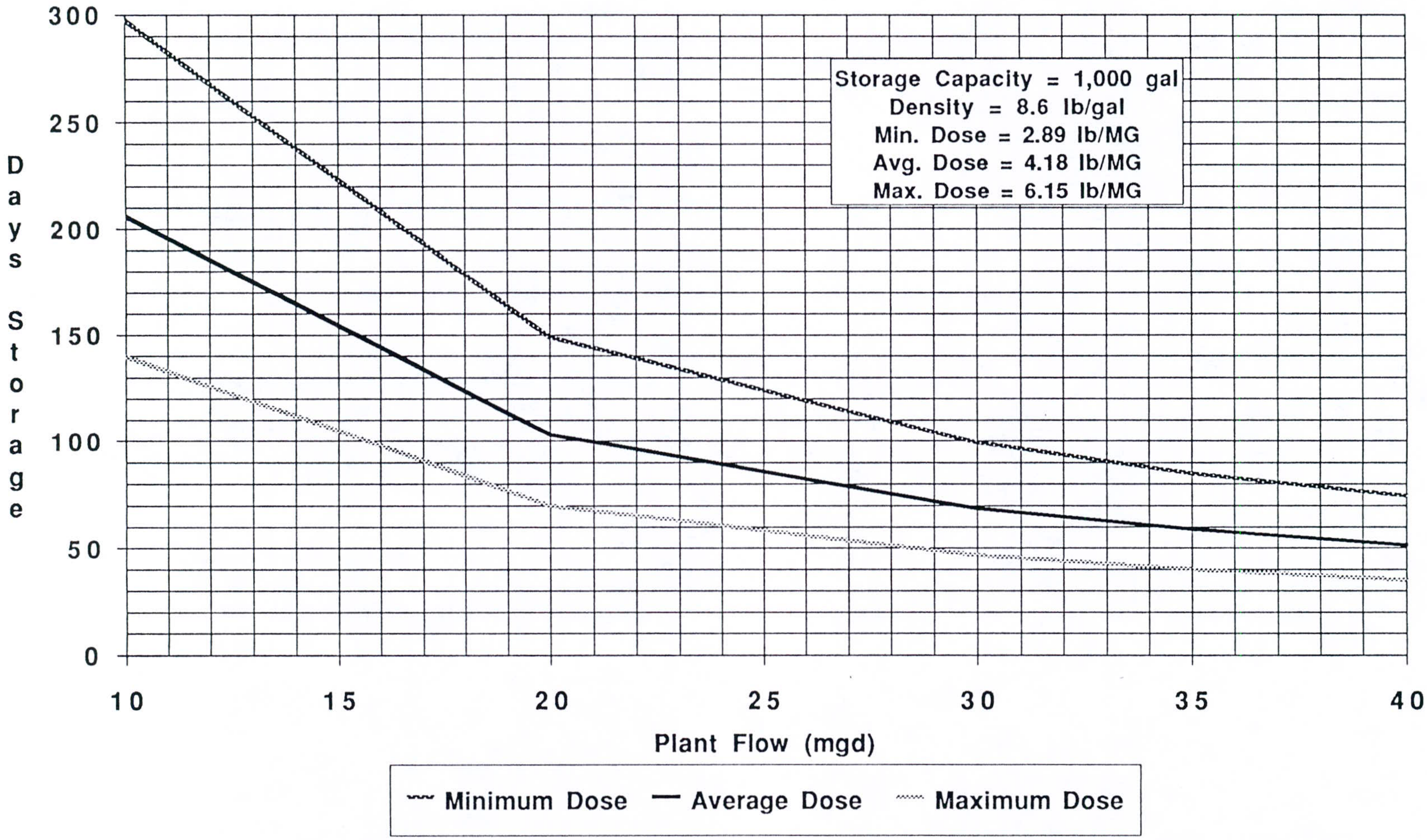


FIGURE 2-17
Polymer On-Site Storage Time
Central Plant

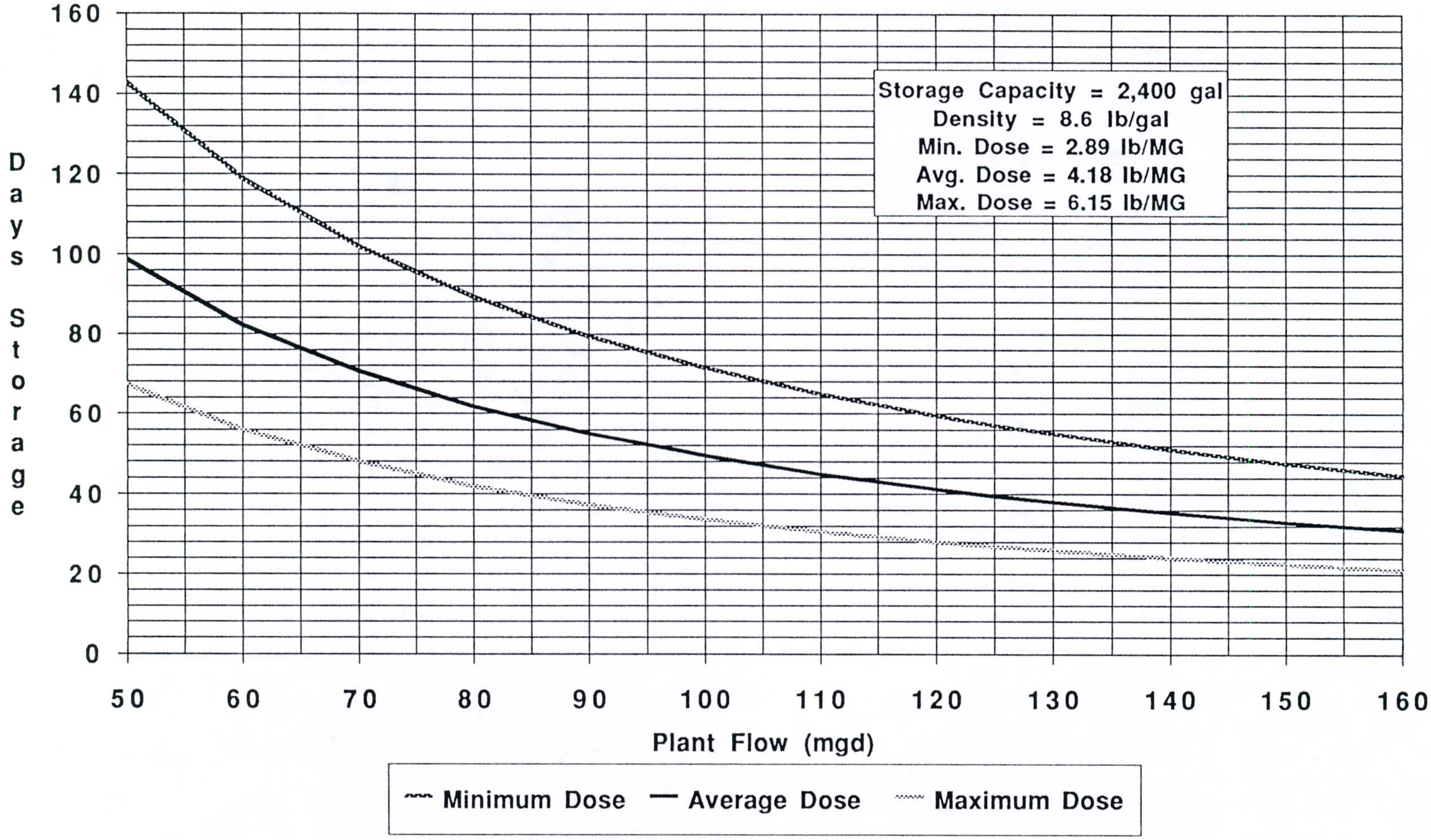


FIGURE 2-18
Chlorine On-Site Storage Time
Combined Central Plant and East Plant

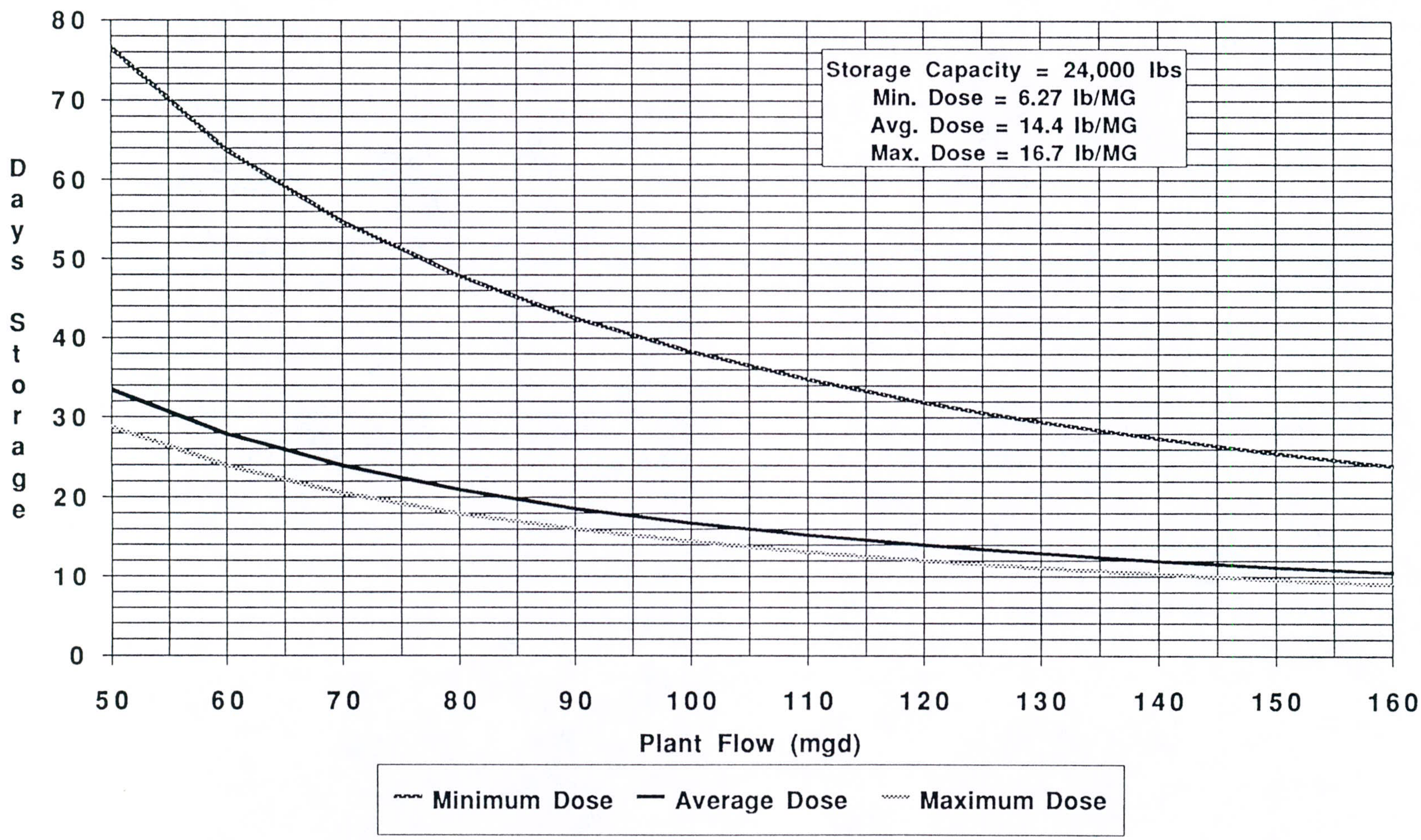


FIGURE 2-19
Ammonia On-Site Storage Time
Combined Central Plant and East Plant

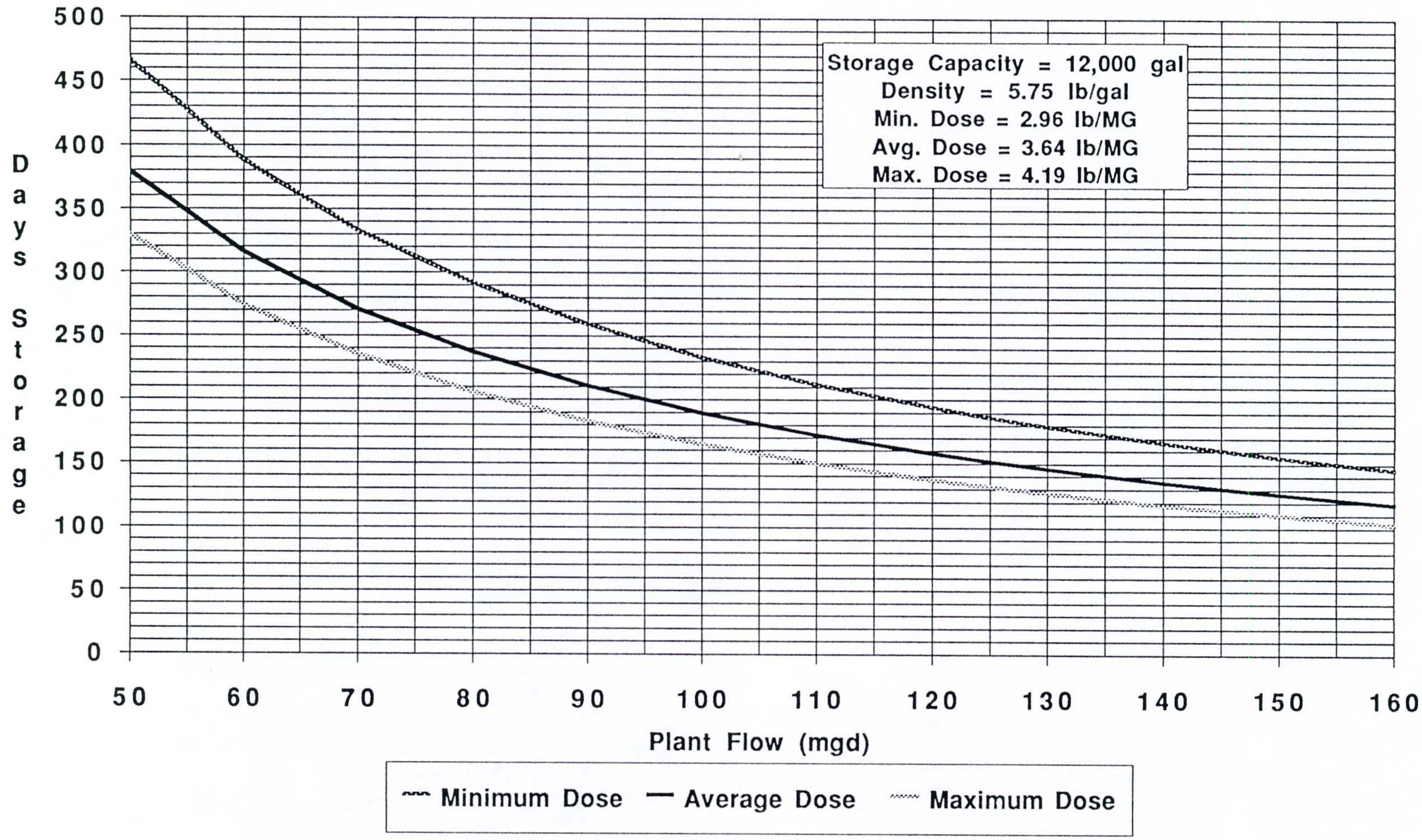
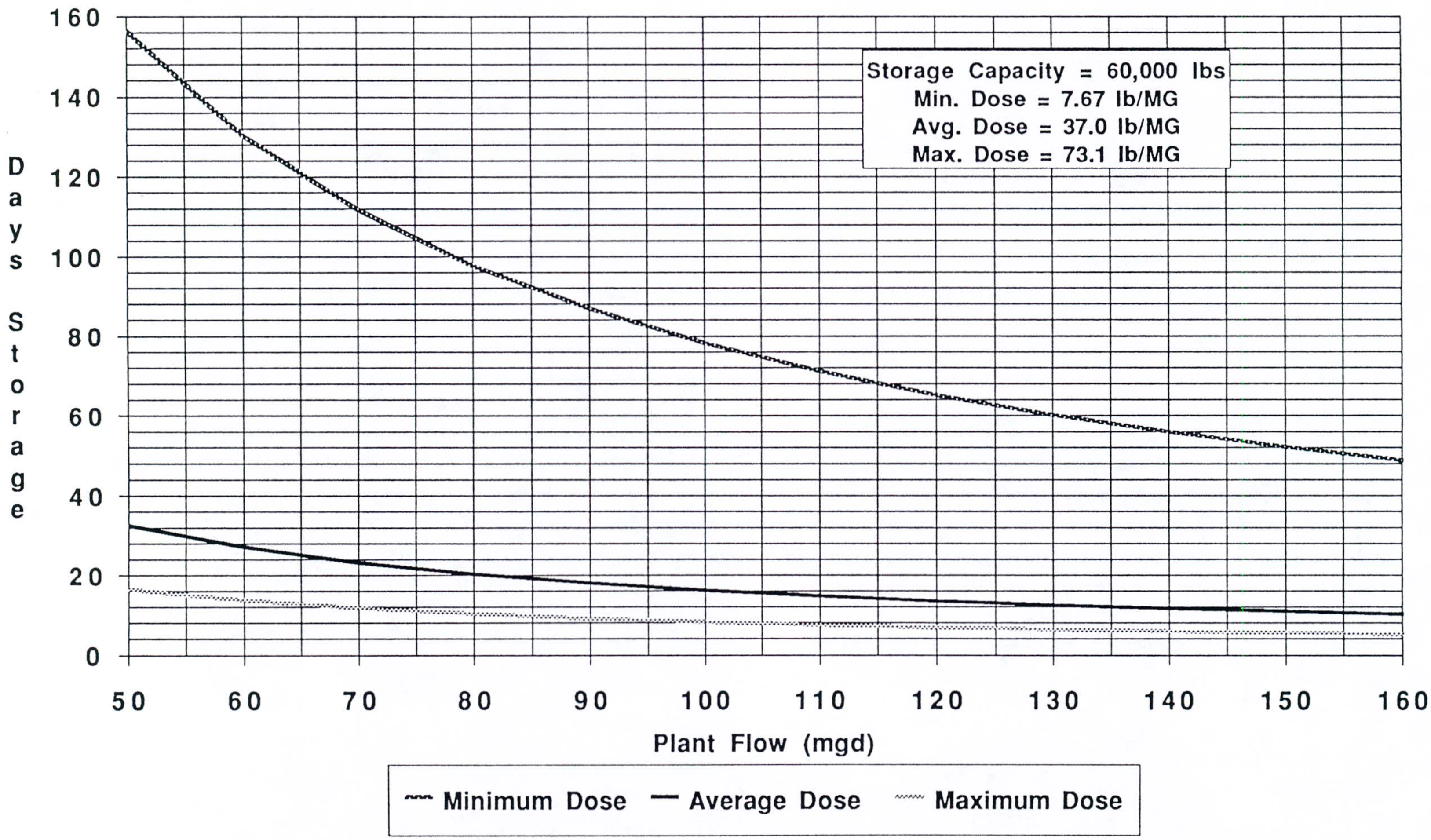


FIGURE 2-20
Carbon Dioxide On-Site Storage Time
Combined Central Plant and East Plant



requirement. Again, the State may allow a variance to this guideline if it can be shown that carbon dioxide suppliers can replenish this supply on a frequent enough basis as required by the plant.

Sodium hexametaphosphate is also used at both Central and East plants for added stabilization and as a corrosion inhibitor in the distribution lines. From the storage curves, shown in Figure's 2-21 and 2-22, it is obvious that the current hexametaphosphate storage is more than adequate.

It is recommended that the carbon dioxide and hexametaphosphate feed systems be upgraded and replaced in the plant improvements.

FIGURE 2-21
Hexametaphosphate On-Site Storage Time
East Plant

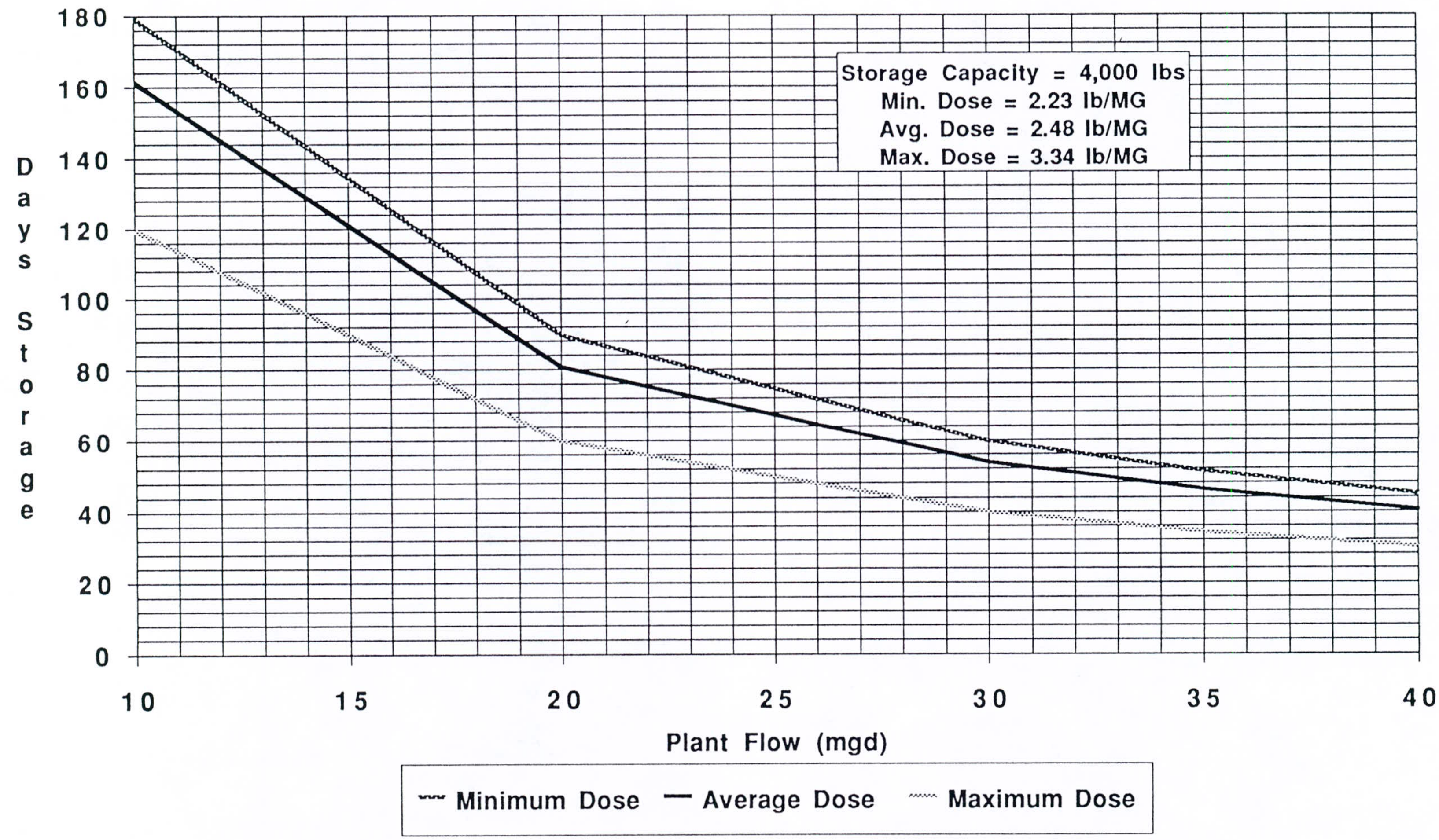
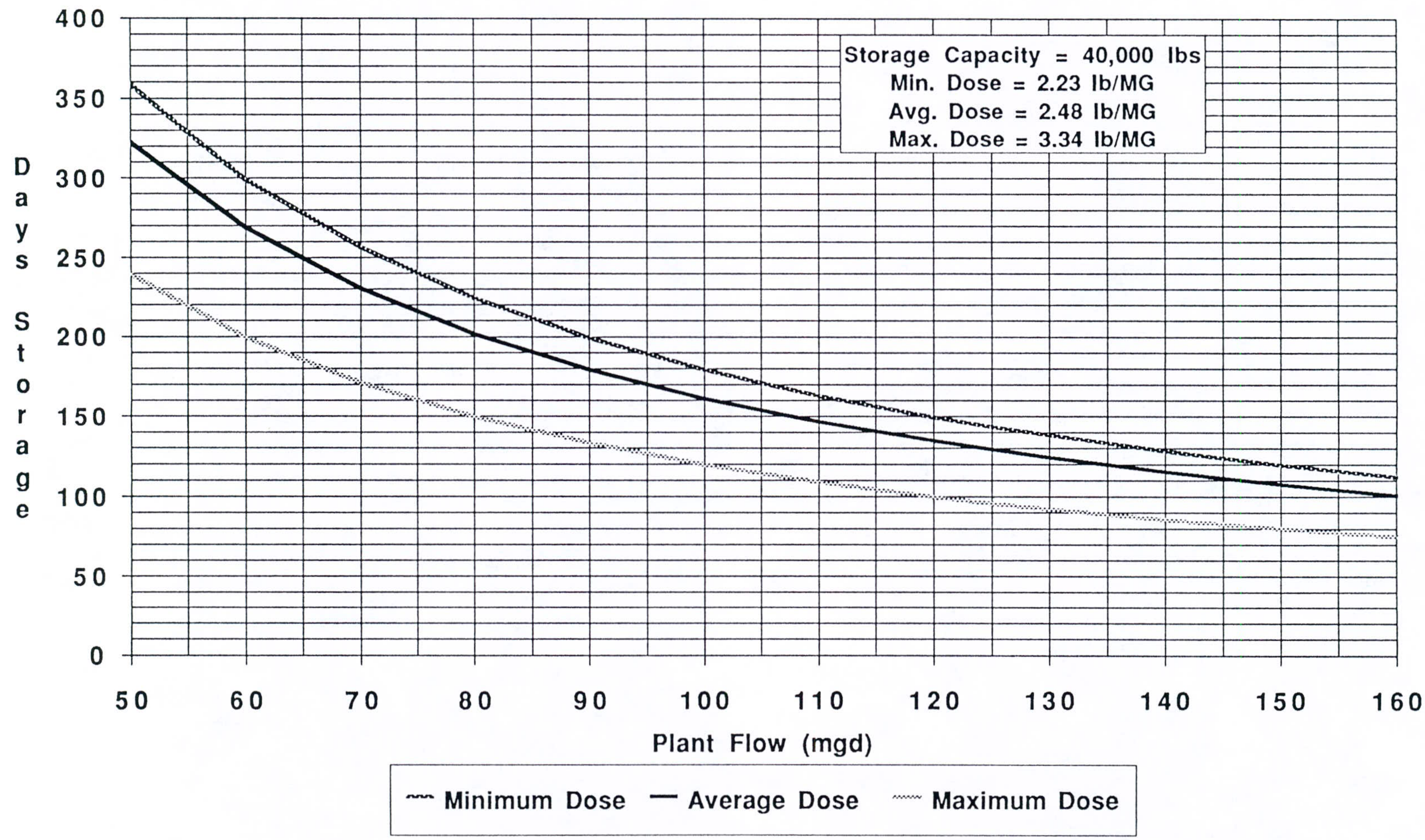


FIGURE 2-22
Hexametaphosphate On-Site Storage Time
Central Plant



2.9 HYDRAULIC IMPROVEMENTS

2.9.1 GENERAL

A hydraulic model for the existing water treatment plants was developed and discussed in Status Report: Treatment Plant Evaluation. This model has been used to determine hydraulic improvements to the existing facilities which must be made in order to effectively treat desired future flows through the plant. The model was run to evaluate 30 mgd through the East plant and 130 mgd through the Central plant, as well as running 160 mgd through the Central plant. Results of the model for these two runs are presented in Appendix A. Hydraulic improvements discussed below are dependent upon the flow to be treated through the two plants. These improvements will be incorporated into the alternative analyses presented in Section 3.0.

2.9.1 EAST PLANT

Inlet Piping

The raw water flows to the East plant through a 48-inch pipeline and is measured with a 48-inch venturi meter with a 20-inch throat. The piping downstream of the venturi results in high velocities at a flowrate of 30 mgd. When operating the East plant at high flows, excessive noise is created due to these high velocities. In order to reduce the velocities through the inlet piping and subsequent headlosses and noise problems, it is recommended to replace the existing 24-inch piping and control valve located in the basement area with 30-inch size. Figure 2-23 presents the proposed modifications.

Primary Settling Basins

Flow from the flocculator basins travels to the two primary settling basins. These basins have different surface areas and thus different treatment capacities. The theoretical capacity of the newer basin (east basin) is about twice the capacity of the older basin (west basin).

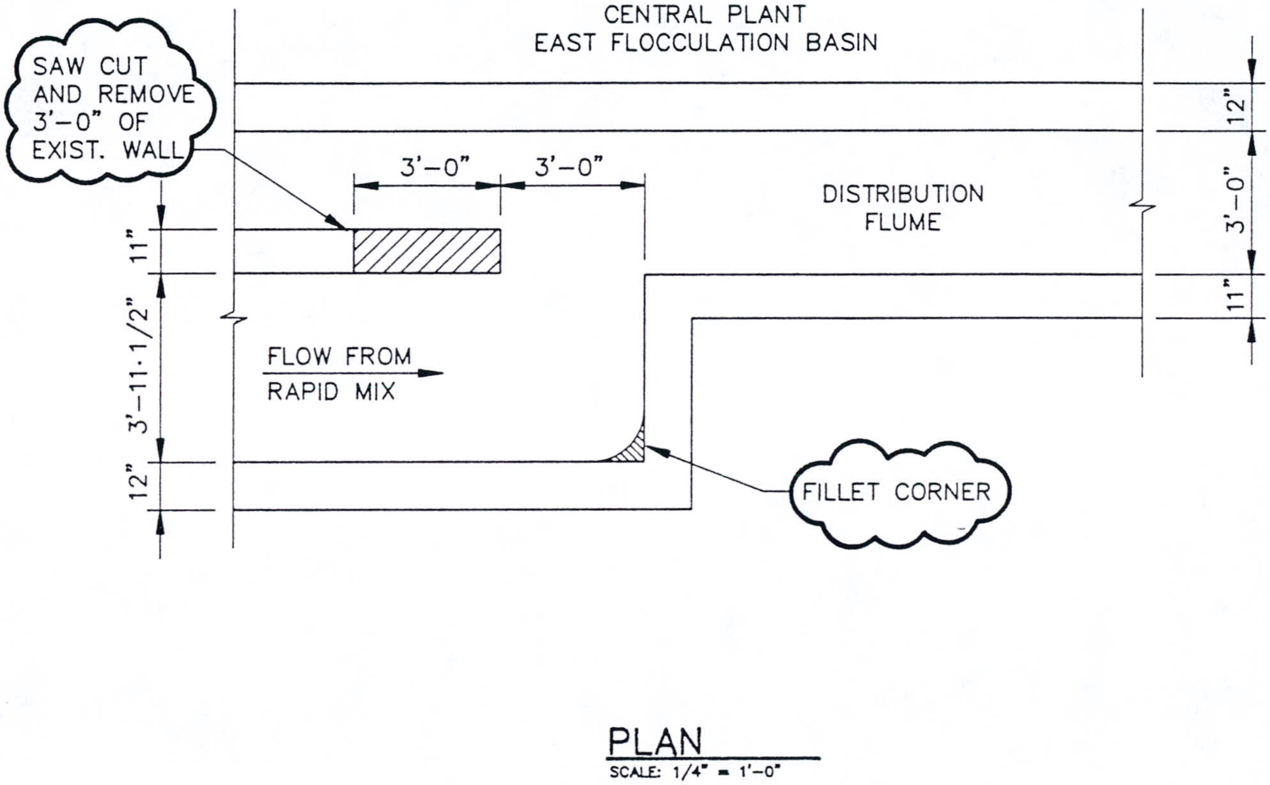


FIGURE 2–23
CENTRAL PLANT
DISTRIBUTION FLUME MODIFICATIONS

Therefore flow should be split such that 1/3 of the flow goes to the west basin and 2/3 of the flow goes to the east basin. The existing plans indicate that weir plates were set to try and accomplish the feat of flow splitting. The condition of the existing weir plates are such that it is recommended to replace the existing weir plates and to set them at the elevations required to achieve desired flow splitting. A control gate may also be added to fine tune the flow split. Currently, the inlet gate to the west basin is used to attempt flow splitting.

Secondary Basin Inlet Weir

Settled water from the primary settling basins is collected and travels over a weir into the secondary basin. This weir has no apparent purpose, but causes flooding of the upstream primary basin launders at high flows. It is recommended to core portholes in this weir wall which will reduce total headloss through the plant, as well as encourage plug flow through the secondary basin.

2.9.3 CENTRAL PLANT

Flocculation Basin Flow Splitting

The flow from the rapid mix basins to the flocculation basins must be split evenly to insure relatively equal flow through the two process trains. This split is necessary to prevent overloading of the flocculation/sedimentation process of one of the trains. The existing arrangement of rapid mix channels and distribution flumes will not achieve an even flow split at current or future flow rates. Therefore modifications to the channel upstream of the rapid mixers will be modified to assure proper flow splitting.

Flocculation Basin Inlet Flume

Considerable headlosses are encountered at inlets to the flocculation distribution flumes from the rapid mix channel. At 80 mgd, a headloss of approximately 10 to 12 inches will occur. Headlosses of up to 18 inches will occur at flows of 160 mgd. The inlet conditions must be modified for all alternatives for expansion to reduce these headlosses. Figure 2-24 presents the proposed modifications.

Sedimentation Basin Launders and Weirs

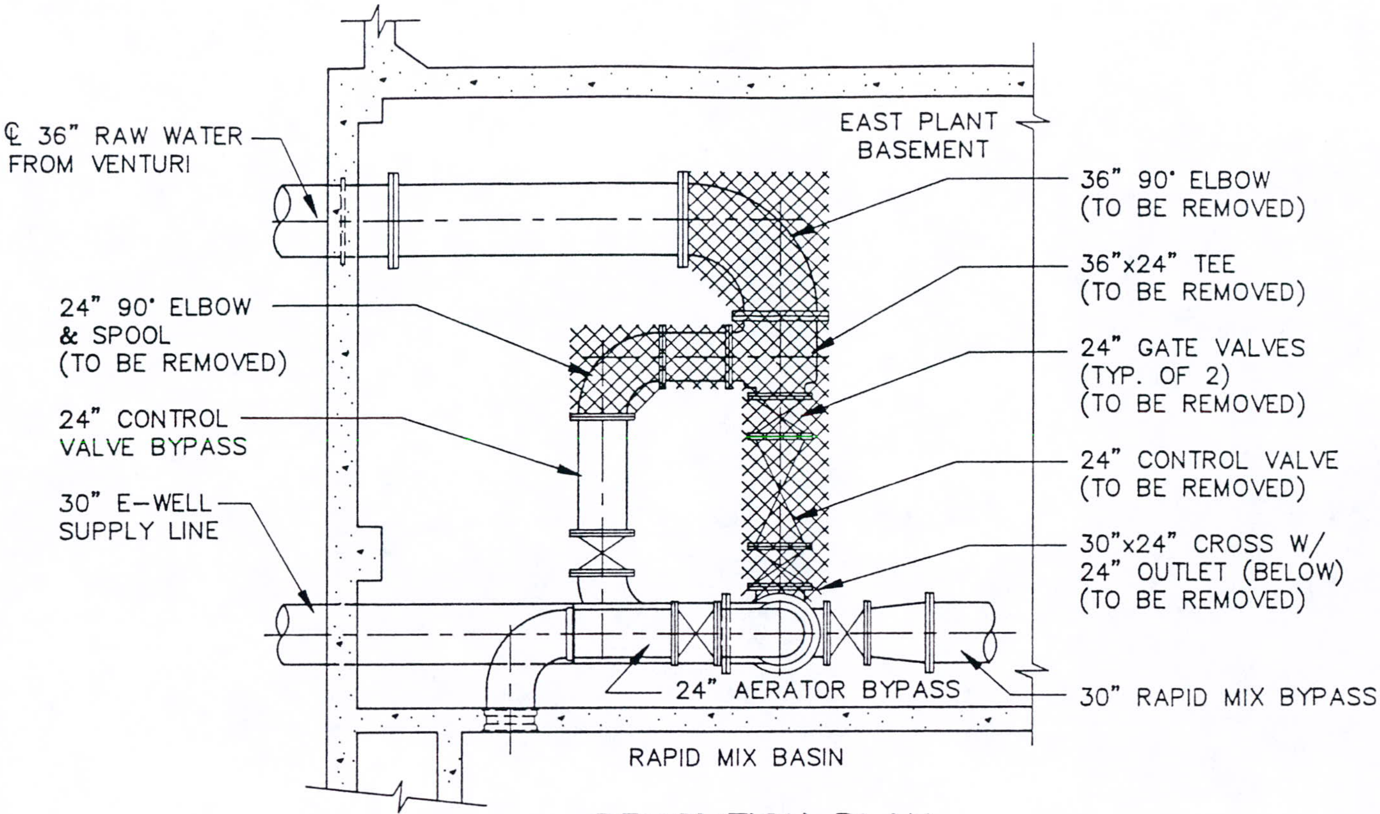
The existing outlet launders and weirs were designed for a total flow of 80 mgd. At flows approaching 80 mgd or greater, the weirs operate in a submerged condition, which further reduces the effectiveness of the sedimentation basin process. As discussed in preceding sections, removal of these launders and provisions for an outlet baffle wall will improve sedimentation. It will also reduce the overall plant headloss caused by the launders.

The launders are also flooded during high flows because of downstream headlosses associated with the 84-inch settled water line. This flooding condition will occur in the outlet channel even when the outlet baffle wall replaces the launders. Therefore, modifications must be made. A parallel 84-inch pipeline is recommended for alternatives where 160 mgd is to be treated by the Central plant. A parallel 60-inch pipeline is recommended for alternatives where 130 mgd is to be treated by the Central plant.

2.9.4 FILTERS

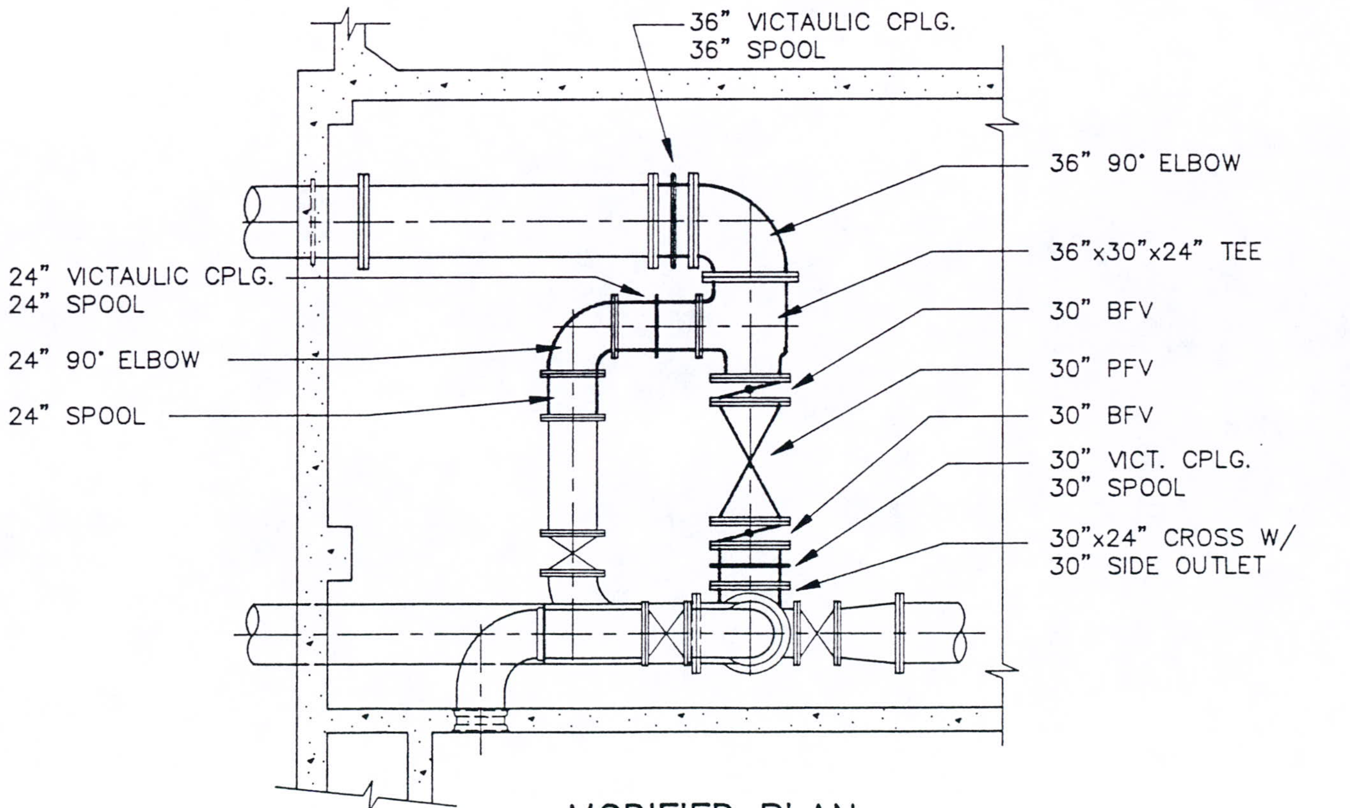
The filters and their design hydraulics were discussed in a preceding section. However, two additional hydraulic concerns must be addressed. The existing filter inlet channels to filters 1-6 and filters 7-14 are connected by a 60-inch pipeline. This restriction causes a headloss of about three inches when 60 mgd is treated through the Central plant and no flow through the East plant. The headloss is accentuated when greater flow is treated through the Central plant. Headlosses much greater than this cannot be tolerated if efficient filter operation is to be expected. It should be noted that when the East plant is providing a portion of the flow to the filters, the headloss at the filters is less since less flow passes through the 60-inch connection. It is recommended to modify the existing filter inlet channel by removing the 60-inch connection and connecting the two channels as one.

The existing filter and clearwell design presents a hydraulic problem between plant operation and storage capacity. The filter control weir, located in the clearwell is set at elevation EL. 108.00 and the maximum water level for the clearwell is El. 113.50. If the clearwell and downstream reservoirs are maintained at a level of approximately El. 110.00 or higher, the driving head through the filters will be significantly reduced, resulting in shorter filter run times. This situation has occurred recently during high flows, while maintaining relatively full reservoirs. The addition of the new 10 million gallon reservoir will significantly help the storage problem and allow the reservoirs to be maintained at a lower level. The filter operation and reservoir levels will be an operational decision.



DEMOLITION PLAN

SCALE: 1/8" = 1'-0"



MODIFIED PLAN

SCALE: 1/8" = 1'-0"

FIGURE 2-24
EAST PLANT
INLET PIPING MODIFICATIONS

3.0 ALTERNATIVE EVALUATION

3.1 GENERAL

The process alternatives developed in Section 2.0 have been incorporated into three plant expansion alternatives. Each alternative plant design will include modifications and improvements to upgrade the existing facilities to a design treatment capacity of 160 mgd. The three expansion alternatives were developed around the following concepts:

- Abandon the East plant and upgrade and modify the Central plant to treat 160 mgd. (Alternative A)

- Abandon the East plant and add additional process units to the Central plant to treat 160 mgd. (Alternative B)

- Upgrade and modify the East plant to treat 30 mgd, and upgrade and modify the Central plant to treat 130 mgd. (Alternative C)

The modifications and upgrades to the existing facilities will include replacing existing equipment to ensure a useful life of 20 years; improvements to piping and channels to provide sufficient hydraulic capacity; basin modifications to insure sufficient detention times and sizes for the appropriate design criteria; and addition of new equipment to provide necessary process design criteria.

Each of the three alternatives are discussed in greater detail below. A comparison cost evaluation is provided following the discussion of alternatives to assist in the decision making process for the recommended alternative. The recommended alternative will be developed in greater detail in the Final Report.

3.2 ALTERNATIVE A

This alternative consists of abandoning the East plant and upgrading the Central plant to treat 160 mgd. Increased plant flexibility would be created through the addition of two new internal walls down the center of each flocculation and sedimentation basin, as well as a new distribution channel feeding to each sedimentation basin, as shown in Figure 3-1. In the event one train in either the flocculation or sedimentation basins is taken out-of-service, the remainder of the plant could still be utilized without having to shut down an entire process train. The distinct advantage of the four train concept over the two train concept is that when part of or an entire process train is taken out-of-service, the maximum reduction in water treatment capacity is only 25 percent (versus 50 percent for the two train concept). The general modifications to each treatment process/facility relating to this alternative are discussed below in greater detail. A cost summary for Alternative A is presented in Section 3.5.

The existing Central plant aerators would have a surface loading rate of 13.1 gpm/sf for an increased plant flow of 160 mgd. A reduction in the efficiency of the aeration process is not desired, for reasons mentioned in Section 2.2. A design surface loading rate of approximately 10 gpm/sf is recommended. This rate also agrees with the surface loading guideline established by the State. To achieve a reduction in the surface loading rate, it is recommended that an additional aerator tray be added to each of the 19 aerators, as shown in Figure 3-2. A new distribution plate with larger orifices would be added to decrease headloss across the aerators. It is also recommended to add a wire mesh screen to collect the Asian clam shells.

As was mentioned in Section 2.3, the major disadvantage of the existing Central plant rapid mix basin is that there is only one basin. Since Alternative A utilizes only Central plant, the rapid mix facility should incorporate greater operational flexibility. For this reason, it is recommended that a second rapid mix basin be added which would operate in parallel with the existing basin, as shown in Figure 3-3. The two basins

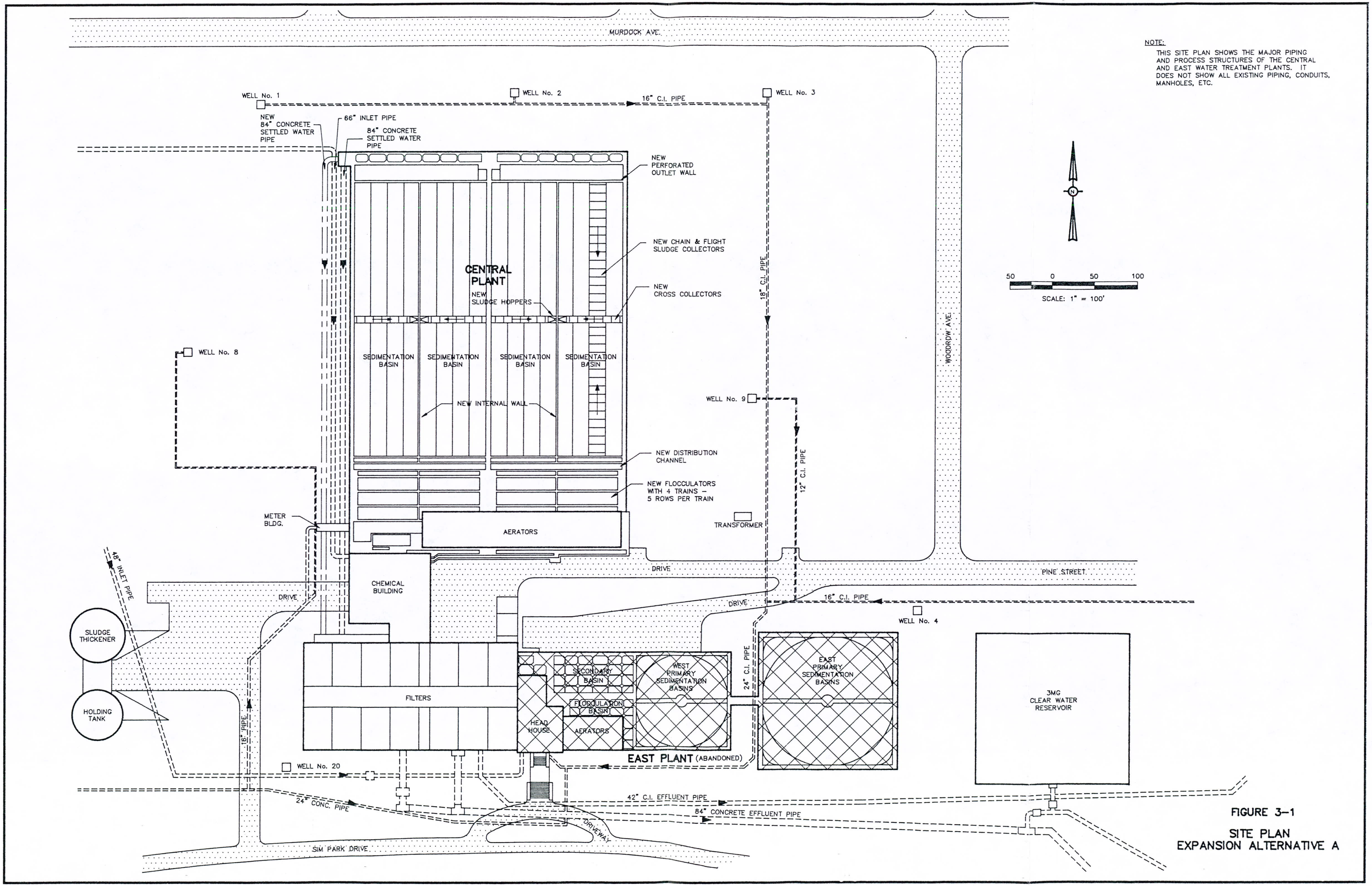


FIGURE 3–1

SITE PLAN
EXPANSION ALTERNATIVE A

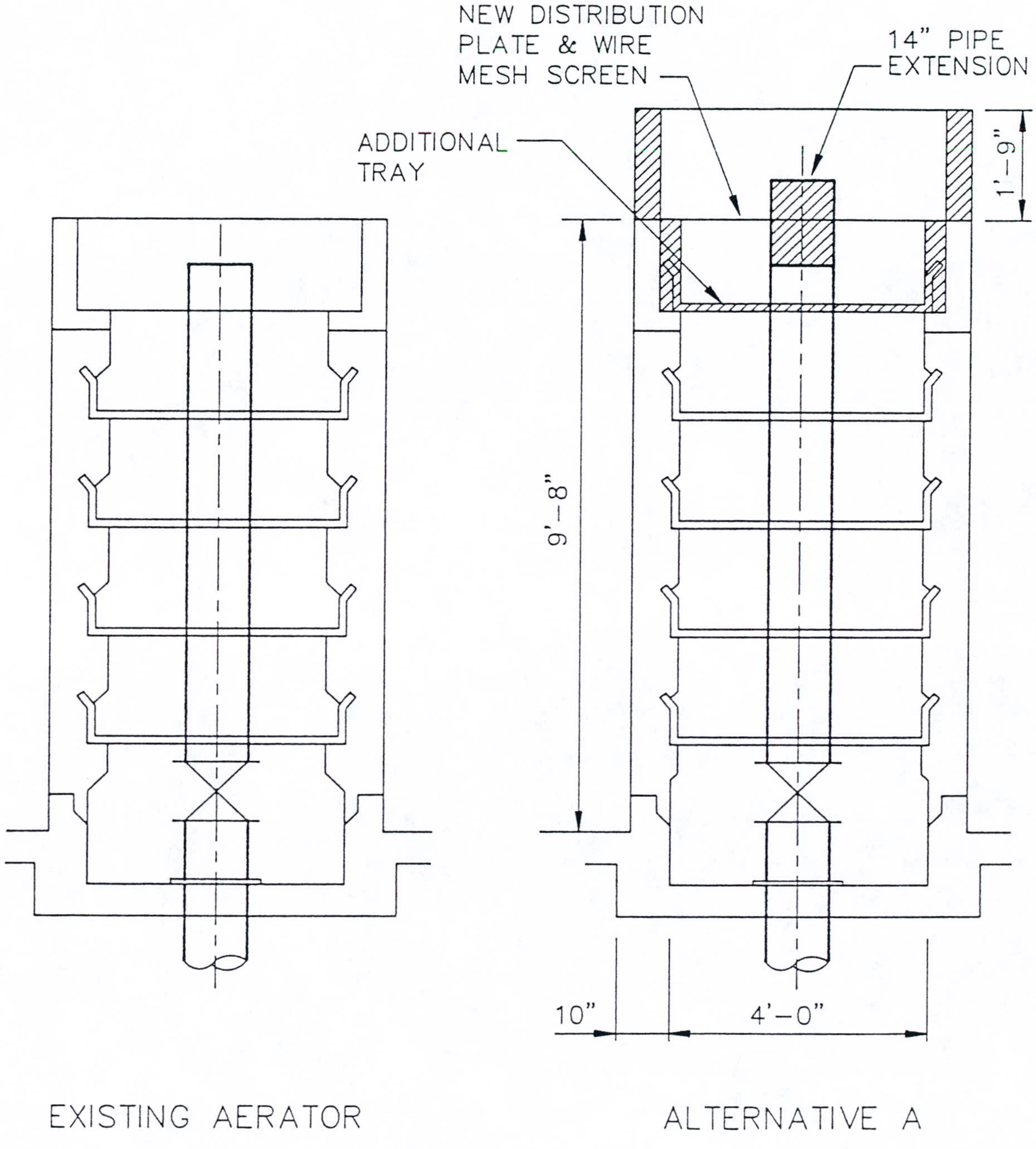


FIGURE 3–2

CENTRAL PLANT AERATORS

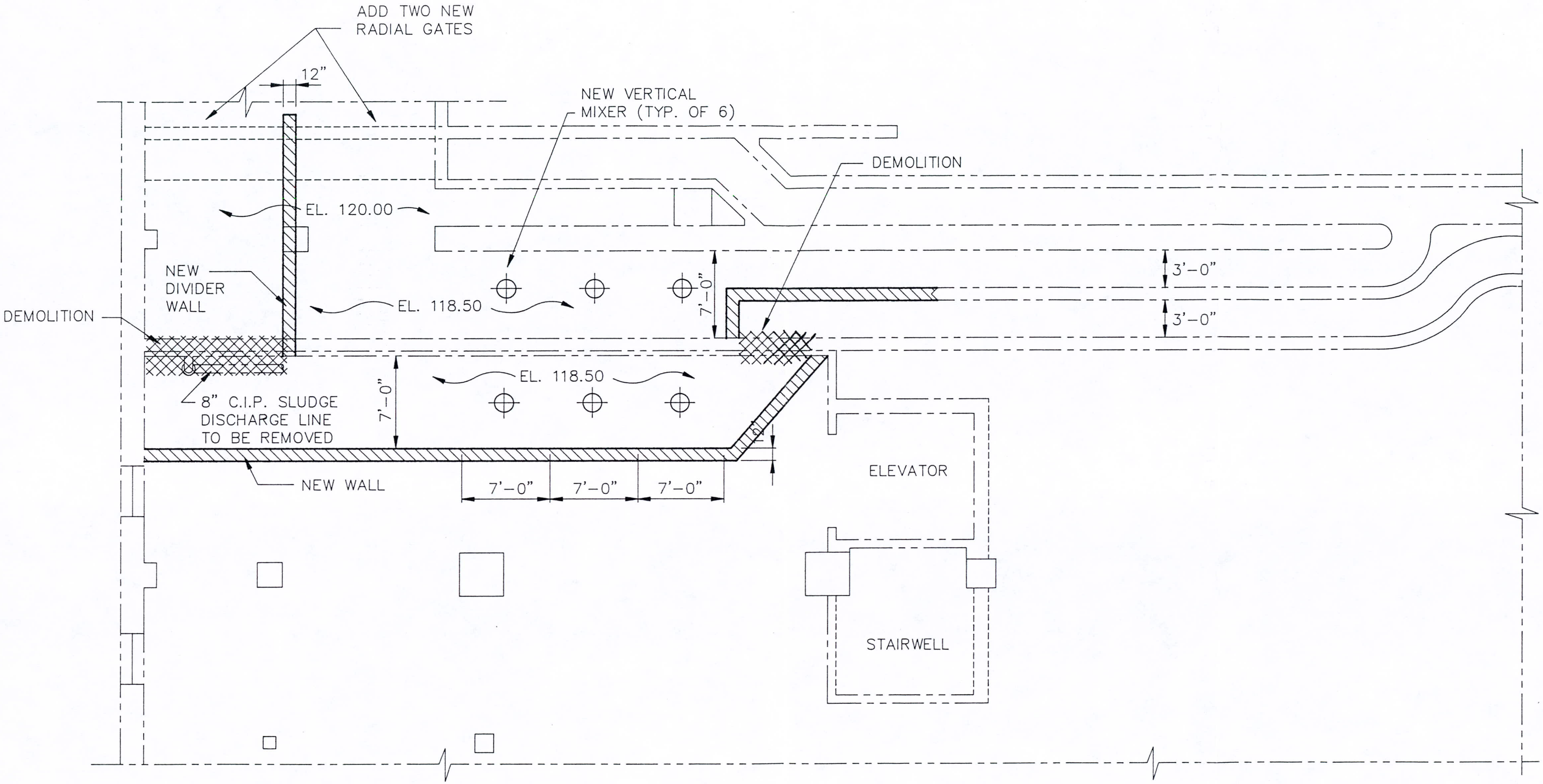


FIGURE 3-3

MODIFICATIONS TO EXISTING CENTRAL PLANT RAPID MIX BASIN

SCALE: 1/8" = 1'-0"

"FLRPLN1"

"FLRPLT1"

would have a total detention time of 10 seconds at 160 mgd. A total of six new "rotating impeller type" mixers would also be installed, as discussed in Section 2.3.

The existing flocculation basins at Central plant have a detention time of 16 minutes for a plant flow of 160 mgd. As was discussed in Section 2.4, a flocculation time of 20 to 30 minutes is recommended. For this reason, two additional rows of flocculators are recommended for this alternative, as shown in Figure 3-1. The detention time through the modified basins would be 25 minutes at 160 mgd. A new distribution channel with four sluice gates (one for each process train) would also be added between the flocculation and sedimentation basins to aid in operational flexibility. Sluice gates would be required in the influent distribution channel to the flocculation basins to provide flow splitting between the four process trains. New paddle wheel flocculators with variable speed drives would be installed to provided tapered flocculation, and wooden baffle walls would be added to reduce short-circuiting and enhance tapered mixing. Concrete fillets would be added in each flocculation stage to reduce dead space and improve mixing.

The sedimentation basins would be modified to provide sludge collection over the full floor area, as shown in Figure 3-1. The existing circular mechanisms and effluent launders would be removed and the basin floor grouted to provide a near-level surface. Chain and flight sludge collectors would then be installed. Due to the length of the sedimentation basin (both primary and secondary basins), it is proposed that two sets of sludge collectors be provided, each roughly half the length of the basin and sweeping towards the middle. Alternative arrangements such as using one long chain and flight collector with high strength chain will be evaluated in more detail in the design phase. For the arrangement depicted in Figure 3-1, the sludge collectors would sweep to a central cross collector, which would then sweep the sludge to a single sludge hopper in each basin.

The new internal walls in the sedimentation basins would provide two benefits: 1) the increased length to width ratio will reduce short circuiting in the basins, and 2) as discussed above, it will be possible to take one of the four sedimentation basins out of service with only a 25 percent loss of sedimentation facilities and no impact on the flocculation basins. Perforated inlet and outlet walls would be constructed in the sedimentation basins to provide a uniform flow pattern in to and out of the basins.

The detention time of the modified sedimentation basins at 160 mgd is 1.7 hours. The surface loading rate is 1,510 gpd/sf for a 160 mgd plant flow. This corresponds to a particle settling velocity of 4.3 cm/min. A typical settling velocity for a heavy lime floc is 9 cm/min. Hence the factor of safety for settling of a representative lime floc provided by the modified sedimentation basin at a 160-mgd plant flow is about 2.0.

A new 84-inch concrete settled water pipe would be installed parallel to the existing pipe for hydraulic improvements.

The existing 14 filters would be upgraded as described in Section 2.6. The existing underdrains and sand would be removed and replaced with a new underdrain capable of air and water backwashing and a new media. Two air blowers would be installed and, if necessary, the existing backwash pumps would be rehabilitated. The filter control system instrumentation would be replaced with a new system and individual continuous turbidity monitors would be provided. The filtration rate of the 14 filters at 160 mgd is 5.7 gpm/sf with all filters operating and 6.1 gpm/sf with one filter out of service for backwashing.

3.3 ALTERNATIVE B

This alternative includes abandoning the East plant and making additions to Central plant to treat 160 mgd. A third process train would be added west of the existing plant, as shown in Figure 3-4. Similar to Alternative A, the creation of an additional process train would increase the plants operational flexibility. In addition to a third process train, a new

NOTE:

THIS SITE PLAN SHOWS THE MAJOR PIPING AND PROCESS STRUCTURES OF THE CENTRAL AND EAST WATER TREATMENT PLANTS. IT DOES NOT SHOW ALL EXISTING PIPING, CONDUITS, MANHOLES, ETC.

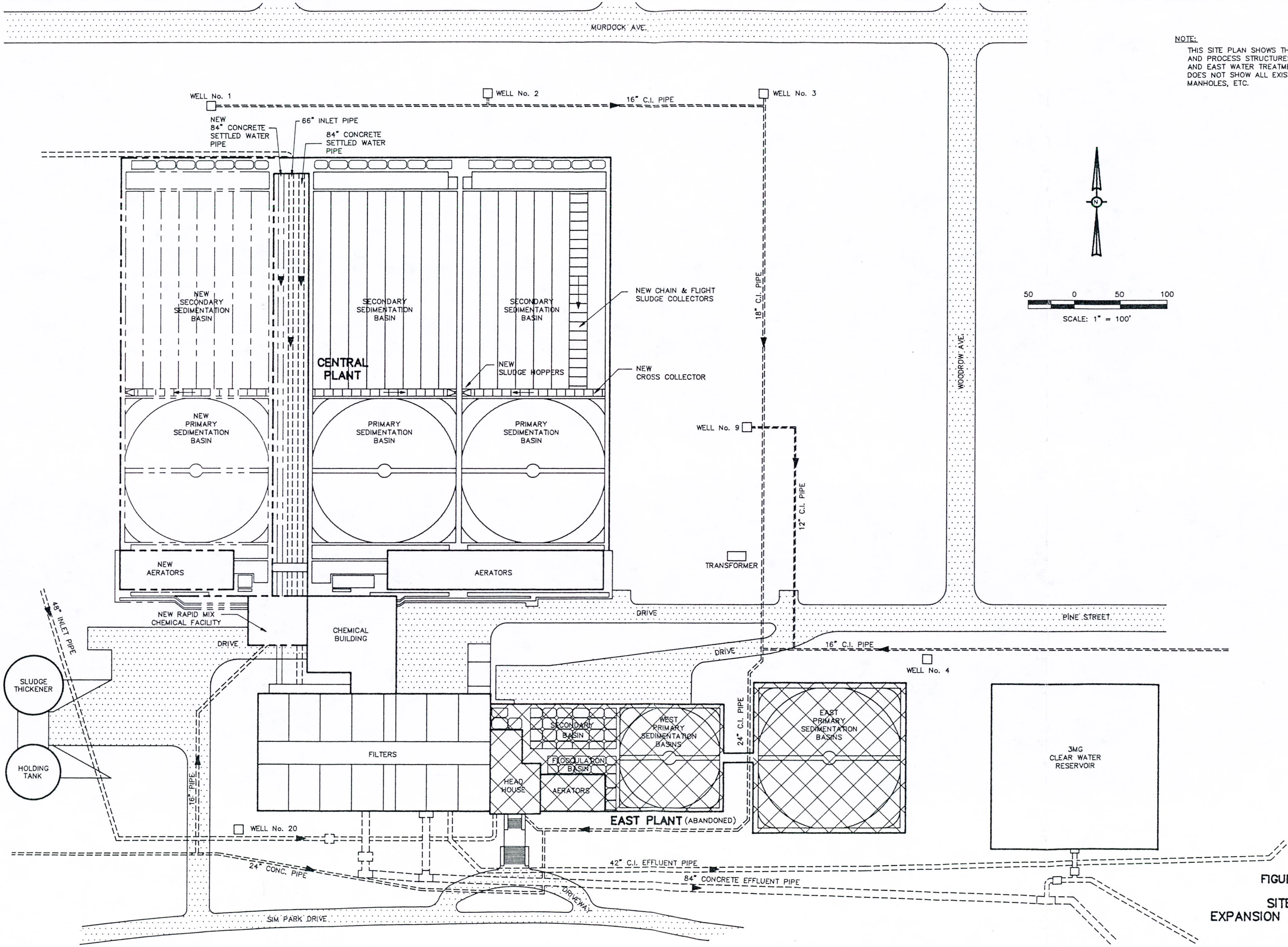


FIGURE 3-4

SITE PLAN
EXPANSION ALTERNATIVE B

aeration building, rapid mix basin, and chemical facility would also be provided. The general modifications to each treatment process/facility relating to this alternative are presented below. A cost summary for Alternative B is provided in Section 3.5.

As was stated under Alternative A, the existing aerators cannot not adequately treat 160 mgd. Because a third process train would be provided, a new aerator building with nine multiple tray aerators would also be added, for a total of 28 aerators. The surface loading rate would be 8.9 gpm/sf for 160-mgd plant flow. Modifications to the existing aerators, such as a new distribution plate and the addition of a wire mesh screen to collect the Asian clam shells, would also be required.

The existing rapid mix basin would be modified as discussed under Alternative A. A new rapid mix basin would also be added to treat the water for the third process train. A total of nine new "rotating impeller type" mixers would be installed - six for the modified basin(s) and three for the new basin. The detention time through the new and modified basins would be 15 seconds at a plant flow of 160 mgd.

The existing flocculation basins would only require minor modifications since a third flocculation train would be added. The new flocculation basin and the existing basins would incorporate the following: 1) new paddle wheel flocculators with variable speed drives, three rows per train: 2) tapered flocculation; 3) wooden baffle walls between each row of flocculators; 4) concrete fillets in each flocculation stage; and 5) a detention time of 24 minutes for a plant flow of 160 mgd.

For the sedimentation basins the existing circular mechanisms would be replaced and a third circular mechanism installed in the primary sedimentation basin of the third train. Chain and flight sludge collectors would be installed in the secondary sedimentation basins of all three trains. These sludge collectors would sweep towards the inlet end and a chain and flight cross collector would move the sludge to new sludge hoppers located at the edge of each basin.

As for Alternative A, the effluent launders would be removed and a perforated outlet wall installed in each basin. The detention time of the three sedimentation basins at 160 mgd is 3.5 hours. The surface loading rate is 810 gpd/sf. This corresponds to particle settling velocity of 2.3 cm/min, which yields a factor of safety of about 4.0 for settling of a typical heavy lime floc.

A parallel 84-inch concrete settled water pipe would be installed beside the existing settled water pipe to reduce the high head losses that would occur at a 160-mgd plant flow.

The filter improvements would be the same as for Alternative A.

Because a new rapid mix basin would be provided for the third process train, an additional lime bin and slaker/feeder would also be required to best feed the lime into this basin. The new lime bin and slaker/feeder would be located in the "New Rapid Mix/Chemical Facility" shown on Figure 3-4.

3.4 ALTERNATIVE C

This alternative consists of providing improvements to the East plant to treat 30 mgd and upgrading the Central plant to treat 130 mgd as shown in Figure 3-5. The general modifications to each plant's treatment process/facility are presented below. A cost summary for Alternative C is provided in Section 3.5.

3.4.1 EAST PLANT

The East plant inlet piping would be modified as described in Section 2.9 to overcome the problem of high velocities. The inlet piping and fittings in the basement would be replaced with a larger size.

As was stated in Section 2.2, the existing aerators are limited in the amount of water they can treat due to hydraulic constraints. To overcome this problem, it is recommended that a new distribution plate with larger

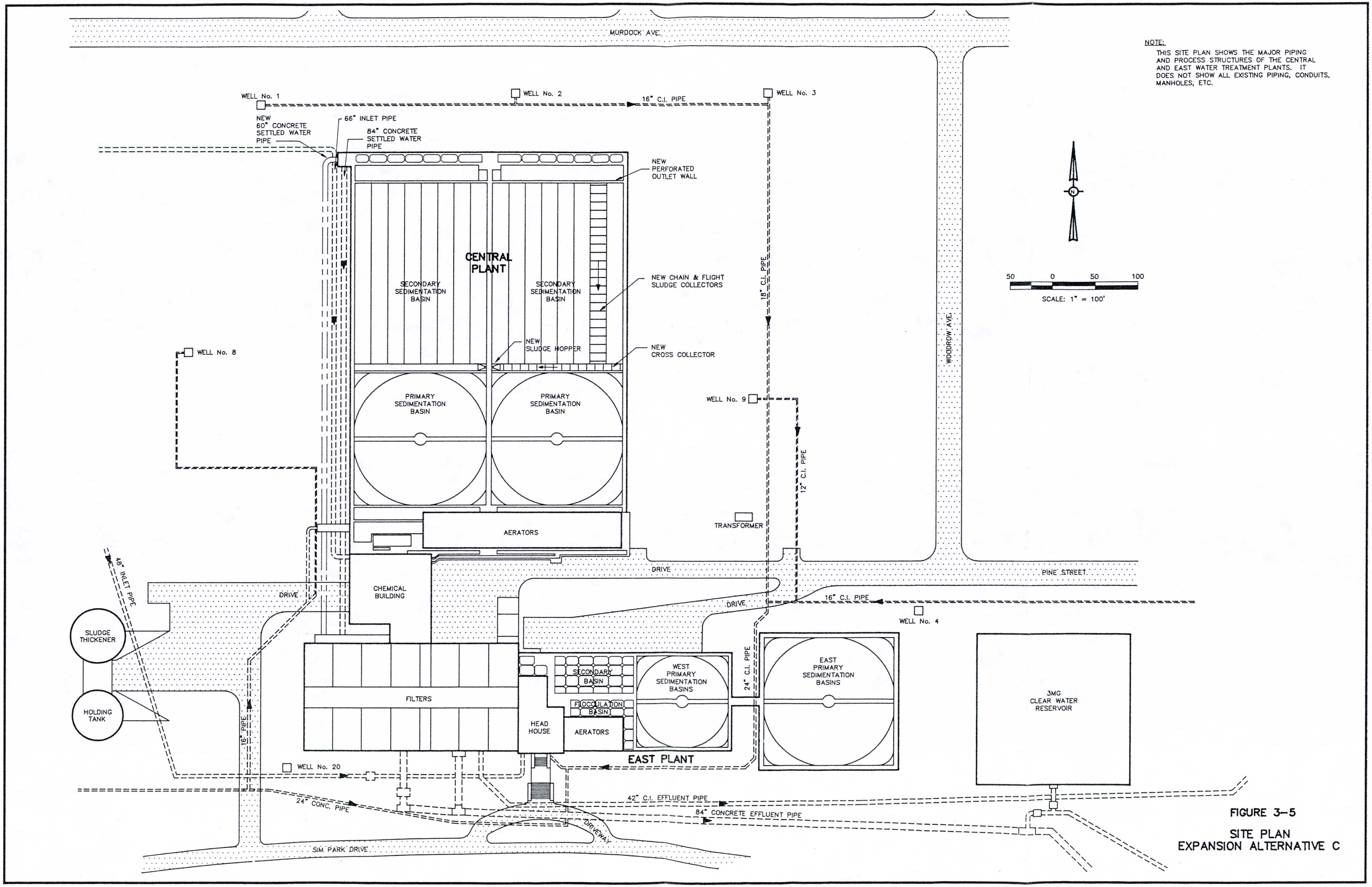


FIGURE 3–5

SITE PLAN
EXPANSION ALTERNATIVE C

diameter orifices be provided. For a plant flow of 30 mgd, the existing aerators have a surface loading rate of 6.2 gpm/sf.

The continued use of a paddle wheel type rapid mixer is recommended for East plant due to the physical constraints of the mix basin. For this alternative, a new paddle wheel mixer would be installed. The detention time through the rapid mix basin is 64 seconds for a plant flow of 30 mgd.

The existing flocculation basins would be upgraded, similar to the Central Plant flocculators in Alternatives A and B, to incorporate the following: 1) tapered flocculation; 2) new paddle wheel flocculators with variable speed drives, five rows; 3) wooden baffle walls between each row of flocculators; and 4) concrete fillets in each flocculation stage. The detention time through the existing basins for a plant flow of 30 mgd is 30 minutes.

The East plant sedimentation basins would be retained essentially as they are with minor modifications as follows. The existing circular mechanisms are both over 40 years old and would be replaced. The perimeter weir plates in the primary sedimentation basins would be replaced with new ones. The inlet weir for the secondary basin would be removed and core holes drilled in the existing wall to provide a perforated inlet wall.

3.4.2 CENTRAL PLANT

For an increased plant flow of 130 mgd, the existing aerators have a surface loading rate of 10.6 gpm/sf. The existing aerators should adequately treat the increased flow. For this alternative, it is recommended that each aerator be modified as follows: 1) add a new distribution plate with larger orifices to decrease headloss, and 2) add a wire mesh screen to collect the Asian clam shells.

The rapid mix basin would be expanded and modified, as discussed under Alternative A, to provide a second parallel basin. The new basins would have a total detention time of 13 seconds at a plant flow of 130 mgd.

The flocculation basin would be modified as discussed under Alternative B. The upgraded basin would incorporate the following: 1) new paddle wheel flocculators with variable speed drives, three rows per train; 2) tapered flocculation; 3) wooden baffle walls between each row of flocculators; 4) concrete fillets in each flocculation stage; and 5) a detention time of 20 minutes for a plant flow of 130 mgd.

The modifications to the sedimentation basins and filters are the same as for Alternative B, except 1) there are two trains of sedimentation basins instead of three and 2) the new parallel settled water pipe would be a 60-inch pipe instead of an 84-inch.

The detention time of the two sedimentation basins at a 130-mgd flow through the Central plant would be 2.9 hours and the surface loading would be 990 gpd/sf.

The filter improvements would be the same as for Alternatives A and B.

3.5 COMPARATIVE CONSTRUCTION COST ESTIMATES

Tables 3-1, 3-2, and 3-3 present comparative construction cost estimates for the three alternative treatment schemes discussed above. These costs represent construction cost items which are used to evaluate one alternative against another. The estimated cost is not the actual total cost for the overall improvements for the plant expansion. Such items as electrical and instrumentation costs and chemical feed modification costs, which are similar for all alternatives, are not included in these comparative estimates. The overall project cost estimate will be presented in the Final Report.

A summary of the comparative construction cost estimates for each alternative is presented below:

Alternative A	$10,704,000
Alternative B	$14,153,000
Alternative C	$ 9,412,000

TABLE 3-1
ALTERNATIVE A - COMPARATIVE COST ESTIMATE

Item Description	Cost
A. EAST PLANT	
Plant abandoned, no improvements	$ 0
B. CENTRAL PLANT	
AERATORS	
Add extra tray to existing aerators	$ 40,000
Miscellaneous improvements	150,000
RAPID MIX BASIN	
New rapid mix basin	$ 160,000
Install 6 new rapid mixers	90,000
FLOCCULATION BASINS	
Replace existing flocculators	$ 585,000
Install new flocculation basin walls	300,000
Install new flocculators	390,000
Install slotted baffle walls	256,000
Install concrete fillets	407,000
SEDIMENTATION BASINS	
Remove effluent launders	$ 100,000
Remove exist. circular mechanisms	50,000
Install perforated outlet walls	100,000
Grout exist. basins to give level floor	1,605,000
Install chain & flight sludge collectors	960,000
Install chain & flight cross collectors	120,000
Install sludge hoppers and sludge piping	100,000
SETTLED WATER PIPE	
Install new 84" pipe	$ 171,000
FILTERS	
Modify existing filters	$ 5,120,000
COMPARATIVE COST TOTAL	$10,704,000

TABLE 3-2
ALTERNATIVE B - COMPARATIVE COST ESTIMATE

Item Description	Cost
A. EAST PLANT	
Plant abandoned, no improvements	$ 0
B. CENTRAL PLANT	
AERATORS	
Construct new aerator building	$ 350,000
Construct new aerated water channel	185,000
Install new aerators	135,000
Miscellaneous improvements	150,000
CHEMICAL BLDG. EXPANSION & RAPID MIX BASIN	
Construct new building & rapid mix basin	$ 780,000
Construct rapid mix basin in old bldg.	200,000
Install new rapid mix units	135,000
Misc. inlet & interconnection piping	800,000
FLOCCULATION BASINS	
Install slotted baffle walls	$ 192,000
Replace flocculators	450,000
Construct new flocculation basin	350,000
Install new flocculators	225,000
Install concrete fillets	340,000
SEDIMENTATION BASINS	
Remove exist. effluent launders	$ 100,000
Remove exist. circular mechanisms	50,000
Install new circular mechanisms	720,000
Install perforated outlet walls	100,000
Grout exist. basins to give level floor	480,000
Construct third sed. basin structure	2,040,000
Install chain & flight sludge collectors	720,000
Install chain & flight cross collectors	90,000
Install sludge hoppers and sludge piping	100,000
Miscellaneous metals & handrail	170,000
SETTLED WATER PIPE	
Install new 84" pipe	$ 171,000
FILTERS	
Modify existing filters	$ 5,120,000
COMPARATIVE COST TOTAL	$14,153,000

TABLE 3-3
ALTERNATIVE C - COMPARATIVE COST ESTIMATE

Item Description	Cost
A. EAST PLANT	
INLET PIPINT	
Replace piping/fittings with larger size	$ 85,000
AERATORS	
Miscellaneous improvements	$ 25,000
RAPID MIX BASIN	
Replace rapid mixer	$ 75,000
FLOCCULATION BASINS	
Install slotted baffle walls	$ 40,000
Replace flocculators	180,000
Install concrete fillets	88,000
SEDIMENTATION BASINS	
Modify inlet hydraulics to primary basins	$ 100,000
Modify secondary basin inlet weir	20,000
Remove exist. circular mechanisms	50,000
Install new 150′ dia. circular mechanism	240,000
Install new 105′ dia. circular mechanism	170,000
Install new weir plates for launders	50,000
East Plant Subtotal	$ 1,123,000

TABLE 3-3 (Cont.)

B. CENTRAL PLANT

AERATORS	
Miscellaneous improvements	$ 150,000
RAPID MIX BASIN	
New rapid mix basin	$ 160,000
Install 6 new rapid mixers	90,000
FLOCCULATION BASINS	
Replace existing flocculators	$ 450,000
Install slotted baffle walls	128,000
Install concrete fillets	227,000
SEDIMENTATION BASINS	
Remove effluent launders	$ 100,000
Remove exist. circular mechanisms	50,000
Install new circular mechanisms	480,000
Install perforated outlet walls	100,000
Grout exist. basins to give level floor	480,000
Install chain & flight sludge collectors	480,000
Install chain & flight cross collectors	60,000
Install sludge hoppers and sludge piping	100,000
SETTLED WATER PIPE	
Install new 60" pipe	$ 114,000
FILTERS	
Modify existing filters	$ 5,120,000
Central Plant Subtotal	$ 8,289,000
COMPARATIVE COST TOTAL	$9,412,000

3.6 ANALYSIS AND RECOMMENDATION

Based on the initial comparative cost analysis presented in Section 3.5, Alternatives A and C are significantly lower than Alternative B. We therefore can conclude that the final option shall be either Alternative A or Alternative C.

Table 3-4 presents a comparative present worth cost analysis of Alternatives A and C from an overall plant expansion viewpoint.

- Additional construction costs will be incurred for improvements to all alternatives. (e.g. electrical, instrumentation, and sludge handling.)

- Annual operating costs will be incurred for all alternatives and are evaluated.

Both cost and qualitative factors should be considered in formulating the recommended plan. There are several qualitative advantages of Alternative A over Alternative C. Alternative C provides for the operation of two plants, which will require:

- Coordination for operations staff.
- Additional record keeping.
- More frequent chemical deliveries and associated noise.
- More overall plant control.
- Additional maintenance for start up of East Plant.

Based on all factors herein, it is recommended that Alternative A - Upgrade the Central Plant to 160 mgd and abandon the East Plant - be implemented to expand the Wichita Water Treatment Plant to 160 mgd capacity. This recommendation is based on the following:

- Present worth costs for Alternatives A and C are, for all practical purposes, the same.

- The qualitative benefits of Alternative A, which requires the operation of only one plant.

Consideration should be given to making minor hydraulic modifications to the East Plant to insure that this facility could be utilized if future need requires. These improvements would cost approximately $250,000.

TABLE 3-4
PRESENT WORTH COST COMPARISON

		ALTERNATIVE A	ALTERNATIVE C
	CAPITAL COSTS		
	1. Construction Cost of Comparative Improvements	$ 10,704,000	$ 9,412,000
*	2. Construction Cost of Additional Items	3,200,000	3,800,000
	3. Contingencies	2,100,000	2,100,000
	TOTAL CONSTRUCTION COST	$ 16,004,000	$ 15,312,000
**	ANNUAL COSTS		
	1. Power	$ 85,000	$ 90,000
	2. Chemicals	800,000	800,000
	3. Staff	1,010,000	1,070,000
	TOTAL ANNUAL COST	$ 1,895,000	$ 1,960,000
	PRESENT WORTH COSTS		
	1. Capital	$ 16,004,000	$ 15,312,000
***	2. Annual	20,076,000	20,764,000
	TOTAL PRESENT WORTH COST	$ 36,080,000	$36,076,000

---

* Additional items, including such costs as electrical, instrumentation, and sludge removal are based on a percentage of construction cost:

Alt. A - 20%
Alt. C - 25% (Additional due to provisions for two plants)

** Annual costs are based on historical numbers for the Wichita Plant projected over a twenty year period assuming a linear increase in flow over the twenty year period. Staffing and power will be less for one plant (Alt. A) than for two plants (Alt. B)

*** Present Worth of Annual Cost is based on a 7% discount rate for a 20 year period.

APPENDIX A

HYDRAULIC MODEL

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
HYDRAULIC PROFILE
21-Nov-90

PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT

****EAST PLANT HYDRAULIC PROFILE:
****PLANT FLOW= 0 MGD

VENTURI TO AERATOR:
===================

Venturi Meter, 48x20	1	0	0	0.00	12.57	0.00		134.33	0.00	134.33
(contract/expand)					2.18	0.00	0.09	134.33	0.00	134.33
48" 90d Bend	1	0	0	0.00	12.57	0.00	0.23	134.33	0.00	134.33
48x36 Reducer	1	0	0	0.00	7.07	0.00	0.08	134.33	0.00	134.33
36" 11 1/4d Bend	1	0	0	0.00	7.07	0.00	0.06	134.33	0.00	134.33
36" 90d Bend	1	0	0	0.00	7.07	0.00	0.20	134.33	0.00	134.33
36" Tee (run)	1	0	0	0.00	7.07	0.00	0.24	134.33	0.00	134.33
36x24 Reducer (sudden)	1	0	0	0.00	3.14	0.00	0.28	134.33	0.00	134.33
24" Gate Valve	1	0	0	0.00	3.14	0.00	0.10	134.33	0.00	134.33
24" Flow Contr (plug)	1	0	0	0.00	3.14	0.00	0.22	134.33	0.00	134.33
24" Gate Valve	1	0	0	0.00	3.14	0.00	0.10	134.33	0.00	134.33
24x30 Enlargr (sudden)	1	0	0	0.00	3.14	0.00	0.13	134.33	0.00	134.33
30" Tee Specl (branch)	1	0	0	0.00	4.91	0.00	0.72	134.33	0.00	134.33
30" Tee (branch)	1	0	0	0.00	4.91	0.00	0.72	134.33	0.00	134.33
30" Outlet to Aerator	1	0	0	0.00	4.91	0.00				
Distr. Flume					21.00	0.00	1.00	134.33	0.00	134.33
Contr. Aerator Flume	1	0	0	0.00	21.00	0.00				
					9.00	0.00	0.20	134.33	0.00	134.33
Flume to 14" Inlet	12	0	0	0.00	1.07	0.00	0.50	134.33	0.00	134.33
14" Gate Valve	12	0	0	0.00	1.07	0.00	0.10	134.33	0.00	134.33
Aerator Discharge	12	0	0	0.00				134.33	0.00	134.33

[Discharge uses weir equation Q=CLH^1.5 @ L=2piR, R=7", C=3]

Distribution Plate	30510	0.00	0.00	0.00	0.00	0.00	0.61	133.33	0.00	133.33

[Uses orifice eq, 6 trains @ 5085 1/4" orifices ea: Q=CA(2gh)^0.5]

AERATOR THROUGH RAPID & SLOW MIX TO CLARIFIER WEIRS:
===

Contr Flume @ Slakers*	1	0	0	0.00	25.54	0.00	0.10	123.44	0.00	123.44
R Mix Entrance*	1	0	0	0.00	11.63	0.00	1.50	123.44	0.00	123.44
R Mix Exit to Dstr Fl*	1	0	0	0.00	27.39	0.00	2.50	123.44	0.00	123.44
Distr Flume Slots (ea)	24	0	0	0.00	1.63	0.00	2.50	123.44	0.00	123.44
Coll Flume Slots (ea)*	29	0	0	0.00	1.55	0.00	2.50	123.44	0.00	123.44

[Floor el.125.50] *denotes flux area dependent on WSEL

HGL BALANCE BETWEEN CLARIFIERS: 1939: 1947:
HGL at Collection Flume: 123.46 123.42

A. FLOW TO 1939 CLARIFIER: 0 MGD SLR= 0 GPD/SF
==========================

Tunnel to Clarifier	1	0	0	0.00	36.00	0.00	1.50	123.46	0.00	123.46
Clar. Weir (v-notch)	424	0	0	0.00				123.46	0.00	123.46

[Uses weir eq Q=2.4381 H^2.5 for 424 v-notches] [Top of weir=123.67, depth=0.2083]

B. FLOW TO 1947 CLARIFIER: 0 MGD SLR= 0 GPD/SF
==========================

Tunnel Entrance	1	0	0	0.00	25.00	0.00				

HYDRAULIC PROFILE / 21-Nov-90 WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT

PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
					42.00	0.00	1.00	123.42	0.00	123.42
Tunnel to Clarifier	1	0	0	0.00	42.00	0.00	1.00	123.42	0.00	123.42
Clarif Weir (v-notch)	605	0	0	0.00				123.42	0.00	123.42

[Uses weir eq Q=2.4381 H^2.5 for 605 v-notches] [Top of weir=123.625, depth=0.2083]

FLOW TO FINAL SETTLING BASIN ENTRANCE WEIR: (Weir plate removed)
==

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
HGL below 1939 Weir		0	0	0.00	45.50	0.00	2.50	122.83	0.00	122.83
HGL below 1947 Weir		0	0	0.00	25.00	0.00	1.50	122.83	0.00	122.83
Final SB Entrance Weir		0	0	0.00				122.83	0.00	122.83

[Uses weir eq Q=3LH^1.5, where L=44]

FINAL SETTLING BASIN TO FILTER FLUME:
=====================================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
SB Coll Fl Slots (ea)*	33	0	0	0.00	1.05	0.00	2.50	122.55	0.00	122.55
Coll Fl to Filter Fl*	1	0	0	0.00	25.25	0.00				
					37.88	0.00	1.00	122.55	0.00	122.55

****CENTRAL PLANT HYDRAULIC PROFILE:

****PLANT FLOW= 160 MGD

YARD TO AERATOR:
================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
66" Tee at 48" (run)	1	160	111111	247.57	23.76	10.42	0.23	169.06	0.39	168.67
66" 90d Bend	1	160	111111	247.57	23.76	10.42	0.24	168.67	0.40	168.26
66x60 Reducer	1	160	111111	247.57	19.63	12.61	0.01	168.26	0.02	168.24
60" 30d Bend	1	160	111111	247.57	19.63	12.61	0.10	168.24	0.25	167.99
60" 30d Bend	1	160	111111	247.57	19.63	12.61	0.10	167.99	0.25	167.75
60" Tee Spcl (branch)	1	160	111111	247.57	19.63	12.61	0.50	167.75	1.23	166.51
60" Butterfly Valve	1	160	111111	247.57	19.63	12.61	0.30	166.51	0.74	165.77
60x30 Reducer	1	160	111111	247.57	4.91	50.44		165.77	2.50	163.27
30" Tee (run)	1	160	111111	247.57	4.91	50.44	0.24	163.27	9.48	153.79
30" Butterfly Valve	1	160	111111	247.57	4.91	50.44	0.30	153.79	11.85	141.94
30x60 Enlarger	1	160	111111	247.57	4.91	50.44	0.09	141.94	3.55	138.38
60" 90d Bend	1	160	111111	247.57	19.63	12.61	0.24	138.38	0.59	137.79
60" 90d Bend	1	160	111111	247.57	19.63	12.61	0.24	137.79	0.59	137.20
60x84 Enlarger	1	160	111111	247.57	19.63	12.61	0.06	137.20	0.15	137.05
84" to 14" Inlet	34	4.70	3267.9	7.28	1.07	6.81	0.50	137.05	0.36	136.69
14" 90d Bend	34	4.70	3267.9	7.28	1.07	6.81	0.26	136.69	0.19	136.50
14" Gate Valve	34	4.70	3267.9	7.28	1.07	6.81	0.10	136.50	0.07	136.43
Aerator Discharge	34	4.70	3267.9	7.28				136.43	0.76	135.67

[Discharge uses weir equation Q=CLH^1.5 @ L=2piR, R=7", C=3]

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Distribution Plate	216801	0.00	0.51	0.00	0.00	3.35	0.61	136.30	0.47	135.83

[Uses orifice eq, 19 trains assuming 12753 1/4" orifices ea: Q=CA(2gh)^0.5]
[Top of aerator distribution wall=137.67; plate @ 135.83]

AERATOR THROUGH RAPID MIX:
==========================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
R Mix Entrance*	1	160	111111	247.57	78.17	3.17	1.00	128.06	0.16	127.91
R Mix Exit*	1	160	111111	247.57	40.27	6.15	1.00	127.91	0.59	127.32

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
HYDRAULIC PROFILE
21-Nov-90

PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT

[Floor el.128.0] *denotes flux area dependent on WSEL

HGL BALANCE BETWEEN CLARIFIERS: West: East:
HGL at Rapid Mix exit: 127.32 127.32

A. SLOW MIX TO WEST CLAR WEIRS: 85.60 MGD PRIMARY SLR= 3333.3 GPD/SF

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Flume Split*	1	85.6	59444.	132.45	18.69	7.09	1.40	127.32	1.09	126.23
Distr Flume Slots (ea)	100	0.85	594.44	1.32	1.38	0.96	2.50	126.23	0.04	126.19
Clarifier Baffle Wall	53	1.61	1121.5	2.50	14.50	0.17	2.50	126.19	0.00	126.19
Wooden Baffle Wall	96	0.89	619.21	1.38	8.45	0.16	2.50	126.19	0.00	126.19
Effl Weir (v-notch)	2020	0.04	29.427	0.07				126.19	0.00	126.19

[Uses weir eq Q=2.4381 H^2.5 for 2020 v-notches] [Top of weir=124.5, depth=0.125] [OF 125.0]
CHECK TO SEE IF WEIRS SUBMERGED BY LAUNDER DEPTH

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS				UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Effl Launder Upstream	10	8.56	5944	13.25	[Uses Benefield eq]			128.82	2.63	126.19
Effl Launder Cr Depth	10	8.56	5944	13.25	Crit Depth at Launder Outlet=				1.52	122.13

B. SLOW MIX TO EAST CLAR WEIRS: 74.40 MGD PRIMARY SLR= 2897.1 GPD/SF

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Flume Split*	1	74.4	51666.	115.12	17.56	6.56	2.20	127.32	1.47	125.85
Distr Flume Slots (ea)	100	0.74	516.66	1.15	1.38	0.84	2.50	125.85	0.03	125.83
Clarifier Baffle Wall	53	1.40	974.84	2.17	14.50	0.15	2.50	125.83	0.00	125.83
Wooden Baffle Wall	96	0.78	538.19	1.20	8.45	0.14	2.50	125.83	0.00	125.83
Effl Weir (v-notch)	2020	0.04	25.58	0.06				125.83	0.00	125.83

[Uses weir eq Q=2.4381 H^2.5 for 2020 v-notches] [Top of weir=124.5, depth=0.125] [OF 125.0]
CHECK TO SEE IF WEIRS SUBMERGED BY LAUNDER DEPTH

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS				UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Effl Launder Upstream	10	7.44	5167	11.51	[Uses Benefield eq]			128.22	2.39	125.83
Effl Launder Cr Depth	10	7.44	5167	11.51	Crit Depth at Launder Outlet=				1.38	122.13

CO2 BASIN TO OUTLET BOX:

HGL BALANCE BETWEEN CO2 BASINS: West: East:
HGL in CO2 Basins Upstream: 126.19 125.83
FLOW THROUGH SLGATE/PIPE: 96 64

EAST SECTION: [Overflow el.124.5]

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
72" Inlet fr CO2 Basin	1	64	44444.	99.03	28.27	3.50	0.70	125.83	0.13	125.69
72" Butterfly Valve	1	64	44444.	99.03	28.27	3.50	0.30	125.69	0.06	125.63
2-72" 90d Bends	1	64	44444.	99.03	28.27	3.50	0.24	125.63	0.09	125.54
72" Outlt to CO2 Basin	1	64	44444.	99.03	28.27	3.50	1.00	125.54	0.19	125.35

WEST SECTION:

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
84x60 Sluice Gate	1	96	66666.	148.54	35.00	4.24	1.50	126.19	0.42	125.77
84x60 Sluice Gate	1	96	66666.	148.54	35.00	4.24	1.50	125.77	0.42	125.35
Contraction @ Screens	1	160	111111	247.57	136.00	1.82	2.00	125.35	0.10	125.25

OUTLET BOX TO FILTER FLUME:

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
84" Entrance	1	160	111111	247.57	38.48	6.43	0.50	125.25	0.32	124.93

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
HYDRAULIC PROFILE
21-Nov-90

PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Recarb turbulence	1	160	111111	247.57	38.48	6.43	2.00	124.93	1.29	123.64
84" 30d Bend	1	160	111111	247.57	38.48	6.43	0.10	123.64	0.06	123.58
84" 30d Bend	1	160	111111	247.57	38.48	6.43	0.10	123.58	0.06	123.51
84" Exit to Flume	1	160	111111	247.57	38.48	6.43	1.00	123.51	0.64	122.87
Flume Contraction	1	160	111111	247.57	38.48	6.43	0.50	122.87	0.32	122.55

FILTER GALLERY:
===============

Filter Gallery contains three types of filters:

1. Filters 1 to 4: 24" ROF — 18" gravel, 36" media — 8.25 lf weir per filter, pipe lateral underdrains
2. Filters 5 & 6: 20" ROF — 18" gravel, 36" media — 8.75 lf weir per filter, pipe lateral underdrains
3. Filters 7 to 14: 20" ROF — 10" gravel, 33" media — 8.25 lf weir per filter, Leopold underdrains

FLOW FROM EAST PLT TO FILTERS: 0 MGD
FLOW FROM CENTRAL PLT TO FILTERS: 160 MGD
TOTAL FLOW TO FILTERS: 160 MGD

1. FILTERS 1 TO 4:
==================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
48" Entrance	14.285	11.2	7777.7	17.33	12.57	1.38	0.50	122.55	0.01	122.53
48" Gate Valve	14.285	11.2	7777.7	17.33	12.57	1.38	0.10	122.53	0.00	122.53
48" Outlet to Filter	14.285	11.2	7777.7	17.33	12.57	1.38	1.00	122.53	0.03	122.50
Clean Media (3'/0.5mm)	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	122.50	5.56	116.94
Gravel Underdrain	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	116.94	0.16	116.78
Underdrn Pipe Latrls	130000	0.00	0.8547	0.00			0.61	116.78	0.26	116.52

[Uses orifice eq Q=19.636C(d^2)(H^0.5), w/9100 3/8" holes/fltr] [Lateral CL=114.75]

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
3x4 Tee (branch)	1857.1	0.08	59.829	0.13	0.05	2.72	1.06	116.52	0.12	116.40
3x6 Enlarger	1857.1	0.08	59.829	0.13	0.05	2.72	0.27	116.40	0.03	116.37
Outlet to Filter Flume	1857.1	0.08	59.829	0.13	0.20	0.68	1.00	116.37	0.01	116.37
36" Inlet	14.285	11.2	7777.7	17.33	7.07	2.45	0.50	116.37	0.05	116.32
36" Cross (branch)	14.285	11.2	7777.7	17.33	7.07	2.45	0.70	116.32	0.07	116.25
36x24 Reducer (sudden)	14.285	11.2	7777.7	17.33	3.14	5.52	0.28	116.25	0.13	116.12
24" Gate Valve	14.285	11.2	7777.7	17.33	3.14	5.52	0.10	116.12	0.05	116.07
24" ROF Cntrllr	14.285	11.2	7777.7	17.33	3.14	5.52	14.33	116.07	6.77	109.30
24" 90d Bend	14.285	11.2	7777.7	17.33	3.14	5.52	0.20	109.30	0.09	109.21
24" Outlet to Clrwell	14.285	11.2	7777.7	17.33	3.14	5.52	1.00	109.21	0.47	108.74
Clearwell Weir	14.285	11.2	7777.7	17.33				108.74	0.74	108.00

[Uses weir eq Q=3.33LH^1.5, where L=8.25 per filter]

2. FILTERS 5 & 6:
=================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
48" Entrance	14.285	11.2	7777.7	17.33	12.57	1.38	0.50	122.55	0.01	122.53
48" Gate Valve	14.285	11.2	7777.7	17.33	12.57	1.38	0.10	122.53	0.00	122.53
48" Outlet to Filter	14.285	11.2	7777.7	17.33	12.57	1.38	1.00	122.53	0.03	122.50
Clean Media (3'/0.5mm)	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	122.50	5.56	116.94
Gravel Underdrain	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	116.94	0.16	116.78

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
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PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Underdrn Pipe Latrls	130000	0.00	0.8547	0.00			0.61	116.78	0.26	116.52
[Uses orifice eq Q=19.636C(d^2)(H^0.5), w/9100 3/8" holes/fltr] [Lateral CL=114.75]										
3x4 Tee (branch)	1857.1	0.08	59.829	0.13	0.05	2.72	1.06	116.52	0.12	116.40
3x6 Enlarger	1857.1	0.08	59.829	0.13	0.05	2.72	0.27	116.40	0.03	116.37
Outlet to Filter Flume	1857.1	0.08	59.829	0.13	0.20	0.68	1.00	116.37	0.01	116.36
36" Inlet	14.285	11.2	7777.7	17.33	7.07	2.45	0.50	116.36	0.05	116.32
36" Cross (branch)	14.285	11.2	7777.7	17.33	7.07	2.45	0.70	116.32	0.07	116.25
36x20 Reducer (sudden)	14.285	11.2	7777.7	17.33	2.18	7.94	0.35	116.25	0.34	115.91
20" Gate Valve	14.285	11.2	7777.7	17.33	2.18	7.94	0.10	115.91	0.10	115.81
20" ROF Controller	14.285	11.2	7777.7	17.33	2.18	7.94	6.05	115.81	5.93	109.88
20" 90d Bend	14.285	11.2	7777.7	17.33	2.18	7.94	0.20	109.88	0.20	109.69
20" Outlet to Clrwell	14.285	11.2	7777.7	17.33	2.18	7.94	1.00	109.69	0.98	108.71
Clearwell Weir	14.285	11.2	7777.7	17.33				108.71	0.71	108.00
[Uses weir eq Q=3.33LH^1.5, where L=8.75 per filter]										

3. FILTERS 7 TO 14:
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PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
48" Entrance	13.793	11.6	8055.5	17.95	12.57	1.43	0.50	122.56	0.02	122.54
48" Butterfly Valve	13.793	11.6	8055.5	17.95	12.57	1.43	0.25	122.54	0.01	122.54
48" Outlet to Filter	13.793	11.6	8055.5	17.95	12.57	1.43	1.00	122.54	0.03	122.50
Cl Media (2.75'/0.5mm)	13.793	11.6	8055.5	17.95	1450	5.56	=gpm/sf	122.50	5.09	117.41
Gravel Underdrain	13.793	11.6	8055.5	17.95	1450	5.56	=gpm/sf	117.41	0.09	117.32
Underdrain - Block	13.793	11.6	8055.5	17.95				117.32	0.50	116.82
36" Inlet	13.793	11.6	8055.5	17.95	7.07	2.54	0.50	116.82	0.05	116.77
36" Cross (branch)	13.793	11.6	8055.5	17.95	7.07	2.54	0.70	116.77	0.07	116.70
36x20 Reducer (sudden)	13.793	11.6	8055.5	17.95	2.18	8.23	0.35	116.70	0.37	116.33
20" Butterfly Valve	13.793	11.6	8055.5	17.95	2.18	8.23	0.30	116.33	0.32	116.02
20" ROF Controller	13.793	11.6	8055.5	17.95	2.18	8.23	5.71	116.02	6.00	110.01
20" 90d Bend	13.793	11.6	8055.5	17.95	2.18	8.23	0.20	110.01	0.21	109.80
20" Outlet to Clrwell	13.793	11.6	8055.5	17.95	2.18	8.23	1.00	109.80	1.05	108.75
Clearwell Weir	13.793	11.6	8055.5	17.95				108.75	0.75	108.00
[Uses weir eq Q=3.33LH^1.5, where L=8.25 per filter]										

COMPARISON BETWEEN FILTER GROUPS --

	DEPTH @ FLUME:	OVER WEIR:	
1. FILTERS 1 TO 4:	122.55	0.74	
2. FILTERS 5 & 6:	122.55	0.71	AVE FL DEPTH= 122.55
3. FILTERS 7 TO 14:	122.56	0.75	

Note: Depth at filter flume should be approximately the same, except for loss through 60" diam connection between Central & East.

	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	HEAD LOSS, FT
FLOW VIA 60" CONN TO EAST=	67.2	46666.	103.98	19.63	5.30	4.00	1.74

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
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PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT

****EAST PLANT HYDRAULIC PROFILE:
****PLANT FLOW= 30 MGD

VENTURI TO AERATOR:

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Venturi Meter, 48x20	1	30	20833.	46.42	12.57	3.69		149.46	7.25	142.21
(contract/expand)					2.18	21.28	0.09	142.21	0.63	141.58
48" 90d Bend	1	30	20833.	46.42	12.57	3.69	0.23	141.58	0.05	141.53
48x36 Reducer	1	30	20833.	46.42	7.07	6.57	0.08	141.53	0.05	141.48
36" 11 1/4d Bend	1	30	20833.	46.42	7.07	6.57	0.06	141.48	0.04	141.44
36" 90d Bend	1	30	20833.	46.42	7.07	6.57	0.20	141.44	0.13	141.30
36" Tee (run)	1	30	20833.	46.42	7.07	6.57	0.24	141.30	0.16	141.14
36x24 Reducer (sudden)	1	30	20833.	46.42	3.14	14.78	0.28	141.14	0.95	140.19
24" Gate Valve	1	30	20833.	46.42	3.14	14.78	0.10	140.19	0.34	139.86
24" Flow Contr (plug)	1	30	20833.	46.42	3.14	14.78	0.22	139.86	0.75	139.11
24" Gate Valve	1	30	20833.	46.42	3.14	14.78	0.10	139.11	0.34	138.77
24x30 Enlargr (sudden)	1	30	20833.	46.42	3.14	14.78	0.13	138.77	0.44	138.33
30" Tee Specl (branch)	1	30	20833.	46.42	4.91	9.46	0.72	138.33	1.00	137.33
30" Tee (branch)	1	30	20833.	46.42	4.91	9.46	0.72	137.33	1.00	136.33
30" Outlet to Aerator	1	30	20833.	46.42	4.91	9.46				
Distr. Flume					21.00	2.21	1.00	136.33	1.31	135.02
Contr. Aerator Flume	1	30	20833.	46.42	21.00	2.21				
					9.00	5.16	0.20	135.02	0.07	134.95
Flume to 14" Inlet	12	2.5	1736.1	3.87	1.07	3.62	0.50	134.95	0.10	134.85
14" Gate Valve	12	2.5	1736.1	3.87	1.07	3.62	0.10	134.85	0.02	134.83
Aerator Discharge	12	2.5	1736.1	3.87				134.83	0.50	134.33

[Discharge uses weir equation Q=CLH^1.5 @ L=2piR, R=7", C=3]

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Distribution Plate	30510	0.00	0.68	0.00	0.00	4.46	0.61	134.16	0.83	133.33

[Uses orifice eq, 6 trains @ 5085 1/4" orifices ea: Q=CA(2gh)^0.5]

AERATOR THROUGH RAPID & SLOW MIX TO CLARIFIER WEIRS:

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Contr Flume @ Slakers*	1	30	20833.	46.42	25.54	1.82	0.10	124.11	0.01	124.10
R Mix Entrance*	1	30	20833.	46.42	14.09	3.29	1.50	124.10	0.25	123.85
R Mix Exit to Dstr Fl*	1	30	20833.	46.42	28.40	1.63	2.50	123.85	0.10	123.74
Distr Flume Slots (ea)	24	1.25	868.05	1.93	1.63	1.19	2.50	123.74	0.05	123.69
Coll Flume Slots (ea)*	29	1.03	718.39	1.60	1.59	1.00	2.50	123.69	0.04	123.65

[Floor el.125.50] *denotes flux area dependent on WSEL

	1939:	1947:
HGL BALANCE BETWEEN CLARIFIERS:		
HGL at Collection Flume:	123.65	123.65

A. FLOW TO 1939 CLARIFIER: 10 MGD SLR= 884.43 GPD/SF

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Tunnel to Clarifier	1	10	6944.4	15.47	36.00	0.43	1.50	123.65	0.00	123.64
Clar. Weir (v-notch)	424	0.02	16.378	0.04				123.64	0.19	123.46

[Uses weir eq Q=2.4381 H^2.5 for 424 v-notches] [Top of weir=123.67, depth=0.2083]

B. FLOW TO 1947 CLARIFIER: 20 MGD SLR= 873.29 GPD/SF

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Tunnel Entrance	1	20	13888.	30.95	25.00	1.24				

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
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PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
					42.00	0.74	1.00	123.65	0.02	123.64
Tunnel to Clarifier	1	20	13888.	30.95	42.00	0.74	1.00	123.64	0.01	123.63
Clarif Weir (v-notch)	605	0.03	22.956	0.05				123.63	0.21	123.42

[Uses weir eq Q=2.4381 H^2.5 for 605 v-notches] [Top of weir=123.625, depth=0.2083]

FLOW TO FINAL SETTLING BASIN ENTRANCE WEIR: (Weir plate removed)
==

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
HGL below 1939 Weir		10	6944.4	15.47	45.50	0.34	2.50	123.34	0.00	123.33
HGL below 1947 Weir		20	13888.	30.95	25.00	1.24	1.50	123.37	0.04	123.33
Final SB Entrance Weir		30	20833.	46.42				123.33	0.50	122.83 weir

[Uses weir eq Q=3LH^1.5, where L=44]

FINAL SETTLING BASIN TO FILTER FLUME:
====================================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
SB Coll Fl Slots (ea)*	33	0.90	631.31	1.41	1.06	1.33	2.50	122.65	0.07	122.58
Coll Fl to Filter Fl*	1	30	20833.	46.42	25.25	1.84				
					37.88	1.23	1.00	122.58	0.03	122.55

****CENTRAL PLANT HYDRAULIC PROFILE:
****PLANT FLOW= 130 MGD
YARD TO AERATOR:
================

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
66" Tee at 48" (run)	1	130	90277.	201.15	23.76	8.47	0.23	158.48	0.26	158.22
66" 90d Bend	1	130	90277.	201.15	23.76	8.47	0.24	158.22	0.27	157.95
66x60 Reducer	1	130	90277.	201.15	19.63	10.24	0.01	157.95	0.02	157.94
60" 30d Bend	1	130	90277.	201.15	19.63	10.24	0.10	157.94	0.16	157.77
60" 30d Bend	1	130	90277.	201.15	19.63	10.24	0.10	157.77	0.16	157.61
60" Tee Spcl (branch)	1	130	90277.	201.15	19.63	10.24	0.50	157.61	0.81	156.79
60" Butterfly Valve	1	130	90277.	201.15	19.63	10.24	0.30	156.79	0.49	156.31
60x30 Reducer	1	130	90277.	201.15	4.91	40.98		156.31	2.26	154.05
30" Tee (run)	1	130	90277.	201.15	4.91	40.98	0.24	154.05	6.26	147.79
30" Butterfly Valve	1	130	90277.	201.15	4.91	40.98	0.30	147.79	7.82	139.97
30x60 Enlarger	1	130	90277.	201.15	4.91	40.98	0.09	139.97	2.35	137.62
60" 90d Bend	1	130	90277.	201.15	19.63	10.24	0.24	137.62	0.39	137.23
60" 90d Bend	1	130	90277.	201.15	19.63	10.24	0.24	137.23	0.39	136.84
60x84 Enlarger	1	130	90277.	201.15	19.63	10.24	0.06	136.84	0.10	136.74
84" to 14" Inlet	34	3.82	2655.2	5.92	1.07	5.53	0.50	136.74	0.24	136.50
14" 90d Bend	34	3.82	2655.2	5.92	1.07	5.53	0.26	136.50	0.12	136.38
14" Gate Valve	34	3.82	2655.2	5.92	1.07	5.53	0.10	136.38	0.05	136.33
Aerator Discharge	34	3.82	2655.2	5.92				136.33	0.66	135.67
[Discharge uses weir equation Q=CLH^1.5 @ L=2piR, R=7", C=3]										
Distribution Plate	216801	0.00	0.42	0.00	0.00	2.72	0.61	136.14	0.31	135.83

[Uses orifice eq, 19 trains assuming 12753 1/4" orifices ea: Q=CA(2gh)^0.5]
[Top of aerator distribution wall=137.67; plate @ 135.83]

AERATOR THROUGH RAPID MIX:
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PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
R Mix Entrance*	1	130	90277.	201.15	54.01	3.72	1.00	126.92	0.22	126.70
R Mix Exit*	1	130	90277.	201.15	33.83	5.95	1.00	126.70	0.55	126.15

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
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PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT

[Floor el.128.0] *denotes flux area dependent on WSEL

HGL BALANCE BETWEEN CLARIFIERS: West: East:
HGL at Rapid Mix exit: 126.15 126.15

A. SLOW MIX TO WEST CLAR WEIRS: 70.80 MGD PRIMARY SLR= 2757.0 GPD/SF

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Flume Split*	1	70.8	49166.	109.55	14.94	7.33	1.40	126.15	1.17	124.98
Distr Flume Slots (ea)	100	0.70	491.66	1.10	1.38	0.80	2.50	124.98	0.02	124.96
Clarifier Baffle Wall	53	1.33	927.67	2.07	14.50	0.14	2.50	124.96	0.00	124.96
Wooden Baffle Wall	96	0.73	512.15	1.14	8.45	0.14	2.50	124.96	0.00	124.95
Effl Weir (v-notch)	2020	0.03	24.339	0.05				124.95	0.00	124.95

[Uses weir eq Q=2.4381 H^2.5 for 2020 v-notches] [Top of weir=124.5, depth=0.125] [OF 125.0]
CHECK TO SEE IF WEIRS SUBMERGED BY LAUNDER DEPTH

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Effl Launder Upstream	10	7.08	4917	10.96	[Uses Benefield eq]			127.27	2.31	124.95
Effl Launder Cr Depth	10	7.08	4917	10.96	Crit Depth at Launder Outlet=				1.34	122.13

B. SLOW MIX TO EAST CLAR WEIRS: 59.20 MGD PRIMARY SLR= 2305.2 GPD/SF

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Flume Split*	1	59.2	41111.	91.60	14.19	6.45	2.20	126.15	1.42	124.73
Distr Flume Slots (ea)	100	0.59	411.11	0.92	1.38	0.67	2.50	124.73	0.02	124.71
Clarifier Baffle Wall	53	1.12	775.68	1.73	14.50	0.12	2.50	124.71	0.00	124.71
Wooden Baffle Wall	96	0.62	428.24	0.95	8.45	0.11	2.50	124.71	0.00	124.71
Effl Weir (v-notch)	2020	0.03	20.35	0.05				124.71	0.00	124.71

[Uses weir eq Q=2.4381 H^2.5 for 2020 v-notches] [Top of weir=124.5, depth=0.125] [OF 125.0]
CHECK TO SEE IF WEIRS SUBMERGED BY LAUNDER DEPTH

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Effl Launder Upstream	10	5.92	4111	9.16	[Uses Benefield eq]			126.77	2.05	124.71
Effl Launder Cr Depth	10	5.92	4111	9.16	Crit Depth at Launder Outlet=				1.19	122.13

CO2 BASIN TO OUTLET BOX:

HGL BALANCE BETWEEN CO2 BASINS: West: East:
HGL in CO2 Basins Upstream: 124.95 124.71
FLOW THROUGH SLGATE/PIPE: 78 52

EAST SECTION: [Overflow el.124.5]

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
72" Inlet fr CO2 Basin	1	52	36111.	80.46	28.27	2.85	0.70	124.71	0.09	124.62
72" Butterfly Valve	1	52	36111.	80.46	28.27	2.85	0.30	124.62	0.04	124.59
2-72" 90d Bends	1	52	36111.	80.46	28.27	2.85	0.24	124.59	0.06	124.53
72" Outlt to CO2 Basin	1	52	36111.	80.46	28.27	2.85	1.00	124.53	0.13	124.40

WEST SECTION:

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
84x60 Sluice Gate	1	78	54166.	120.69	35.00	3.45	1.50	124.95	0.28	124.68
84x60 Sluice Gate	1	78	54166.	120.69	35.00	3.45	1.50	124.68	0.28	124.40

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Contraction @ Screens	1	130	90277.	201.15	136.00	1.48	2.00	124.40	0.07	124.33

OUTLET BOX TO FILTER FLUME:

PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
84" Entrance	1	130	90277.	201.15	38.48	5.23	0.50	124.33	0.21	124.12

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
HYDRAULIC PROFILE
21-Nov-90

PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Recarb turbulence	1	130	90277.	201.15	38.48	5.23	2.00	124.12	0.85	123.27
84" 30d Bend	1	130	90277.	201.15	38.48	5.23	0.10	123.27	0.04	123.23
84" 30d Bend	1	130	90277.	201.15	38.48	5.23	0.10	123.23	0.04	123.19
84" Exit to Flume	1	130	90277.	201.15	38.48	5.23	1.00	123.19	0.42	122.76
Flume Contraction	1	130	90277.	201.15	38.48	5.23	0.50	122.76	0.21	122.55

FILTER GALLERY:

Filter Gallery contains three types of filters:

1. Filters 1 to 4: 24" ROF — 18" gravel, 36" media — 8.25 lf weir per filter, pipe lateral underdrains
2. Filters 5 & 6: 20" ROF — 18" gravel, 36" media — 8.75 lf weir per filter, pipe lateral underdrains
3. Filters 7 to 14: 20" ROF — 10" gravel, 33" media — 8.25 lf weir per filter, Leopold underdrains

FLOW FROM EAST PLT TO FILTERS: 30 MGD
FLOW FROM CENTRAL PLT TO FILTERS: 130 MGD
TOTAL FLOW TO FILTERS: 160 MGD

1. FILTERS 1 TO 4:

48" Entrance	14.285	11.2	7777.7	17.33	12.57	1.38	0.50	122.55	0.01	122.53
48" Gate Valve	14.285	11.2	7777.7	17.33	12.57	1.38	0.10	122.53	0.00	122.53
48" Outlet to Filter	14.285	11.2	7777.7	17.33	12.57	1.38	1.00	122.53	0.03	122.50
Clean Media (3'/0.5mm)	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	122.50	5.56	116.94
Gravel Underdrain	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	116.94	0.16	116.78
Underdrn Pipe Latrls	130000	0.00	0.8547	0.00			0.61	116.78	0.26	116.52
[Uses orifice eq Q=19.636C(d^2)(H^0.5), w/9100 3/8" holes/fltr] [Lateral CL=114.75]										
3x4 Tee (branch)	1857.1	0.08	59.829	0.13	0.05	2.72	1.06	116.52	0.12	116.40
3x6 Enlarger	1857.1	0.08	59.829	0.13	0.05	2.72	0.27	116.40	0.03	116.37
Outlet to Filter Flume	1857.1	0.08	59.829	0.13	0.20	0.68	1.00	116.37	0.01	116.37
36" Inlet	14.285	11.2	7777.7	17.33	7.07	2.45	0.50	116.37	0.05	116.32
36" Cross (branch)	14.285	11.2	7777.7	17.33	7.07	2.45	0.70	116.32	0.07	116.25
36x24 Reducer (sudden)	14.285	11.2	7777.7	17.33	3.14	5.52	0.28	116.25	0.13	116.12
24" Gate Valve	14.285	11.2	7777.7	17.33	3.14	5.52	0.10	116.12	0.05	116.07
24" ROF Cntrllr	14.285	11.2	7777.7	17.33	3.14	5.52	14.33	116.07	6.77	109.30
24" 90d Bend	14.285	11.2	7777.7	17.33	3.14	5.52	0.20	109.30	0.09	109.21
24" Outlet to Clrwell	14.285	11.2	7777.7	17.33	3.14	5.52	1.00	109.21	0.47	108.74
Clearwell Weir	14.285	11.2	7777.7	17.33				108.74	0.74	108.00
[Uses weir eq Q=3.33LH^1.5, where L=8.25 per filter]										

2. FILTERS 5 & 6:

48" Entrance	14.285	11.2	7777.7	17.33	12.57	1.38	0.50	122.55	0.01	122.53
48" Gate Valve	14.285	11.2	7777.7	17.33	12.57	1.38	0.10	122.53	0.00	122.53
48" Outlet to Filter	14.285	11.2	7777.7	17.33	12.57	1.38	1.00	122.53	0.03	122.50
Clean Media (3'/0.5mm)	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	122.50	5.56	116.94
Gravel Underdrain	14.285	11.2	7777.7	17.33	1400	5.56	=gpm/sf	116.94	0.16	116.78

WICHITA WATER TREATMENT PLANT -- EAST PLANT & CENTRAL PLANT
HYDRAULIC PROFILE
21-Nov-90

PLANT LOCATION/ITEM	NO. TRAINS	FLOW PER TRAIN: MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
Underdrn Pipe Latrls	130000	0.00	0.8547	0.00			0.61	116.78	0.26	116.52
[Uses orifice eq Q=19.636C(d^2)(H^0.5), w/9100 3/8" holes/fltr]								[Lateral CL=114.75]		
3x4 Tee (branch)	1857.1	0.08	59.829	0.13	0.05	2.72	1.06	116.52	0.12	116.40
3x6 Enlarger	1857.1	0.08	59.829	0.13	0.05	2.72	0.27	116.40	0.03	116.37
Outlet to Filter Flume	1857.1	0.08	59.829	0.13	0.20	0.68	1.00	116.37	0.01	116.36
36" Inlet	14.285	11.2	7777.7	17.33	7.07	2.45	0.50	116.36	0.05	116.32
36" Cross (branch)	14.285	11.2	7777.7	17.33	7.07	2.45	0.70	116.32	0.07	116.25
36x20 Reducer (sudden)	14.285	11.2	7777.7	17.33	2.18	7.94	0.35	116.25	0.34	115.91
20" Gate Valve	14.285	11.2	7777.7	17.33	2.18	7.94	0.10	115.91	0.10	115.81
20" ROF Controller	14.285	11.2	7777.7	17.33	2.18	7.94	6.05	115.81	5.93	109.88
20" 90d Bend	14.285	11.2	7777.7	17.33	2.18	7.94	0.20	109.88	0.20	109.69
20" Outlet to Clrwell	14.285	11.2	7777.7	17.33	2.18	7.94	1.00	109.69	0.98	108.71
Clearwell Weir	14.285	11.2	7777.7	17.33				108.71	0.71	108.00
[Uses weir eq Q=3.33LH^1.5, where L=8.75 per filter]										

3. FILTERS 7 TO 14:
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PLANT LOCATION/ITEM	NO. TRAINS	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	UPSTRM HGL, FT	HEAD LOSS, FT	DWNSTRM HGL, FT
48" Entrance	13.793	11.6	8055.5	17.95	12.57	1.43	0.50	122.56	0.02	122.54
48" Butterfly Valve	13.793	11.6	8055.5	17.95	12.57	1.43	0.25	122.54	0.01	122.54
48" Outlet to Filter	13.793	11.6	8055.5	17.95	12.57	1.43	1.00	122.54	0.03	122.50
Cl Media (2.75'/0.5mm)	13.793	11.6	8055.5	17.95	1450	5.56	=gpm/sf	122.50	5.09	117.41
Gravel Underdrain	13.793	11.6	8055.5	17.95	1450	5.56	=gpm/sf	117.41	0.09	117.32
Underdrain - Block	13.793	11.6	8055.5	17.95				117.32	0.50	116.82
36" Inlet	13.793	11.6	8055.5	17.95	7.07	2.54	0.50	116.82	0.05	116.77
36" Cross (branch)	13.793	11.6	8055.5	17.95	7.07	2.54	0.70	116.77	0.07	116.70
36x20 Reducer (sudden)	13.793	11.6	8055.5	17.95	2.18	8.23	0.35	116.70	0.37	116.33
20" Butterfly Valve	13.793	11.6	8055.5	17.95	2.18	8.23	0.30	116.33	0.32	116.02
20" ROF Controller	13.793	11.6	8055.5	17.95	2.18	8.23	5.71	116.02	6.00	110.01
20" 90d Bend	13.793	11.6	8055.5	17.95	2.18	8.23	0.20	110.01	0.21	109.80
20" Outlet to Clrwell	13.793	11.6	8055.5	17.95	2.18	8.23	1.00	109.80	1.05	108.75
Clearwell Weir	13.793	11.6	8055.5	17.95				108.75	0.75	108.00
[Uses weir eq Q=3.33LH^1.5, where L=8.25 per filter]										

COMPARISON BETWEEN FILTER GROUPS --

	DEPTH @ FLUME:	OVER WEIR:	
1. FILTERS 1 TO 4:	122.55	0.74	
2. FILTERS 5 & 6:	122.55	0.71	AVE FL DEPTH= 122.55
3. FILTERS 7 TO 14:	122.56	0.75	

Note: Depth at filter flume should be approximately the same, except for loss through 60" diam connection between Central & East.

	MGD	GPM	CFS	AREA, SF	VEL, F/S	HL COEFF	HEAD LOSS, FT
FLOW VIA 60" CONN TO EAST=	37.2	25833.	57.56	19.63	2.93	4.00	0.53