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Water and Air Research, Inc.

TREATMENT OF THE EAST GHOR CANAL WATER FOR AMMAN JORDAN

Prepared By

WATER AND AIR RESEARCH, INC.
Gainesville, Florida

For

Agency for International Development
Washington, D.C.

Contract No. NEB-018-C-00-1021-00
Project No.--

Technical Services - Sub Project Review
Of EGMC - Pipeline and Treatment Plant Design

March 1981

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TREATMENT OF THE EAST GHOR CANAL WATER
FOR AMMAN, JORDAN

1.0 RAW WATER

The source of water for the treatment plant will be the East Ghor Main Canal. The water in the canal comes principally from the drainage of the east bank of the Jordan River. The canal runs parallel to the river and provides irrigation water to a large and rich agricultural area.

From observation, bench scale testing and review of the limited data available there is nothing to indicate any serious treatment problems. The water comes mainly from an area where there is an abundance of limestone so that it is high in alkalinity and relatively hard. As expected it is high in dissolved solids but not excessive. During the rainy season turbidity is high but about half is settleable silt. In general it is the kind of water which could be expected in any stream which is fed by surface runoff. The heavy silt will settle out quickly while a small amount of cationic polymer will accelerate the removal of more of the lighter turbidity in the presedimentation basins. During the dry season there is an indication of algae but the data is not clear in terms of numbers.

The species indicated as present in the canal water did not include the most serious offenders in terms of taste and odor producers. This doesn't mean that they were not present; they were not found in the samples examined. No real assessment of the algae problem will be possible until the hot season when further work can be done on algal counts.

2.0 TREATABILITY TESTING (BENCH SCALE) OF THE EAST GHOR CANAL WATER -

2.1 General

Testing was carried out in the NRA laboratories with the assistance of the NRA staff for space, glassware and use of pH meter, balances and other necessary laboratory equipment. Dr. Joseph Saman and Eng. Yousef Qassar of the JVA cooperated in carrying out the work.

In general, the reaction of the raw water with all coagulants was instantaneous. Minute floc appeared immediately indicating the speed of the reaction. This would be expected of this type of water.

All samples were collected at a point in the canal near the proposed inlet pump station site. They were collected in 20 liter plastic containers and taken to the laboratory where the bench scale testing was carried out. Testing was done in the conventional manner of rapid mixing of the coagulant, followed by flocculation and settling. Tests were run also to determine the "Direct Filtration Potential" of this raw water.

2.2 Flocculation and Settling Tests of Turbid Water

The raw water at this season was very turbid--from 114 to 156. The first samples were collected on March 4, 1981, just after several days of rain. Figure 1 gives the range of results from the settling tests carried out with this water. The results of this testing indicates that $FeCl_3$ gives somewhat better results but requires a dose of about 80 percent of the alum for about equal performance. This would be uneconomical since the cost differential is much more than 20 percent of the difference in consumption. An alum dose of 30 mg/L is about equivalent to 25 mg/L of $FeCl_3$ on the settling velocity distribution curve. The alum would be more economical and should be the coagulant used. Theoretically ferric should have been much more effective but the tests did not confirm this prognosis. An alum dose in the range of 22 to 30 mg/L should produce a satisfactory settled water for filtration for the raw water at this season.

The time required for flocculation is about 20 minutes. In the range of 3 to 5 cm/min. loading all the tests produced a settled water which would filter well without putting an excessive load on the filters. The lower velocity gradients produced a settled water of lower turbidity. Due to the heavy water turbidity and therefore its high mass, the floc formed very quickly and much of it settled out in the jars during flocculation.

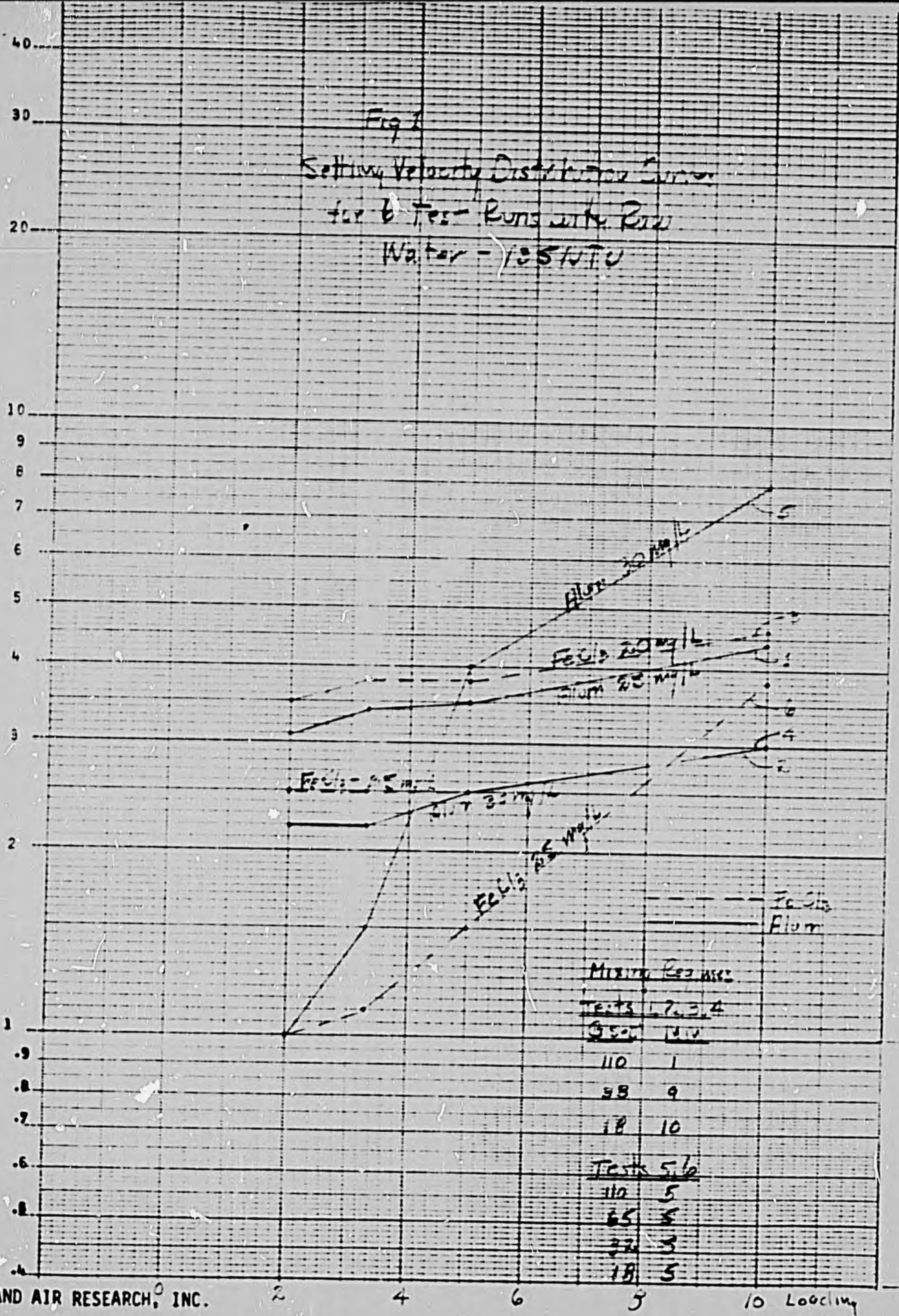


Fig 1
Settling Velocity Distribution Curve
for 6 Test Runs with Raw
Water - 135 NTU
40
30
20
10
9
8
7
6
5
4
3
2
1
.9
.8
.7
.6
.5
.4
2
4
6
8
10
Loading
Alum
FeCl3
Mixing Regimes
Tests 1, 2, 3, 4
110 1
38 9
18 10
Tests 5, 6
110 5
65 5
32 5
18 5

2.3 Flocculation and Settling Tests of Decanted Raw Water

In order to try to simulate the water which will reach the treatment plant after presettling in the tanks near the intake, the samples collected on March 7, 1981 were allowed to settle and water was decanted off the top. This water was in the turbidity range of 12.5 to 17.5 NTU. These results are shown in Figure 2. As might be expected, the lack of the turbid mass to form heavy floc is immediately apparent. Settled water turbidities in the high loading range are high. Also when the alum dose drops off the settled water turbidity immediately goes up into the unsatisfactory range even at low loadings. The test run at 30 min. floc time produced a somewhat better settled water at loadings of 2 to 5 cm/min.

The dramatic effect of the addition of a very small dose of non-ionic polymer (80 NP) is shown in Figure 3. In this test two different polymers were applied to two separate samples. The difference between samples with and without polymer is very great in the high loading ranges. The 80 NP produced a good settled water even at the highest loading while NIOIP was somewhat less effective.

2.4 Testing For Destabilization

Destabilization of colloidal turbidity in raw water for direct filtration is a relatively new development in terms of treatment. This is a process somewhat different than traditional coagulation and flocculation. In the destabilization treatment a type of micro coagulation occurs while settleable floc which is normally obtained in the conventional sense of water treatment, is not formed. At the same time, however, filterable floc is formed. The difference is that in most waters, only a very small coagulatnt dose is required to achieve destabilization while a large dose is needed to obtain settleable floc. Furthermore, in most situations a large part of the treatment units of conventional design such as settling basins, flocculation and sludge handling systems is eliminated. If filter runs of a reasonable length and filtered water quality can be maintained, treatment by destabilization and direct filtration is a very economical method.

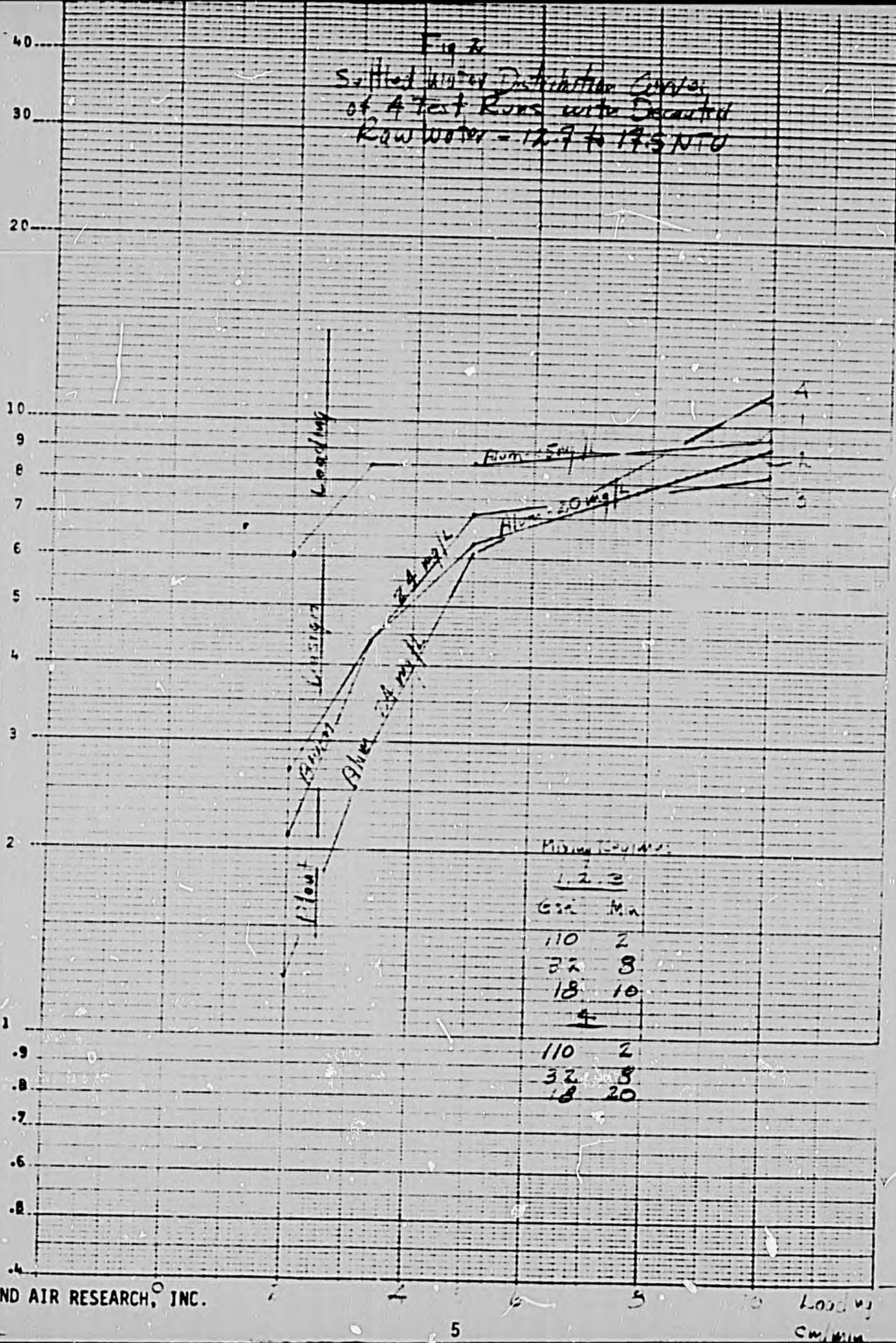


Fig 2
Settled Water Distribution Curves of 4 Test Runs with Selected Raw Water - 12.9 to 17.5 NTU
NTU
40
30
20
10
9
8
7
6
5
4
3
2
1
.9
.8
.7
.6
.5
.4
Loading
Alum - 5mg/L
Alum - 20mg/L
Alum
4
1
2
3
Mixing Regimes
1, 2, 3
110 2
32 8
18 10
4
110 2
32 8
18 20
2
4
6
8
10
Loading
cw/min

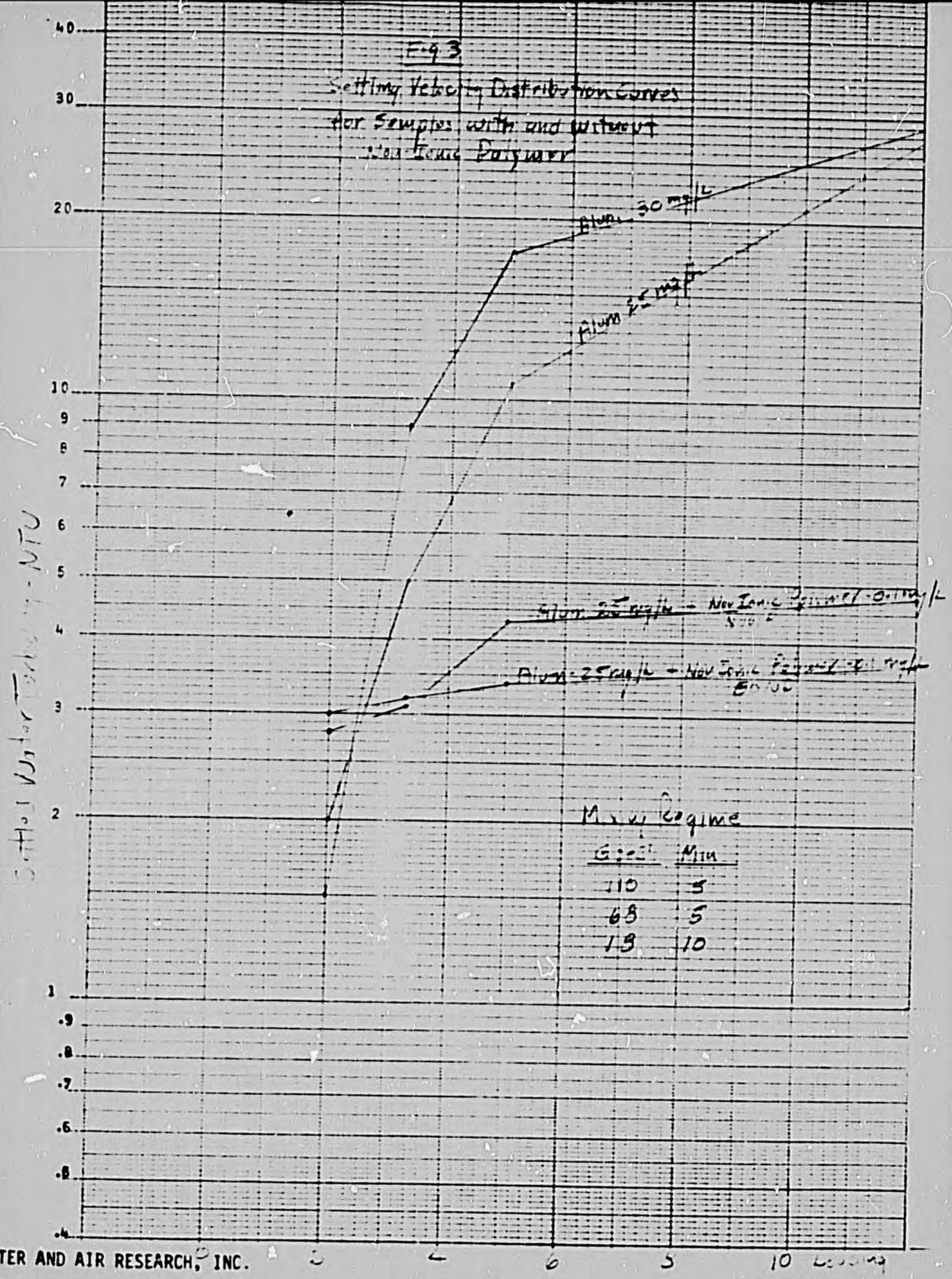


Fig 3
Settling Velocity Distribution Curves
for Samples with and without
Non Ionic Polymer
Alum 30 mg/l
Alum 25 mg/l
Alum 25 mg/l + Non Ionic Polymer
Alum 25 mg/l + Non Ionic Polymer
Mixing Regime
G
110 5
68 5
13 10
40
30
20
10
9
8
7
6
5
4
3
2
1
.9
.8
.7
.6
.5
.4
2
4
6
8
10

Destabilization tests were carried out on both the turbid raw water 135 and 114 NTU as well as the decanted water in the range of 12.5 at 17.5 NTU. Results of these tests are contained in Figures 4 and 5.

These tests were carried out with one liter samples dosed with the quantities and reagents indicated. Stirring was done at the highest speed of the stirring machine, about 120 RPM. After one minute, the speed was reduced to 20 RPM and continued for 3 to 5 minutes. This latter is about the time required for the water to go from the point of mixing to the level of the filter media.

After mixing, the water was filtered through No. 40 Whatman filter paper. This grade of paper has proven to produce filtered water turbidities slightly higher than those from a dual media filter. In other words, the turbidities obtained from filter paper are conservative relative to those which will be produced by the pilot or plant filter.

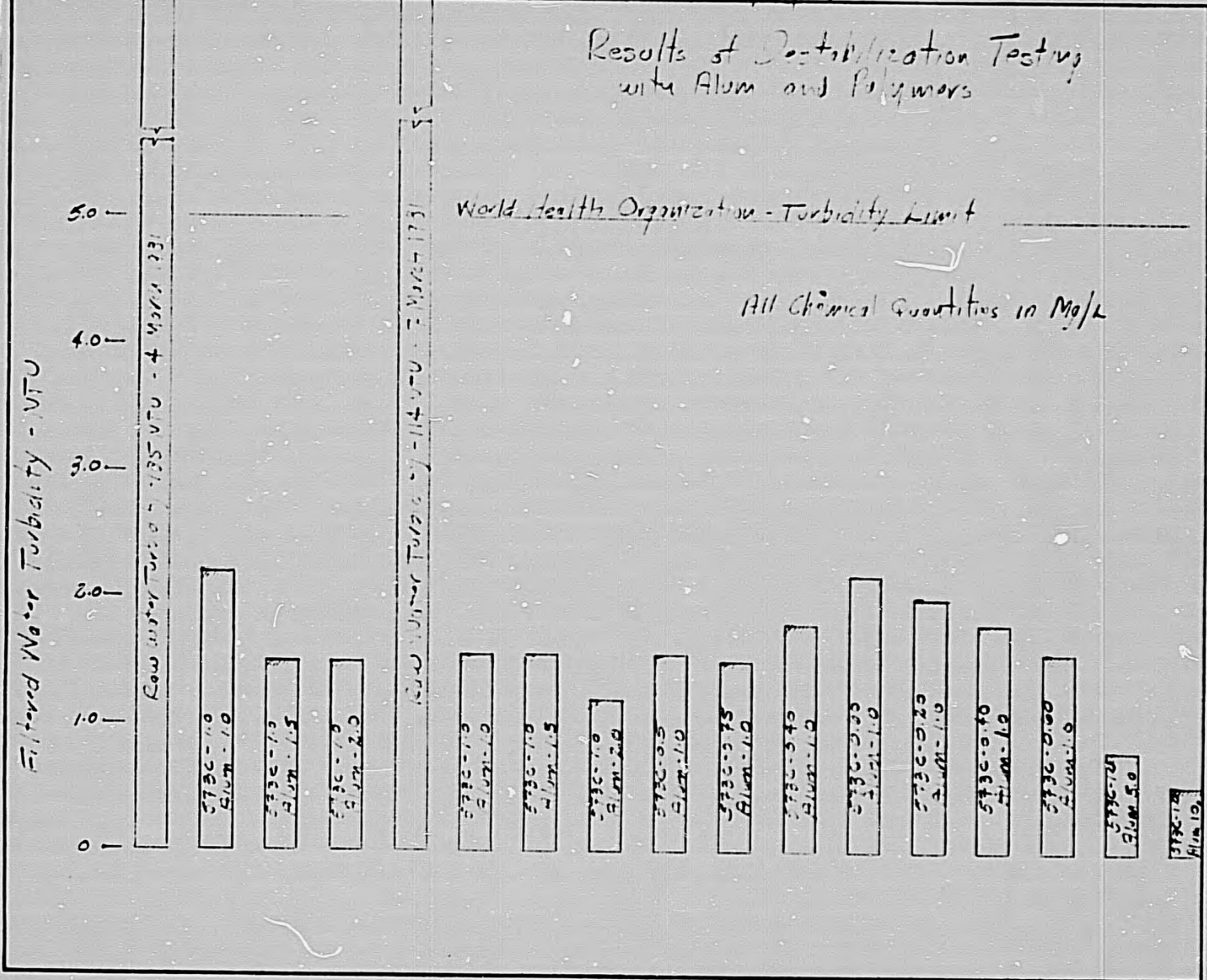
As is clear in Figure 4, even at the highest turbidity of 135 NTU the doses of 1.0 mg/L of alum and 0.2 mg/L of American Cyanamid Cationic Polymer 573C produces a filtered water considerably below the World Health Standard which is that adopted by the Jordanian Government.

In Figure 5 the results of further destabilization tests with polymer only indicates that alum is not required for good destabilization. A dose of 0.2 to 0.5 mg/L of three differed cationic polymers proved satisfactory. One of the polymers 515C proved to be much less effective than the others.

2.5 Testing for Extent of Settling in Pre-Sedimentation Tanks

Two large holding tanks have been planned for construction between the intake and the first pump station. These basins are 30 m in diameter and will provide one hour of settling. A large amount of sediment was observed in the bottom of the beakers in the laboratory after treating with a small polymer dose. This would indicate the value of applying a small amount of polymer at the intake in order to accelerate the removal of solids before the water goes to the treatment plant.

Fig 4



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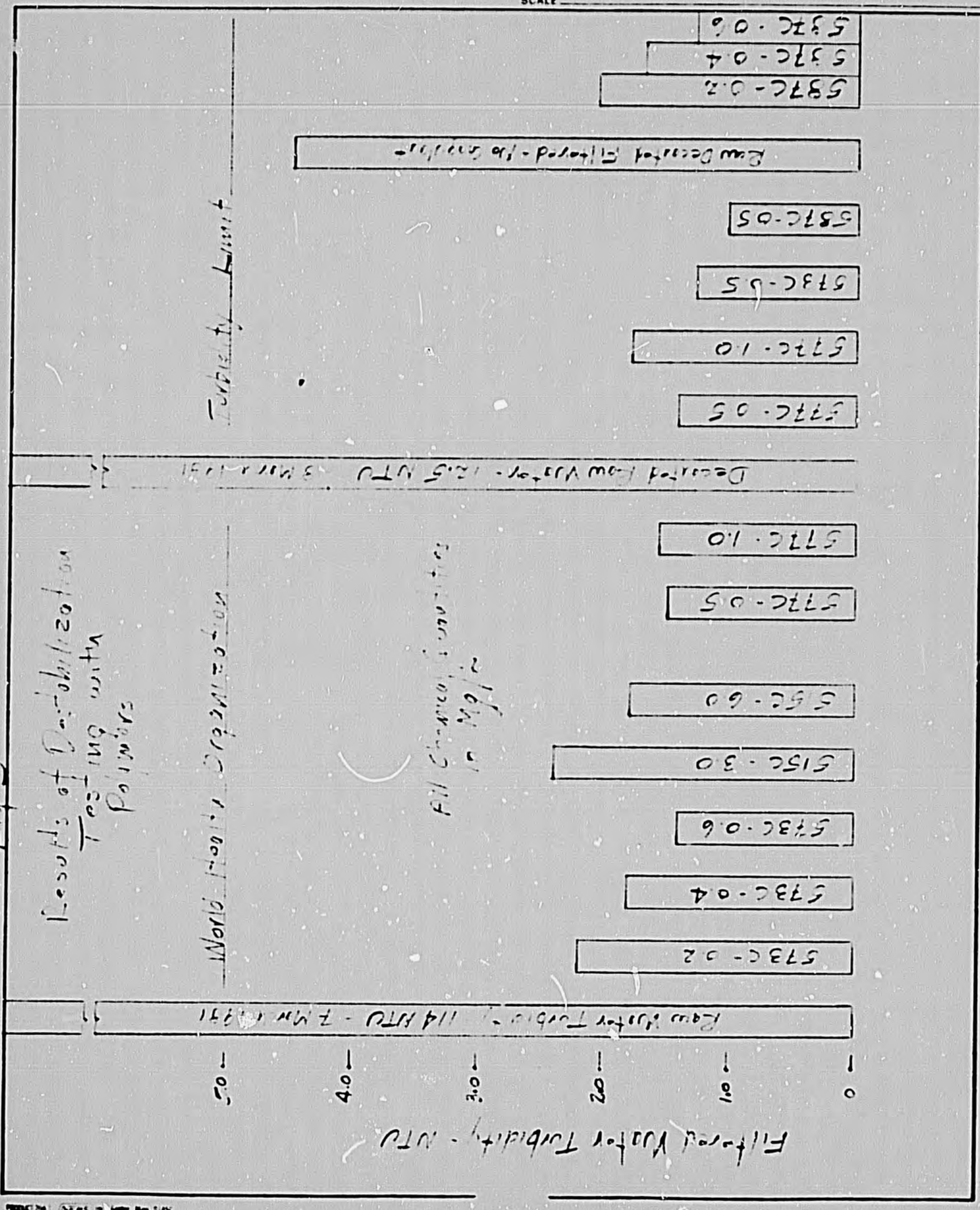

The test was done with two large jars filled to the same level with raw water having a turbidity of 156 NTU. One beaker contained raw water only while the second contained raw water mixed with 0.2 mg/L of cationic polymer. After one hour a sample was drawn from each jar at a point exactly 1/2 the water depth. This point was selected because the large concrete holding tanks are designed so that the bottom half is to provide settling and the outlet will take water off the top half only. The sample taken from the jar containing only raw water had a turbidity of 73 NTU while that which was dosed with 0.2 mg/L of cationic polymer had a turbidity of 15 NTU (see Figure 6).

The supernatent from the latter jar was filtered through No. 40 Whatman filter paper and the resultant turbidity was 4.0. This is a little high for the filtered water, but it indicates that destabilization occurred throughout the entire mass of water with this very small polymer dose. Even though most of the solids had settled out, the remainder still met the World Health Standard for filtered water quality. A slightly higher dose would have reduced the turbidity even more in the settled water and very probably produced a filtered water turbidity from the supernatent in the range of 2.0 NTU or less. In other words, it is probable that no other application of polymer would be necessary prior to filtration in the plant.

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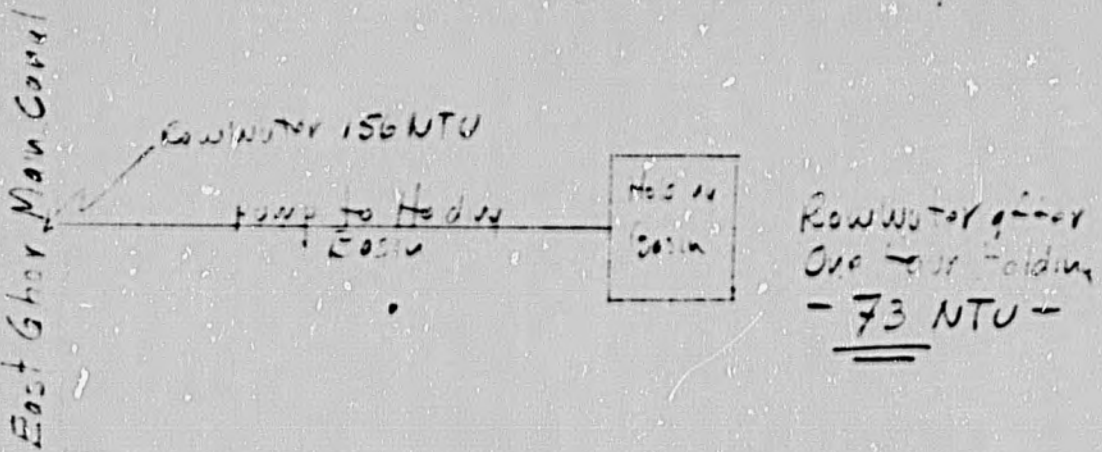
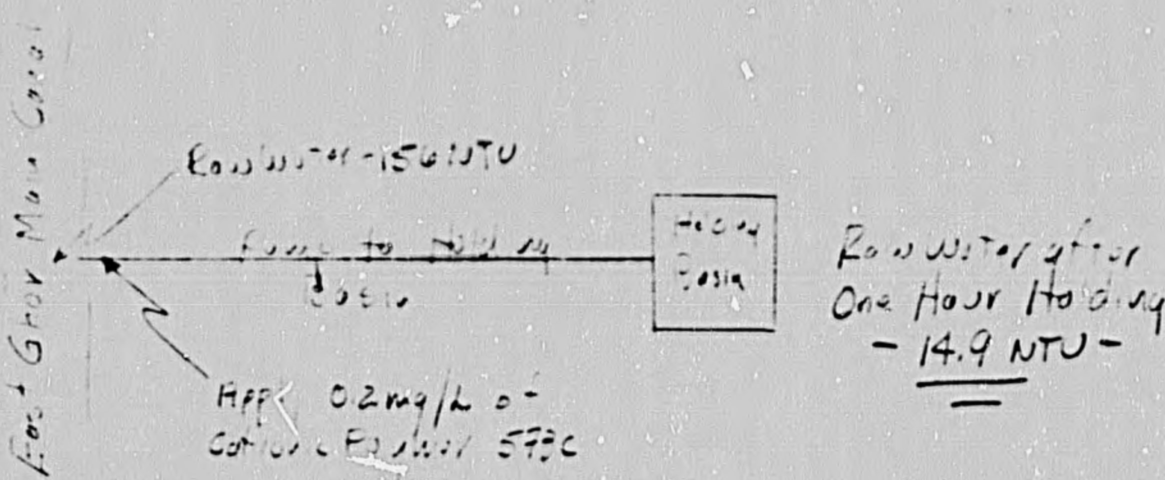
Fig. 6
Bench Scale Simulation of Raw Water Turbidity
Conditions with One Hour Holding at the
Intake STP - March 10 1981
With and Without Polymer





3.0 THE WATER TREATMENT PLANT DESIGN

3.1 General

The plant design conforms to usual practice in the U.S.A. The loadings on all units are conservative and if properly operated, and maintained, the plant will produce a good quality water.

It must be understood that the plant is expensive as judged by the costs developed by the U.S. EPA and published under the title, "Estimating Water Treatment Costs" in August 1979. Some of the higher costs are due to the importation of equipment and supplies which are obviously more expensive in Amman than in the country of origin. A part, however, is the result of the design concept.

The plant is highly mechanized and sophisticated. All chemicals are mechanically mixed and pumped to their respective points of application. All filter valves are electrically operated, and settled sludge is mechanically collected and removed from the settling basins. Controls and instrumentation are electrical and the plant functions are largely automated through a central logic unit. In other words, hand labor and gravity are minimized while pumps, motors and equipment are maximized.

This increases the initial cost and will require very high level competence in operation. The necessity of good maintenance in this type of a plant is obvious. This requires well-trained technicians to maintain the equipment in good operating condition and a sufficient supply of spare parts.

Conservative loadings in the flocculation basins, low-rate plain settling in very large basins have considerably increased the size of the settling basins, and relatively low filtration rates with constant-rate filter controls increases the cost of filters. These costs may not be lost because they provide for excess capacity which if recognized can be utilized later to produce more treated water.

The nominal design is 1.42 m^3/sec while 2.5 m^3/sec can quite readily be treated in the treatment units as now designed. The hydraulic transport pipes, canals, and ports must be sized, however, for the higher flow to avoid floc breakup at some points and excessive head-loss in others. Under this increased load, the flocculation time would be reduced to just under 19 minutes, the settling velocity would be 4.38 cu/min or 63 $m^3/day/m^2$ and the average filter rate would be just over 400 m/day or about 280 $L/min/m^2$. These loadings are being successfully used in many plants around the world which have well designed and operated mixing, flocculation, settling and filter units.

3.2 Recommended Improvements in the Plant as Designed

Regardless of the plant loadings, several details of the existing design should be modified to improve the treatment efficiency, and reduce operating and maintenance costs.

3.2.1 Initial Mixing of Coagulant

The reaction of this raw water with alum is essentially over in a fraction of a second. It is important therefore for all the coagulant to be mixed with all the raw water instantly. The point of dosing of alum is not clear from the information available. Presumably it is immediately prior to the first motor mixer in the channel shown. This channel does not appear to be designed in such a way that instant mixing of all the coagulant with all the raw water will occur. As a matter of fact, a considerable portion will not participate in the immediate mixing because the channel is 2.5 m wide while the mixing will occur in the center portion only.

A better system would be an inline blender type of mixing in the pipeline just prior to entering the open concrete channel. Another possibility would be a diffuser at a weir which could be easily constructed in the channel as designed and it has the advantage that the operators can see the coagulant being applied.

[More treatment problems arise from faulty coagulant dosing than any other single cause.]

3.2.2 Flocculation

Another problem is that of the baffles in the flocculation basins which separate the sections of each basin where the mixers are in operation.

These sections are approximately 6 x 6 m square. They are separated with a baffle made up of alternating fiberglass channels 20 cm wide mounted on each side of a concrete column. In other words, it is a horizontal lattice of fiberglass channels. These baffles should be replaced with solid walls having only one opening between each section. As designed, there will be excessive short-circuiting. They should be redesigned as indicated in Figure 7.

A redesign of the flocculation basins could accomplish better control of the flow and reduce short circuiting as well as reduce the number of agitator motors and mixing mechanisms.

3.2.3 Settling

It is difficult to construct and maintain long outlet weirs in a completely level position. The difference in elevation common in this take-off system is the source of many problems of excessive floc carry over. This can be easily and economically corrected with perforated, submerged launders. These can be made of light weight plastic and the take-off rate is unaffected by small differences in elevation.

There does not appear to be a compelling reason to provide continuous sludge removal from the settling basins. These could well be modified to manual cleaning with the economy of eliminating the sludge removal equipment as well as their continuous maintenance. Obviously such a system will require a basin shut down every 4 to 6 months and manual cleaning.

3.2.4 Settling Basin Entrance Baffle

In order to insure good distribution of flocculated water across the entrance of the basin a specially designed baffle is recommended. This baffle contains a large number of ports which provide a small head- loss between the flocculation and settling basins. The head-loss must be large enough to assure good distribution over the section but at the same time not introduce a velocity gradient that will break up the carefully formed floc. Attached separately are excerpts from H.E. Hudson's new book which describes this baffle design.

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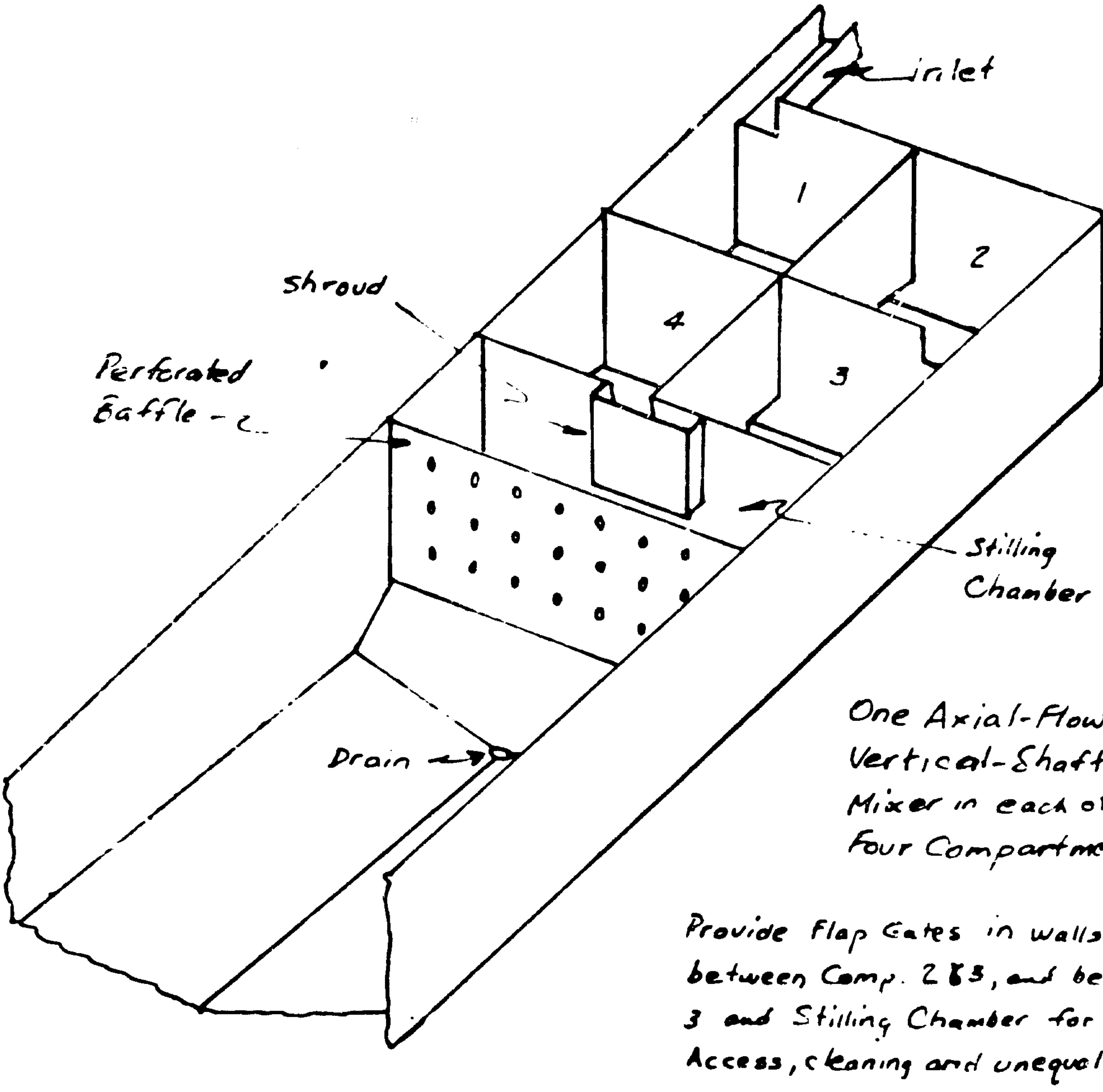

One Axial-Flow Vertical-Shaft Mixer in each of Four Compartments.

Provide Flap Gates in walls between Comp. 2 & 3, and between 3 and Stilling Chamber for Access, cleaning and unequal pressures.

FIG. 7. ISOMETRIC SCHEMATIC OF FLOCCULATOR COMPARTMENT AND SETTLING BASIN INLET.

Not to Scale.

3.2.5 Filters

The design memoranda show about a 5.0 m headloss through the filters which is large. More than half of this loss can be recovered by changing the filter rate control to a declining-rate control system.* This will reduce the headloss, eliminate the expensive constant-rate-of-flow controllers, assure a better quality effluent because the filter flow reduces as the filter becomes clogged, and produce more filtered water during each filtration cycle.

Changing to a declining rate can be accomplished by 1) substituting the rate-of-flow controllers with an orifice plate to limit maximum flow in clean filters; 2) raising the effluent weir structure walls to elevation 886.00 and providing a throttle valve at the chamber outlet to regulate the level in the chamber to provide back-pressure filter control.

The only specifications for filter media in the memoranda were the thickness of each of the media stone, sand and anthracite. It is recommended that the support gravel be "reverse gradation". That is, there should be large diameter gravel on the top section of the support layer as well as the bottom. This was developed by Baylis** and somewhat modified by Hudson*** (excerpt attached separately). This support resists gravel disturbance much better than the conventional support gravel layer.

Since the Jordanian Government has wisely adopted WHO quality standards the filter media can be somewhat more porous. The sand layer in the filter should have an effective size of about 0.6 mm, while the anthracite should be about 1.2 mm. Both sand and anthracite should be highly uniform, with a uniformity coefficient not more than 1.3 to 1.4.

3.2.6 Chemicals and Chemical Feed

Points of chemical application are generally indicated. Contact time is important in many of the applications. For example, Cl_2, K_2MnO_4, and activated carbon may be applied to the raw water at the entrance of the raw water regulating reservoir. There is no detail in the reservoir design to indicate that attention has been given to the matter of short-

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*Chapter 7--AWWA Manual 20126 "Upgrading Existing Water Treatment Plants"

**Chapter 7A--AWWA, "Water Quality and Treatment"

***Chapter 12, pp. 221, "Water Clarification Processes", H.E. Hudson, Jr. Published by Van Nostrand and Reinhold.

circuiting. Since in the control of tastes and odors contact time is very important, the regulating reservoir should be baffled to avoid short-circuiting.

In the design, the use of non-inonic polymer has been anticipated as a flocculation aid. This polymer is best applied in the flocculation basin at a point where the floc has been well-formed but at the same time as early as possible in the flocculation cycle. Also it should be applied at a very high dilution which must be done after the dry polymer is wetted and mixed initially with water at the chemical feed unit. The design should provide for the dilution (not less than 1000:1) as well as the proper point of application (about 4 minutes after flocculation begins).

Chemicals and chemical handling facilities will be required at the intake and pump station No. 1. During some periods of the year copper sulfate will need to be applied to the canal water to control algae growths. Also during the high turbidity season a small dose of cationic poloymer applied at the intake will greatly accelerate the settling of solids in the storage holding basin immediatley prior to pump station No. 1. The pipeline from intake to pump No. 1 will need shock chlorination from time to time. Copper sulfate will obviously be applied by hand at some point in the canal above the intake. The cationic polymer and Cl_2, however, will have to be dosed accurately at the intake pump station.

In order to keep the pipeline free of any possible organic slime, heavy doses of chlorine applied intermittently at pump station No. 1 will be necessary. Possibly once every week or ten days a dose of up to 10 mg/L during one to two hours will be necessary. Whether such a dose will carry a sufficient residual [illegible]l the way to the treatment plant can only be determined after operation begins or in the laboratory by simulating the time lapse from beginning to the end of the pipeline. The chlorine demand will probably vary somewhat throughout the year so that such testing may be necessary on at least bi-weekly basis. The result of such tests would indicate whether or not further chlorination would be required at some intermediate point on the pipeline.

Alum obsered in Africa and the Near East is very lumpy having a large portion in the form of heavy gravel (1" to 2"). This kind of alum may cause serious problems with the type of dosing equipment specified. The alum feed equipment should be able to handle coarse material.

3.2.7 Pre-Settling Tanks Prior to Pump Station No. 1

The purpose of these basins is to remove the maximum amount of solids from the raw water before pumping to the treatment plant. Details of these basins were not available but the design must provide baffling which will reduce the possibility of short circuiting. Unless the basins contain such baffling, some of their effectiveness will be lost.

4.0 DIRECT FILTRATION

The results of the bench scale testing from March 4 to March 10 of the East Ghor Main Canal water indicate clearly and unquestionably that this water can be treated by the destabilization - direct filtration method. This work is described in Section 2.4, "Treatability--Bench Scale Testing". Figures 4 and 5 contain the information on dosages and resultant filtered water turbidity with a wide range of alum and polymer doses as well as those of polymer alone.

Treating the East Ghor Main Canal water by the direct filtration process has several important advantages:

1. The very large settling basins are eliminated.
2. Most of the flocculation system is eliminated.
3. Most of the sludge drying beds are eliminated.
4. Drastically reduce chemical storage and handling.
5. Substantial reduction in chemical costs.
6. With the elimination of the large structures, the possibility of relocating the plant to the original site in the Jordan Valley may be feasible.
7. Relocating the plant to the Jordan Valley site has several further advantages.

    a. The two planned and designed holding basins each 30 m in diameter can serve as the regulating reservoir as well as the pre-settling basins.

b. All the chemical feed and treatment operations will be in the same general area where they will be much easier to control and supervise.

c. The pumps and pipeline will be handling clean water.

d. While there is the possibility of earthquake, the largest structure and probably the most vulnerable will be the planned presedimentation basins. The filter structure is very rigid, full of cross walls and likely to resist any earth tremor much better than the presedimentation basins.

The problem which treatment by direct filtration brings is the requirement of carrying out pilot filter testing. This is the only way in which the filter clogging characteristics can be determined for each loading, dose and filter media. While experience indicates that this method of treatment will be satisfactory, there is no substitute for pilot filter testing. Unfortunately, this will take time. A pilot filter must be obtained and set up. A raw water preparation system is required as well as personnel and equipment for monitoring the results. Also a further disadvantage with the plant at its present site will be the need for a clarifier to separate the solids from the filter wash water. This would substitute the presently planned wash water recovery system. If the site were to move to the Valley, the value of wash water recovery is doubtful.

While the elimination of structures, and in this case the possibility of moving to a better location, are important considerations, the economy in chemical cost and operation may be more important. The alum dose required for conventional treatment of the canal water is from 22 to 28 mg/L. At nominal plant capacity this is 2.7 to 3.5 metric tons per day. At a cost of \$300 per ton, this is \$810 to \$1,050 per day in alum cost. The dose of polymer to eliminate the alum is about .3 to .5 mg/L or 37 to 62 Kg/day at nominal flow. Polymer cost is about \$3.50 per kg. or \$130 to \$217 per day. Based on this calculation, the minimum saving is over \$600 per day, \$220,000 per year, while the maximum would be over \$330,000 per year. At the same time the alum handling and storage cost is practically eliminated.

5.0 CONCLUSIONS

1. With proper operation the treatment plant, as designed, will produce a good quality treated water.

2. The design concept utilizes electric energy, mechanical equipment and automation in contrast to more extensive use of manual labor, hydraulic mixing and gravity which can reduce construction costs and maintenance needs.

3. The plant will require high-level competence in its operation and maintenance. Water treatment skills of this level are not available in Jordan.

4. Loadings on the treatment units are conservative. If channels, ports, and pipes are sized for higher flows the plant can readily treat 2.5 to 3.0 m^3/sec in the future.

5. Bench scale testing on both turbid and clarified raw water confirmed the advantage of alum as the choice for coagulant.

6. A small dose of non-ionic polymer greatly accelerates the clarification process.

7. A small dose of cationic polymer significantly accelerates the settling of suspended solids in the raw water pre-treatment basins.

8. Tests for destabilization and direct filtration of the raw water indicated that there is a good possibility of successfully treating the water by this method.

9. It is obvious from the report that Water and Air Research, Inc. takes a considerably different view of the treatment of this water both in concept and in some of the design details. Because of this difference in approach, and a limited budget, our review of the design is quite general.

6.0 RECOMMENDATIONS

In view of the advantages of direct filtration and the possibility that it will be successful, pilot filter testing should begin immediately. Quite probably, the critical times for testing will be in this high-turbidity season and later when algae may be a problem.

During the initial filter test work, JVA engineers should be intimately involved so they can carry on the work for several months. This work need not be continuous but should be done, perhaps, one day per week. In periods of water quality change it should be carried out for a week or two at a time, until treatability is established.

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Attachment A: Inlet Baffle
Excerpt from "Water Clarification Processes" by H.E. Hudson, Jr., published by Van Nostrand Reinhold.

Conventional settling basins have four major zones: (1) the inlet zone; (2) the settling zone; (3) the sludge storage or sludge removal zone; and (4) the outlet zone.

INLET ZONE

For control of the turbulent zone near the settling basin inlet, various baffling methods have been used for admitting water to the settling basin, but the one most successfully used to date has been the perforated baffle. In the design of perforated baffles, four requirements must be met:

1. The head loss through the ports should be about four times higher than the kinetic energy of any approaching velocities in order to equalize flow distribution both horizontally and vertically.
2. To avoid breaking up floc, the velocity gradient through inlet conduits and ports should be held down to a value close to or little higher than that in the last compartment of the flocculators.
3. The maximum feasible number of ports should be provided in order to minimize the length of the turbulent entry zone produced by the diffusion of the submerged jets from the ports in the perforated-baffle inlet.
4. The port configuration should be such as to assure that the discharge jets will direct the flow toward the basin outlet.

The use of a large number of small ports in accordance with item 3 tends to increase the velocity gradients through the ports, as is shown by Table 4-2.

It is desirable for the port diameter to be no more than the thickness of the permeable-baffle wall, so that the hydraulic behavior will cause the jets to emerge in the proper direction. This is one disadvantage in using timber permeable baffles. It can be overcome by using tubular inserts fastened in the openings of the timber wall, made of plastic or asbestos-cement tubes or of wood construction to train the flow properly. Timber baffles alone are probably not desirable for basin inlet perforated walls. Since there is now knowledge of how to design the inlet system properly, there is no longer any need to use makeshift construction that can easily be modified.

The easiest way to commence the design of permeable baffles is to estimate the kinetic energy of flows approaching or passing along the baffle. Stilling baffles or shrouds can be used to reduce these velocities and the resulting kinetic energies to relatively low values, so that in most cases it is safe to use a velocity through the perforated baffles of about 20 or 30 cm/sec. The head loss through cylindrical ports is 1.7 times the velocity head through the ports. The 1.7 figure is based on a 0.7 entry coefficient plus one velocity head. Thus, for a velocity of 30 cm/sec, the head loss will be approximately 7.8 mm at the nominal design flow. This is a relatively small loss, but if the approach velocities are carefully controlled, it will produce satisfactory uniform flow distribution. From the velocity through the ports the total port area may be calculated.

Consider a basin having a flow capacity of 1 m^3/sec as an example. The design proceeds as is shown in Table 8-1. For this example, a mean port velocity of 30 cm/sec has been selected which, with a port coefficient of 1.7, gives the head loss through the ports of 7.8 mm at nominal flow rate. With flow of 1 m^3/sec and this velocity, the total port area required is found to be 3.33 m^2. For each of a number of different port diameters, the number of ports required is computed.

If the settling basins are 22.5 m wide and the depth of flow in the sedimentation zone is 2 m, the area of the permeable-baffle wall is computed to be 45 m^2. Next, a wall space per port is computed. Finally, the square root of this wall space per port is computed, giving the mean spacing between port centerlines.

Assuming that the diagonal port arrangement shown in Fig. 8-2 will be used, the spacing of horizontal rows of ports is calculated as approximately 0.7*a*

TABLE 8-1. Design of Perforated Inlet Baffle.

Assumptions:

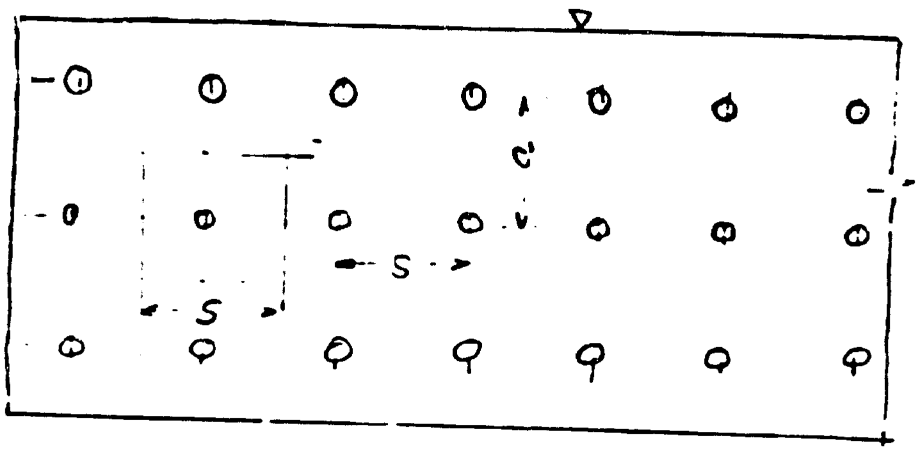
$$Q = 1\ m^3/s,\ V_p = 30\ cm/sec$$

$$\text{Baffle wall: } 2\ m \times 22.5\ m = 45\ m^2$$

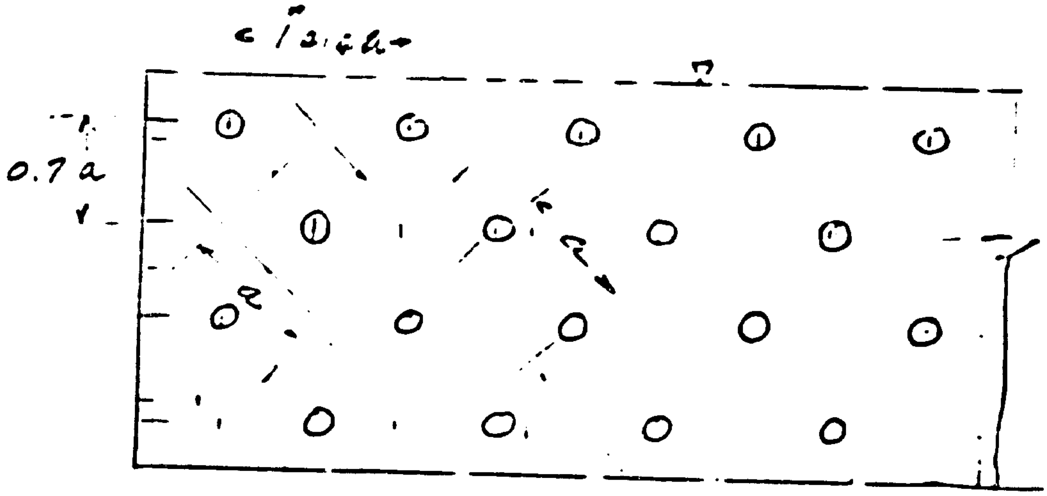
$$\text{Port area} = \frac{Q}{V} = \frac{1.0}{0.3} = 3.33\ m^2$$

$$\text{Head loss} = 1.7 \times \frac{(0.3)^2}{20} = 7.8\ mm$$

Port diameter (in.)	6	8	10	12
Port diameter, cm.	15	20	25	30
Area per port (m^2)	0.01777	0.032	0.05	0.071
No. of ports reqd.	188	104	67	47
Wall space / port (m^2)	0.24	0.43	0.67	0.96
Avg. port spacing, m(a)	0.49	0.66	0.82	0.98
G, Sec^{-1}	9	5	4	3
Horiz. row spacing estimate, m.	0.34	0.47	0.59	0.70
No. horiz. rows	6	5	4	3
No. ports / row	31	21	17	16
Final spacing between rows	0.33	0.40	0.50	0.67
Final distance between ports in a row (diagonal arrangement)	0.73	1.07	1.32	1.40



a. VERTICAL ALIGNMENT



b. INCLINED ALIGNMENT

FIGURE 8-2. JET ARRANGEMENT IN PERFORATED BAFFLE BASIN INLET WALL.

Figure 8-2. Port arrangements in perforated-baffle basin inlet wall.

(where *a* is the centerline spacing of ports), and the number of rows of ports is estimated. The total number of ports is then divided by the number of rows of ports to obtain the number of ports per row. The exact port spacings can then be calculated.

The velocity gradients for each size of port, as taken from Table 4-2, are also noted in Table 8-1. In the example cited, all these values are relatively low, but it is assumed that in the future the basins may have to be operated at double the nominal flow rate. It is seen from Table 4-2 that with doubled flow, the velocity gradient would rise to a range of about 13-90 sec^{-1}. The higher of these values would probably cause floc breakage, impairing settling. Preference should therefore be given to the larger sized ports. It is apparent from Fig. 8-2 that with a vertical alignment of 8-in. ports only three rows are feasible, whereas with inclined port alignment four rows may be used. This permits use of maximum feasible number of ports. It is well to prepare a sketch of each arrangement considered in order to see whether any minor adjustments are desirable. It is likely to be found that a small adjustment of a number of ports will be desirable in order to make even spacing feasible. The end ports may be located as close to the side walls as one-half the calculated horizontal spacing.

If this basin were designed for a settling velocity loading of 4 cm/min, the settling zone would have a length of 67 m and the total tank length would be 70 m. From information given in Chapter 4, assuming the use of 15-cm ports, the turbulent entrance zone would be 4.2% of the tank length, not an unreasonable proportion.

SETTLING ZONE

A second important zone in the basin is the settling zone. It has been the usual practice to allow more than 2 m of depth for this zone, but the trends of today suggest that, wherever conditions permit, this zone need be only 1-2 m deep.

It has been traditional to design settling basins that are long and narrow, partly because of lack of knowledge of effective inlet and outlet design, partly to help overcome unsatisfactory pretreatment provisions, and partly to help control density currents. The rule of thumb has been to make each basin at least three times as long as its width, sometimes six or more times.

In the settling zone of a properly designed settling basin the flow is turbulent but relatively quiescent. For purposes of illustration, let us consider two specific cases of basins designed for a settling velocity of 4 cm/min and a rate of flow of 1 m^3/sec, the first being a 38.7-m square and the second being a 22.5 × 67-m rectangular unit. If the depth of the settling zone is 2 m, the time of flow from the beginning to the end of the settling zone will be 50 min. Because of the greater velocity in the rectangular basin, it will provide more drag to help overcome density currents by reducing the gradient used in the density-current computation described in Chapter 7.

A long and narrow basin also has some advantage in reducing the likelihood of short-circuiting. It also has an advantage in minimizing the proportion of the length occupied by the turbulent inlet zone. The rectangular settling basin has a further advantage over the square one in that equipment cost can be reduced by installing cleaning mechanisms in only the first part of the basin, leaving the latter part for periodic manual cleaning.

Attachment B: Reverse Gradation Filter Support
Excerpt from "Water Clarification Processes" by
H.E. Hudson, Jr., published by Van Nostrand Reinhold.

Head losses in the clay-tile Leopold system were measured in a hydraulic laboratory,[5] where the losses through the blocks were shown to be

$$H_L = 0.01V^2$$

where H_L is head loss in feet, and V is vertical wash rate (gpm/ft^2). At the customary wash rate of 15 gpm/ft^2, head loss was 2.3 ft of water. This relation does not include the entry and velocity head losses at the inlet to the system. The latter are functions of the total flow entering each lateral, which depends on both wash rate and lateral length. Head loss was found to be 1.9 velocity heads for bottom-entry systems. The entry loss is equal to the block losses at a wash rate of 15 gpm/ft^2, when the length of lateral is 47 ft. With shorter lengths entry loss is less, and with longer ones entry loss is greater. The entry losses are approximately the same whether the flow enters through the end of the lateral or through a bottom entry, as in Fig. 12-3.

In 1978, the Leopold Division introduced modified dual-parallel underdrain blocks fabricated of high-density unbreakable extruded plastic. The new blocks have greater burst strength but less compressive strength than the former tiles. Preliminary hydraulic testing showed them to have head loss characteristics that are 35-70% lower than those of the tile blocks. The modified configuration, using triangular passages, lends itself to use with air wash or water wash. For use with air wash, a reverse gravel gradation is recommended, and indeed, the reverse gradation appears desirable with water wash, to eliminate gravel movement in any system requiring gravel above the underdrains.

A reverse gravel gradation that has been found to be safe against movement of the top gravel layer in model tests done for the F. B. Leopold Division can be used with either type Leopold bottom. It is

Location	Particle Size (in.)	Thickness (in.)
Top	1/4 × 1/2	3
	1/8 × 1/4	2
	10 × 1/8	2
	1/8 × 1/4	2
	1/4 × 1/2	2
Bottom	1/2 × 3/4	3
	Total	14

Two inches is about the thinnest layer that can be placed well. Even this requires level chalk lining on the walls, setting screed boards, striking off, and checking the surface by bringing water up to the desired elevation. Placement is done by men working from wide boards to prevent footprints.