CLEAN WATER

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Combined Sewer Overflow Seminar Papers

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U.S. DEPARTMENT OF THE INTERIOR ● FEDERAL WATER POLLUTION CONTROL ADMINISTRATION

Combined Sewer Overflow Seminar Papers

A compilation of technical papers and
discussions presented at a Seminar at
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U.S. Department of the Interior
Federal Water Pollution Control Administration
Office of Research and Development
Division of Applied Science and Technology
Storm and Combined Sewer Pollution Control Branch

CONTENTS

OPENING REMARKS

by

William A. Rosenkranz

As Mr. Dewling indicated, my purpose at this particular point of the program is to give you a general idea as to how the program works, some information about the background of its initiation and how you would go about implementing a project with the assistance of grant or contract through the Federal Water Pollution Control Administration. Some of you may already be familiar with the history but I will give you a bit of it to keep you on board.

Back in 1964, FWPCA, at that time Public Health Service, completed a general assessment of the storm and combined sewer problem in the United States. The report included an estimate that it would cost about $30 million to correct this problem on a national basis by means of sewer separation. The principal recommendation was that alternatives to separation of sewers be studied to find ways to do the job at less cost with the same or better efficiencies. As a result of that report and other information available to the Congress, the Water Quality Act of 1965 included the establishment of a demonstration grant program which was at that time called Facilities Demonstration Grants. It was set up on a basis of 50% grants. Contracts were also authorized and the program was actually initiated on an active scale during the spring of 1966. The first grant was made in June of 1966 and we have gone on from there.

The program was changed in the Fall of 1966 with the passage of the Clean Water Restoration Act of 1966. At that time the amount of participation in individual projects by means of grants was changed from 50% to 75%. Additional demonstration grant programs were authorized at the same time in the fields of advance waste treatment; industrial waste treatment and joint municipal industrial treatment. Since there was no appropriation made at that time, the Congress authorized the funds originally provided for the Storm and Combined Sewer Program to be used during this first year and thereafter for implementation of some of these programs to get them off the ground. The first year of the Clean Water Restoration Act saw some of the funds that had been allocated to the Storm and Combined Sewer Program used for the first grant projects and contracts in the other technical areas. The additional programs were funded the following year.

At the present time we have initiated 82 projects, the project cost involved is $65 million and the Federal grant and contract funds supporting these projects is about $28 million. So you can see that the program is active, there is a lot of work going on and we are still looking for additional demonstration and developmental projects to carry us further down the road.

Once the program was initiated, it was obvious that additional work was needed to bring the assessment of the problem on a national basis more up to date. The information and data contained in the 1964 report that I referenced earlier actually was data prior to 1964 and it was felt that we ought to update not only the assessment of the impact of combined sewers, their location and similar information, but also update the estimate of cost for remedial measures. Therefore, in 1967 a contract was initiated with the American Public

Works Association to do this job for us. As a result of that study the APWA estimated that it would cost $48 billion to provide separate storm and sanitary sewers in areas now served by combined sewers. This included the work that would be required on private property to separate plumbing from the combined sewers and reconnect to sanitary sewers. The report also pointed out specific areas where additional research and development is needed. Research and demonstration projects to look into these particular areas are being implemented as fast as it can be done.

With regard to procedures, let me say first that with regard to demonstration grants the Federal funds are usually used to support full scale projects. We are looking for projects that take a new and/or improved method and apply it to a community problem at full scale, so that the community does achieve a significant improvement in their sewer system, a significant level of water pollution control is obtained and data is obtained and evaluated so that other people, such as yourselves, will have this information available and be able to apply it on jobs that you may have. The participation, as indicated earlier, is at the level of a maximum of 75% federal participation in the project. In return, we are actually buying information so that we can disseminate it to others that have the need to use it. We do participate in construction costs on the projects, however, this is not our prime objective. This is not a construction program, it is a research and demonstration program. Therefore, from other standpoints, we prefer to keep participation in construction costs as low as possible. We recognize that we must build a facility to evaluate it, therefore, we do participate in construction costs to the extent possible, retaining compatability with the demonstration objective. Grants can be made only to official government bodies. We can make them to sanitary districts, municipalities, metro organizations, state agencies, and counties; but we cannot make grants in this program to profit making industrial organizations or firms. Grantees must be responsible for either the construction or operation of maintenance of a system.

As Mr. Dewling mentioned, we have kits here today, including the application forms, instructions for completing them, copies of the regulations involved and other pertinent information. If you are interested in implementing a demonstration project with Federal assistance we will give you the kits for filing an application. The applications must be approved by the official water pollution control agency in the state where the project is to be done. In this way, it functions very similar to the more normal construction grant program, which requires state approval as well. The only difference being that there is no state allocation of funds. As you know, in the construction grant program each state is given a funding allocation. In the research and demonstration program this is not done.

Contracts can be utilized to deal with government agencies, industry, consulting firms, universities--almost anyone who has a good project proposal. An organization, for example, the American Public Works Association, can be either profit or non-profit to be eligible for contract work. Contracts are generally used for developmental work--investigations to explore difficult technical areas. Or, in some cases, non-technical areas such as the nationwide study done by American Public Works Association. We hope that if we have a viable process come out of a developmental contract we can then follow it up and demonstrate the method at full scale by means of a grant. This approach permits us to develop a piece of equipment, a technique or method we can study at pilot scale and move right into full scale operations by means of a suitable demonstration grant project.

Special forms are not needed for contract proposals. It is up to the person with the idea to state his idea, tell us how he is going to carry it out, what his objectives are, what manpower he needs, what kind of water quality sampling, monitoring or lab work he will require together with his estimates of total cost that will be involved in completing the proposed work.

You are probably interested in some of the technical areas in which we are looking for work. When preparing material for the Seminar, I came up with a list of something over 60 individual kinds of projects that we would like to initiate. I am not going to read the entire list to you this morning, but I would like to mention several of them to give you an idea of some of the things we would like to do. These are areas where work has not been done and areas in which we have no real data for field use. Potential project areas include removal of storm flow and infiltration from sanitary sewers, thru elimination of illicit storm water connections and similar approaches. Is there a good way to go about this task efficiently and at reasonable cost to a community?

We need to assess the extent and significance of discharge of sanitary wastes to storm sewers. Control techniques, if such control is really warranted by an assessment of the problem, need to be developed and applied. Is this problem significant enough to warrant extensive work?

We are interested in looking at special conveyance systems such as pressure or vacuum sewers as an alternate means of sewer separation. The American Society of Civil Engineers has examined pressure sewers for us. The report is now at the printers and the concept is being explored further by means of a demonstration grant with the State of New York.

We need to do some work in infiltration control. I think of you working on projects, especially the consulting engineers and municipal people, find that infiltration is a major problem. The American Public Works Association has found that this is true in their earlier work and are now doing an assessment of the state-of-the-art involved. Improved sealing materials have been explored and you have a report on our first attempt at the conference tables. Additional work is needed to further develop the materials and their application.

A close look at improved materials and construction practices with regard to all types of construction, including sewers, is needed. Faulty bedding, poor jointing and inspection are causes of major sewer problems. New construction materials and methods for tanks, housing and all types of facilities are needed to reduce cost of remedial measures. System analysis techniques are now being developed and you will hear a paper on that later today. We want to demonstrate the use of systems analysis in designing and investigating problems in the sewerage system and implement an integrated control system for an entire drainage area.

A need exists to demonstrate full scale some of the new methods of treatment now under development. Here we are looking at treatment methods and techniques which will be of a much higher through-put rate than we now have. We are evaluating bio-disc treatment by means of a contract pilot operation, dissolved-air flotation with high surface loading rates, screening and microstraining, high-rate filtration (both physical and biological). We think that sufficient progress has been made in several of these methods to permit application to combined sewage treatment on a full-scale basis and at reasonable cost at individual overflow points. Considerable work is needed to field develop and apply system regulators. We will shortly have a published report on the development of a "Fluidic Regulator" which we think will have the potential to operate much more efficiently than existing mechanical types of regulators. Many of you, especially if you are in a municipal or consulting field, recognize that our capabilities for measuring flow are not very good. They are difficult to apply, costly, require a great deal of equipment. A major impact on the field could be made if we could come up with more efficient and accurate flow measuring techniques which can be easily and economically applied. We need to improve our water quality monitoring capability, including sampling techniques.

Thus far I have talked almost entirely about combined sewage. Perhaps I ought to indicate that we are also interested in storm water control and treatment. Very little work is going on in this area at this time. We would like to see some good projects aimed at treating and/or controlling urban runoff. Within our over-all Research and Development programs, our particular Branch is also charged with the responsibility for determining the nature and extent of problems existing and what potential remedial measures can be applied to non-sewered runoff. This is runoff that does not reach the collection system--storm, sanitary or combined.

In approaching and executing storm and combined sewer projects, particularly the engineering investigations and determination of the way to go, the municipalities and the engineers involved should keep in mind that while we need to apply conventional engineering practice, the problem itself doesn't lend itself to a straightforward engineering solution. The problem is more complicated and difficult in that we need to keep in mind that we have to apply originality and ingenuity in solving these problems. We need good knowledge of the type and magnitude of the local problem in order to properly jell a project which may be feasible from both technical and economic standpoints.

OVERVIEW OF CONTROL METHODS

by

Francis J. Condon

Most of what we will talk about this morning is the control problems of combine sewers. The predominant pollutional sources are in the combined sewer area and, therefore, to date, the predominance of our efforts have been in solving these problems. A little closer definition of combined sewers as we use the term should be given. The classic definition is well known, but there are ostensibly separate systems which are in fact combined mainly due to large infiltration problems. There are separate systems built in outlying areas from the old urban areas which flow into combined sewers and then there are the problems of construction such as the cross connections which were made for expediency. Finally there is poor construction practices. The result is that many of the so-called separate systems are cross connected with combined or act as combined sewers.

Strangely enough in many parts of the country there is a reluctance to enforce the ordinances necessary to make a separate system separate. Examples of this are downspouts and foundation drains that are connected into separate systems. The local populace want it that way and the ordinances aren't enforced. Actually there are some combined sewers yet being constructed although the general practice is supposed to be that all new construction will be separate systems.

The design of the recent collection systems and some of our current interceptor designs, which take both separate and combined sewage, is done by old practices. The ratio of three to seven wet weather flow to dry weather flow is usually used when the design considerations are weighed, the governing factor is economics. The data are interpreted so that they indicate that the flow ratio selected, in that range, is the proper ratio to use in order to intercept most of the storm flows. We have shown in many cases where that rule of thumb of selecting a three or five to one ratio is grosely in error. It is not uncommon to have 50 to 200 times dry weather flow in urban runoff.

Before we talk about where control is needed and what can be done, we should briefly discuss the reason for those flows. The first and most obvious is the rain event itself. Going back to the ratio of flows we can use a specific example to illustrate the point. There is a New England state which has a large number of middle sized cities all of which were sewered with combined sewers and very few of which have treatment plants. They discharge raw into the receiving waters. The intent of the state is to build treatment plants with some auxiliary facility to handle excess combined flows. The justification for our entering into a demonstration grant with one of the cities was that the design criteria for the state would be set on our demonstration project for use in future treatment plant construction thruout the state. To establish the volumes and rates which could be expected and verify current runoff estimating practices, it was decided to cross check estimating techniques. As a result there were three groups whom we asked to calculate the rate and amount of runoff during a rain event. First was the design engineers, a large and reputable firm. They would naturally have to design for the volumes of flows that would have to be taken by both the treatment plant and the auxiliary facility. Then we asked the city engineer to make his calculations. Finally we had a research and development group, who are doing pre-construction evaluation, to actually measure and back calculate the runoff factor. It was extremely interesting, everyone knew what the other fellow was doing. They all did a thorough job. I believe the engineering firm used a modified rationale method; the city engineer used the Chicago method. The research and development firm actually measured all flows and back calculated over a full year. There are so many elements which enter in to how much runoff you get that the old simplified equations were inadequate. The design engineers calculations of volume for a given rain event, on the average, was about 70% less than what was actually measured. The peak rates were off by more than 100%. You can see what that would do to design criteria. Therefore, a very important point is actually knowing the total amount and rates of flow you will receive.

We have several model studies going on to help solve this problem and I believe we have one paper on that subject today.

The next big source of the excess flows is infiltration. Mr. Rosenkranz touched on this a little this morning. This is a very wide spectrum: how do you measure it? how do you control it? Remember most of the time we are dealing with in situ problems. We have projects where we are attempting to develop better construction materials, better construction practices and a very simple yet difficult thing to come by, good inspection practices. But what do we do about the in situ problem? Sewer sealants and the development of reliable instrument devices were given our earliest attention. One of the toughest problems is that the majority of flow comes from the service connections. Not only the joints in the sub-system, the mains, and the laterals but as indicated from one recent study about 70% of the volume came from the length of line from the house to where it connected to the lateral. How do you repair instead of replacing 4 inch service lines? We are looking diligently for a project in this area and that will be a major accomplishment if we can make any gains towards that problem solution.

Another source of excess flow is malfunctioning regulators. Again operations people in the room know the tough problem that they present. The mechanical systems need constant maintenance and looking after; even then they don't often perform as they were designed. This morning there was mentioned the fluidic regulator which we have great hopes for and there is another project in New York City utilizing the Ponsar regulator.

Another result of the excess flows is the bypass at the treatment plant and just as important the plant upset. So control at the plant site itself with devices or facilities to handle excess flow is another area which we are investigating. To summarize the sources or causes of flow problems: one, it's underestimated runoff from a given rainfall; two, infiltration from many sources; and three, the regulators and four under designed treatment plants. Therefore, we now come to what methods are being presently investigated and what further needs to be done.

Firstly, drainage area control, this is a Pandora's box. The term control applies both in the hydraulic volume and pollutant load. We have projects in this vein and there will be a paper today on one technique. The method to be discussed is up-system storage, urban lakes, and surface lakes, which include recreation. In our urban areas this is a very important element. Sub-surface storage in caverns or tunnels and utilizing the geology itself is another method. The so-called geological hidden valleys which are areas of high permeability and void space may be used for storage. The problem in utilizing this method of permeable stratum is polluted water in the ground and, if needed, taking it back out. Barriers to keep the polluted water from moving after it is in sub-surface storage is an area to be investigated.

Another area of the investigation is the collection system control and here again we have a wide range of ideas and projects which we could look at. Special conveyance systems of pressure and vacuum lines were one method already mentioned.

Catch basin improvements is an area to be researched and for a while I thought that catch basins were no longer being constructed. But we have found that they are still being designed in collection systems. They serve a purpose in some cases, the idea is to make them better and more functional.

Reducing the infiltration and ex-filtration as we mentioned is a large problem but that is also part of the collection system control.

In-line storage routing is a very interesting area. There are projects in Milwaukee, Detroit, Minneapolis and St. Paul where they utilize the storage concept in the collection system itself. The purpose is to route the sewage and have it hit the plant not as a large slug but as a slowly built-up volume that the plant can handle without upsetting.

There are some areas in instrumentation, monitoring, sensing devices, and automated real-time control where much has been learned.

In-line detention also means flushing. Flushing becomes important not only where we have the flat grades but also if we are going to use in-line storage. In holding the sewage in large sewers we have primary settling. Then the problem is after we have stored how do we get those settled polluting materials back to plant again without a slug effect.

There are flow additives to increase the capacity of the flow characteristics. We have a very interesting paper on this today. The use of polymers, whereby the flow is increased at the same head is the concept utilized. There appears to be reduction of internal fluid friction and perhaps a boundary layer effect. This is a most interesting phenomenon.

As mentioned there are many spinoffs from these projects and it is difficult to categorize and talk about them because each project incorporates so many of the side issues. Instrumentation, sampling devices, sampling techniques and the methods are part of almost all of our projects.

Another method of control is control at the overflow point. We have projects, and these were our first cut efforts, such as tanks or storage facilities where they were above ground, below ground or underneath the water. These tanks would hold and take the first surge with the heavy pollution load, if that is the case, then treat what they had to bypass. The stored volume would be fed back into the system. The treatment is usually aeration and chlorination when the excess is discharged. An interesting point in the storage devices, whatever they may be, is the idea that we have only a short time to hold because you couldn't economically design to retain every storm. We want to get maximum treatment in a short time, so with say a ten minute detention time how can we get the maximum sedimentation. This is usually the treatment that goes on at the detention facility. Our experience indicates that primary sedimentation tank design is really far behind. There are many things that could be done, and we are attempting to develop projects now to improve settling tank design. We believe it would be a major step if we could improve primary sedimentation in a shorter holding period. There are also chemical and mechanical treatment processes we are developing in this respect.

Now the last area. There are the modifications or additions to existing treatment plants to contain or retreat excess flows both by biological and mechanical techniques. There are several interesting projects, I believe. One which is listed in your handbook is Kenosha, Wisconsin. There we used a biological treatment method. The concept is to keep a bio-mass viable during a dry period and have it available for treatment of the excess flows as activated sludge.

Another concept is to concentrate the pollutants which actually do go to the treatment plant. We had one of our very early projects in this area where we attempted to use the pipe itself as a filter. The idea was when the fluid head on the pipe caused surcharging, the pipe would expand and in expanding it would become permeable. The excess water would seep out the sides, it would be captured and chlorinated but the solids would remain concentrated in the smaller, contained stream which would go to the treatment plant thereby not hydraulically upsetting the plant. Unfortunately, that one has more application to some industrial waste projects and was not feasible in combined sewage. We couldn't get the self-cleaning aspect to the high degree which was needed on that concept. The project did introduce many new ideas for other people.

Many of these treatment and control methods we talked about with respect to combined sewage apply also to storm water. Urban runoff itself is surprisingly loaded with polluting materials. One could expect, especially in the COD readings, that urban runoff is heavily polluted. In addition, coming from urban areas are pesticides and insecticides which induce a fairly high toxic level in the receiving waters. So that the storm runoff itself needs a good deal of the attention in treatment especially in control measures.

In summary it can be said that:

(1) Work is currently going on in predicting more accurately the volume and pollutional load of excess combined sewage and urban runoff. Verification of the methods being developed is still needed.

(2) Projects are active for in-system routing and storage. There needs to be refinement in what has been developed to date.

(3) Projects are active in off-system and outfall storage and treatment. This is a very broad area and the ideas which could be applied here have not been exhausted. The combination of pollution abatement and recreational use could certainly be explored further.

(4) There are activities in bettering construction materials and practices in addition to in-situ repair of pipe. This is an area where not only pollution control but economic benefits or gains could be very large. A large amount of research awaits future efforts.

(5) Dual use treatment concepts at treatment plants where facilities for treatment of excess flows could be utilized for tertiary treatment of dry-weather flow is a wide open research and development area.

(6) Appurtenance development such as improved regulators, tide gates, catch basins, volume or rate measuring devices, sampling devices, instrumentation in general, and many other related items need further investigation.

There followed an open discussion of the fluidic regulator.

STORAGE AND TREATMENT OF COMBINED SEWAGE

AS AN ALTERNATE TO SEPARATION

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INTRODUCTION AND ACKNOWLEDGEMENT:

The City of Chippewa Falls, Wisconsin was confronted with the need to complete a program of separating storm water from its sanitary sewage and waste collection and treatment facilities or to provide a method of treating the combined sewage and wastes.

The State Regulatory Agency basically had requested separation, although, upon questioning, would approve an "alternate to separation" if the ultimate objectives could be achieved.

A thorough investigation and study was undertaken, which indicated substantially the same apparent objective could be achieved either by storage and treatment of the combined sewage and wastes or by separation.

In evaluating the two possible procedures, the comparison was made on the basis of complying with a regulatory agency order. Certain fringe benefits such as elimination of flooding of basements during heavy rainfall and the occasional extreme high river water in the Spring were not considered in the evaluation, although these advantages became apparent during the course of the studies necessary to reach a conclusion. The fringe benefits which resulted were an extra bonus.

This paper will present the alternatives, the recommended project, and what results have been achieved.

Too much cannot possibly be said about the complete and enthusiastic support and assistance provided by the City officials and staff and especially Superintendent

of Public Utilities, Clyde Lehman.

BACKGROUND:

The City of Chippewa Falls, Wisconsin was faced with the problem that many of its sewers were of the combined type; the City was required to establish a system of "separate" sewers. This problem is the same one confronting many of the older cities throughout the United States today, whether large or small. The financial impact upon any city is substantially the same, regardless of size, when related to the number of taxpayers and the tax base. In many respects, the development of the various centers of population throughout the United States probably followed the same general pattern as that in Chippewa Falls. A brief background of the City appears appropriate.

The City of Chippewa Falls has been incorporated for 105 years. Its development and initial reason for establishment was due to the lumbering industry. Of course, lumbering in the area is now almost non-existent. The City is situated on the Chippewa River, with about 40 per cent of the area being south and 60 per cent being north of the river. That portion of the City lying north of the river is bisected by Duncan Creek. The Chippewa River in the vicinity of Chippewa Falls is controlled by two hydroelectric dams, one of them being in the City. At one time both Duncan Creek and the Chippewa River were used to float logs downstream to sawmills located in Chippewa Falls. As development occurred, a further use of the river and of Duncan Creek was to receive and carry away surface runoff and sewage and industrial wastes. The matter of water pollution was never considered. Thusly, prior to about 1935, practically all of the sewers constructed in Chippewa Falls were of the combined type which discharged directly into either the Chippewa River or Duncan Creek.

In 1937 the Wisconsin State Board of Health strongly urged the City to provide treatment of its sanitary sewage; in 1939, the City commenced construction of intercepting sewers on both sides of Duncan Creek, which, when completed, would prevent all dry weather flows from entering the Creek. A plan had also been developed for the construction of intercepting sewers on the north bank of the Chippewa River, and a site for a waste water treatment plant was obtained.

World War II resulted in the stoppage of all sewer construction in the City and no further progress was made until 1950, at which time the State Board of Health and the Wisconsin Committee on Water Pollution issued an order requiring the completion of intercepting sewers and the construction of a primary waste water treatment plant. Construction of this was completed in 1952. Since the end of World War II all sewer construction within the City has been of the "separate" type.

In 1954 the State Board of Health and the Committee on Water Pollution requested the City to prepare a master plan for storm sewer separation. This plan was completed and, as a result, the City began a program to construct separate storm sewers and elimination of surface water entering the combined system. Each year the Director of Public Works would include in his budget a sum of money for implementation of the separation program. However, occasionally the need for constructing sanitary sewer extensions occurred and some of the separation was not done. Obviously, separation in the "downtown" area would be more costly and inconvenient per "amount of separation" than in the residential areas. In 1965 the State Regulatory Agency directed that the City establish a definite time schedule for the completion of the separation program. Substantially, all of the separation in the residential areas had been completed by this time, but the "downtown" area contributed the vast majority of surface runoff tributary to the combined system.

The same order requiring that the City establish a definite timetable for completion of the separation program also included a requirement that improvements be made to the waste water treatment plant to provide the degree of treatment intended. As a point of information, the "degree of treatment intended" was primary treatment.

By State interpretation "primary treatment" was a minimum of 30 per cent removal of 5 day BOD and suspended solids. Industrial development and population increase has occurred beyond expectation. Our firm designed the original intercepting sewers and waste water treatment plant; the City again engaged us to assist them in this project. In reviewing the treatment required by the State Agency, it became obvious that improving the facilities at the waste water treatment plant to provide the minimum 30 per cent removal would not be a sound approach because it was anticipated that secondary treatment would be required within two years. Accordingly, a program was recommended to include secondary treatment incorporating the activated sludge process to provide in excess 90 per cent removal of 5 day BOD and suspended solids.

It must be realized that great emphasis in Wisconsin is being placed on recreational use of many of the rivers, including use for whole body contact. The whole theory of using the assimilative capacity of the receiving streams can no longer be used in determining the degree of treatment to be provided. While the waste water treatment plant, per se, may not appear a part of the storage and treatment of combined sewage, certain parts of the plant could be affected if large volumes of combined sewage were to be treated therein.

F.W.P.C.A. DEMONSTRATION PROGRAM:

At the Water Pollution Control Federation convention in Kansas City, F.W.P.C.A. Commissioner Quigley announced that the F.W.P.C.A. had been authorized, and money appropriated, an amount of $20,000,000 for demonstration projects for alternates to storm water separation.

This information was presented to the Chippewa Falls Common Council, with a possible program of an alternate to separation. The City agreed to finance preliminary investigations and feasibility study for such an alternate.

The results of the study indicated that it would be feasible to construct a large storage pond to store combined sewage and wastes which would otherwise bypass to the Chippewa River and Duncan Creek during periods of surface runoff. It was also found feasible to construct separate storm sewers.

The estimated cost of storage, with certain minor separation still being required and construction of certain trunk sewers being required, was only slightly less than the cost of separation. Comparative cost estimates were prepared for both programs. In considering separation, it was assumed that the City could program the separation over about ten years and pay for each year's work from the annual budget; no financing and interest costs would be involved. It was also assumed that construction costs would increase between two and two and one half per cent per year.

It was therefore recommended that the alternate to separation was advantageous to the City only if a major grant-in-aid could be obtained, otherwise, the conventional separation program should be adopted.

The City made an application for a F.W.P.C.A. Demonstration Grant. The grant was reduced from the authorized 75 per cent to 55 per cent because of a rash of "last minute" applications. However, the State of Wisconsin also has a grant-in-aid plan of 25 per cent, for which the City applied and received. Hence, the net cost to the City was 20 per cent of the cost plus the cost of land.

SELECTED PROJECT:

The project selected consisted of certain combined trunk sewers, increasing the pumping capacity of the Bay Street Sewage Pumping Station, certain minor separate storm sewers, a combined sewage (storm water) pumping station, and a combined sewage storage pond.

In addition, certain conduit and sewage pumping capacity at the Waste Water Treatment Plant had to be increased and each of the final two settling tanks at the plant were increased from 55 foot diameter to 65 foot diameter. No increase in size was made to the primary settling tanks, aeration tanks, blower capacity or chlorine contact tank.

The total cost of this project, including the enlargements at the Waste Water Treatment Plant, was $620,701. The general overall project is shown in Figure 1.

Studies by McKee in Boston have shown that for a rainfall intensity of 0.1 inches per hour the percentage of sanitary sewage escaping from the sewers was as much as 80 per cent of the total sanitary sewage, when the intercepting sewer had a capacity of two (2) times average dry weather flow. When the rainfall intensity was 0.5 inches per hour, the percentage of sanitary sewage escaping increased to 95 per cent.

Although studies of the overflow points in the Chippewa Falls sewer system have not been undertaken, casual observances tend to indicate that such studies would probably show results similar to the foregoing. Practically every rainfall, no matter how low the intensity may be, would have caused overflowing from the combined sewer system.

The first problem was locating a storage pond site where interception of the overflow from the combined sewers could be done without long distances of large diameter pipe being required. A problem of equal importance was to determine the size of the pond.

There was a low area lying between the downtown business district and the Chippewa River. The main overflow point for the downtown area combined sewers was adjacent to this site and the former overflow was carried across this low area in a 42 inch corrugated metal pipe discharging into the Chippewa River. There is a railroad trackage along one edge of the low area but at an elevation high enough so as not to be endangered by the storage of storm water.

This was the only feasible location for the pond. By using this location, the maximum size of the pond was established by geographical features of the area. Approximately three acres was available for pond construction and the average area within the pond could be about 1.33 acres.

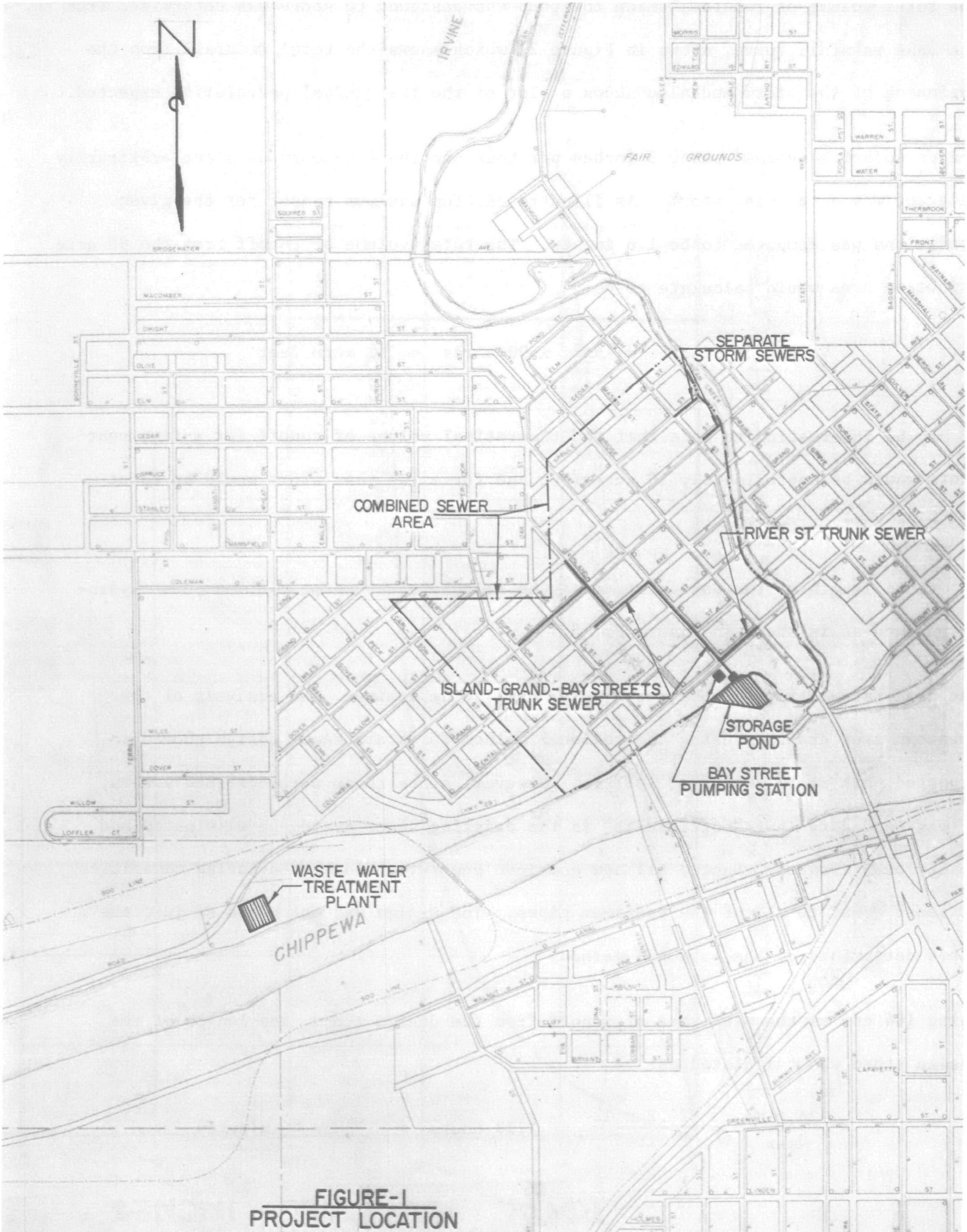

FIGURE-1
PROJECT LOCATION

The total volume of rainfall which the pond was designed to store was determined from the mass rainfall curve, shown in Figure 2, which shows the total rainfall from the beginning of the storm and also shows a plot of the theoretical percolation expected.

The percolation assumed was 0.3 inches per hour and the frequency of storm arbitrarily selected was a ten year storm. As illustrated, the maximum runoff for the given conditions was computed to be 1.6 inches. The total volume of runoff from the 90 acre tributary area would calculate to be:

$$\text{Volume of runoff} = \frac{1.6}{12} \times 90 \text{ acres} = 12 \text{ acre feet}$$

It may be interesting to note that the theoretical volume of runoff for a five year storm would be approximately 10 acre feet and for a two year storm would be about 7.5 acre feet.

The total length of the design storm may be calculated by using a three point hydrograph such as is shown in Figure No. 3.

The peak rate of flow to the pond (Q_{max}) must be determined. An analysis of the downtown area drainage using the rational method for storm sewer design showed an expected peak storm runoff of 164 cfs. However, since these were combined sewers, it was necessary that "bottlenecks" in the existing sewer system be eliminated and no new ones created. Hence, all new combined sewers were designed having capacities at least equal to all of the upstream pipes. The design was not based on just the flows determined by the rational method.

Using 164 cfs as the peak rate of runoff from the design storm, the length of the design storm was calculated as:

$$T = \frac{24.2 \text{ Vr}}{Q_{max}} = \frac{24.2 \times 12}{164} = 1.77 \text{ hours} = 1 \text{ hour } 46 \text{ minutes}$$

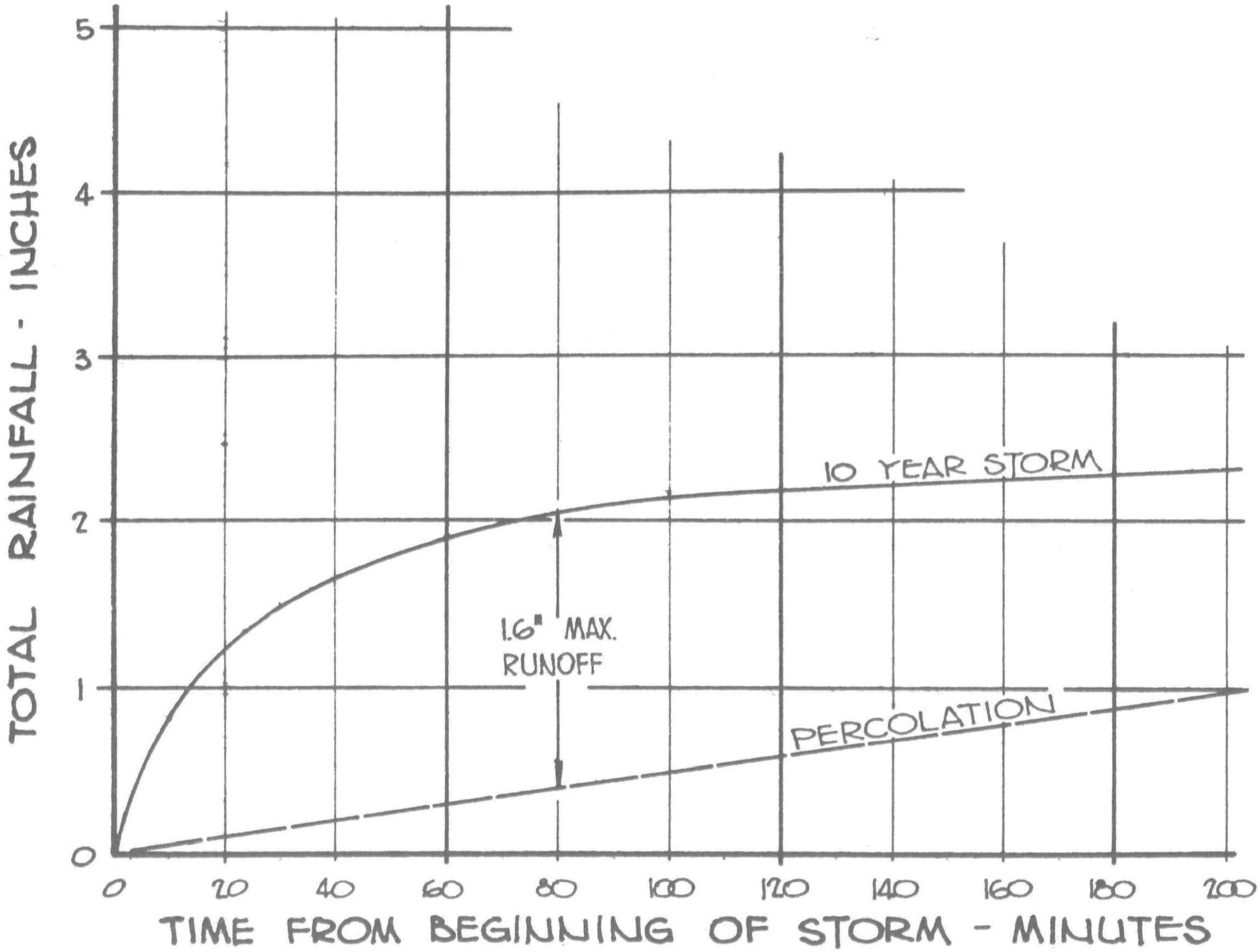

FIGURE -2-

MASS RAINFALL CURVE

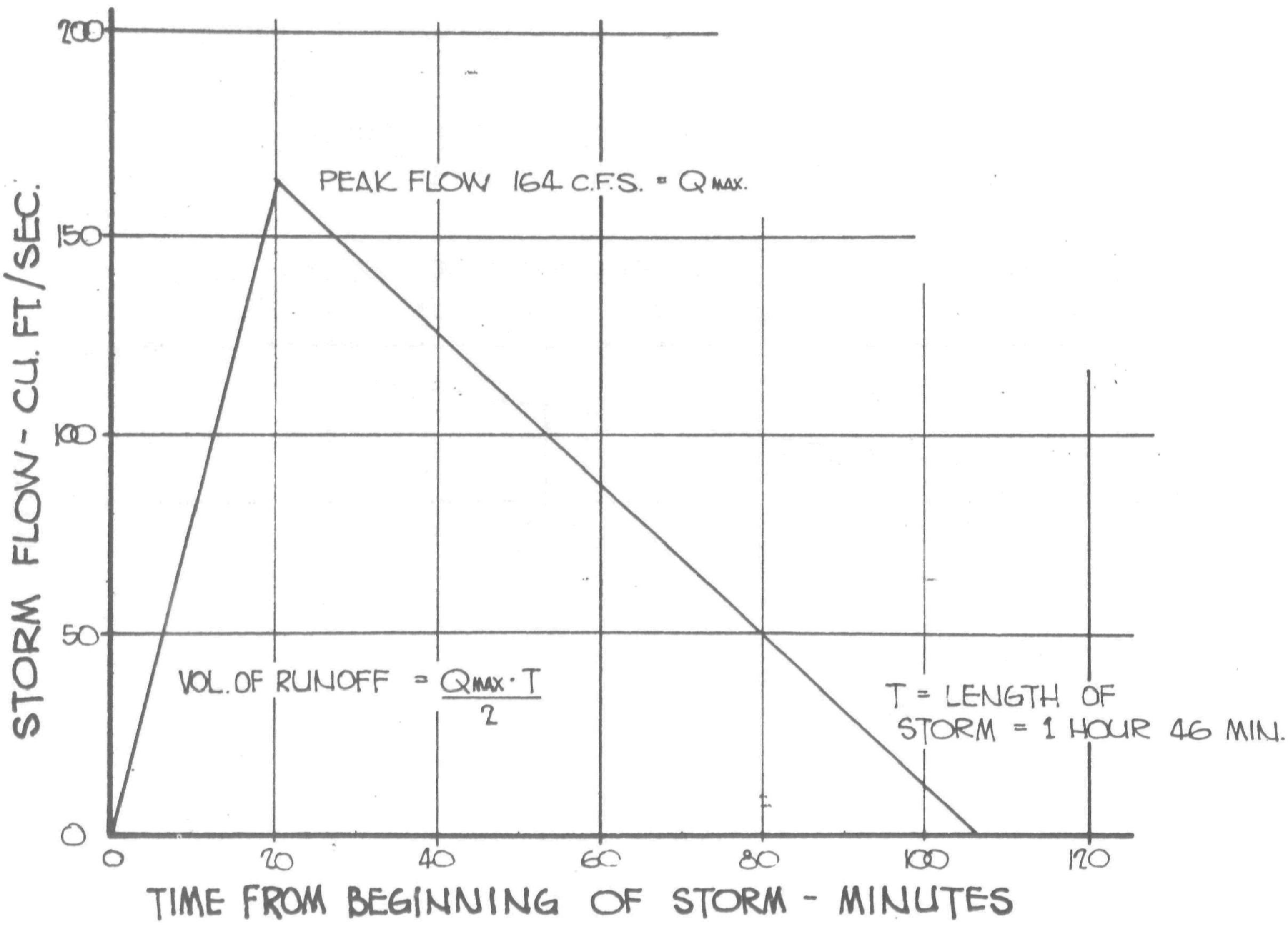

FIGURE -3-

3 · POINT HYDROGRAPH

The existing Bay Street Sewage Pumping Station had a pumping capacity of 4000 gpm but the maximum rate permitted by the force main and intercepting sewer could be 6000 gpm. Hence, the pump capacity was increased to 6000 gpm. The estimated peak flow of domestic sewage was 2000 gpm, so that up to 4000 gpm of the storm runoff could be pumped to the intercepting sewer and the waste treatment plant without overflowing to the storage pond.

During the period of the storm, the sewage pumping station will deliver:

4000 gpm x 106 minutes = 424,000 gallons of storm water to the Treatment Plant,

which is 1.3 acre feet and represents the amount by which the total volume of runoff could be reduced when calculating the size of the pond.

Thus, in this case, we designed the pond for a volume of 12.0 - 1.3 = 10.7 acre feet. This requires a water depth in the pond of 8 feet.

The elevation of the invert of the trunk sewers where they enter the Bay Street Pumping Station is only 0.4 feet above normal river level. Gravity flow to an above ground storage could not be obtained. Therefore, a combined sewage pumping station was constructed to pump all combined sewage to the storage pond. This station has a capacity of 75,000 gpm, which is the total capacity of the tributary trunk sewers.

The storm water pond was constructed of earth dikes with the top of the dike at one foot above the top of the overflow elevation.

The design provided that, after the pond is emptied, the bottom would be flushed with river water or from a fire hydrant to wash solids to the inlet to the Bay Street Pumping Station to minimize the leaving of solids after draining. The pond interior was surfaced with a bituminous mat to facilitate cleaning and to allow vehicular traffic within the pond for maintenance of structures and removal of grit.

Relief valves were placed in the pond bottom so that when the river level rises above the pond bottom, it will flood rather than being in danger of floating. The earth dikes adjacent to the Chippewa River are fully protected by riprap to prevent erosion during the periods of high river level.

The entire volume of combined sewage from the pond flows by gravity, through a regulating valve, to the Bay Street Sewage Pumping Station and thence to the waste treatment plant for treatment and disposal.

The Waste Water Treatment Plant now has secondary treatment utilizing the activated sludge process. The aeration tanks were designed on the basis of 50 pounds of 5 day BOD per 1000 cubic feet of volume. There are four separate aeration tanks so designed that they can be operated utilizing conventional activated sludge, contact stabilization or step aeration.

The final settling tanks were designed on a solids basis, not the usual overflow or detention basis. In the design, once it had been decided that combined sewage would be treated during periods of runoff, further consideration to the final settling tank size was given. It was decided that no increase in size would be required if the increased flow could be passed through the plant within about three hours. It was agreed that higher flows beyond this time would "flush out" all of the activated sludge.

No change in chlorine contact tank size was made.

For information, the plant was designed for an average dry weather flow of 3.5 mgd. The characteristics of the sewage and waste used in the design provided for a BOD of 320 mg/L (8500 pounds) and suspended solids of 290 mg/L (7500 pounds).

The present connected population is about 13,500 persons within the City plus 3,500 persons at the Northern Wisconsin Hospital and Training School, which is about one mile east of the City and is the reportedly largest single institution in the State.

The present BOD load is about 6000 pounds per day. There are four "wet" industries which must be considered: Peters Packing Company, Leinenkugel Brewing Company, Bowman Dairy and Clover Dairies. The wastes from the Brewery and Clover Dairies are tributary to those sewers which overflow to the combined sewage storage pond. The wastes from Peters Meat Packing Company and Bowman Dairy enter the intercepting sewer near the Waste Water Treatment Plant, and are not in any way tributary to the pond.

RESULTS:

In any new or different type of project, the results are the major consideration. The entire project was designed using sound and proven engineering principles, except that they had never all been put together in this manner in a single project.

Figure 4 is a tabulation of preliminary data to date. The table gives the total precipitation on a given day, but not, at this time, the duration or intensity of precipitation. Samples of the sewage and wastes entering the pond are collected at five minute intervals by an automatic sampler. These are subsequently analyzed in the laboratory at the Waste Water Treatment Plant. A similar program of sampling the pond overflow to the river also is accomplished. All sewage and wastes tributary to the pond are metered through a flume. The volume of sewage and wastes overflowing the pond are currently not metered, although this could be done.

It will be noted that if the pond was not present, combined sewage would have overflowed to the river in excess of sixteen times between April 20 and September 29, 1969. At the time this paper is written, data has not been tabulated as to how many occurrences, in excess of sixteen, would have overflowed to the river. With an average maximum dry weather flow to the Bay Street Pumping Station of 2000 gpm and a new pumping capacity of 6000 gpm, it is obvious that the first 4000 gpm of surface runoff never entered the pond. Between April 8 and September 29, 1969 there were 32 days having a measurable precipitation. It is not known at this time whether there would have been a discharge

DATA ON DAYS OF PRECIPITATION

DATE	PRECIPITATION IN INCHES	DURATION OF DISCHARGE TO POND IN MIN.	B.O.D. TO POND AVG	1ST SAMPLE	2ND SAMPLE	3RD SAMPLE	4TH SAMPLE	5TH SAMPLE	LAST SAMPLE	POND OVERFLOW TO RIVER TIME IN MIN.	BOD AVG	PLANT SEWAGE FLOW MGD.	ROW BOD	FINAL BOD
April 8	.07	-								-		2.7	177	53
14	0.14	-								-		2.2	398	69
20	0.20	25	140	196						-		1.9	462	22
26	0.13	25	224				252			-		2.0	260	32
27	0.48	-								-		1.6	242	21
May 1	.81	20	211	223						-		3.4	204	52
		305	117	191						-				
5	.05	-								-		2.2	369	15
6	.17	-								-		2.4	315	12
10	.02	-								-		2.2	364	25
17	.66	-								-		3.2	369	15
19	.22	-								-		2.0	315	12
21	.12	-								-		1.9	366	30
26	.10	-								-		1.8	354	22
31	.21	-								-		1.5	343	20
June 11	.88	90	151	179	177	195				-		4.3	347	22
12	.53	-								-		3.0	240	23
22	.48	50	156	72	135	212				-		2.8	199	32
25	.68	80	182	229	175					-		3.2	196	8
July 2	.69	60	55							-		2.8	213	13
4	.15	-								-		2.6		
8	1.01	90	125							-		3.8	294	26
14	2.53	115	78	81	112	44	129			120	61	4.9	178	4
24	.03	-								-		3.0	272	14
26	.24	55	170	141	197	227				-		3.6	209	6
Aug. 4	.4	110	98	142	144	140	169	260	59	-				
	1,97	110	110	142						60	27	7.5	138	25
29	.14	-										2.5	250	36
Sept. 14	.18	60	383	368	246	296	317	323		-		2.6	262	26 leaves
22	.36	65	315	261	188	233	271	483		-		3.3	311	17
25	.22	-								-		2.5	353	11
29	.15	35	154	121	154	184	315			-		2.4	174	22

FIGURE 4

to the pond if the pumping capacity of the Bay Street Pumping Station did not take about the first 4000 gpm of runoff.

It must be realized that this is an unusually low frequency of rainfall. The area experienced a severe drought in August and September. It would be nice if the weather were more co-operative when an evaluation program is undertaken.

It will be noted, however, that on only two occasions did the pond overflow. One of these occurred on July 14 when a total rainfall of 2.53 inches occurred and 1.45 inches of this occurred in about 35 minutes. This overflow lasted for two hours and the five day BOD of the overflow was 61 mg/L. The second overflow occurred on August 4, 1969 when there were two separate rainfalls. The first started at about 9:30 A.M. and entered the pond starting at about 9:45 A.M.. The pond had not been emptied when a second rainfall of 1.97 inches started at about 9:20 P.M.. The second rainfall lasted about one hour and about 1.35 inches fell in 40 minutes. The five day BOD on this date was 27 mg/L.

The original worry of problems of sludge deposits on the pond bottom has been overcome. The total time to clean the pond since April has been as follows:

April	15 Manhours
May	3 Hours - Street Sweeper
June	21 Manhours
July	22-1/2 Manhours
August	18 Manhours
September	3-1/2 Hours - Street Sweeper

Experience has shown that the quickest and most economical means of cleaning sludge from the pond is using a street sweeper. However. availability of the unit and operator is sometimes a problem.

Only two of the four aeration tanks are in use. The use of the other two has not been required to date. Figure 4 also shows the 5 day BOD of the sewage and wastes tributary to the Waste Water Treatment Plant and the final effluent on days of precipitation. A review of this data indicates that the introduction of combined sewage and treatment thereof has not been deleterious to the plant efficiency or quality of final effluent when it is discharged to the plant at reasonable rates.

Attention is directed to the relatively poor effluent equality in April and early May. The new facility was placed in operation in February From the start, and until the first week in August, the return activated sludge pumps were not operating properly or at capacity. This was a combination of faulty pumps and motors and poor co-ordination between the motor and control manufacturers. The pumps were variable speed units.

For information, the average volume of sewage and wastes, 5 day BOD thereof, and final effluent from the plant have been:

MONTH	BOD INFLUENT	BOD EFFLUENT	% REMOVED
February	446	27	93.9
March	401	38	90.5
April	345	35	89.8
May	327	26	92
June	236	21	91.1
July	229	18	92.1
August	193	21	89.5

In the Spring of 1969 much of the upper Midwest experienced the second worst floods in history. Chippewa Falls also had extreme high water , and the pond was flooded with river water to prevent damage. After the river receded and the pond was drained a fibrous material appearing to be similar to papermill wastes covered the pond bottom.

This material was about 1/4 to 3/8 inch thick. It was readily removed in pieces, some as large as about a square yard.

Two "bonus" results have resulted. The first of these resulted from the new trunk sewers, which removed all "bottlenecks". Previously, whenever a rainfall, some of lesser intensity than those encountered this year, many basements flooded because of sewer "backup". There has not been a single flooded basement because of sewer backup.

A second bonus became evident during the Spring flood. Previously, whenever there was a flood, basements in buildings near the river flooded because of sewer backup. In the Spring of 1969, not a single basement was flooded.

Some discussion appears appropriate concerning the characteristics of the combined sewage entering the pond and which would otherwise overflow to the river. It was once a general opinion that the "first flush" of runoff flow would produce the highest BOD. Later some authorities have proposed that the flow sometime after the "first flush" would produce the highest BOD. Because about the first 4000 gpm of flow from runoff in this instance is not sampled, the characteristics of the first flush are not known. It appears, however, that the 5 day BOD of the flow tributary to the pond increases for up to about 25 minutes. This would substantiate the theory that the "first flush" is not the problem but rather a sustained flow.

The project has now been accepted by the general public as a major improvement. Initially, the public was convinced that odors would result. There have been no odors. Basement flooding has been eliminated. The public is happy about it. The newspaper editor has had a sign prepared and the combined sewage storage pond is now named "LAKE LEHMAN" for Superintendent of Public Utilities, Clyde Lehman.

CONCLUSIONS:

A project of this type will achieve the required results if properly located, designed and operated. Its use must be studied, based on land availability, feasibility and economics. Of course, the requirements of the State Regulatory Agency must be considered.

It must always be remembered that any program of storm water separation can probably never be 100 per cent accomplished if there has once been a combined system. Probably the only way it could be accomplished would be to test every catch basin, televise every sewer, and demolish every building where roof drains do not discharge above ground and where it is positively known that there are not any footing and foundation drains. There still remains those buildings having leaking basements and the water goes to a floor drain.

The F.W.P.C.A. appointed a special task committee to review the project and observe the sampling and testing program. At the time of writing this paper, the task review committee has not submitted its report. However, the chairman of the committee has advised that the only physical change they would recommend is the installation of a baffle preceding the pond overflow structure. This baffle theoretically would minimize the floating solids from overflowing to the river. The committee expects to recommend some changes in the tests now being conducted, especially the obtaining of D.O. in the river.

Polymers for Sewer Flow Control

by

George A. Kirkpatrick, P.E.*

"Polymers for Sewer Flow Control" is a report completed in August 1969 by The Western Company of Richardson, Texas. It describes the work performed during a 29-month contract period to develop and demonstrate the use of high molecular weight polymers to reduce pipe friction and, thereby, increase flow rates in sewers. The additives thus used were tested for toxicity and their effect on aquatic life. The effects of the polymers on sewage parameters such as dissolved oxygen, biochemical oxygen demand, change in settleable solids, and sludge drying were studied. Limited work was performed to determine their effects on sedimentation, filtration, and sludge drying in an actual wastewater treatment plant.

For those not familiar with them, polymers can be defined as products resulting from the joining together of a number of identical molecules of a simple substance. Rubber is an example of a polymer which is made up of a long chain of isoprene molecules. It has been learned that water can be polymerized to form a so-called "polywater", a substance 40 percent more dense than normal water. In the wastewater treatment field, certain polymers are used to induce coagulation of colloids and assist in sedimentation and filtration processes.

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The chemical and physical changes which polymers impart to fluids to reduce viscous friction have not been fully explained. A simplified explanation given by The Western Company is: "...that polymers probably tend to act as 'turbulence dampers' and, in effect, damp out the very irregular paths of the fluid particles near the wall and extend the laminar boundary layer further into the turbulent flow core. This damping effect causes the laminar sublayer to increase in thickness, resulting in a reduction in the wall velocity gradient and shear stress gradient which provides a reduction in the frictional resistance to flow, since the action of wall shear stress is to slow down the fluid near the wall."

Based on a literature survey, and on the Contractor's previous experience, six polymers were selected for evaluation. Preliminary tests of these potentially "best" polymers were made in an existing small-scale test rig, and five of them indicated sufficient potential to warrant further testing.

A six-inch diameter, 100-ft long, asbestos-cement sewer pipe, with one transparent section of pipe, was constructed to evaluate the effects of polymers under different flow conditions of sewage. (See figure 1). Provision was made for varying the slope of the facility between 0.3 and 2.0 percent, controlling the temperature of the sewage between 38^{o} and 90^{o} F, varying the flow rate from 0 to 750 gpm, and the polymer concentration from 0 to 1,500 mg/l. Sewage concentration could be varied as required.

To disperse the polymers throughout the fluid for rapid absorption, it was found necessary to first prepare them in a slurry form. For this

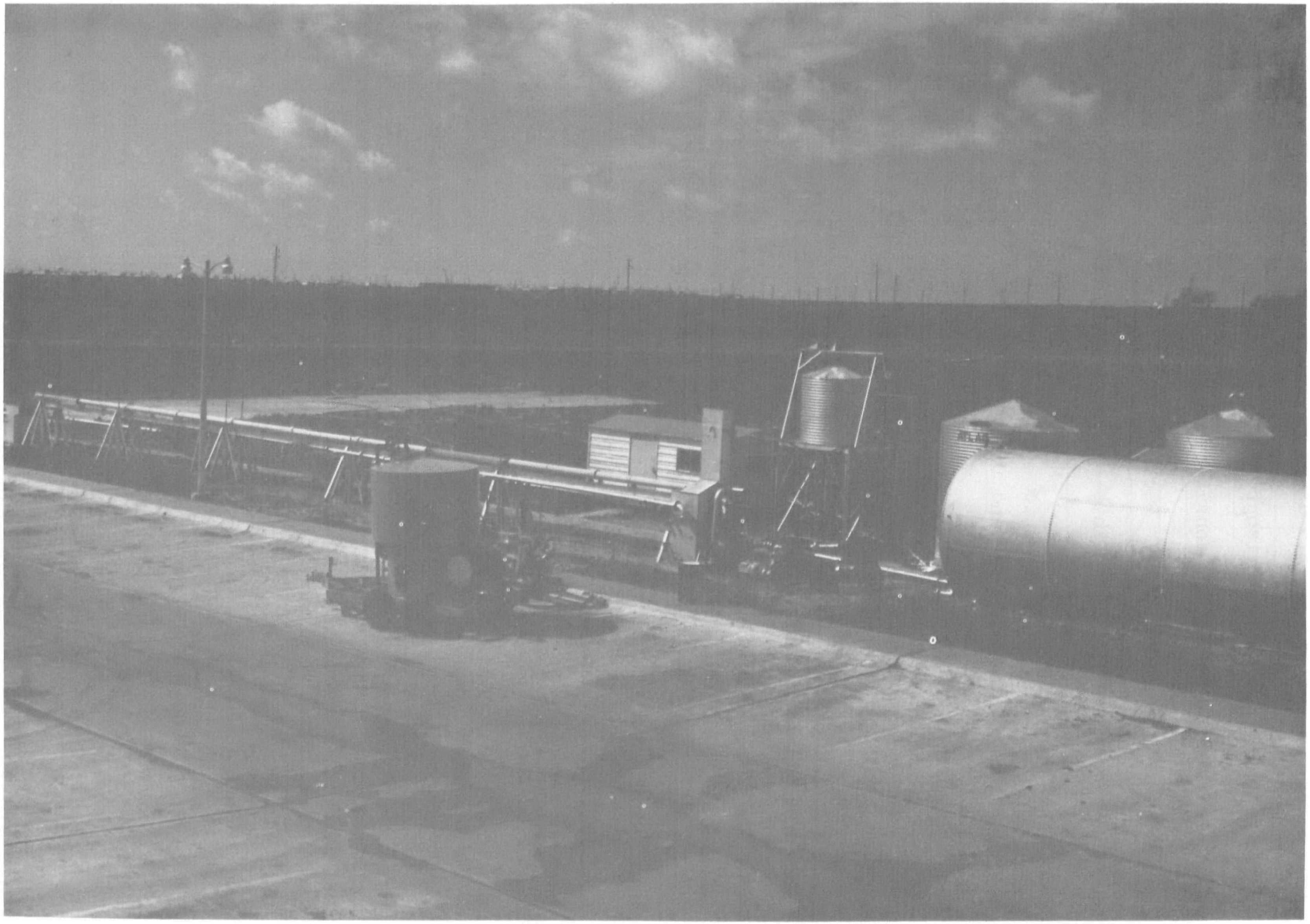

Figure 1. Overall View of 6-Inch Diameter Flow Test Facility at Richardson, Texas

purpose they were predispersed in a nonsolvent, either a product called Cellosolve or anhydrous isopropanol, which was jelled with a cellulose ether. The slurry consisted of 40% polymer, 59.5% nonsolvent, and 0.5% gelling agent. Because one of the five candidate polymers did not lend itself to this treatment, it was eliminated from further testing.

Pressure drop, temperature, and flow rate through a 30-ft test section were measured in the test facility. During each test the flow rate was held constant. Each polymer was tested at various flow rates, polymer concentrations, sewage concentrations, and temperatures.

Results of these tests are presented in terms of percentage flow increase (see figure 2), which was derived from measured pressure drop using curves of relationship between pressure drop and flow rate, as shown in figure 3. The three most effective polymers with respect to flow increase at constant pressure drop, in order of decreasing effectiveness, are Polyox Coagulant - 701 and WSR - 301 (both supplied by Union Carbide Co.), and AP - 30 (supplied by Dow Chemical Co.). Flow increases of more than 140 percent (2.40 times original flow) were attained with polymer concentrations of 150-200 ppm. Effectiveness of these polymers varies significantly with sewage temperature and solids concentration, with AP - 30 being least effected by temperature and solids. (See figures 4-6).

Following tests in the six-inch sewer line, a section of a 24-inch interceptor sewer line in the City of Dallas, Texas was instrumented as shown in figure 7 for testing under actual field conditions. This

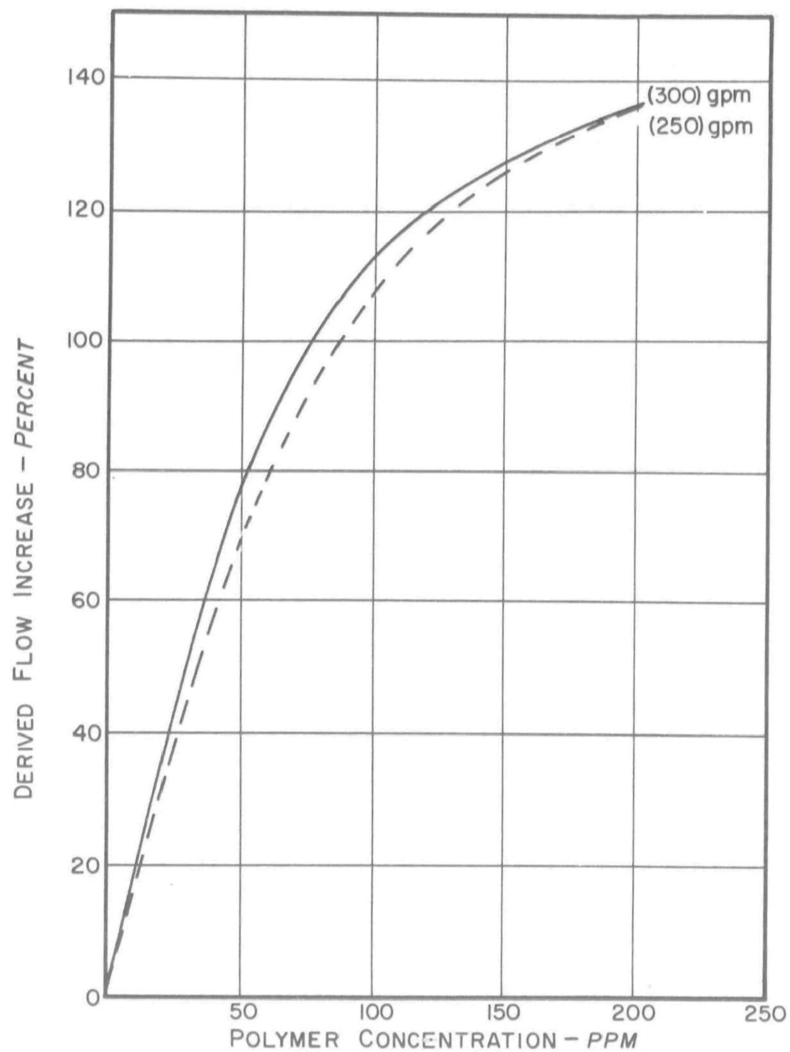

Figure 2. Polyox Coagulant-701 in Six-Inch Test Facility at 73°F in Water.

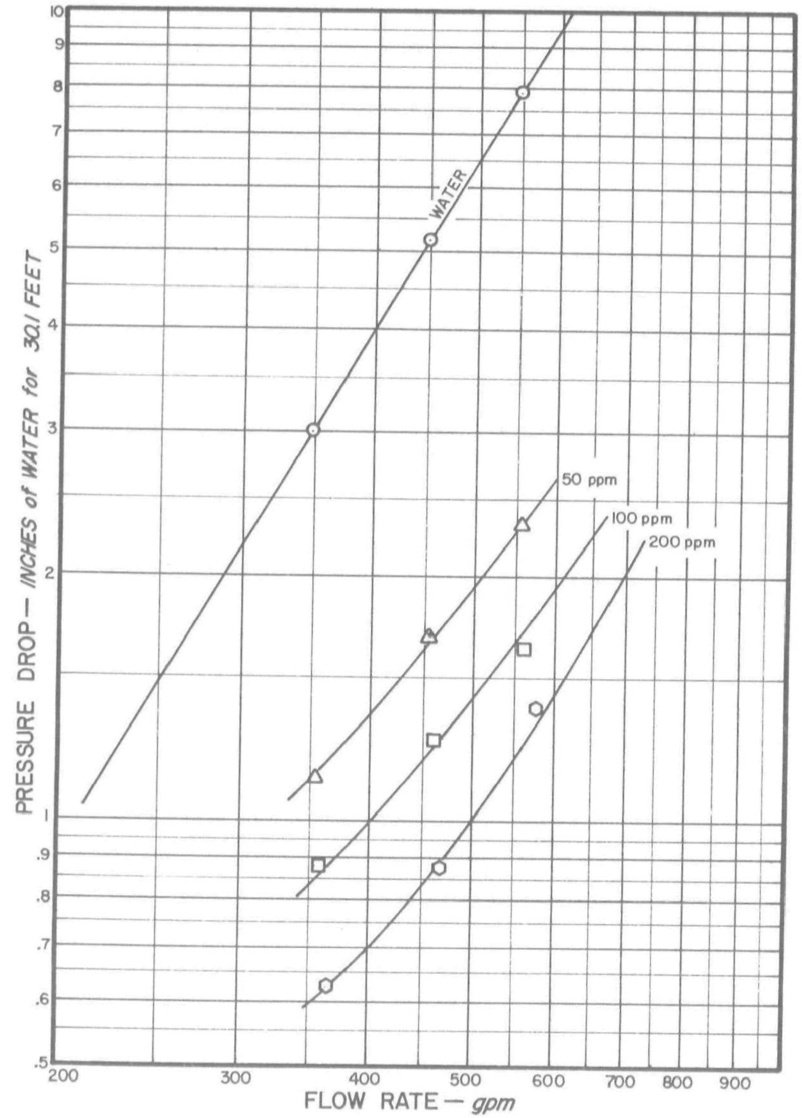

Figure 3. Polyox Coagulant-701 in Six-Inch Test Facility at 73°F in Water.

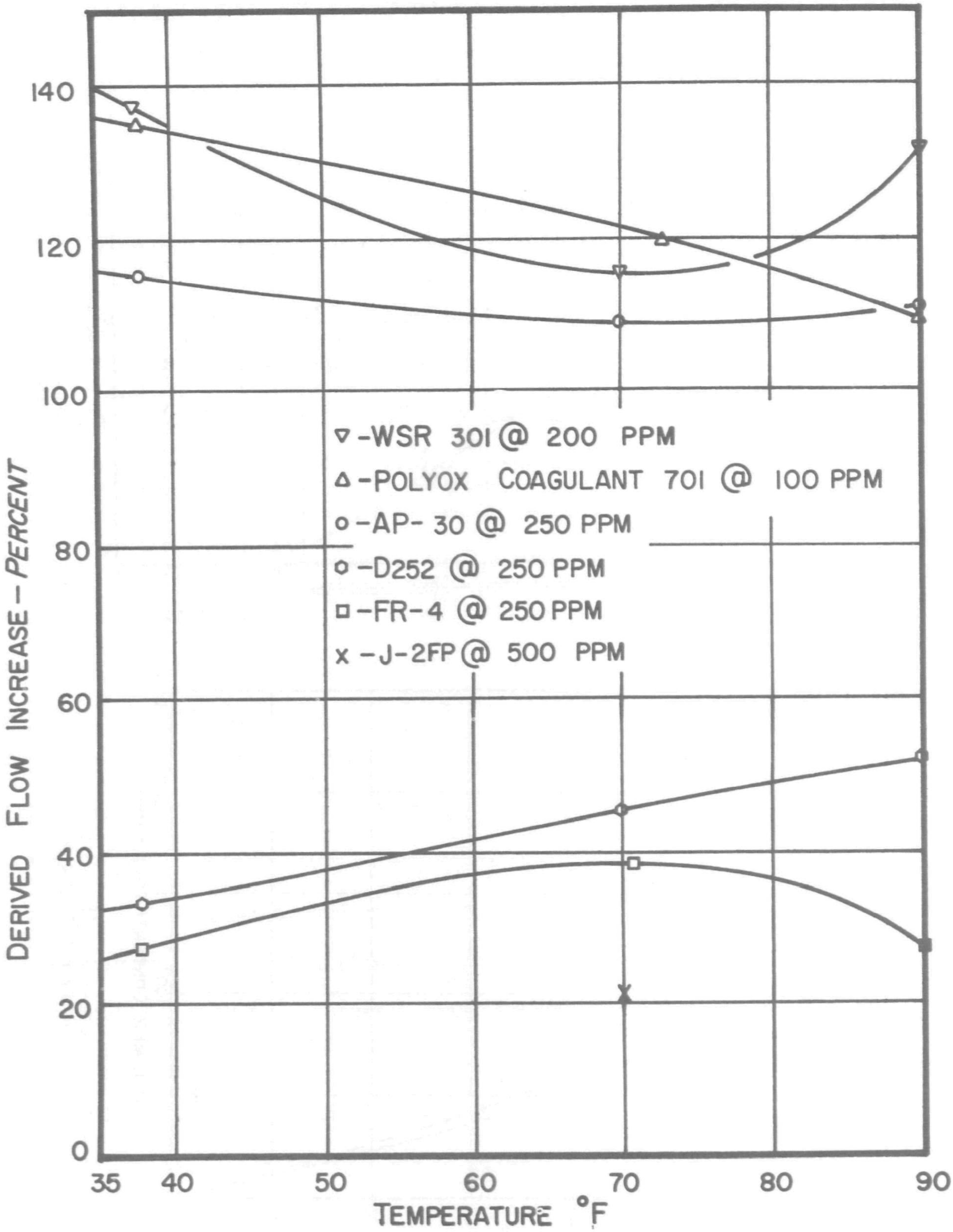

Figure 4. Comparison of the Effectiveness of Six Additives in Water as a Function of Temperature.

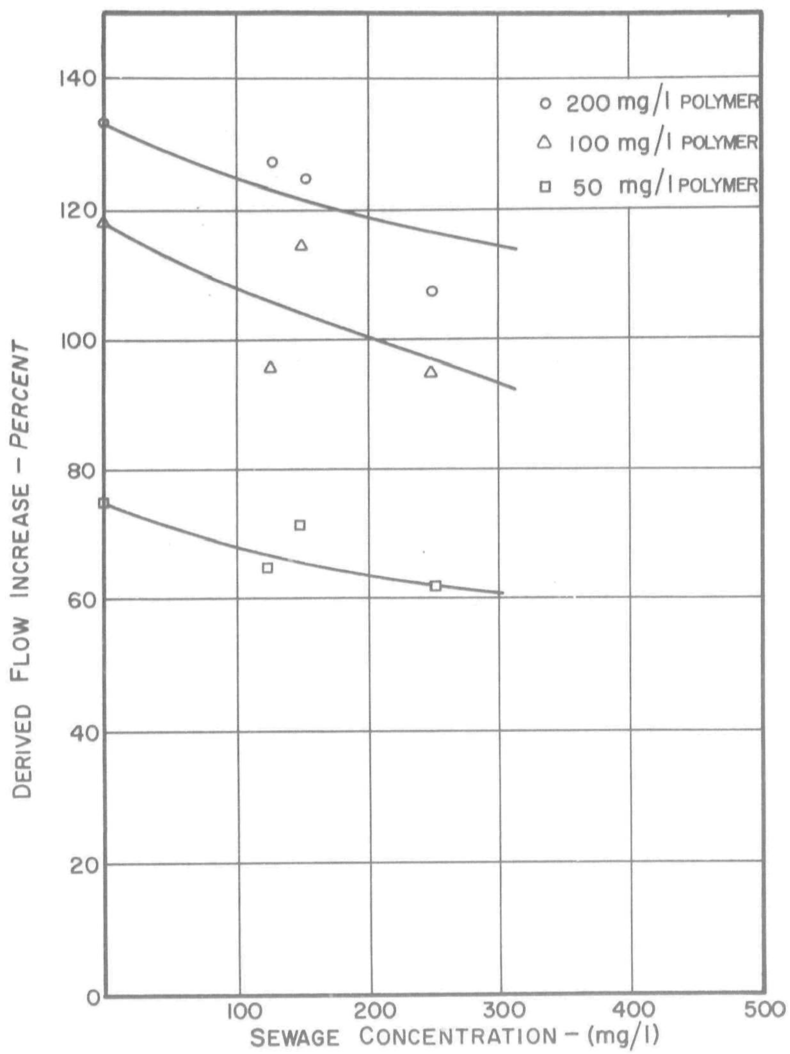

Figure 5. Percentage Flow Increase vs Sewage Concentration (mg/l) Polyox Coagulant-701 Polymer.

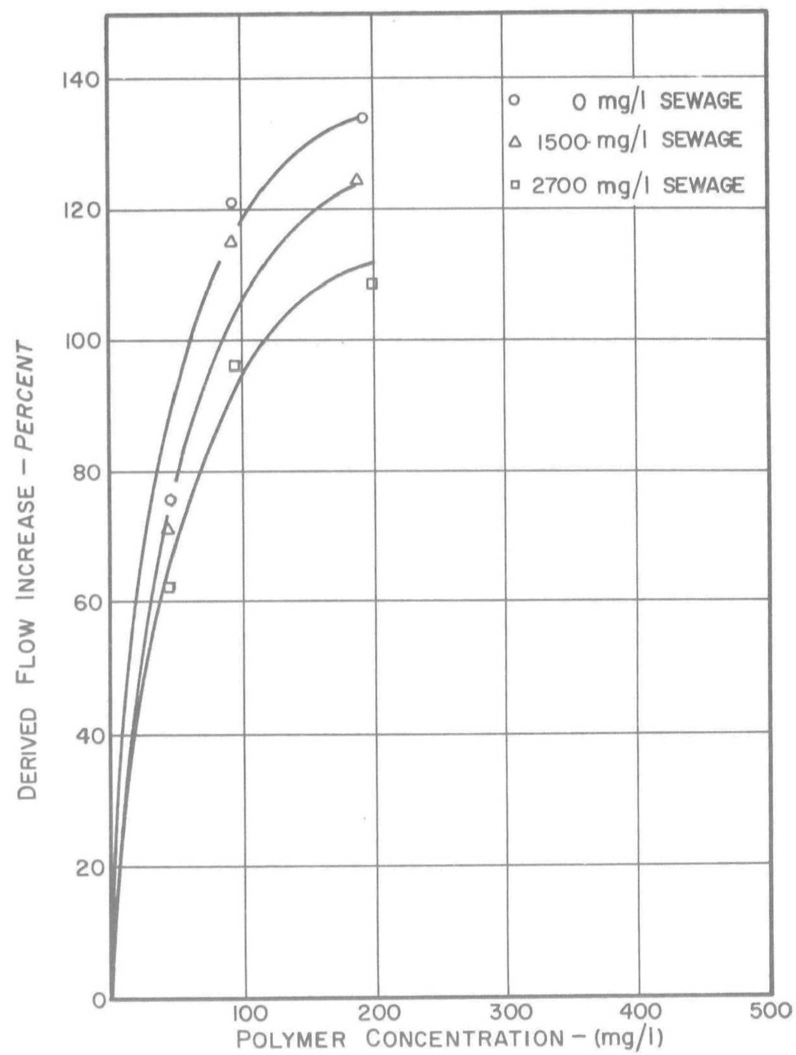

Figure 6. Polymer (mg/l) vs Percent Increase With a Given Sewage Concentration, Polyox Coagulant-701 Polymer.

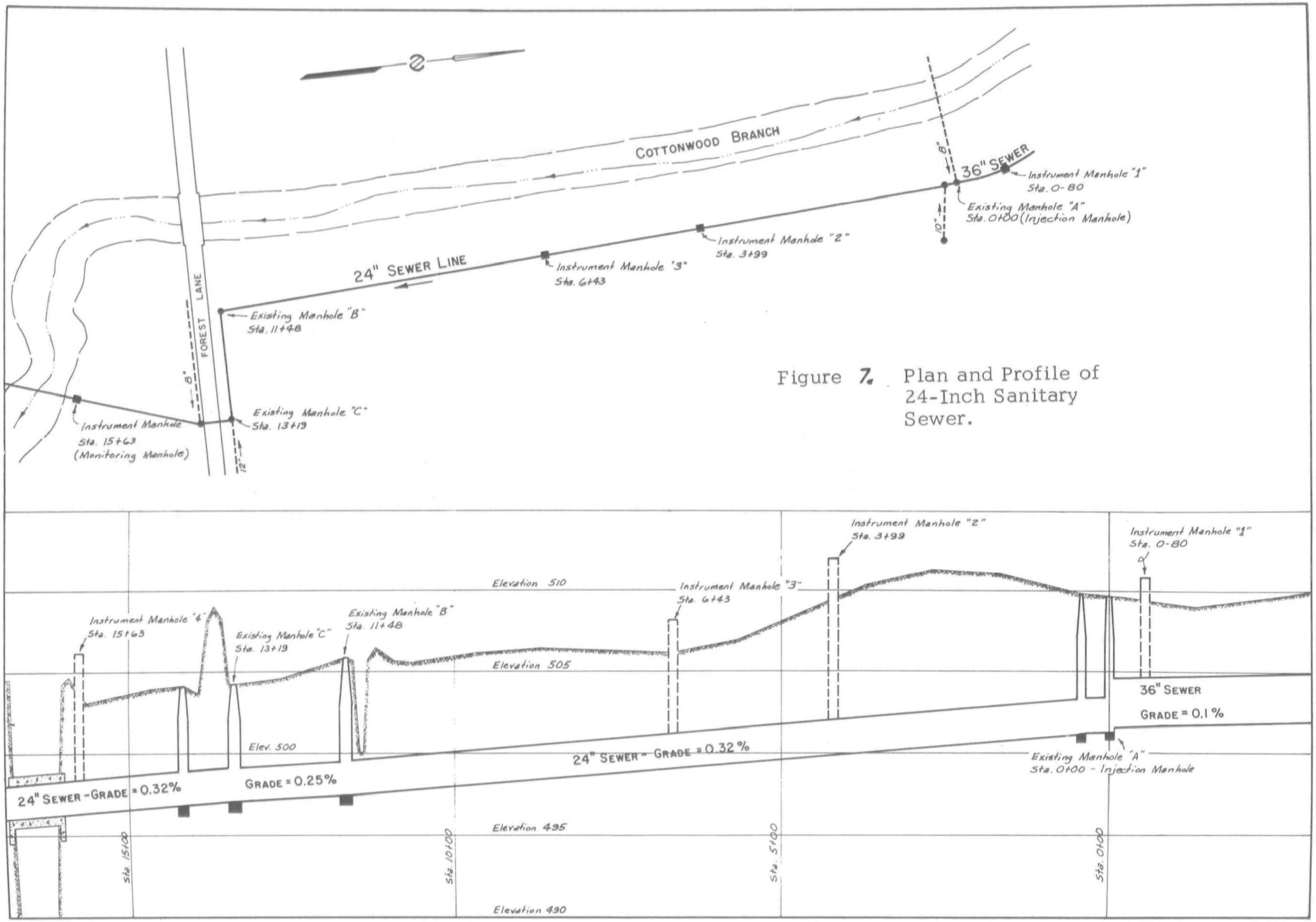

Figure 7. Plan and Profile of 24-Inch Sanitary Sewer.

sewer receives flow from a 36-inch interceptor line and discharges into a 30-inch diameter sewer downstream. During peak daily flow periods, the line is surcharged to heights between four and eight feet above the top of the pipe.

Seven manholes were used in a 1,563-foot length of the sewer line, and of the 36-inch line just upstream. Piezometers were installed in six of the manholes - three to be read manually and three equipt with level recorders. Provision was made for taking temperatures and collecting samples of the sewage. Sewage flow was measured by using a dye tracer and a fluorometer for determining dye concentration.

Two mobile units for mixing, transporting, and injecting polymers were constructed for the tests. Figure 8 shows the mixing and transporting unit, which is primarily a 1,140-gallon tank. The second unit containing the injection device with a capacity of 250 gallons per minute, measured with a magnetic flow meter, is shown in figure 9. Polymers were injected as a slurry consisting of 69.25% isopropyl alcohol, 0.75% gelling agent, and 30% polymer. In figure 10, the slurry mixing and storage tank, the injector, and auxiliary equipment are shown connected to the 24-inch sewer line.

Four tests runs each were made to test Polyox WSR - 301 and Polyox Coagulant - 701, using polymer concentrations varying between 35 and 100 mg/l. In each test, polymer injection was stopped when head on the line was reduced enough to eliminate a surcharged condition. Both polymers were effective in obtaining the desired head reduction, although Polyox Coagulant - 701 provided a more rapid head reduction for the same

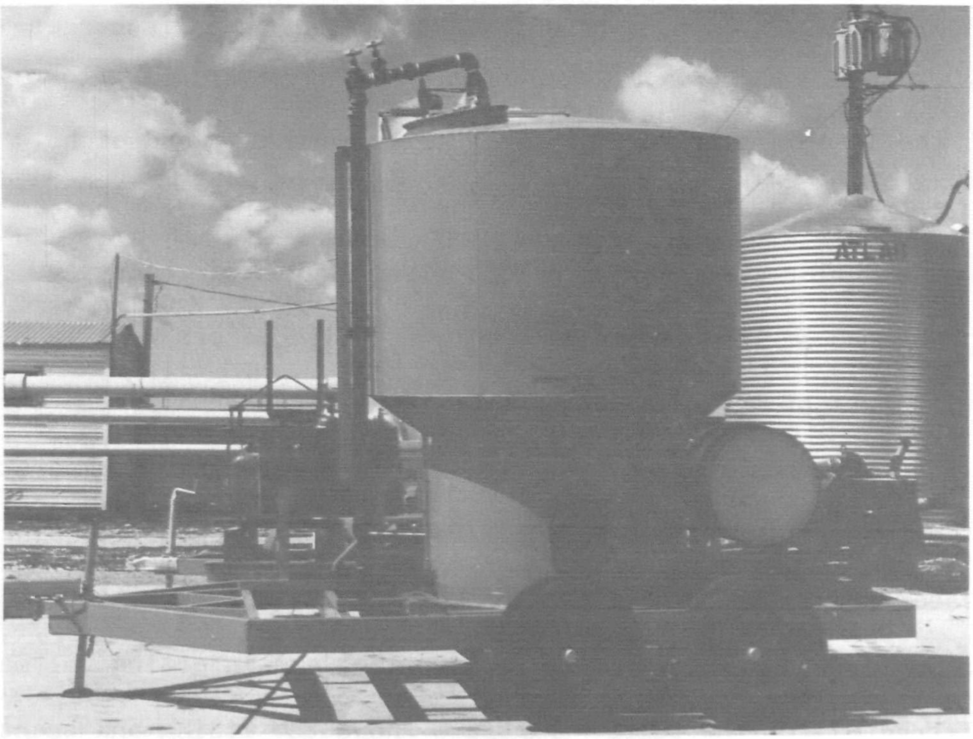

Figure 8. Slurry Mixing Tank

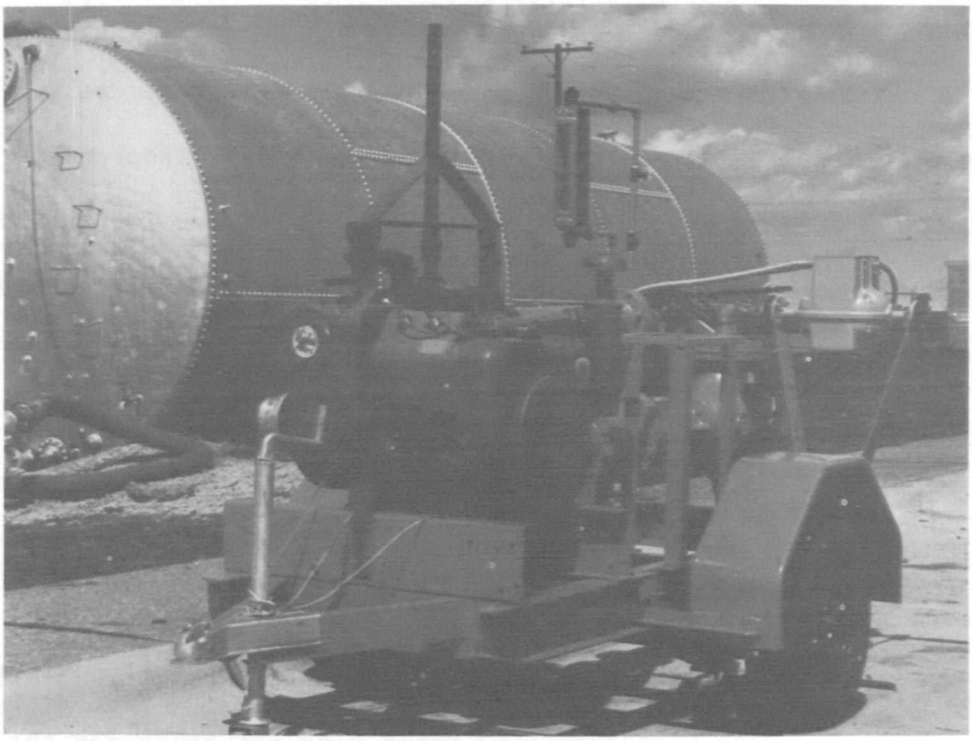

Figure 9. Injection Unit

Figure 10. The Injector, Slurry Mixing and Storage Tank, and Auxiliary Equipment Connected to the 24-Inch Sewer Line

polymer concentration. A hydrograph of flow and changes in surcharge elevation before, during, and after injecting polyox coagulant - 701 are presented in figure 11.

Based on information gained from these tests, and on an analysis of frequency and intensity of rainfall in the area, the annual cost of using polymers to control overflows from a 15-inch sewer at Garland, Texas, was estimated to be about $6,400 per year. Based on actual bid costs for construction, and on average sewer operation and maintenance costs in Garland, the estimated cost of a relief sewer would be about $27,000 per year, or more than four times as great.

While tests were being run to demonstrate the effectiveness of polymers on flow increase, laboratory tests were conducted on the originally selected six polymers to determine their effects on aquatic life. Concentrations of polymer of up to 500 mg/l were used. Tests were made using polymers in both a slurry and non-slurry form, and using nonsolvents without polymers. From these tests, the following conclusions were reached:

1. The polymers evaluated are nontoxic to bacteria found in raw sewage under the conditions of the tests. Therefore, they should not be detrimental to the micro-biological treatment process in a wastewater treatment plant.

2. The polymers tested have neither toxic or nutrient effect on algae under the concentration and conditions tested.

3. The use of polymers as friction reducers in sewers will not contribute indirectly to lake or stream pollution by having a toxic affect upon fish life.

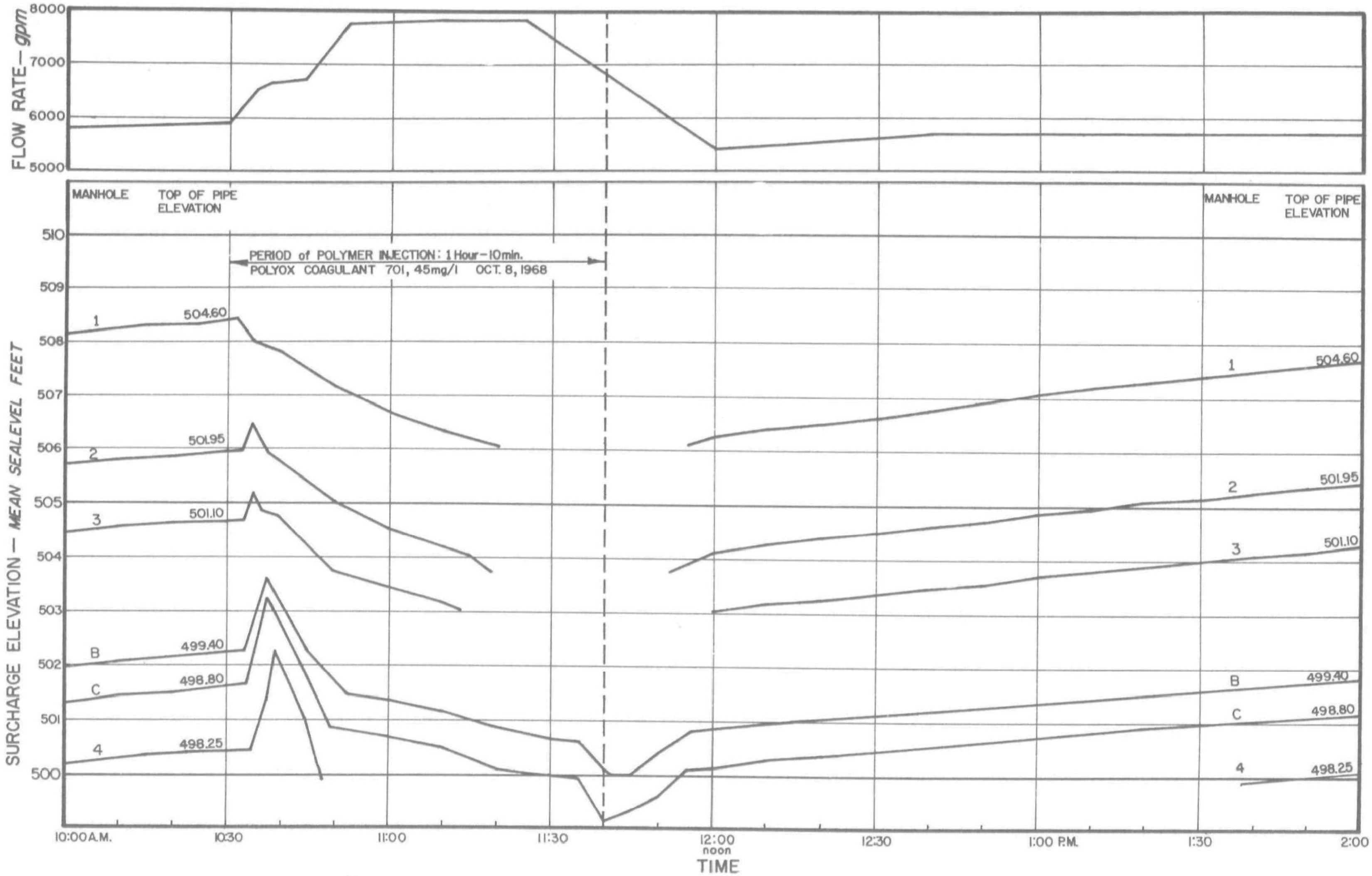

Figure 11. Hydrograph of Flow and Surcharge of Monitoring Manholes Before, During, and After Injecting Polyox Coagulant-701.

Tests of the effects of the polymers on certain sewage parameters resulted in the following conclusions:

1. All the polymers demonstrate the capability to increase sedimentation. Although some loss of polymer injected into a sewer may occur due to sedimentation, this loss would be small because of turbulent conditions when the sewer would be surcharged and the polymers would be used.

2. The 5-day BOD for all polymers averages 1.56 mg/l for a polymer concentration of 500 mg/l. This value of oxygen demand is negligible when compared with that of the raw sewage used in the tests, about 200 mg/l.

3. Use of polymers decreases the water retention capacity of sludge, thereby yielding a dryer sludge cake for earlier disposal. .

The wastewater treatment plant at Lewisville, Texas, was instrumented and tests with polymers were run to determine their effects on sedimentation, filtration, and sludge drying under actual wastewater treatment plant conditions. Unfortunately, due to plant machinery breakdown and other plant operational difficulties during the testing, results of the work are inconclusive. Although no definite improvements in filtration and sedimentation rates could be detected, apparently no adverse effects developed under the conditions of polymer application at the plant.

Additional experimentation with use of polymers for sewer flow control is recommended to include the following:

1. Investigate polymer modification to permit dry feeding directly into the wastewater, without use of a slurry.

2. Study the effects of mechanical agitation, such as pumping, on degradation of polymer effectiveness.

3. Test the effectiveness of polymers on flow increase in pipes of larger diameter.

4. Study the effects of friction reducing polymers on filter-rock biota and the activated sludge treatment process.

5. Determine the effects of various industrial wastes upon the friction reduction capabilities of polymers.

The City of Dallas, Texas, with the assistance of The Western Company, has commenced a project to install permanent equipment to inject polymers into a 30-inch sewer line to increase flow rate and control overflows. Possible degradation of polymer effectiveness when flow passes through a pumping station will be investigated. Nearly 75 percent of the cost of this project is to be funded by an FWPCA demonstration grant.

OVERVIEW OF TREATMENT METHODS

by

Darwin R. Wright

If I were a college professor at this time, I would tell you to open up your text book to the section on combined sewer overflow treatment, but there is one problem. There isn't a book on the treatment of combined sewer overflows. Why? Because we are treating a different waste with a different flow pattern; we are treating a random waste, not a steady-state waste. As we proceed, I think that you will find why this is true.

I would like to make two points. Number 1, as we say in Washington after we got Vince Lombardi and Ted Williams, it is an all new ballgame. Just taking quality alone, it is not unusual to have the suspended solids range from a few mg/l to 2000-5000 mg/l and these changes can occur rapidly. There is also a difference in the COD/BOD relationship, being greater than domestic sewage. As was mentioned earlier, I am not convinced that there is or is not a first-flush phenomenon. The important thing is that you are going to have to treat a varying waste. You are liable to get this "first flush" at any time. The quality will be constantly changing since the flow pattern is constantly changing. You are treating a storm hydrograph. You are not talking about a peak dry-weather flow that may be twice the daily average flow. You may be talking of a peak flow of a hundred to a thousand times dry-weather flow. I was in Philadelphia yesterday and although I don't think you would design for a hundred year storm, one outfall has a dry weather flow of about 25 cfs (intercepted) and over 3000 cfs storm flow was recorded. That is high, but this same sewer is capable of going up 20 or 30 times the normal dry weather flow or peak dry weather flow for a one-year storm. Since we have the two problems of varying quality and quantity, it appears that some type of a storage or surge facility ahead of the treatment unit will be required. One of our speakers this afternoon will discuss, in some detail, the tradeoffs that were made between storage required versus treatment facility size.

We are basically looking at all three of the treatment methods--physical treatment, biological treatment and physical-chemical. Under the physical treatment methods, the two that are going to be talked about after lunch appear at this time to be the most promising. One is screening from bar screens down to micro strainers with 15 to 20 micron size openings. The other promising method is dissolved air flotation. We have gone through a .5 mgd pilot plant scale. We now have a 5 mgd plant, which you will hear about this afternoon, in Milwaukee; and we have the plans and specifications for a 24 mgd plant in San Francisco. When we talk about high-rate filtration we are talking about "high-rate" filtration. "High-rate" filtration will be covered by the Crane Company speaker today, but we're talking now about 45 gallons per minute per square foot. We are attempting to achieve the equivalent of secondary treatment, or 70-80 percent removal of solids. I might point out here that since we are treating a different waste, if we can get out 70 or 80 percent of the solids, we can get maybe 60 or 70 percent BOD removal. A couple of other techniques that we have tried in the filtration area is one in which we used an ultra-sonic filtration system where we could go down to 10 microns pore openings. It turned out that after providing the pre-treatment required, because of the nature of the combined sewer overflows, the plastic filter elements were not effective. We tried also using the diatamaceous earth filter and again that was unsuccessful for the same reason.

The SWECO screen is a vibratory rotating screen which we have successfully tested in Portland on combined sewer overflows. Utilizing mesh screens instead of micro screens, 40 percent to 50 percent removals have been achieved.

When it comes to biological treatment, we suddenly wonder what happens when you have a large influx of flow. Here again we are talking of 15-20 times dry weather flow on a routine basis. The one we have investigated that looks promising for high flow rates, is the Allis Chalmers' bio-disc, the rotating biological contactor method. The dry weather flow going through the plant is 1 mgd and the peak flow is about 24 mgd. As with the other treatment processes, it appears that a surge facility will be required. This particular type of biological treatment involves a large concentration of bio mass. Sloughing is a problem and the bio-mass is very easy to settle. Therefore, a final clarifier is needed.

In New Providence, New Jersey, we have under construction a high-rate rock filter and a plastic media filter. We are attempting to determine how a high-rate filter compared to a standard trickling filter will react to treating storm overflows. Part of this project is a surge facility. New Providence can only discharge into the interceptor sewer one-half mgd per 8 hours. Since normal daily variation of flow just doesn't follow the above condition, the surge facility will be used to enable the City to discharge at a constant rate of 0.5 mgd per 8 hours. The surge facility will also reduce peak storm waters and level out the flows to the treatment facility.

Mention was made earlier of the Kenosha project involving a bio-solids reservoir. There will be a 15 to 30 minute contact tank which will be empty during dry weather conditions. Storm flow will be diverted into the contact tank, as will bio mass, thus providing solids stabilization. Chlorination will also be provided.

We have one project where we have built a 10-acre oxidation pond for treating combined sewer overflows. The results are inconclusive, but tend to be on the negative side. As built, the pond needs further refinement. We have under construction a deep lagoon project in which we will have anaerobic treatment at the bottom and aerobic treatment at the top.

Under the physical-chemical treatment processes, the feasibility of activated carbon adsorption followed by or with coagulation, flocculation or sedimentation is being investigated. It appears that carbon regeneration may be a problem. In-house and contract work for FWPCA indicate that economical methods of carbon regeneration may be available soon.

As far as disinfection is concerned, which would be the final stage, we are investigating in New Orleans chlorination with gaseous chlorine versus sodium hypochlorite. We are attempting to treat 11,000 cfs during a peak storm flow. To provide large quantities of sodium hypochlorite the Grantee built an automatic hypochlorite plant. We are investigating use of ozone in our microstrainer project.

We have also done exploratory studies on some of the other ones like bromine. This report will be out soon.

An interesting question is, "What degree of treatment do we want?" We haven't really formulated a policy yet on what degree of treatment is required. The key issue depends upon the existing water quality standard. To protect a beach area, reduction of bacteriological pollution would be the important parameter. This is basically the problem which we face now with our San Francisco dissolved-air flotation grant. Of prime concern is cleaning up a beach area, making it safe from a bacteriological standpoint and removing the flotable solids--grease balls and other visible materials.

If you have a stream which already is overloaded from any oxygen demanding material or unsitely sludge deposits, the BOD or solids must be controlled. Another problem in analyzing your system is industry wastes or some other toxic material which may be contained in the overflow. The other thing to consider in the control of overflows is what effect will this control have on the existing sewage treatment plant. If you store all the waste from 30 overflows, where are you going to treat it? If you already have an overloaded sewage treatment plant, you are going to have to provide additional facilities. Consideration should be given in the planning for making the facilities multiple purpose. The facilities could operate during dry weather flow to provide a higher overall degree of treatment. During periods of runoff you would treat the overflows. The net result is a greater overall **system** efficiency. In planning new treatment facilities, such as new dry weather treatment plants or additions, consideration must be given to the storm water overflow problem. If you do provide storage facilities, you are going to have to treat that stored waste somewhere. One of the gaps which we still have in our program, because we have not completed all our projects, is the cost data, as pointed out in the storage problem at Chippewa Falls, the alternate solution was nearly equal to what the separation cost was except for the intangible benefits. One thing with that particular project or with all projects, if you separate you will have to live with adverse quality of the separate storm water. Ten to 15 of our demonstration studies have sampled straight storm water. For highest water quality uses associated with the water quality standards storm water discharges impart a significant load. This load would probably not meet the existing standards. One of the problems which needs resolving is the intangible costs or benefits. To come up with a true cost, both tangible and intangible costs must be considered. There seems to be a problem as to how to express the combined sewer abatement cost. The typical terminology used for treatment costs is cents per thousand gallons. Is this a realistic cost to use when you are only talking about operating these facilities maybe 3% of the time? In this general area here in the Northeast it rains about 3 percent of the time, overflows occur 3 to 5 percent of the time. Should treatment be expressed in terms of dollars per acre, which would give an equivalent separation cost. Regardless of how the cost is expressed, we must accept the fact that there is not an economical solution utilizing either treatment or in storage. The route we are taking on these projects is the development of cost curves based upon flow rates or treatment efficiencies.

MICROSTRAINING - WITH OZONATION OR CHLORINATION - OF COMBINED SEWER OVERFLOWS

PRELIMINARY REPORT

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ABSTRACT

Microstraining[(C)], using a nominal 23 micron aperture Microstrainer screen, has removed up to 98% of the suspended solids from a combined sewer overflow. The sewer, which has an average sanitary sewage flow of 1,000 gph, serves a residential area of 11 acres in the City of Philadelphia. The maximum combined sewer flow recorded during rainstorms in one year of operation has been 305,000 gph.

Volatile suspended solids removals with the 23 micron Microstrainer screen have averaged 68% and 71% during different test periods.

BOD removals, as measured by BOD tests, and coliform bacteria concentrations in the Microstrained effluents have varied widely. Postulations as to the effects of Microstraining on these results are given.

Results to date indicate that there is a slightly better colon group bacterial kill with chlorine in the Microstrainer effluents than when ozone is used, when both are used at an initial nominal concentration of 5 ppm with 5-12 minutes detention time. However, chlorine application has been better controlled and it has not been possible to optimize requirements for these chemical feeds.

Preliminary estimates have been made for the costs of treatment for a combined sewer via the Microstraining process. It is estimated that the costs per acre of drainage for a full scale plant in our test area would range from approximately $9,500 to $11,800 for Microstraining alone, $10,500 to $12,800 for Microstraining plus chlorination, $18,000 to $21,300 for Microstraining plus ozonation. These costs compare favorably with other techniques that have been proposed; e.g., the costs associated with construction of separate storm and sanitary sewers have, in several cases, been estimated to range between $20,000 and $23,000 per acre.

Cost estimates at a higher confidence level for Microstraining could be derived through additional investigation at the higher throughput rates, the consideration of which was begun during the latter part of the program. We have also performed preliminary calculations, which show that larger installations; e.g., 10 x the above, may produce costs 20% to 30% lower than these estimates, on a per acre basis.

Moreover, should the market for Microstrainers substantially increase over present levels, it is probable that a higher production volume would result in lower production costs, which could be passed on in savings to users.

[(C)] Copyrighted Trade Name - Crane Co., Glenfield & Kennedy Div.

This work has been conducted with the cooperation of the City of Philadelphia under Contract No. 14.12.136 from the Federal Water Pollution Control Administration, U.S. Department of the Interior.

The pollution problems associated with combined sewer overflows in our cities have multiplied and grown enormously over the past 20-30 years. The increased concentration and growth of urban activities have brought this about, and these problems have been subjected to much technical and economic study. The studies have been intensified and broadened, particularly over the past 15 years or so, because of increased public awareness of the severity of the overall problem of pollution of our streams and coastal waters.

Exact figures are difficult to obtain -- and they are certainly variable over a wide range in relation to such factors as sewer design and service, time of day, intensity and frequency of rainfall and character of the drainage area -- but the contribution to pollution of the combined overflows is of considerable magnitude. An example is data reported from Buffalo, N.Y.(1), which indicated that one third of the city's annual production of sewage solids overflowed without treatment, although only 2-3% of the sewage volume actually overflowed. And, in a special 2 to 3 year study conducted at Northampton in England(2) it was estimated that the overflow loads of suspended solids, COD and BOD discharged in a year of average rainfall would be respectively 132, 62, and 35 times the daily dry weather load with an overflow setting of 3 times the dry weather flow. Further, work accomplished at Detroit, Mich.(3) is reported to show that receiving stream bacterial contamination effects persist for several days after discharge has ceased, with the duration of effects increasing with an increase in storm intensity.

Heavy debris and sludge deposits in receiving streams below sewer outfalls are common, and are considered to be long term contributors to pollution due to the fact that only their surface layers are readily accessible to natural purification forces.

Among the possible solutions to the problem that have been proposed, and in some cases implemented, are separate and parallel storm and sanitary systems, separate sanitary systems piped within larger combined systems, retention tanks for combined flows, in-system storage, surface drainage area control, various treatment processes, and combinations of these.

The costs of sewer separation for the entire country have been variously estimated at 25-30 billion dollars(1) and higher(4), depending on the factors considered in arriving at the estimates. And many municipalities indicate that the practical possibility of changing all combined sewers to separate is remote(5).

In a more recent estimation, it is suggested that street refuse can present a significant pollution load(6). There is thus a growing body of opinion that the quality of separated urban storm water is such that this separated storm water should be treated in some manner prior to ultimate discharge.

It now seems obvious that, for a variety of reasons, the separate storm and sanitary system concept is not, at least by itself, the answer to the problem. Rather, conclusions from information gathered in an extensive survey[5] point out that different solutions and combinations of solutions will be required in different localities depending upon local circumstances. These circumstances include not only the discharge systems, rainfall, areal characteristics, etc., but also the desired character of the effluents as they relate to the receiving stream or body of water.

The work reported here should provide a basis for the application of additional tools that can be employed in combatting a most complex pollution problem.

SUMMARY

The information developed in this work to date preliminarily indicates that treatment of combined sewer overflows via Microstraining can furnish a high degree of solids removal for a per acre cost of approximately 40-50% of the cost of sewer separation in cities where separation has been considered, such as Washington, D.C., Philadelphia, and Chicago. Treatment of an actual overflow in a residential area of Philadelphia has produced solids removals of up to 98%. Limited data, for a fine Mark "0" (23 micron) screen, under relatively high throughput conditions, show removal figures ranging from 78% to 98%, with an average of 91%. Figures for a larger number of tests made with lower throughputs show a solids removal range of 62% to 96%, with an average of 80%.

Volatile suspended solids removals have roughly parallelled the experience with total suspended solids. These removals for the Mark "I" (35 micron) screen averaged 47% and, for three modes of operation using the Mark "0" screen, have averaged 68%, 71% and 71%.

Bacteriological results measured across the Microstrainer screens exhibit anomolies, both reductions and increases in total and fecal coliform being measured. Further major total coliform reductions can, of course, be achieved with chlorine or ozone. Our results, with both ozone and chlorine, although again anomolous in some instances, indicate a slightly better performance with chlorine. Both chemicals have been used at a nominal 5 ppm feed rate, the chlorine detention time being varied at 5 and 10 minutes, and the ozone reaction period regulated at about 12 minutes. Average values for total coliform residuals after treatment were (per 100 ml) 166,000, 129,000 and 619,000 respectively. These values for fecal coliform residuals were 41,000, 81,000 and 42,000. We attribute the ostensibly better performance of the chlorine to a more positive mode of chlorine addition than has been possible with the ozone.

BOD removals across the Microstrainer have been difficult to measure. In those cases where reductions have been recorded, the average reduction has been 65% across the Mark "0" screen. However, in 8 of the 17 measurements, increases in BOD are shown across the Microstrainer.

Several postulations have been made for the observed increases:

1. Natural predators for bacteria are largely removed by Microstraining and are thus not present in large number on the discharge side.

2. Large colonies of bacteria are subdivided into larger numbers by passage through the screen.

3. The bacterial food supply is made more available -- more surface

area is produced on the escaping solids -- by the screening process and growth kinetics are enhanced. This is perhaps reflected in the observed BOD increases.

We lean toward the last-named explanation, and believe that such effects of the Microstraining process can be desirable in the treatment of storm water:

1. Smaller particles will have a lesser tendency to occlude bacteria. They are thus more vulnerable to attack from ozone or chlorine.

2. If the BOD peaks largely over an early, shorter time period, downstream effects are likely to be less persistant.

This work has consisted of the design, installation, operation and evaluation of Microstraining equipment, and of ozonation and chlorination at a combined sewer overflow judged to be typical. At the time of this report, operation has not been concluded and complete evaluations have not been made. However, enough information is available to permit preliminary conclusions regarding Microstrainer operation on this type of combined sewer overflow.

Initially, the Microstrainer was fitted with a Mark "I" screen. Results indicating effluents of intermediate quality were obtained from 9 rainfalls of utilizable intensity and duration over a 6-month period.

A finer, Mark "0" screen, was then fitted to the Microstrainer. Effluent qualities, with respect to removals of total and volatile suspended solids increased measurably.

After an additional 8 useable rainfalls, the Microstrainer controls were altered so as to produce a pre-established constant differential head across the Microstrainer screen. Results from 6 sets of samples during 3 different rainfalls indicated still further improvement in suspended solids removals.

Finally, the differential head has been increased well above the normal level noted above by blanking off much of the screen area designed into the Microstrainer model employed. This reduction in screen area amounts to about 80% of that available for filtering and has resulted in high quality results in terms of suspended solids removals. These results are most significant because they point the way to higher hydraulic loadings with attendant lower capital costs.

These last tests, although few in number, indicate that the Microstrainer Mark "0" screen in this service is superior to the Mark "I" coarser screen, and that the Mark "0" performs well at a higher hydraulic loading. No evidence of screen pluggage has been observed at any time.

"Pretreatment" of the Microstrainer influent by means of a heavy solids trap and a bar screen are recommended for full scale installations. Early approximate estimates for installed capital costs for such an installation, and based on our 11 acre drainage area are:

1.	Bar Screening and Microstraining	\$ 9,500 - \$11,800 per acre
2.	Bar Screening, Microstraining plus Chlorination @ 5-20 ppm	\$10,500 - \$12,800 per acre
3.	Bar Screening, Microstraining plus Ozonation @ 5 ppm	\$18,000 - \$21,300 per acre

We hope to do further work to further define Microstrainer performance at higher, heretofore unexplored ratings, and to optimize chlorine and ozone requirements.

EXPERIMENTAL
EQUIPMENT & TEST SITE

Microstrainer

The test system incorporates Microstraining for the removal of suspended solids and associated impurities, followed by ozonation and/or chlorination for disinfection. The Microstrainer comprises a 5 ft diameter by 3 ft long drum, fitted on the periphery with a specially woven wire fabric of stainless steel, having microscopic apertures. In this work, two different types of screen have been employed, the Mark "I" (nominal aperture 35 microns) and the Mark "0" (23 microns). In operation, the drum is submerged in the flowing water to approximately two-thirds of its depth. Raw water enters through the upstream end of the drum and flows radially outwards through the microfabric, leaving suspended solids deposited on the inside of the mesh. The drum rotates continuously, at variable speeds, carrying the dirty fabric out of the water and under backwashing jets mounted across the top of the drum.

Intercepted solids are flushed into a receiving hopper fitted inside the drum, with its lip above the top water level. In a full scale project, these solids would be returned to the interceptor sewer for disposal to the nearest sewage treatment plant.

Microstrainers of this type have been employed since 1945 for the filtration of municipal and industrial water supplies, and more recently for "tertiary" treatment of sewage effluents[7]

Chemical Equipment

After water passes through the Microstrainer, it is collected in a 1,200 gallon storage tank, before ozonation or chlorination. Ozone, is generated in an Otto* Plate Type (Model 3-63) Ozonizer. This ozonizer has 15 plate type elements and is rated at 300 grams of ozone/hour at a concentration of 20 grams/cubic meter of air at a maximum power load of 7 kw. Supply to the high voltage electrodes is variable over a 7,000 - 15,000 volt range. The maximum cooling water requirement is 11 gal/min at 15 foot head. Air drying equipment including a refrigerator and dessicator, and electrical control panels are also provided. The air is supplied by a 1/2 hp blower and is filtered. It is cooled to 2-5^{o} C prior to dessication in silica gel columns to a dew point of -40^{o} C. The concentration of ozone in air and the amount of ozone introduced into the water can be varied by adjusting the air flow and the voltage of ozone production.

*Supplied by La Compagnie des Eaux et de l'Ozone (CEO) of Paris, France.

In the CEO Otto system that we are using (Figure 1), hydraulic injectors are used to mix ozonized air with the water to be treated in two contact columns. The water is pumped to a first injector, where it mixes with residual ozone and air from the second contact column. Both water and ozonized air travel down through a centrally located ($1\frac{1}{2}$ in D) pipe in the deep (17' L x 12" D) column, in which the water level is regulated at about 16 ft in depth, exiting at the bottom of this pipe and passing upward through the first column. Air and unused ozone exhaust at the top of this column. The effluent water from the first column is pumped to a second injector for absorption of the initially generated ozone, and this gas mixture passes through a second identical contact column. The finally treated water is discharged to an inspection tank and then to the surface stream.

It can be seen that this system can be described as a combination co-current -- counter-current contactor.

In an actual plant, where operation is intermittent, it would seem desirable from a capital cost standpoint to use an oxygen, rather than an air, supply to the ozonizer. Using oxygen, the concentration of ozone in the ozonizer effluent gas is twice that with air. Thus any ozone generator will produce twice as much ozone from oxygen as from air.

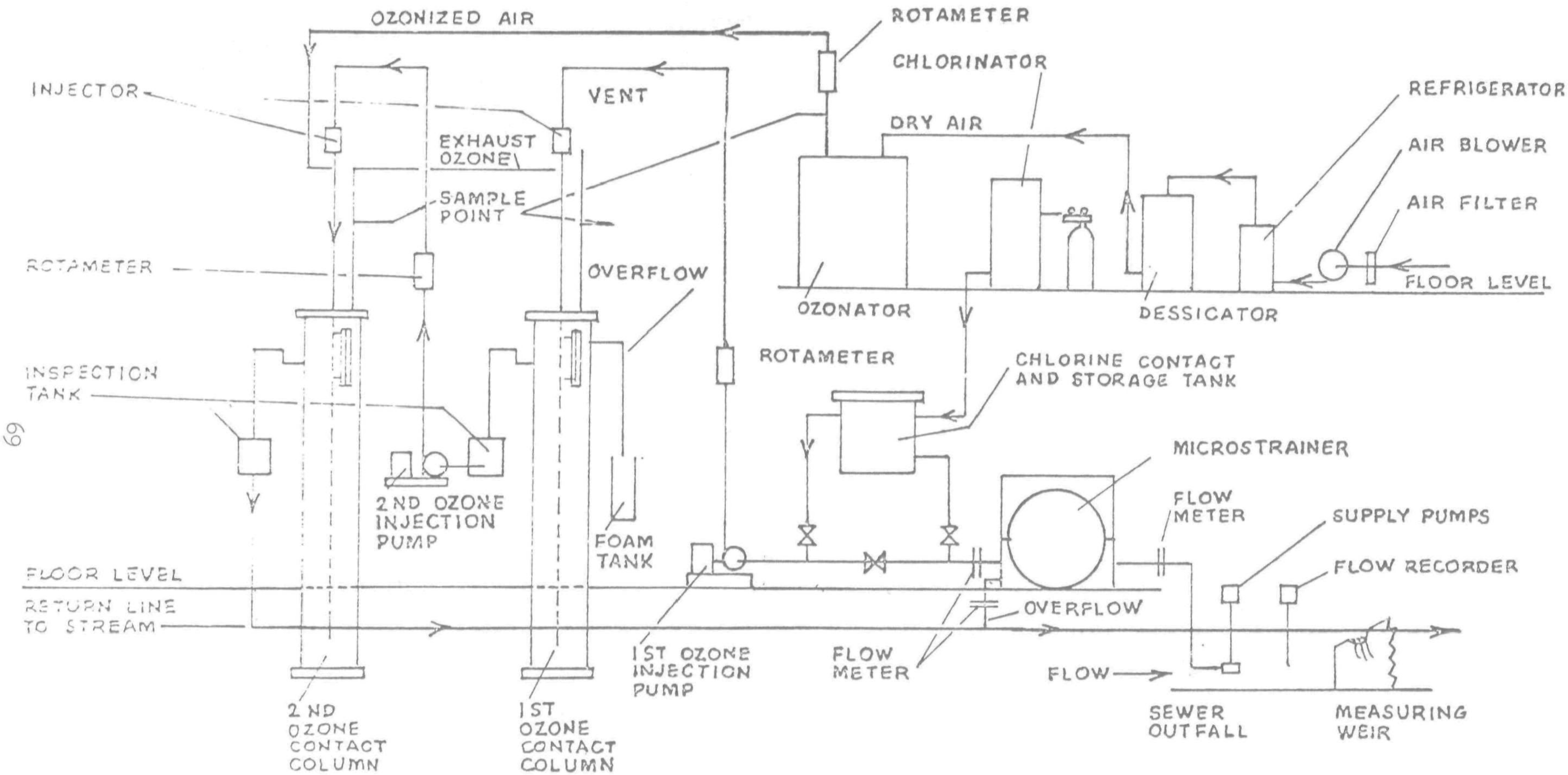
We suggest that oxygen would be used on a once-through basis, with no oxygen recycle.

Chlorination equipment supplied for the plant consisted of a gaseous addition system**. Originally, water was treated by means of this system, and attempts were made to retain the chlorinated effluent for varying periods of time. However, the short duration of very many of the useable rainstorms created metering and regulation problems. This, coupled with the need for a supply of water for relatively long periods of time for operation of the ozonator, forced a change in the method of chlorine treatment. Manual addition of a solution of sodium hypochlorite to samples of Microstrainer effluent from the holding tank was adopted. Close control of chlorine addition was then possible and the residual, -- after chosen retention times -- was destroyed by the addition of thiosulfate prior to refrigerated storage while awaiting analysis.

Test Site

The test site is located on the western side of Philadelphia on a sewer outfall which enters a tributary stream of Cobbs Creek, flowing eventually into the Delaware River. The outfall serves an area of

**Wallace and Tiernan



EQUIPMENT INSTALLATION-SCHEMATIC

Figure 1

approximately 11.2 acres, principally dwelling houses with paved roads and sidewalks (Figure 2). Dry weather flow in the sewer averages 1,000 gallons per hour, and at the setting employed during the tests, the interceptor will collect up to four times this flow. In Figure 2, the dotted lines define sub-drainage areas, and the solid lines connecting the small circles (catch basins) are the sewer lines. The outfall is located at elevation 148, about 3 feet below the 150.7" intercepting elevation.

Overflows normally take place when storms in the area exceed a rate of 0.1 in/hr which occurs approximately 40 times a year, mostly during the spring and summer. However, our plant is such that it is usually not properly activated unless the rate reaches 0.2-0.3 in/hr for about 1 hour. The rate of flow into the outfall can reach as much as one million gallons per hour during an intense storm of six inches per hour, which is attained on average once every five years.

The sewer outfall was modified to incorporate a collection sump (Figure 3) from which the storm water runoff is pumped into the test installation. Rate of flow from the outfall is measured by means of a weir fixed at the sump outlet, and is continuously recorded. A baffle wall was constructed in front of the weir to prevent surges of water upsetting the measurement of flow rate.

Two Microstrainer supply pumps are installed between the baffle wall and the measuring weir one having a maximum flow capacity of 12,000 gph, the other having 5,000 gph capacity. These pumps have been used together and separately so as to supply water at rates of 5,000 and 17,000 gph, with some intermediate and lower rates, depending on the supply heads available. Intakes to the pumps are protected by a screen, having 1/2" square openings.

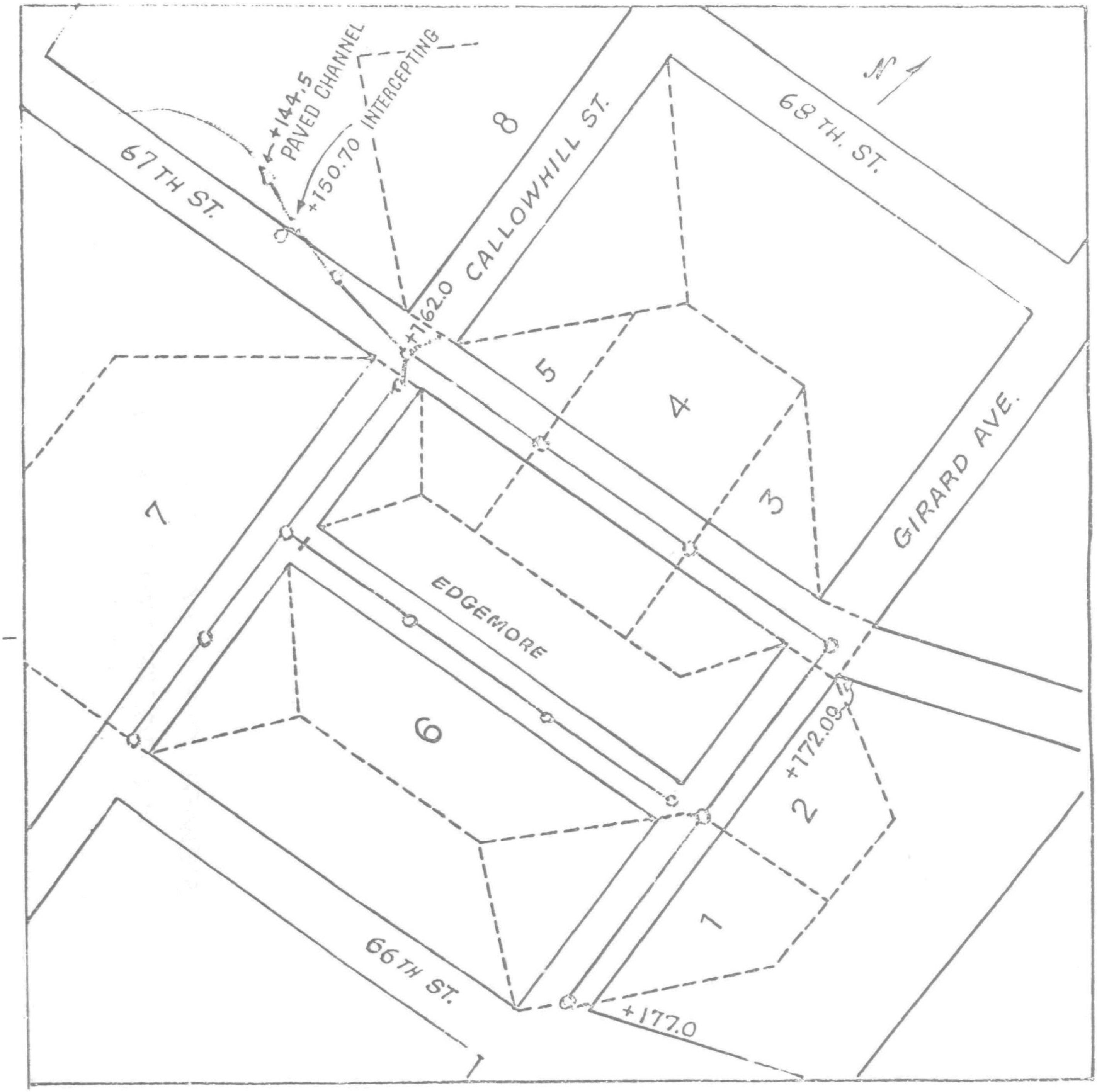
Operation

As water enters the collection sump, the level rises starting the pump(s) and initiating a timer connected to the sampling devices inside the test installation.

The rate of flow pumped into the Microstrainer is recorded continuously. The pumped flow from the sump, together with the recorded overflow yield an indication of the total storm flow.

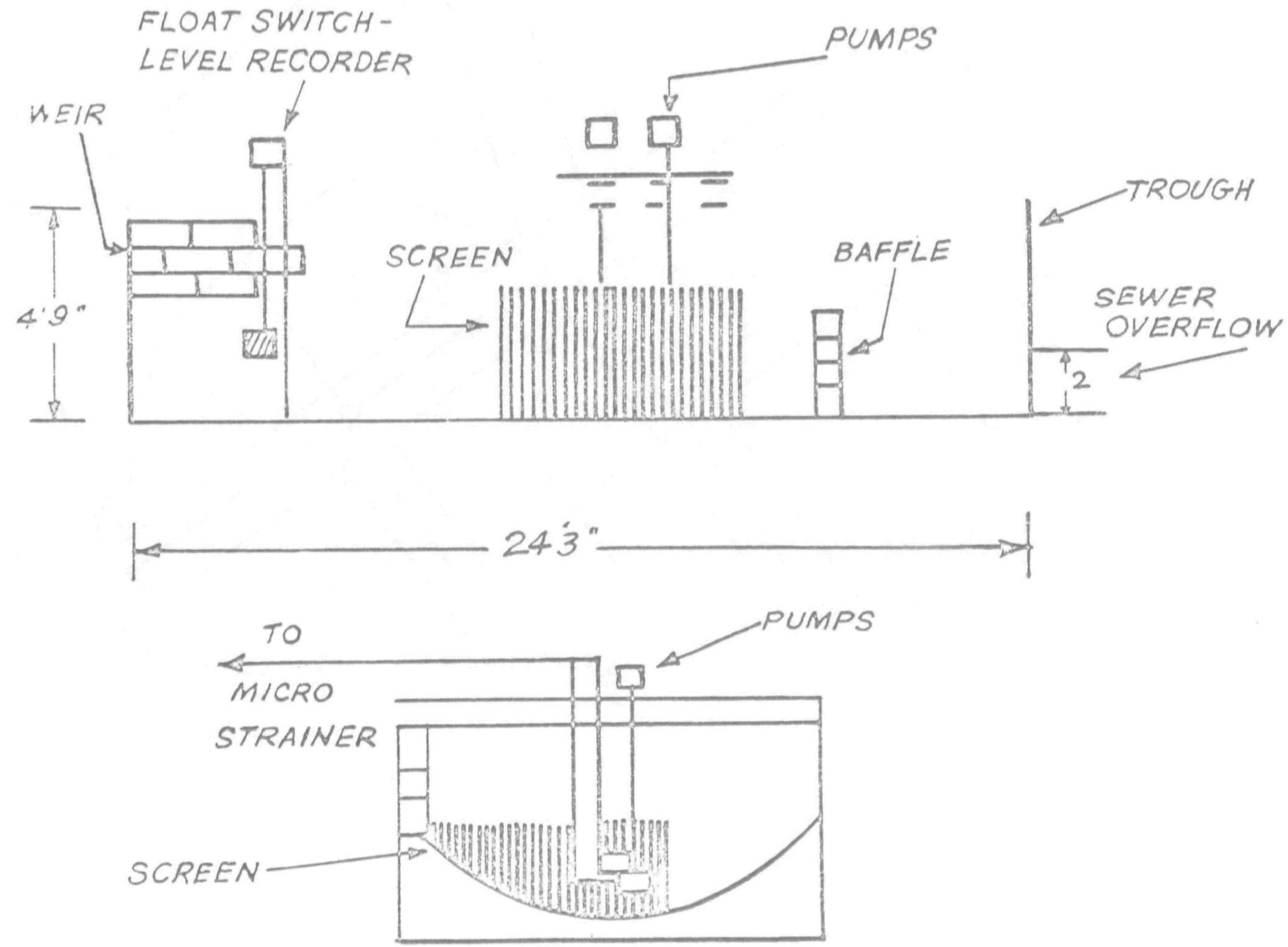
As water enters the Microstrainer drum, head loss through the fabric increases causing the drum speed to increase by means of an automatic control system. Water for backwashing is drawn from the downstream side of the strainer by means of a small pump, kept supplied during dry periods from a storage tank containing city water. The Microstrainer thus commences its filtering action at the beginning of a storm, passing

Figure 2



OUTFALL

67TH & CALLOWHILL



SEWER OVERFLOW TROUGH—
MICROSTRAINER SUPPLY
Figure 3

strained water into the collection tank. Water not stored for further tests is bypassed and returns to the stream.

Sampling

Composite samples of the raw and strained water are extracted automatically during a storm and stored in refrigerated containers from where they are collected and tested by the Philadelphia Water Department. Ozonation and chlorination are carried out as soon as possible, and further samples are taken before and after treatment.

The Philadelphia Water Department performs the laboratory analysis of samples, maintains the recording rain gauge and cleans the outfall sewer after each overflow.

RAINFALL AND RUNOFF

Rainfall Intensity-Return Frequency curves for the City of Philadelphia are approximated in Figure 4, as furnished by the City Water Department.

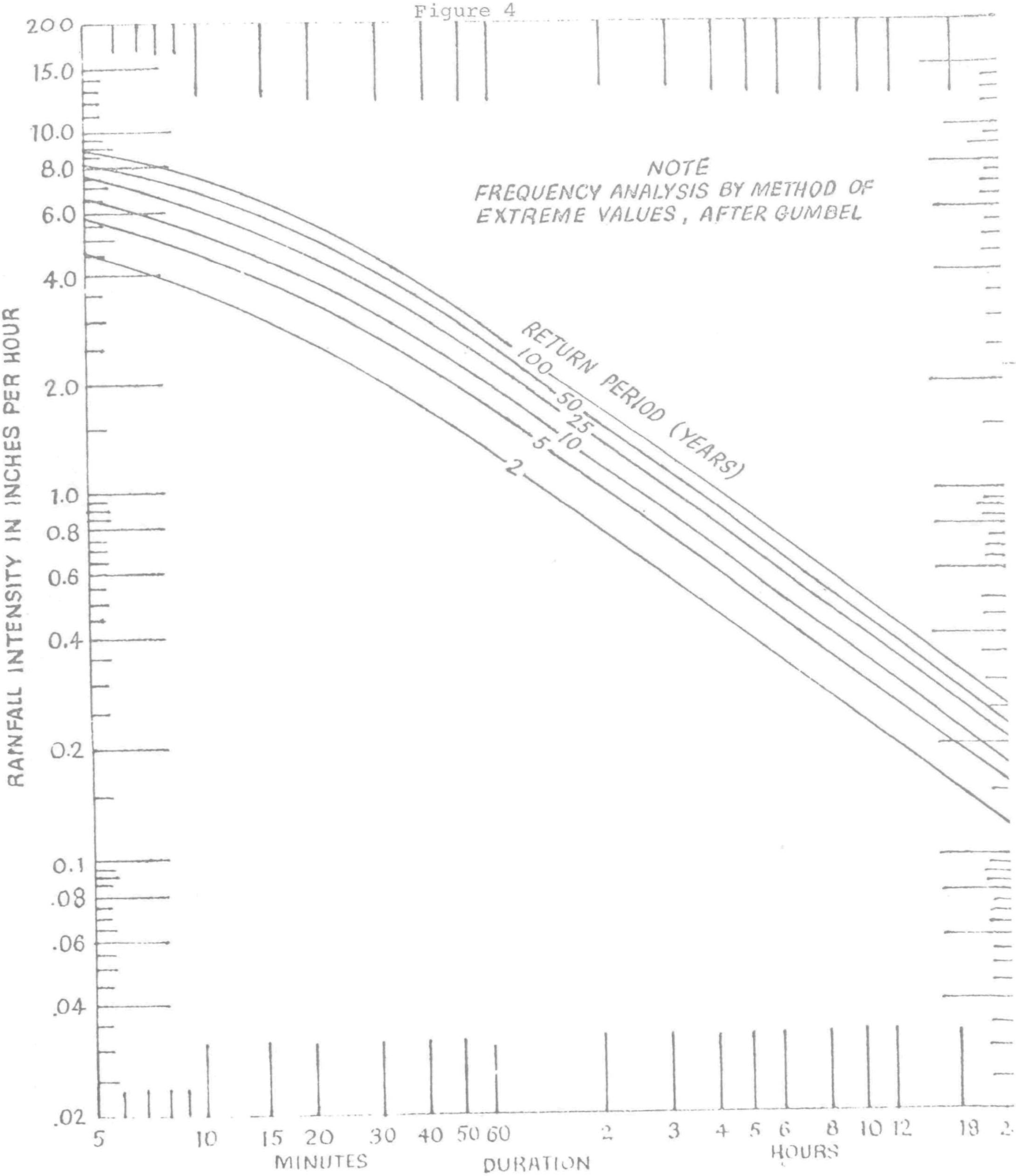
Figure 5 shows the rainfall intensities and durations that we have measured during the course of the testing from a rain gage located about 100 yds from the test site, along with calculated corresponding runoff coefficients. Total flows are measured by a rectangular weir mounted at the discharge of the combined sewer trough and the metered quantities that are pumped to the Microstrainer. The sanitary flow is subtracted, according to the corresponding flow expected during the same period, along with the pre-calibrated portion that flows into the interceptor through a "drop" weir preceding the outfall. In relation to our higher total flows, this constitutes a minor correction, since the drop weir is set to accept only 4 x the average dry weather flow (1,000 gallons/hour).

It can be seen that the highest rainfall recorded during our work has been about 3.3 in/hr for 10 minutes but that the highest runoff coefficients (0.8) do not coincide with these periods. We also have shown, for some points in Figure 5, the intervals in days since the previous rainfall. These figures do not appear to be adequate for interpretation of the differences in the runoff coefficients.

The highest flow at which the combined sewer discharged into its receiving sump was thus approximately 300,000 gal per hour for about .08 hours. The corresponding runoff coefficient was calculated at 0.5. For the 11 acre area involved, which has an imperviousness factor of 61%, this runoff amounts to 450 gal per min per acre.

The lowest runoff figures recorded for which operating test data were acquired, were approximately 10,000 gal per hour.

Figure 4

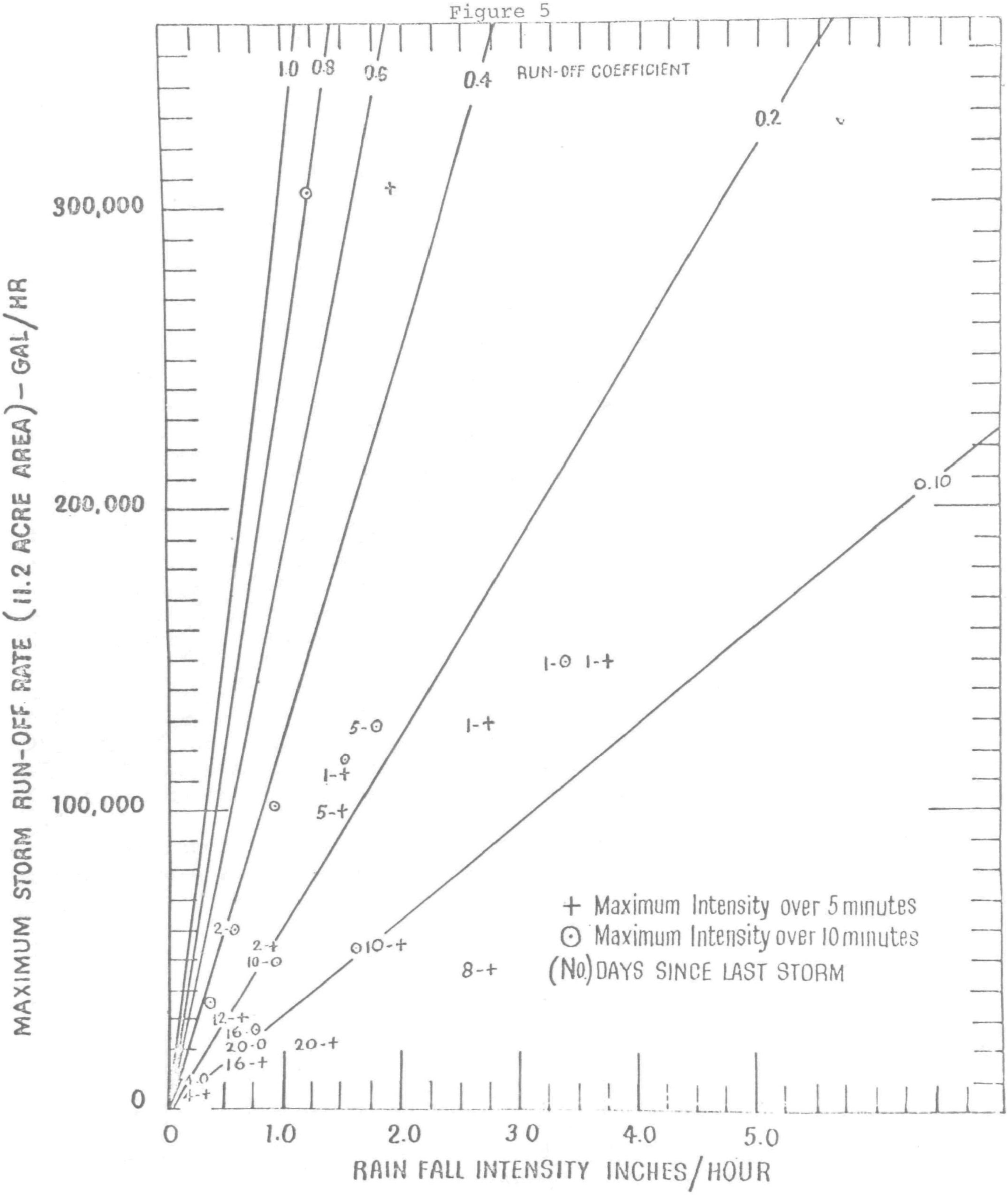


RAINFALL INTENSITY

PHILADELPHIA, PENNSYLVANIA

1903 - 1951

Figure 5



MAXIMUM STORM RUN-OFF
VS
RAINFALL INTENSITY

As expected, our data show that the quality of the overflow tends to change with both the quantity and the duration of the rainfall. For example, in Table 1, for the storm of 7/23/69, it is seen that the suspended solids concentration of the Microstrainer influent was 55 ppm during the early storm period, increasing to 97 ppm at a second later sampling, and then falling to a lower 21 ppm nearer the end of the last period. The same phenomena are shown for the storm of 7/28/69. Figure 6, which combines elements of both time and rainfall intensity, illustrates the relationship between overflow flow rate and suspended solids over a larger number of storms for which both flow rate and suspended solids data are available. From this limited information there is a direct relationship. These data were accumulated over relatively short periods, and it would seem that, with high intensities for longer periods, this relationship will not hold. Unfortunately, data for varying flow vs individual suspended solids information within storm periods are not available.

Fecal coliform results are generally higher at the beginning or toward the middle of a storm and lower at the conclusion (Table 2). BOD results tend to follow the same course that of the total suspended solids for 7/28/69 and 9/3/69, as do volatile suspended solids (Table 3).

SOLIDS AND BOD, MICROSTRAINER INFLUENT, EFFLUENT

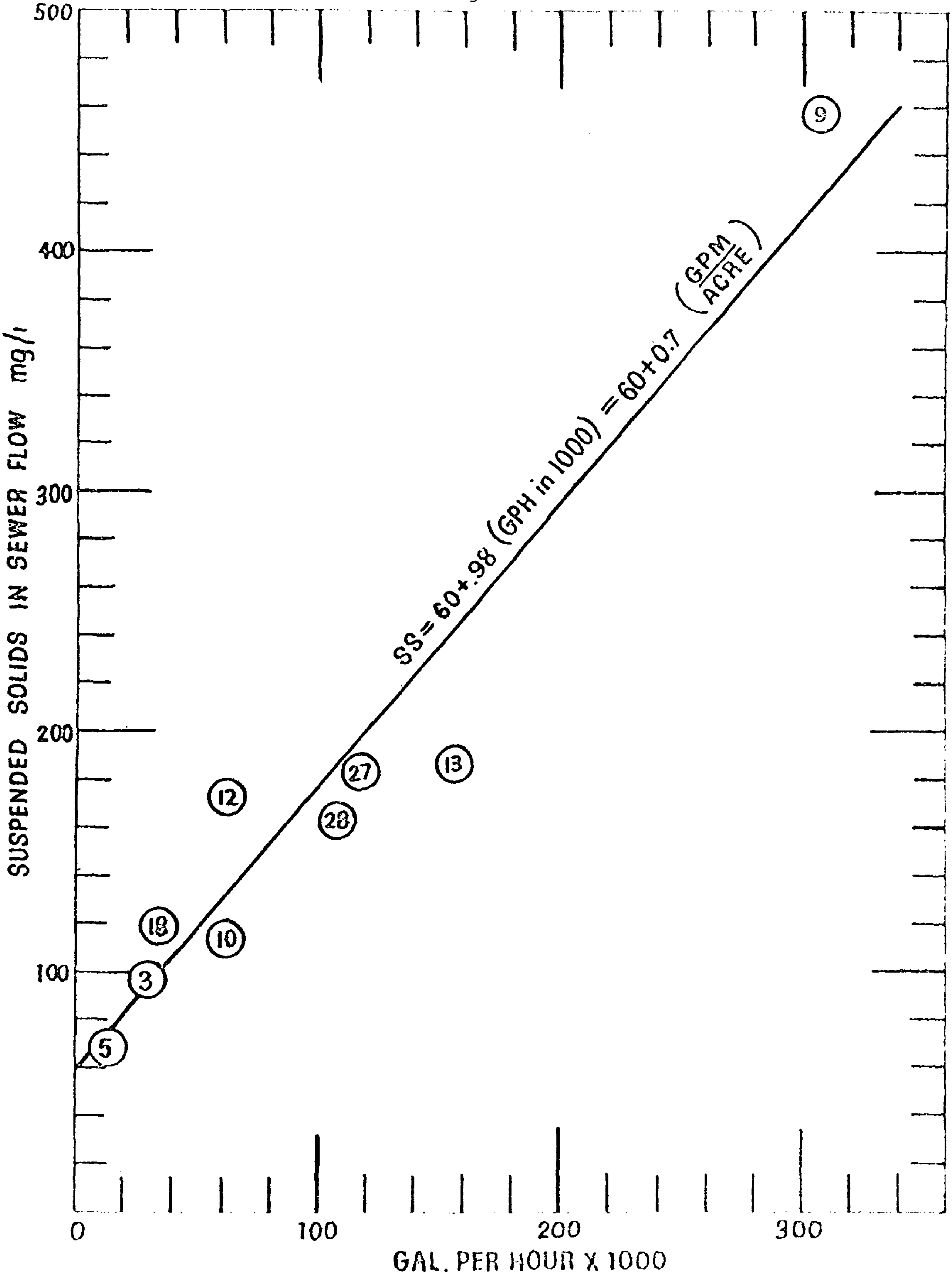
Date	Suspended Solids, mg/l In	Suspended Solids, mg/l Out	Suspended Solids, mg/l % Reduction	BOD, mg/l In	BOD, mg/l Out	BOD, mg/l % Reduction
MARK "I" SCREEN						
12-3-68	104	57	45	21	18	14
1-23-69	71	62	13	17	782	Incr.
4-11-69	202	90	55	36	23	36
4-18-69	223	150	33	29	21	28
4-19-69	457	251	45	27	20	26
4-21-69	115	71	38	40	12	70
5-9-69	108	44	59	39	18	54
5-19-69	173	89	49	43	26	53
5-20-69	372	139	63	44	252	Incr.
AVG.	203	106	44	33	130	-
MARK "0" SCREEN						
6-15-69	107	71	34	20	9	55
6-18-69	103	17	84	112	38	66
6-23-69	159	48	70	11	15	Incr.
6-25-69	157	24	85	38	6	84
7-7-69	118	49	58	30	48	Incr.
7-23-69	55	29	47	41	3	93
7-23-69	97	43	56	135	7	95
7-23-69	21	17	19	5	4	20
AVG.	102	37	57	49	16	-

TABLE 1 (continued)

SUSPENDED SOLIDS AND BOD, MICROSTRAINER INFLUENT, EFFLUENT

	Suspended Solids, mg/l			BOD, mg/l		
Date	In	Out	% Reduction	In	Out	% Reduction
MARK "0" SCREEN Control change, Max. differential increased.						
7-28-69	175	66	62	8	6	25
7-28-69	498	55	89	385	76	80
7-28-69	288	72	75	13	210	Incr.
7-29-69	139	50	64	14	16	Incr.
7-29-69	189	17	91	260	370	Incr.
8-4-69	163	6	96	438	584	Incr.
AVG.	242	44	80	186	211	-
MARK "0" SCREEN Filter Area Reduced						
9-3-69	111	2	98	135	740	Incr.
9-3-69	419	17	96	740	208	72
9-3-69	69	15	78	13	45	Incr.
AVG.	200	11	91	296	331	-

Figure 6



INFLUENCE OF RUN-OFF RATE ON SUSPENDED SOLIDS

TABLE 2

FECAL COLIFORM[1]

Date	In	Out	After Chlorination 5 ppm-5 min	After Chlorination 5 ppm-10 min	After Ozonation 5 ppm[2]	Residual O_3, ppm
MARK "I" SCREEN						
12-3-68	330	655	-	-	-	-
1-23-69	1,300	900	-	-	-	-
4-11-69	510	670	5	-	-	-
4-18-69	1,610	1,630	-	-	-	-
4-19-69	1,460	1,940	-	-	-	-
4-21-69	690	5,700	-	-	-	-
5-9-69	9,000	8,800	100	670	140	-
			-	-	110	-
5-19-69	28,000	28,000	-	-	-	-
	1,500	2,500	-	-	33	1.9
5-20-69	3,000	4,800	-	-	0.2	1.3
	2,100	3,000	-	-	0.3	-
MARK "0" SCREEN						
6-15-69	13,400	6,200	77,000[3]	58	32,000[3]	1.6
			-	-	23,000[3]	-
6-18-69	4,400	2,600	77	0	44	1.9
			-	-	23	-
6-23-69	100	2,800	63	22	200	0.6
			-	-	57	-
6-25-69	6,700	590	91	8.4	5,700	1.3
			-	-	2,800	-
7-7-69	27,000	30,000	-	-	-	-
		11,000	0	1	6.8	-
			-	-	4.3	-
7-23-69	2,200	2,700	-	-	-	-
	3,700	810	-	-	-	-
	2,800	1,100	0.1	25	0.6	0

(1) Per 100 ml x 1,000

(2) Nominal Feed Rate

(3) Values Not Used in Calculation of Averages

TABLE 2 (continued)

FECAL COLIFORM [1]

Date	In	Out	After Chlorination 5 ppm-5min	After Chlorination 5 ppm-10 min	After Ozonation 5 ppm [2]	Residual O_3, ppm
MARK "0" SCREEN Controls changed to produce fixed relation between differential and drum speed. Max. differential increased.						
7-28-69	240	190	-	-	-	-
	120	0	-	-	-	-
		11	0.5	5.4	0.6	0.3
			-	-	1.8	-
		0	76	25	7	-
			-	-	2.6	-
	25	31	-	-	17	-
			-	-	19	-
7-29-69	5	90	-	-	-	-
	110	120	-	-	-	-
8-4-69	200	18	0	0	2.3	0.6
			-	-	25	-
SCREEN FILTERING AREA REDUCED						
9-3-69	5,200	3,900	-	-	-	-
	7,300	6,000	-	-	-	-
	2,600	3,800	-	-	-	-

(1) Per 100 ml x 1,000

(2) Nominal Feed Rate

TABLE 3

VOLATILE SUSPENDED SOLIDS, MICROSTRAINER

Date	mg/l In	Out	% Reduction
12-3-69	60	60	0
1-23-69	33	27	18
4-11-69	41	21	49
4-18-69	63	38	40
4-19-69	111	52	53
4-21-69	44	22	50
5-9-69	51	21	59
5-9-69	69	27	61
5-19-69	79	38	52
5-19-69	42	20	52
5-20-69	90	30	67
5-20-69	42	17	60
AVG.	60	31	47

MARK "0" SCREEN

Date	In	Out	% Reduction
6-15-69	34	12	65
6-18-69	35	4	89
6-23-69	31	12	61
6-25-69	81	8	90
7-7-69	53	28	47
7-23-69	21	7	67
7-23-69	39	13	67
7-23-69	9	4	56
AVG.	38	11	68

TABLE 3 (continued)

VOLATILE SUSPENDED SOLIDS, MICROSTRAINER

Date	mg/l In	Out	% Reduction
MARK "0" SCREEN - Controls Changed			
7-28-69	37	9	76
7-28-69	63	13	79
7-28-69	48	22	46
7-29-69	44	19	57
7-29-69	38	9	76
8-4-69	54	3	94
AVG.	47	12	71
MARK "0" SCREEN - Area Reduced			
9-3-69	21	9	57
9-3-69	42	7	83
9-3-69	18	5	72
AVG.	27	7	71

MICROSTRAINING RESULTS

Total Suspended Solids

As initially started up, the Microstrainer was fitted with the Mark "I" screen (nominal aperture size - 35 microns). As work progressed, it became evident that the backwash jets, in conjunction with slime prevention by means of the ultra violet light employed, would prevent pluggage and fouling of the screen, and that the influents that were received could be more than adequately handled. Accordingly, after about 6 months of operation, the finer Mark "0" screen was installed to determine if increased quality of the effluent could be realized without pluggage.

Furthermore, after an additional approximately 2 months of operation, the Microstrainer controls were altered so that it would operate at a drum speed more closely related to differential head. And, finally, 80% of the filter screen area was blanked off by inserting plastic film inside of the screen.

At the same time the backwash jets normally serving the blanked off area were turned off.

These last-named steps were taken to increase the Microstrainer hydraulic rating -- important because of possible reduction in capital cost of a full scale installation -- and to determine the effects of this increase on the quality of the effluent.

As can be seen in Table 1, and Figures 7, 8, 9, the removals of total suspended solids ranged from 13% to 98%, the higher values being characteristic of the Mark "0" screen, and better-regulated drum speeds. Although the data are scattered, regressions are shown for suspended solids in the feed vs % reduction of suspended solids in Figures 7 and 8. These illustrate the improvement in performance gained through the use of the Mark "0" screen, and also show the tendency for increased suspended solids removal efficiency with an increase in the influent suspended solids concentration. Over the range of data that we acquired, straight line regressions offered the best fit.

The results for removal of volatile suspended solids are shown in Table 3 and Figure 10. These results parallel those for total suspended solids, ranging to an average value of 71% removal for the Mark "0" screen and the higher differentials.

BOD

Removals of BOD are scattered (Table 1, Figure 11).

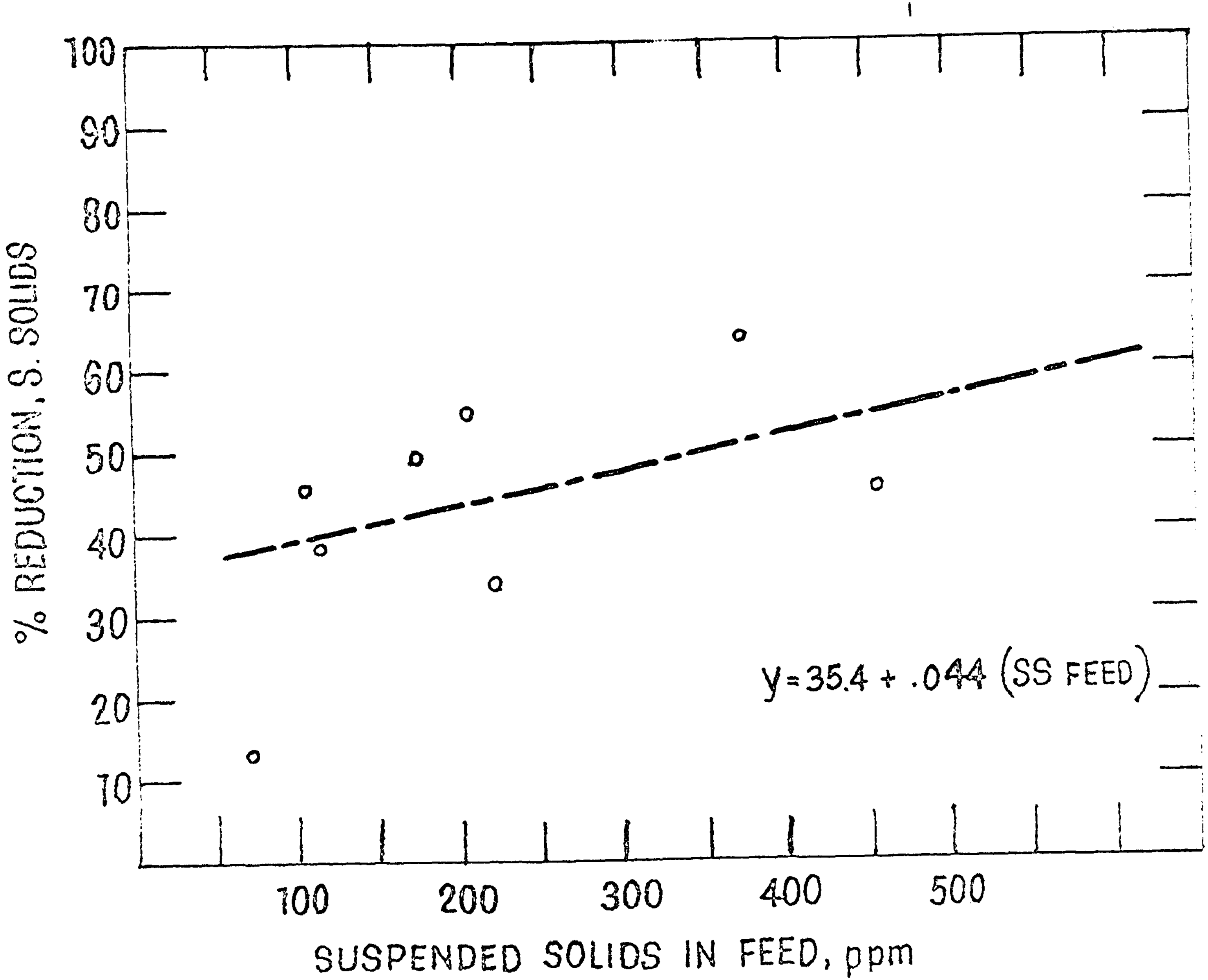
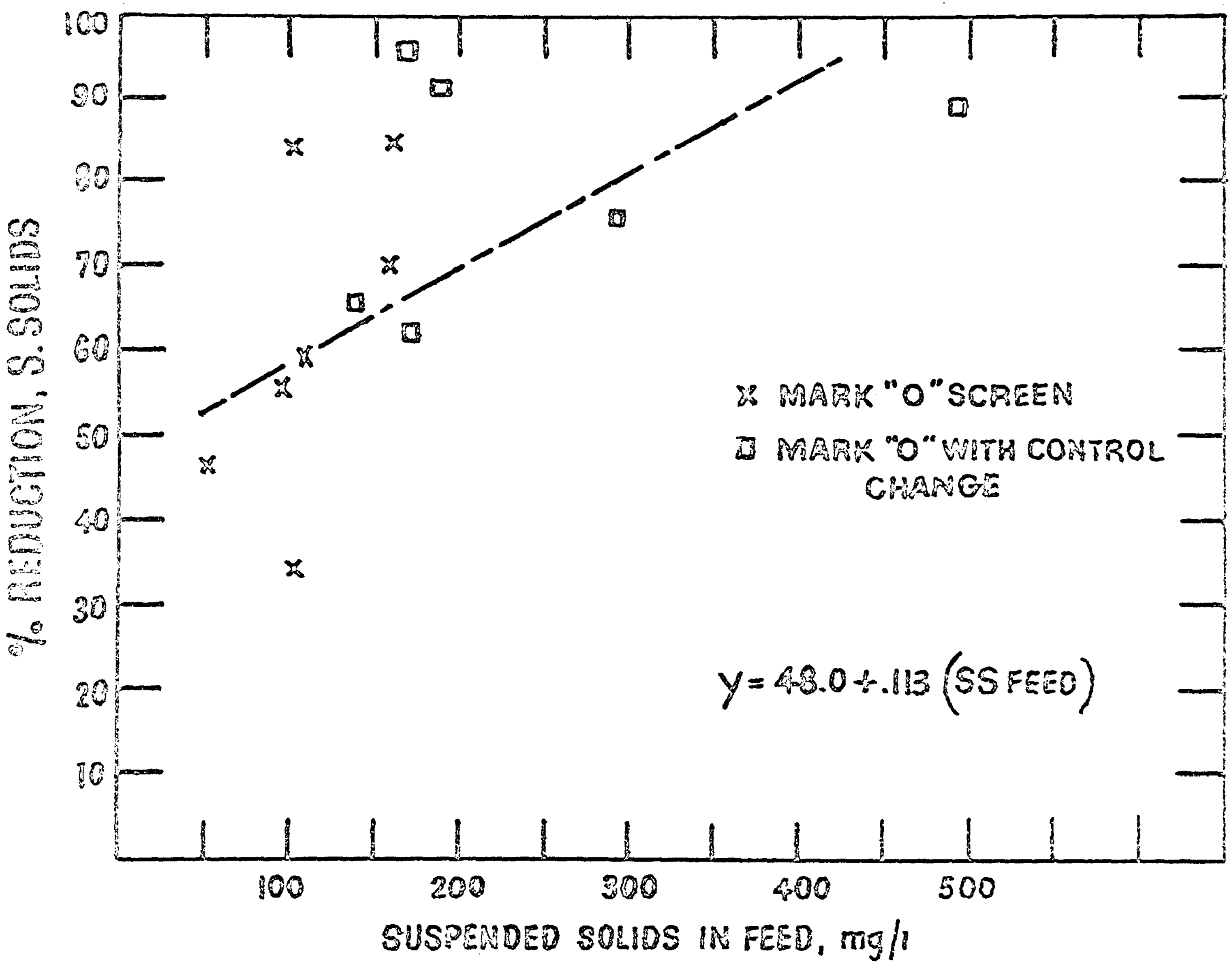


Figure 7

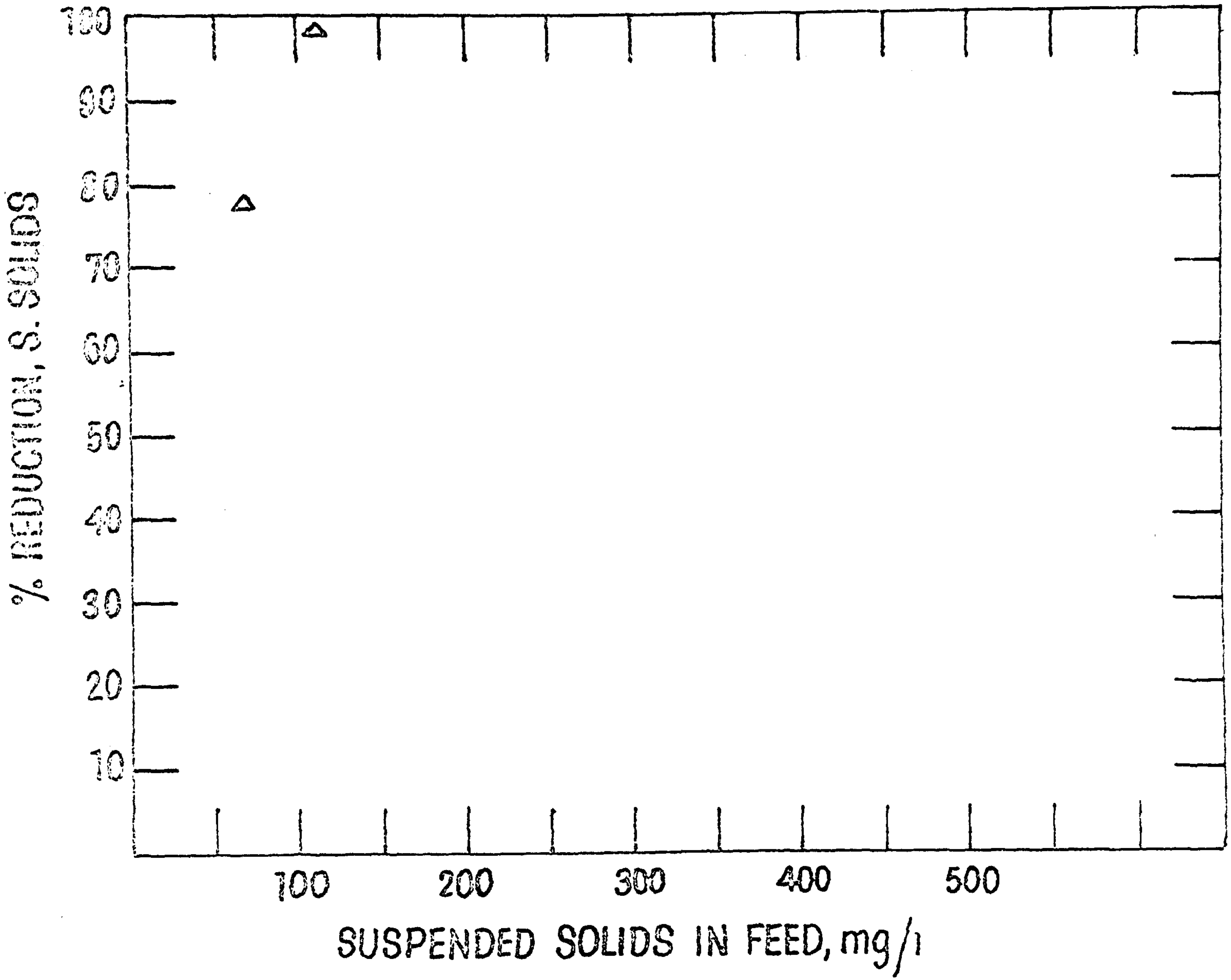
SUSPENDED SOLIDS REDUCTION-
MARK I SCREEN



SUSPENDED SOLIDS REDUCTION - MARK 'O' SCREEN

Figure 8

Figure 9



SUSPENDED SOLIDS REDUCTION
MARK 'O' SCREEN. REDUCED AREA

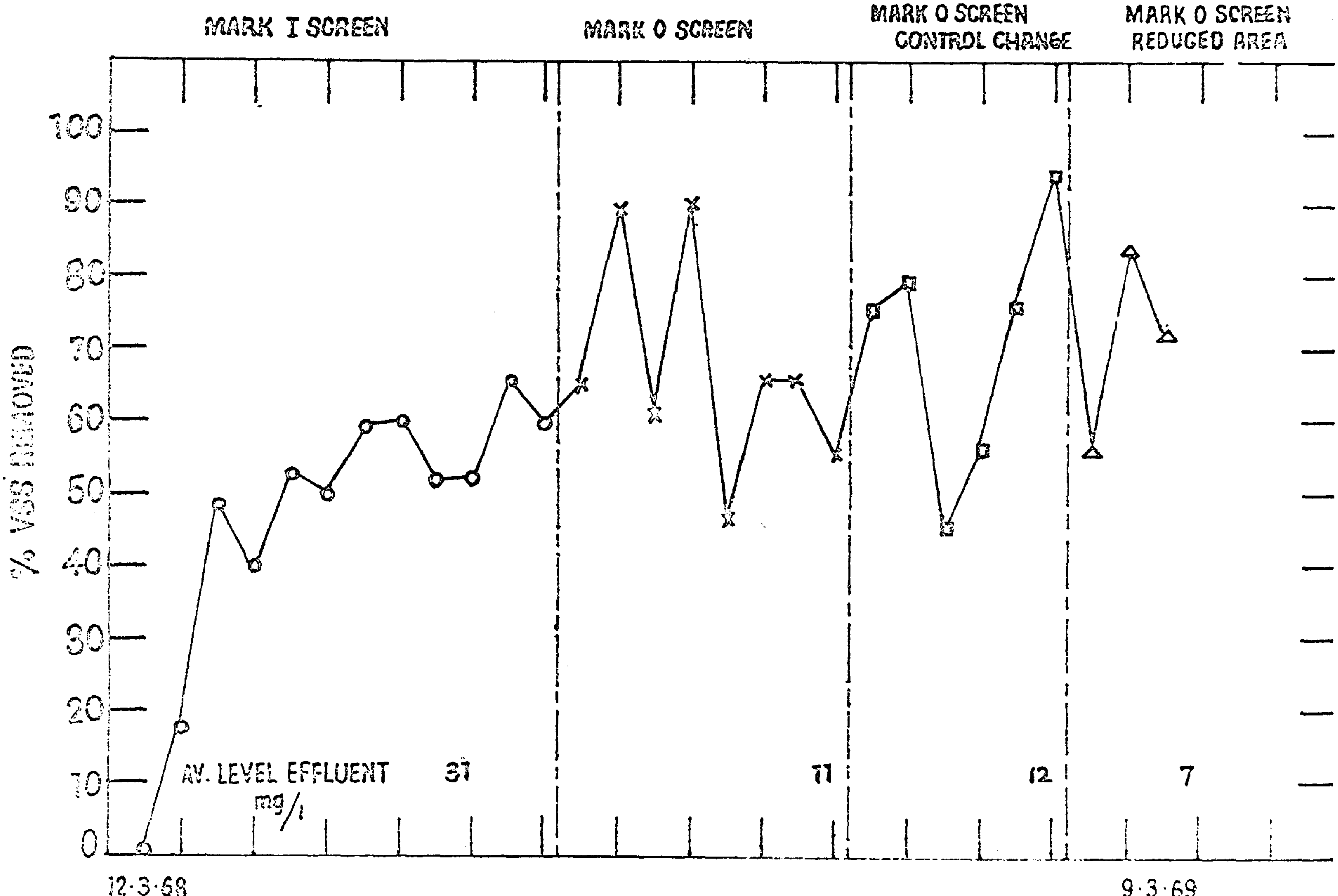


Figure 10

VOLATILE SUSPENDED SOLIDS REMOVAL-MICROSTRAINER

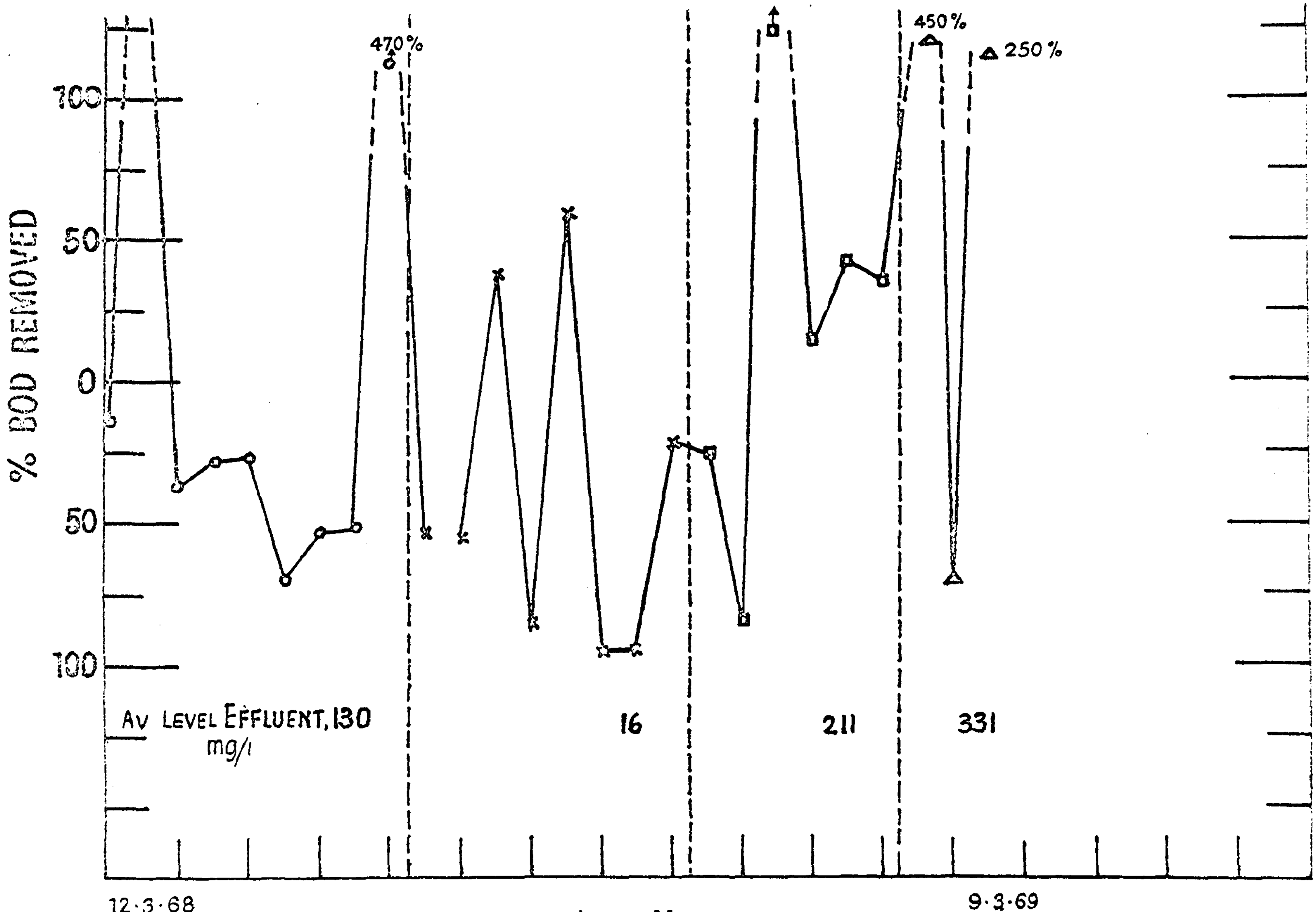


Figure 11

BOD REMOVALS - MICROSTRAINER

We postulate that the volatile suspended solids that pass the screens are present in the effluent in a much more finely divided form. We further suggest that the resulting increased surface area of these solids may serve as a more rapid and more efficient growth medium for bacteria.

It is thus probable that downstream effects, after Microstraining, will be less persistent -- and particularly so in view of the major reduction in the volatile suspended solids fraction. Moreover, it appears certain that post-treatment with chlorine or ozone, if practiced, should be markedly enhanced, considering the reduction in the number of larger particles that tend to occlude organisms, protecting them from the action of these treatments.

Fecal and Total Coliform

As shown in Tables 2 and 4, both fecal and total coliform bacteria quite frequently exhibit increases in their concentrations in the Microstrained effluent. This phenomenon has previously been noted by Boucher[8]. Clearly, no net "removal" of these organisms can be attributed to the Microstraining process.

Several postulations have been made for these observed increases:

1. Natural predators are largely removed by Microstraining and are thus not present in large numbers on the discharge side.

2. Large colonies of bacteria are subdivided into larger numbers by passing through the screen.

3. The bacterial food supply is made more available by the screening process and the growth kinetics during the 5 day measurement period are enhanced.

As related above, we tend to accept the last-named postulation, but it must be emphasized that the question has not been resolved.

TABLE 4

TOTAL COLIFORM[1]

Date	In	Out	After Chlorination 5 ppm-5min	After Chlorination 5 ppm-10 min	After Ozonation 5 ppm[2]	Residual O_3, ppm
MARK "I" SCREEN						
12-3-68	666	740	-	-	-	-
1-23-69	1,607	1,280	-	-	-	-
4-11-69	720	840	3	6	-	-
4-18-69	2,600	2,970	-	-	-	-
4-19-69	2,310	2,380	-	-	-	-
4-21-69	1,310	9,800	-	-	-	-
5-9-69	10,300	8,500	800	760	330	-
			-	-	580	-
5-19-69	100,000	93,000	-	-	-	-
	8,700	4,000	-	-	33	1.9
5-20-69	2,700	3,600	-	-	0.2	1.3
	5,200	6,700	-	-	0.4	-
MARK "0" SCREEN						
6-15-69	19,900	8,600	98,000[3]	130	60,000[3]	1.6
			-	-	36,000[3]	-
6-18-69	8,600	5,900	290	0	100	1.9
			-	-	120	-
6-23-69	1,200	14,000	240	79	500	0.6
			-	-	220	-
6-25-69	10,000	860	150	18	7,600	1.3
			-	-	3,900	-
7-7-69	28,000	11,000	5.1	100	18	-
			-	-	13	-
7-23-69	1,800	1,100	0.2	110	4.8	0

(1) Per 100 ml x 1,000

(2) Nominal Feed Rate

(3) Values Not Used in Calculation of Averages

TABLE 4 (continued)

TOTAL COLIFORM[1]

Date	In	Out	After Chlorination 5 ppm-5 min	After Chlorination 5 ppm-10 min	After Ozonation 5 ppm[2]	Residual O_3, ppm
MARK "0" SCREEN Controls changed to produce fixed relation between differential and drum speed. Max. differential increased.						
7-28-69	170	0	330	200	30	-
			-	-	8	-
	44	11	0.8	12	0.5	0.3
			-	-	5.4	-
	78	200	-	-	-	-
		31	-	-	100	-
			-	-	23	-
7-29-69	130	330	-	-	-	-
	150	230	-	-	-	-
8-4-69	15	8	0	0	3.8	0.6
			-	-	32	-
SCREEN FILTERING AREA REDUCED						
9-3-69	12,000	16,000	-	-	-	-
	20,000	13,000	-	-	-	-
	12,000	20,000	-	-	-	-

(1) Per 100 ml x 1,000

(2) Nominal Feed Rate

Average total coliform concentrations for the final effluents in all of our work, under varying conditions imposed on the Microstrainer, using 5 ppm of chlorine for 5 and 10 minute retention times, were 166,000/100 ml and 129,000/100 ml, respectively. For fecal coliform concentrations, these values, in the same order, were 41,000 and 81,000. Similar results for ozone at a nominal concentration of 5 ppm and a detention time of about 12 minutes were 619,000 and 42,000. The corresponding total coliform results for the Microstrainer effluent (prior to chemical treatment) ranged from "0" (in one instance) to a high of 93,000,000, and the fecal coliform counts ranged from "0" to 30,000,000.

Ozone, of course, is more desirable should a colorless final effluent be desired, or in those cases where a less stable, less persistent downstream chemical residual is needed.

Higher chemical feeds and/or longer detention times are indicated for a more complete bacterial kill. In this treatment situation it is obvious that the former is more desirable because of the increased cost associated with provision of storage for detention. Whether additional "detention time" would be available in the discharge, downstream of an actual plant of this type, would depend on individual circumstances.

Table 5 gives average values for final effluent coliform concentrations.

Time has permitted the investigation of the use of larger amounts of chemicals with shorter detention times to only a limited degree, but some results with chlorine* indicate the probability of greater bacterial kills with larger amounts of chlorine and shorter detention times. For example: in one test on different portions of the same sample, total coliform were 110,000/100 ml for 10 ppm - 2 minutes and 7,500/100 ml for 15 ppm - 2 minutes.

*These last-acquired results are not listed in any of the Tables.

TABLE 5

TOTAL COLIFORM, FINAL EFFLUENT
AVERAGE VALUES (per 100 ml x 1,000)

CHLORINATION (5 ppm)		OZONATION (5 ppm)
5 min	10 min	
166	129	619

FECAL COLIFORM, FINAL EFFLUENT
AVERAGE VALUES (per 100 ml x 1,000)

CHLORINATION (5 ppm)		OZONATION (5 ppm)
5 min	10 min	
41	81	42

The possible solutions to the combined sewer overflow problem appear to be varied, depending upon individual circumstances[5]. Among these circumstances can be listed such items as the character of the existing collection system, types of receiving waters, population density, rainfall, land use factors (i.e., residential, commercial, industrial) type of catchment area and size of catchment area.

In many cases, it would appear that large areas are not available for the construction of holding basins. And, in some cases, the prospect of retaining large volumes of sewage for the times required for discharge at low rates either to a receiving stream or to the sewer system and a disposal plant, would appear unattractive from both aesthetic and practical standpoints.

Although large detention basins, such as have been mentioned for Columbus, Ohio, and Boston, Mass.[4], will presumably continue to be employed where huge overflow volumes are involved, in instances where large amounts of land are not available, and where ultimate disposal is difficult, or where the local environment is not suitable for detention tank installation, the Microstrainer can be considered.

In this connection, a recent publication[9] points out that 25% of the catchment areas in Washington, D.C., are 25 acres or less in size, and that a similar survey of Milwaukee, Wis., revealed that 50% of these areas are of 25 acres or less. There is no intent to imply that the use of Microstraining should be limited to the smaller areas, but these figures illustrate the number of smaller subdivisions of a drainage basin that might be handled locally.

The cost analysis quoted below illustrates the expenditure that could be expected for a drainage area of the type for which this program was conducted.

Plant design for Microstraining only envisions the treatment of 540x average dry weather flow. Where chlorination (in 2 minutes retention time) would be employed with Microstraining, the design contemplates chlorination of an additional 540x; that is, when the overflow occurred at 540x or below, both Microstraining and chlorination would be used, and when the flow exceeded 540x, the excess up to a total of 1,080x would be only chlorinated. The 540x, at the target plant, is 20 cfs. Flows over 1,080x would bypass the entire plant.

A similar provision for ozonation capacity for the flows between 540x and 1,080x was not included.

It is calculated that, at the 540x figure, about 95% of the overflows occurring in one year would be fully treated[10]

These capital cost estimates include equipment, installation, and engineering costs for Microstraining, chlorination via sodium hypochlorite or ozonation, the last-named operation being carried out with an oxygen rather than an air feed. The use of oxygen -- on a once-through basis -- eliminates the first costs and maintenance changes associated with air preparation or oxygen recycle equipment.

Dollar estimates for plant operation and maintenance have not been made. However, operation costs of Microstrainer-chlorination plants should be low. On a single plant basis, for Microstraining -chlorination, we feel that maintenance labor costs should not exceed 4 hours per week for 2 employees. Where multiple plants were installed, this estimate would be materially reduced.

Where ozonation is used, operation and maintenance costs would be expected to be somewhat higher.

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Plant design and its early operation were carried out by Messrs. E. W. J. Diaper, Manager of Municipal Water and Waste Treatment and Mr. J. D. Reilly, both of the Cochrane Division,Crane Co.

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(10) City of Philadelphia Data.

THE USE OF SCREENING/DISSOLVED-AIR FLOTATION FOR TREATING COMBINED SEWER OVERFLOW

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INTRODUCTION

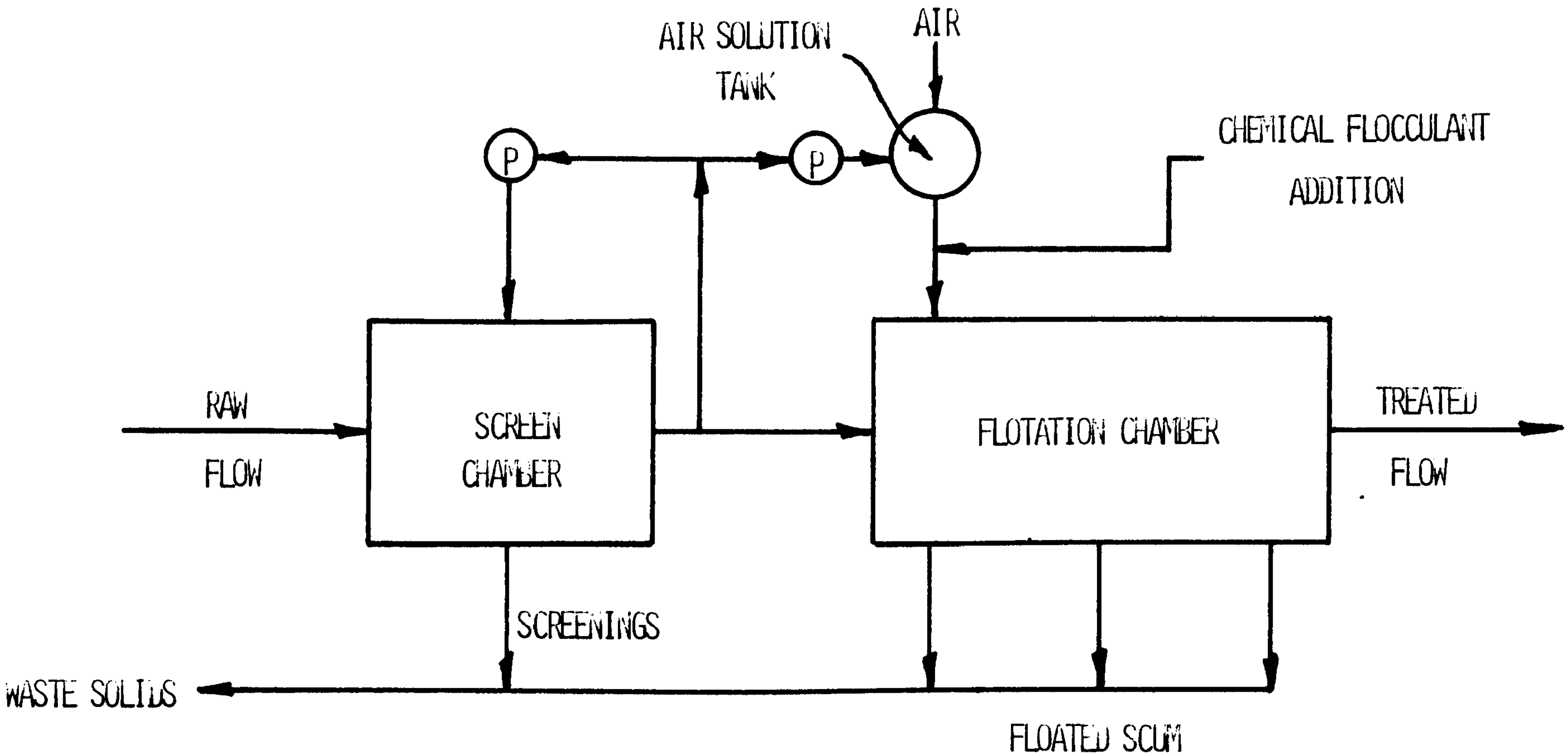
The pollutional characteristics of combined sewer overflow are being documented through the many federally sponsored projects which are now underway. Preliminary results indicate that the majority of the pollutional substances present in combined sewer overflow are in the form of particulate matter. This indicates that a high degree of treatment could be obtained by utilizing an éfficient solids/liquid separation process. The objectives of this project (FWPCA Contract #14-12-40) are to determine the design criteria, effectiveness, and economic feasibility of using screening and dissolved air flotation to treat combined sewer overflows.

The project is currently underway. Completion is expected by late spring or early summer of 1970. The following discussion is a review of the results obtained to date, tentative design criteria, and expected removal rates.

DESIGN OF TEST FACILITY

During the fall and spring of 1967, the Hawley Road Combined Sewer in Milwaukee, Wisconsin was monitored. A total of 12 overflows were sampled. Laboratory scale testing on these samples included screening with various size media, chemical oxidation, flotation, and disinfection. Laboratory analyses on the untreated overflow as well as the effluents from the laboratory bench tests were analyzed for BOD, COD, SS, VSS, and disinfection requirements. It was determined from this testing that chemical oxidation did not appear technically feasible (1). However, encouraging results were obtained from the screening and flotation tests. These tests served as input data in the design of a test facility utilizing screening and dissolved air flotation. A process flow sheet for the system is shown in Figure 1.

The system basically consists of a screen chamber and a flotation chamber. The screen is an open ended drum into which the raw waste flows after passing a 1/2" bar rack. The water passes through the screen media and into a screened water chamber directly below the drum. The drum rotates and carries the removed solids to the spray water cleaning system where they are flushed from the screen. Screened water is used for flushing. The spray water and drum rotation are controlled by liquid level switches set to operate at 6 inches of head loss through the screen. The flotation chamber is a rectangular basin with a surface skimming system to remove floated scum. Screened water is pressurized and mixed along with air in an air solution tank. The liquid becomes saturated with air and when the pressure is reduced, minute air bubbles (less than 100 micron diameter) are formed. This air-charged stream is then mixed with the remaining screened water flow. The bubbles attach to particulate matter and float it to the surface for subsequent removal by the skimmers. Chemical flocculants may be added to enhance the removal efficiency of finely divided particulate matter.


AIR SOLUTION
TANK
AIR
P
P
CHEMICAL FLOCCULANT
ADDITION
RAW
FLOW
SCREEN
CHAMBER
FLOTATION CHAMBER
TREATED
FLOW
SCREENINGS
WASTE SOLIDS
FLOATED SCUM

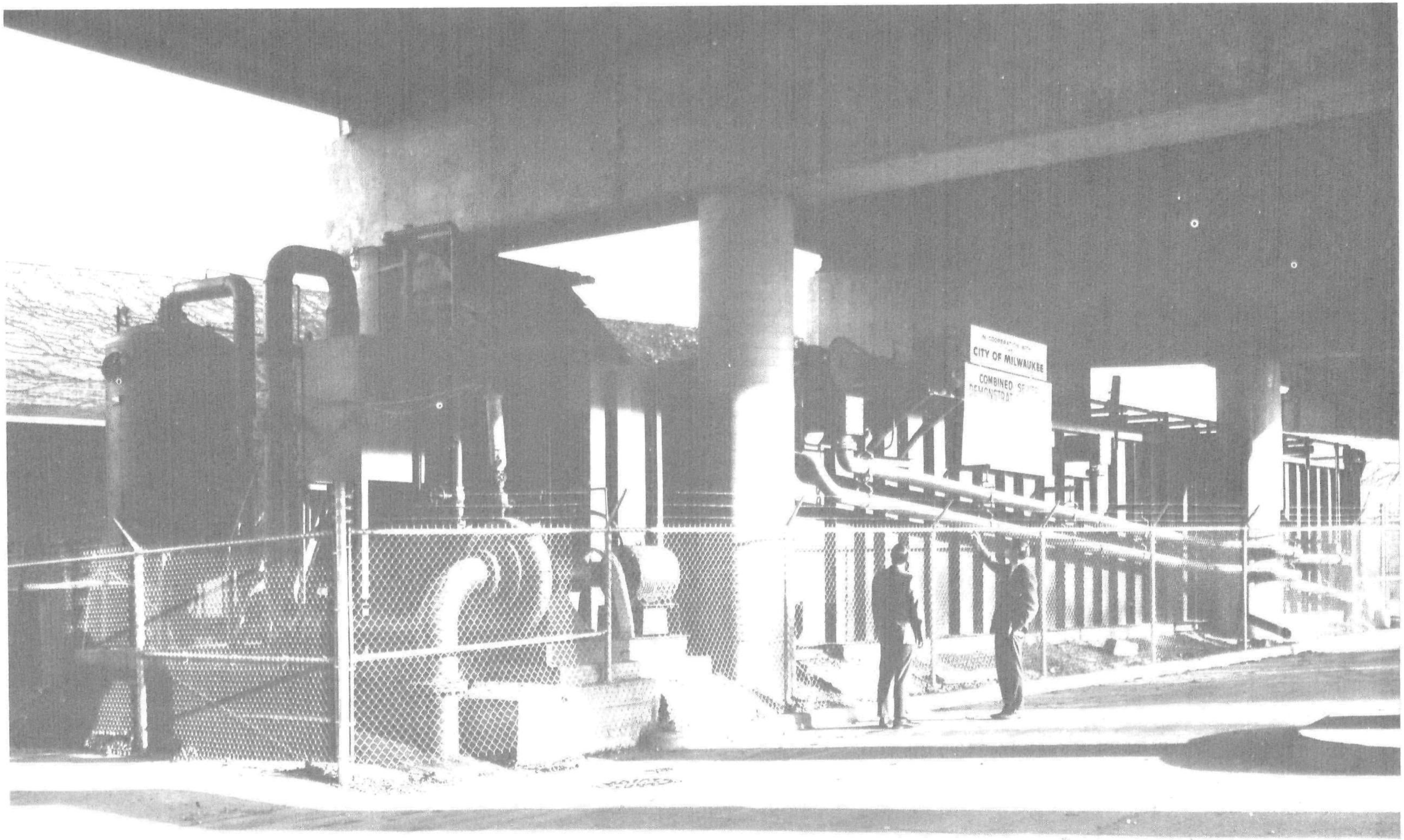


Figure 1a. Screening and Dissolved-Air Flotation Unit

The design criteria utilized in the design of the test facility are shown in Figure 2. These criteria provide the wide flexibility necessary in a test facility. More precise design criteria will be given later. The system was designed to treat 5 MGD of combined overflow.

All pumps and auxiliary equipment were sized on this flow. The flotation tank is compartmentalized to allow variation in the surface loading without changing the raw flow rate. Pressurized flow rate and operating pressures can be maintained over a wide range of values.

RESULTS OF OPERATION

The test facility was completed and put on stream in May of 1969. Since that time, 28 overflows have been monitored. It has been observed that about 25% of these overflows have high pollutional load during the first portion of the overflow. This period of first flushes has never lasted longer than one hour and has been as short as 10-15 minutes. After these flushes pass, the characteristics of the overflow become quite constant. This period has been called the extended overflow period. The range of pollution parameters measured for these 28 storms at the 95% confidence level is shown in Figure 3. It may be observed that the first flushes data has quite a wide range of values, while the extended overflow data has a relatively narrow range. All laboratory analysis were performed according to Standard Methods (2). The

SCREEN

1.	RAW FLOW RATE	3500 GPM
2.	HYDRAULIC LOADING	50 GPM/SQ FT
3.	SCREEN SIZE	50 X 50 297 MICRON OPENINGS
4.	SCREEN WASH	150 GPM MAXIMUM

FLOTATION TANK

1.	FLOW RATE	3500 GPM
2.	SURFACE LOADING	3-9 GPM/SQ FT
3.	HORIZONTAL VELOCITY	3 FPM
4.	PRESSURIZED FLOW RATE	400-1100 GPM
5.	OPERATING PRESSURE	40-70 PSIG
6.	MINIMUM PARTICAL RISE RATE	0.5-1.5 FPM

FIGURE 2

GENERAL DESIGN CRITERIA
FOR DEMONSTRATION SYSTEM

	FIRST FLUSHES
COD	500-765
BOD	170-182
SS.	530-848
VSS	221-495
TOTAL N	17-24

	EXTENDED OVERFLOWS
COD	113-166
BOD	26-53
SS	115-174
VSS	58-87
TOTAL N	3-6

ALL VALUES IN MG/L AT 95% CONFIDENCE LEVEL

COLIFORM 310 X 10^3 TO 1.5 X 10^3 PER ML

FIGURE 3

CHARACTERISTICS OF COMBINED SEWER

OVERFLOW FROM HAWLEY ROAD SEWER

data presented correlates well with combined overflow data from the Detroit Milk River Study (3) and other published data (4).

The operation of the previously described test facility during the spring, summer and fall of 1969 has provided valuable data on operational characteristics and removal rates. Figure 4 shows the data associated with operational variables. The average run had a length of 1-4 hours. Approximately 1/2 hour is required to allow the flotation tank to come to equilibrium. The flow rate for these runs was held constant at 3500 gpm. Pressurized flow was varied over the range of 400-900 gpm and the operating pressure from 40-60 psig. Of considerable importance in the design of this type of system is the volume of residual solids produced during operation. As shown in Figure 4, the volume of water required to backwash and clean the screen ranges from 0.29 to 0.64 percent of the raw flow rate, while the volume of floated scum ranges from 0.43-0.85 percent at the 95 percent confidence level. Solids concentrations in these streams generally is in the range of 1 to 2 percent, and at this concentration they easily flow by gravity. Disposal methods utilized for these solids streams should be sufficient to handle the upper limit of the expected sludge volumes. Under the current contract, we are disposing of these streams via an interceptor sewer which directs them to the sewage treatment plant.

LENGTH OF RUN HOURS	RAW FLOW RATE GPM	SCREEN WASH AS % OF FLOW[1] %	FLOATED SCUM AS % OF FLOW[1] %	PRESSURIZED FLOW RATE GPM	OPERATING PRESSURE PSIG
1-4	3500	0.29-0.64	0.43-0.85	400-900	40-60

(1) AT 95% CONFIDENCE LEVEL

FIGURE 4

OPERATION DATA FROM HAWLEY ROAD DEMONSTRATION SYSTEM

The efficiency of contaminant removal experienced for the overflows monitored to date is shown in Figure 5. Two time periods are shown -- spring storms and summer/fall storms. By observing the screen data in Figure 5, it may be seen that during the spring storms removals ranged from 23-33 percent for all listed parameters. This was consistent with the preliminary data collected the previous year. During the summer/fall storms, however, COD removals decreased indicating a change in the characteristics of the overflow. It was determined that an increase in soluble COD had occurred which was the probable cause for the noted decrease in COD removal across the screen. The mechanical operation of the screen has been very satisfactory. The media utilized was type 304SS. No permanent media blinding has been experienced. No build-up of greases or fats has occurred. Some clogging problems have been experienced with the spray nozzles, but this was caused by a sealing problem around the screen which allowed unscreened water to pass into the screened water chamber.

The overall removals, <u>i.e.</u> screening plus flotation, are also shown in Figure 5. Removals are shown with and without the addition of chemical flocculants. The chemical flocculants, when utilized, were a cationic polyelectrolyte (Dow C-31) and a flocculant aid (Calgon A25). The polyelectrolyte dosage was 4 mg/1 and the coagulant aid dosage was 8 mg/1. Contaminant removal without chemical addition was about 50% for all parameters as shown in Figure 5. Adding chemicals caused an increase

	SCREENING		SCREENING AND FLOTATION	
	SPRING	SUMMER-FALL	W/O CHEMICAL FLOCCULANTS (SPRING)	W/CHEMICAL FLOCCULANTS (SUMMER-FALL)
BOD	23.4 ± 9.3	20.3 ± 6.5	48.4 ± 15.7	50.8 ± 12.5
COD	33.9 ± 10.7	22.4 ± 5.0	52.9 ± 8.7	53.4 ± 8.6
SS	28.8 ± 10.5	24.9 ± 9.8	53.7 ± 11.7	68.3 ± 8.4
VSS	28.2 ± 13.6	24.4 ± 13.2	51.0 ± 15.9	64.8 ± 10.0

NOTES: REMOVALS AS % @ 95% CONFIDENCE LEVEL

SCREEN OPENINGS 297 MICRONS

SURFACE LOADING 3 GPM/SQ FT

FIGURE 5

CONTAMINANT REMOVALS IN PERCENT BY SCREENING AND FLOTATION in SS and VSS removals to around 70%. COD and BOD removals, however, did not increase significantly. This was probably a result of the increase in soluble organics associated with the summer/fall overflows. Chemical addition also provided a strengthening effect on the floated sludge blanket, which is very desirable from the materials handling aspect. Mechanical operation of the flotation tank has been excellent. No mechanical problems have been experienced. Maintenance on the entire system is limited to periodic lubrication and requires less than 6 man hours per month.

Another important aspect in the treatment of combined overflow is disinfection. Figure 6 shows the effect of chlorination on total coliform density from various overflows. In storms 5 through 11, chlorine was added in the pressurized flow line prior to blending with the remainder of the flow in the flotation tank. The dosage was 10 mg/l. The dosage may have actually been lower in some of the runs, since sodium hypochlorite was utilized as the source of chlorine and this solution decreases in strength over a relatively short period of time. Introduction of the chlorine in the pressurized flow allowed approximately 15 minute contact time before discharge from the unit. In storms 19 through 22, chlorine was added to the effluent from the flotation basin and allowed to react for a ten minute period. The chlorine was then deactivated with sodium sulfite and coliform analyses were performed. It may be observed in Figure 6 that coliform reduction was related to initial coliform density when using a constant chlorine dosage. In the spring and early summer when coliform

STORM #	RAW COLIFORM DENSITY PER ML	CHLORINE DOSAGE MG/L	CONTACT TIME MIN.	EFFLUENT COLIFORM DENSITY PER 100 ML
5	36,000	10	15	0
6	5,700	10	15	0
7	1,300	10	15	0
8	7,800	10	15	0
9	6,200	10	15	2
11	20,000	10	15	10
19	310,000	10	10	600
20	160,000	10	10	400
21	55,000	10	10	0
22	82,000	10	10	1500

FIGURE 6

DISINFECTION DATA FOR COMBINED OVERFLOWS AT HAWLEY ROAD densities were low, good disinfection was obtained. However, in late summer when coliform density increased, the effluent contained increased numbers of coliform organisms. Chlorine demand tests were run on some storms. The chlorine demand was generally in the range of 13 to 17 mg/l.

SUMMARY AND CONCLUSIONS

Based on the data taken during 28 overflows, Figure 7 presents the recommended design criteria for screening and dissolved air flotation systems treating combined sewer overflow. This criteria is tentative, since the project has not yet been completed. The most important criteria associated with screen design include hydraulic loading and solids loading. The recommended values are those which were found satisfactory in the operation of the Hawley Road facility.

With regard to the flotation design criteria, the surface loading variable is the only one which has not been fully evaluated. Higher rates will be investigated, and the effect of these rates on removal efficiencies will be evaluated. The other criteria for flotation shown in Figure 7 have been thoroughly evaluated and proven adequate for combined sewer overflow treatment.

The cost of a flotation system for treating combined overflows is directly related to the surface loading variable which is still under investigation. Based on a 3 gpm/sq ft surface loading and the other design parameters of Figure 7, capital cost of a screening/ flotation system should be in the range of $5,000 to $8,000 per

SCREENS

MEDIA - 50 x 50 (297 MICRON OPENINGS)

HYDRAULIC LOADING - 50 GPM/SQ FT

HEAD LOSS CAPABILITY - 14 INCHES WATER

SOLIDS LOADING - 0.14# DS/100 SQ FT

CLEANING WATER - 0.75% SCREENED FLOW

FLOTATION

SURFACE LOADING - 3 GPM/SQ FT[1]

HORIZONTAL VELOCITY - 3 FPM

PRESSURIZED FLOW - 15%

OPERATING PRESSURE - 50 PSIG

FLOATED SCUM VOLUME - 0.95% OF FLOW

PROVISIONS FOR TOP AND BOTTOM SKIMMING

CHEMICAL FLOCCULANT ADDITION

(1) THIS VALUE MAY BE CONSERVATIVE, HIGHER VALUES NOW BEING TESTED.

FIGURE 7

RECOMMENDED DESIGN CRITERIA FOR SCREENING AND FLOTATION

MGD capacity for large capacity plants (>50 MGD). Detailed cost analysis have not yet been performed and these costs are therefore only ball park figures which could increase or decrease as more information is obtained. These cost estimates do not include land costs, which could vary considerably.

Operating costs for a screening and flotation system will be low due to the expected periodic usage when treating combined overflow. Chemical costs should be in the range of 2 to 2.5 ¢/1000 gallons, while operating, maintenance and power costs are expected to be less than 2 ¢/1000 gallons.

In summary, it appears that dissolved air flotation can be utilized as a partial solution to the combined sewer overflow problem. Significant removals of BOD, COD, SS, and VSS can be obtained utilizing screening/flotation. While a detailed cost analysis has not yet been completed, preliminary cost information appears to justify the economic feasibility of the system.

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OVERVIEW OF COMBINED CONTROL AND TREATMENT METHODS

by

William A. Rosenkranz

The discussions thus far have dealt with several methods of control, or treatment. We have discussed the containment or hydraulic control of the flow. Implementation involves modifying an over-all system with some sort of treatment device or control devices or utilization of combinations of these.

Today's papers and discussions thus far illustrate or bring to our attention several important points:

> First of all, there is likely to be no single method of either control or treatment applicable as a complete answer to combined sewer problems. I believe that it is safe to say that this is true even when considering sewer separation as a corrective measure. We have to think in terms of individual outfall control and treatment. Over-all systems which would achieve control over individual drainage areas is of great importance. Systems which can handle the entire problem for a given community or even perhaps a metropolitan area must be considered and evaluated. In other words, drainage area approach must be applied when investigating solutions to overflow problems. While the point source control or treatment application must also be utilized, they will fail unless the entire

drainage area problems are assessed. This is applicable to the entire community or metropolitan area as well as the individual portions within the area.

Proper planning must consider and evaluate all of the different kinds of treatment that might be utilized in an integrated control/treatment system. We need to carefully examine the compatibility of the methodology used in order to properly develop integrated and coordinated systems.

The concept of storage that we use in our program involves any kind of storage that you want to talk about--concrete tanks, design and construction of buildings or modification of roof designs to permit use of roof-top storage, off-shore storage by means of retention basins,underwater bags, many surface ponds, deep tunnels or lined caversn--many different methods and in many different configurations. Each of these methods may have a particular application to a given situation due to the geology of the area, the topography of the land, the location of the sewers, type of receiving waters, water quality required and many other factors.

Engineering studies must consider all potential alternatives when seeking to determine what is <u>most</u> applicable, what is efficient and what is economical. In order to achieve an "optimal" system physical control by storage must be considered in conjunction with potentially applicable treatment methods. This would include as possible solutions the use of storage or surge basins in combination with screening, dissolved air flotation, bio-disc treatment, high rate biological filtration or any other treatment configuration. In-system flow control, flood routing in the system, and storage of waste waters from the system - off the system. In other words, remove wastewater from the system--perhaps even in the upper portions of the drainage area for feed back into the system when the storm is over. Improved regulators, remote control, surge tanks combined with existing or expanded sewage treatment plants, surface storage or retention basins and microstrainers should be examined as possibilities. For example: Infiltration control to achieve reduction of flows, in-system storage or control combined with controlled release to a sewage treatment plant might make a workable system. I am sure that in your own minds you can dream up many possible combinations. The big problem, of course, is the one that is the center of focus for our program activities. That is the determination of what is feasible, what is economical and how it (or they) can be used. Other papers to be presented during the afternoon will deal with this aspect of the problem and the slides which I will show will serve to illustrate some of the techniques and devices currently being studies or demonstrated.

ASSESSMENT OF ALTERNATIVE METHODS FOR CONTROL/TREATMENT OF COMBINED SEWER OVERFLOWS FOR WASHINGTON, D.C.[1]

by
John A. DeFilippi, P.E.[2]

INTRODUCTION

The majority of United States cities today are served by both combined sewers and separated sanitary and storm sewers. The District of Columbia follows this pattern with an area of approximately 20 square miles being served by combined sewers. These, of course, are sewers which carry sanitary sewage during dry periods and, during periods of precipitation, carry sanitary and storm sewer flows. The hydraulic capacity of the system is often exceeded during times of precipitation and raw sewage mixed with surface runoff is spilled into the water courses of the District.

An investigation, sponsored by FWPCA, is now being completed which deals with the assessment of alternative methods for control/treatment of combined sewer overflows for the District of Columbia. The investigation, as presented herein, had three major components: (1) problem definition, (2) the study of the feasibility of high-rate filtration for treatment of combined sewer flows and, (3) the study of alternative methods of solution. Problem definition dealt with attempting to define hydraulic properties and water quality characteristics of combined and separated storm sewer flows. This was accomplished by both desk top hydrology and hydraulic studies and by field sample collection. The second major area of study, high-rate filtration, was investigated by bench-scale laboratory

[1]Research upon which this publication is based was performed pursuant to Contract Number 14-12-403, with the Federal Water Pollution Control Administration, Department of Interior.

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experiments. The third part of the investigation, the study of alternatives, was accomplished by analyzing various approaches used in other parts of the country relative to their applicability to the Washington, D.C. system.

This paper will present a discussion of the three major portions of the investigation. The approaches will first be described and then appropriate conclusions presented.

PROBLEM DEFINITION

Problem definition must first concern itself with an inventory of basic data of the combined sewer area. The basic data must include the following types of information: schematic and detailed maps of the system, drainage basin delineations, land use characteristics, slopes and hydraulic capacities of collector lines and interceptors, overflow points, diversion points, etc. Once this basic inventory of information is available, more precise problem definition can follow. Fortunately, in the case of the District of Columbia this basic information was readily available.

Following the inventory, attention was directed at attempting to quantify flow rates and flow patterns which prevail in the system. The first approach which was used was the Rational Method. The Rational Method can only estimate peak flow rates; it cannot provide the second necessary component of flow determination, hydrographs of flow.

In order to determine the hydrographs, two methods were initially attempted. The first was a method developed in the City of Chicago and reported quite extensively in the literature. It will not be dwelled upon here except to say that it is a relatively elaborate procedure which relates rainfall patterns to resultant flow hydrographs in sewers. The second attempt at defining hydrographs used the unit hydrograph method. When both of these methods were attempted, they did not agree nor did they match the results which were obtained by using the Rational Method. Therefore, a third, more simplified approach was used.

This approach used the Rational Method to predict the peak flow rate; this peak flow rate was plotted at the time of concentration for each drainage basin. Following this, the volume of runoff for a particular storm was estimated by the total amount of rainfall and the runoff coefficient assumed for the drainage area. Knowing the peak runoff and the total volume of runoff, a simple triangular hydrograph was assumed and plotted.

It was realized that this would not provide highly accurate depictions of flow patterns in the sewers. However, it was felt that the hydrographs would be sufficiently accurate for the purposes and goals of the study. As it turned out when actual field data was collected, the assumed hydrographs rather closely estimated actual flow conditions.

The simplified triangular hydrograph approach was quite appropriate to the investigation. The hydrographs were quickly compiled and reflected, with sufficient accuracy, the actual flow conditions. Moreover, they could easily be routed along interceptor routes because of their fixed geometric properties. Routing was accomplished very simply by graphical methods. The hydrographs were plotted along the x-axis (time axis) and lagged by the assumed flow times between individual drainage basin discharge points. The resultant hydrographs at any particular point along an actual or proposed interceptor could then be compiled by simple addition of the cumulative ordinates.

WATER QUALITY DETERMINATION

Water quality determination proved to be significantly more difficult. Prior water quality data in the literature had dealt primarily with composite samples which were collected over the entire duration of a storm. In this particular investigation, it was necessary to define water quality characteristics at discrete time points during the course of a storm. To accomplish this, completely automated monitoring stations were constructed and operated in the field.

Prior to constructing the monitoring stations, drainage basins for sampling were selected. The selection was critical in order that representative data might be collected. The choice of drainage areas for sampling was based upon the following criteria:

1. The size of the drainage area had to be sufficiently large to be representative of the system but it also had to be small enough to be monitored economically.

2. Population density and land use within a monitored basin should be representative of the entire combined sewer area.

3. In order to have a valid correlation between runoff and rainfall, multiple diversion or a large number of intercepting points were not desirable.

4. The geographical configuration at each proposed monitoring site should be flat and accessible in order that monitoring equipment could be installed.

5. Traffic and public impact had to be kept to a minimum.

6. The size of the sampling sewer had to be sufficiently large to allow the installation of equipment.

7. Extensive underground utilities could not be present to prohibit excavation.

Applying each of these criteria, three drainage areas were selected for sampling. Two of the drainage areas were served by combined sewers. The third drainage area was served by a separated storm sewer. The storm sewer was sampled to act as a control and to provide a comparative basis.

When the sampling basins and sites had been selected, construction of the sampling stations began. At this point, it was determined that completely automatic sampling would be required. This was decided because of the extreme difficulty of predicting accurately when rainfall would occur and because of the further difficulties associated with compiling required manpower on short notice. This was a wise decision and it should be strongly urged that further monitoring be accomplished automatically.

Each monitoring station required the construction and installation of equipment at an upstream and a subsequent downstream manhole. The upstream manhole was used to trigger the sampling process and to release a tracer element into the sewer flow for subsequent sampling and measurement at the downstream manhole. By measuring the tracer concentration at the downstream manhole and by knowing at what concentration and rate it was introduced at the upstream manhole, accurate flow measurements could be made. The use of depth of flow measurements and a steady state equation like the Manning Formula are not applicable in this case because flow in combined sewers during times of precipitation is not a steady state phenomena.

The downstream manhole was used for collecting the actual samples. A pump was located in the sewer; this pump lifted wastewaters to a receiving tank in a shed above grade. Samples were removed from the receiving tank at distinct time intervals and stored in a refrigerated sample collector for subsequent analyses. For the majority of the storms, samples were taken at five minute intervals.

Upon installation of the equipment, the systems had to be made operative and reliable during periods of high flow. This proved to be difficult because of the extreme flow ranges encountered and the very destructive debris which finds its way into a combined or storm sewer. However, after much effort, the three monitoring installations were made operative and performed extremely well during the summer of 1969.

There were a total of 150 samples collected and subsequently analyzed. These resulted from 22 storms occurring on the combined sewer drainage areas and 9 storms occurring on the separated storm sewer area.

The data is still in the process of being reduced and organized but the following table provides representative information. It is quite obvious that significant pollution occurs from combined sewer overflows in terms of BOD, solids, COD, nutrients and coliforms. It is perhaps even more surprising to note that significant pollution can occur from separated storm sewers as well.

By integrating the combined sewer quality data, it is estimated that averages of 9.5 million pounds of BOD, 224 million pounds of suspended solids, 3.5 million pounds of total phosphate, and 1.0 million pounds of total nitrogen are discharged annually from combined sewer overflows in the District of Columbia.

Organic contents are lower in the storm sewers than in the combined sewers but solids loadings are much higher in the storm sewer than in the combined sewer. COD loadings are about the same in both cases.

Initial flushing effects were very definitely shown but water quality remained poor throughout the entire duration of the storm. That is, even after the initial flushing effects, there was still significant pollution load being added to the water courses. Furthermore, combined sewer water quality was not necessarily at its worst condition on the initial flush; quality did get worse in several cases on subsequent flushes.

A potential problem with storm sewer flows is that biodegradability may be retarded by the high solids content. The low BOD values may have resulted because high solids hampered bacteria growth and therefore delayed biodegradability. This would result in lower five day BOD values than in the combined sewer data but ultimate BOD may be equally as high.

Table 1

Approximate Ranges of Water Quality Parameters

	Combined Sewer	Storm Sewer
Flow	4,000 - 600,000 gpm	2,000 - 75,000 gpm
BOD	10 - 500 mg/L	10 - 650 mg/L
Suspended Solids	100 - 2,000 mg/L	150 - 11,000 mg/L
Total Solids	400 - 3,000 mg/L	400 - 14,000 mg/L
COD	30 - 2,000 mg/L	40 - 1,500 mg/L
Total Phosphate (as Phosphorus)	0.1 - 8 mg/L	0.4 - 5 mg/L
Total Nitrogen (as Nitrogen)	1 - 17 mg/L	0.6 - 6.5 mg/L
Total Coliforms	60,000 - 6,000,000 counts/100 ml	20,000 - 1,500,000 counts/100 ml
Fecal Coliform	300,000 - 5,000,000 counts/100 ml	0 - 1,300,000 counts/100 ml

A further observation is that severe peaking hydrographs were recorded in the sewers. This adds further proof to the concept that the application of a steady state formula for estimating sewers flows based on flow depths is not applicable.

The data collected in the field has proven to be extremely valuable. It has demonstrated that combined sewer flows exhibit significantly different water quality characteristics than those which are to be found in domestic sewage. Total solids are completely out of the range of expected values; the BOD/COD relationship is very much different than would be expected. It is plain to see that we are dealing with a different set of conditions in combined sewer flows and additional work is necessary to quantify and qualify the quality characteristics of these flows. Assumptions that the water quality parallels that of domestic waste is simply invalid. Treatment schemes and conclusions drawn on this basis cannot help but fall short of their goals.

HIGH-RATE FILTRATION FOR TREATING COMBINED SEWER FLOWS

Having defined peak flow rates and water quality characteristics of combined sewer flows in Washington, D.C., the efforts of the study were then turned to the laboratory analysis of high-rate filtration for the treatment of combined sewer flows. High-rate filtration was defined in this study as filtration rates equal to or greater than 15 gallons per minute per square foot of filter area.

The overall objectives of the filtration study were:

1. To evaluate the applicability of high-rate filtration for the treatment of combined sewer overflows.

2. To determine flocculation materials and procedures which will optimize solids and BOD removal.

3. To provide a design basis for pilot-scale or full-scale treatment units.

In view of the objectives, the variables which were evaluated in the laboratory study are as follows:

1. Filter media including type, depth, size, and arrangement.

2. Flocculant and flocculant aid including types, dosages, and combinations.

3. Filtration rates.

Wastewater characteristics which were studied were size and concentration of suspended solids, BOD concentrations, and temperature. The operating variables were backwash rate and quantity, air scouring rate including duration and pressure, and pressure. Performance was evaluated in terms of effluent quality, length of filter run, suspended solids penetration, and head requirements.

The filtration system consisted of three filters each 4" in diameter and equipped with associated instrumentation to monitor and control filtration rate, operating pressure, head loss, and temperature. The filter columns were designed to have adequate strength to withstand elevated pressures, adequate depth for deep-bed filtration, and could be easily disassembled for the purpose of exchanging and/or modifying the filter media. Wastewater storage facilities of sufficient capacity were also provided to assure a maximum anticipated volume of wastewater which the system would process. Transmission facilities were required between the storage tanks and the filters. The transmission facilities were capable of delivering the wastewater over the desired ranges of flow and pressure without materially affecting the characteristics of the wastewater. Flocculating material supply and injection systems were developed which were capable of delivering and mixing the numerous flocculants and flocculant aids which were under consideration.

A supply of wastewater was provided for testing the filters. The wastewater was developed by diluting domestic sewage and adding clays and silts to provide a waste comparable to the combined sewer flows being measured at the time in Washington, D.C. Finally, overall system safeguards and monitoring and control devices were installed to protect and coordinate the system components.

The equipment was arranged into three separate filtration systems which were parallel and independent of each other. Each filter consisted of a 9 foot jointed glass pipe of 4" inside diameter. Each filter is fed by a pump taking suction from the wastewater storage tanks; the pump maintains a constant operating pressure on the filter. Eleven hundred gallons of storage was provided for each filter.

Three different filter media were investigated. The first media was composed of fiberglass; flow was in a downward direction. A second filter consisted of a 9" gravel base, 3" of coarse garnet, 24" of a garnet/sand mixture, and 36" of anthracite; this was also a down-flow filter. The third filter consisted of a 9" gravel base, 9" of coarse garnet, and 48" of medium garnet; it was designed and operated as an upflow filter.

A total of 40 filter runs were performed with influent solids ranging from 4 to 900 mg/L and influent BOD ranging from 40 to 90 mg/L. For each filter run, all necessary parameters were measured and samples were periodically collected for subsequent laboratory analyses to determine efficiency.

This data is now in the process of being reduced and analyzed but certain general conclusions can be drawn. The upflow filter could only operate satisfactorily between the ranges of 5 and 15 gallons per minute per square foot. Within that range, suspended solids removal were approximately 60 percent and a BOD removal of approximately 45 percent was achieved. However, as the filtration rates were raised above 15 gallons per minute per square foot, efficiency dropped off remarkably.

Filter number two, the tri-media filter, performed very well at a loading of 10 gallons per minute per square foot. At this filtration rate, suspended solids removal of 80 to 95 percent were achieved and BOD removals in the range of 50 to 80 percent were also achieved. Filter runs were approximately of two hours duration before head losses reached a point where backwash was required. As the filtration rate was increased to 20 gallons per minute per square foot, the same approximate efficiency was maintained. However, the length of filtration runs was reduced from approximately two hours to approximately one-half hour.

The third filter media, fiberglass, performed significantly better than the other two granular media. The fiberglass was tested within the range of 15 gallons per minute per square foot up to as high a loading rate as 50 gallons per minute per square foot. At 15 gallons per minute per square foot, suspended solids removals were in excess of 95 percent and BOD removals were in the range of 60 to 90 percent removal. Filter runs lasted from two to five hours and no flocculant or flocculant aid was required.

As the filtration loading rates were increased from 15 to 50 gallons per minute per square foot, suspended solids removals were in the range of 87 to 95 percent. BOD and COD removals ranged between 60 to 75 percent and 50 to 75 percent respectively. Filter runs lasted from one-half to one hour. At this particular point, there were 750 to 1,000 mg/L of suspended solids in the influent.

As mentioned, it was found that flocculants and flocculant aids did not significantly increase the efficiency of the fiberglass filter. However, this was not the case with the granular media. For these two filters, it was found that alum in a dosage of 150 mg/L and C-5 at a concentration of 4 mg/L were the optimum combination of flocculant and flocculant aid.

As a result of the laboratory filtration studies, a number of conclusions can be drawn. First, and of major importance, is the fact that physical treatment can be used to effectively reduce suspended solids and BOD concentrations. This is a significant finding because, as developed in the problem definition part of the investigation, very high flows are to be encountered in relatively short durations of time. This means that biological systems probably cannot be utilized for the treatment of combined sewer flows unless large storage facilities are provided for. Even in this case, it is questionable whether or not a biological system can be kept active and sufficiently alive to adequately treat the wastes. Physical treatment, on the other hand, can be used effectively on an intermittent basis.

Of the various media investigated, there is no question but that the fiberglass media performed best and appears to definitely have an applicability for treating combined sewer flows. Additional laboratory work is needed to develop data on depth of fiberglass bed, desirability of combining granular and fiberglass beds, density gradation of fiberglass, and backwash requirements. However, based upon the results in this study, the laboratory study of fiberglass media should be continued and a pilot-scale facility operated for a final evaluation.

STUDY OF ALTERNATIVES

Having defined the problem and having shown that physical treatment of combined sewer flows is probably possible, the efforts of the study now concentrated upon the analysis of alternative methods which might be applied in solving the combined sewer problem in Washington, D.C.

The various alternative methods which are being considered, separately or in combination, can be classified under four headings:

1. Sewer Separation
2. Off-System Storage
3. In-Line Treatment
4. Miscellaneous

However, as has been shown, separation in itself may not be a solution in view of the high pollutant loads which are delivered to the water courses from urban areas where separated storm sewers exist.

In the study of Washington, D.C., four specific alternatives were developed. These were developed based upon the physical layout of the combined sewer system and applicability of various concepts to the Washington, D.C. system. Capital and annual cost estimates are being prepared to provide a comparative basis.

The alternatives considered generally tunnel storage, local underground tank storage, treatment of combined sewer flows for small drainage areas, and separation. Separation was not studied in detail since it had been studied quite extensively in a previous investigation. The results of the previous investigation were accepted and the earlier cost estimates were brought up-to-date by the use of construction cost indexes.

Storage was studied in terms of volumes required and type of storage facility. In certain cases, for the small drainage areas, local underground reinforced concrete storage tanks were considered. Herein, the tanks would be constructed at the overflow points below grade. Top soil would be added above and a park or other open space use would be made of the land above the tank. For the larger drainage areas, underground tunnels were considered. These would be bored in rock at an approximate depth of 800 feet. In both storage approaches, the combined sewer flows are held during periods of peak runoff and fed back into the system after precipitation has stopped for subsequent treatment. In each storage alternative it was assumed that existing hydraulic capacity of interceptors would be utilized as much as possible. Therefore, during times of precipitation, the interceptors would be flowing full or nearly full and treatment plant capacity would be exceeded. Incremental capacity was assumed to be added at the plant during these periods and cost estimates included.

Specifically, the alternatives were developed as follows:

Alternative I: For the smaller drainage areas, storage would be provided at each individual site by underground storage tanks. For the larger drainage areas, storage would be provided in tunnels. After precipitation had stopped, the stored flows would be pumped back into the system for subsequent treatment at the existing treatment plant.

Since the anticipated plant capacity would still be exceeded during times of precipitation, it was assumed that physical treatment facilities would be necessary at the plant. These facilities would treat the non-stored flows and could act in series with the biological systems during dry weather.

Alternative II: Herein, physical treatment would be provided at overflow points in the system. However, due to the extremely high flow rates on even the smallest drainage areas, storage chambers would be required to act as surge facilities. The same storage system was assumed as in Alternative I.

A number of physical treatment processes are being studied including filtration, microstraining, screening, etc. The research on these processes is still in early phases. However, it has been shown that physical treatment is apparently feasible. In order to derive comparative cost estimates, high rate filtration was assumed for this alternative.

Alternative III: This alternative provided tunnel storage for all overflows with subsequent treatment at the existing plant. It is similar to Alternative I except the overflows at the smaller drainage areas would also be stored in tunnels.

Alternative IV: This method assumed separation and, as pointed out previously, was not studied in detail because of prior studies and the question of effectiveness.

Capital and annual cost estimates are being prepared for each of the alternatives. These are not sufficiently developed to be presented at this time but it appears that the first three alternatives will have capital costs in the range of 100 million to 200 million dollars. Separation, at the present level of construction costs, would have a capital cost in the range of 300 to 400 million dollars.

If physical treatment can be developed as anticipated, the first three alternatives should provide essentially equal pollution control and reduction. Separation would not be as effective.

Operationally, separation would be far simpler once it had been accomplished. Storage facilities will inherently require greater operational efforts, especially sludge handling and removal. Physical treatment installations will require more maintenance and operation but it is felt that, due to the nature of physical treatment, these systems can be automated to a substantial degree.

If the final cost estimates develop as anticipated, one of the first three alternatives will be recommended. Lower cost and increased pollution abatement will outweigh the operational advantages of separation.

SUMMARY

The study has successfully demonstrated an approach in analyzing solutions to combined sewer problems for an urban area. The first two main areas of concentration - problem definition and feasibility of physical treatment - allowed the development of a comprehensive master plan for eliminating raw sewage discharges from a combined sewer area.

The principles and method of approach developed herein can be applied to other combined sewer areas to insure an optimum approach in eliminating combined sewer overflows.

ASSESSMENT OF COMBINED SEWER PROBLEMS

by Richard H. Sullivan
Assistant Executive Director
for Technical Services
American Public Works Association

The water pollution problems which have become the target of public opinion and public official concern are the sins of the past being imposed on the present. We are today racing headlong to catch up with yesterday's custom of using rivers to rid man's environment of his undesirable waste into the waters most convenient to his urban habitat. Now that there is a national desire to clean up the discharge of sewage and industrial waste by construction of treatment plants of adequate processing effectiveness, attention is turned to another sin of the past that is being imposed on the present—the discharge of excess flows from combined sewers everytime it rains.

The problem stems from the early use of storm drains to handle domestic sewage by admitted sanitary flows to these conduits. When sewage treatment was not practiced, the fact that combined sewers spilled their waste water into receiving streams was not a matter of concern, but when treatment was provided for sanitary sewage it becomes necessary to install in combined sewer interceptors, regulator devices which would divert dry weather flow to the treatment plant and during storm run-off period to (excessive flows) receiving waters.

In urban areas where adequate sewage treatment is provided, these periodic overflows stand as a negative effect which minimizes investment in pollution control. A water course that is polluted periodically is only little more usable for most purposes than one that is continuously polluted. As more and more sewage treatment facilities are provided, meeting Federal and State Standards for high degrees of treatment, the anomaly of combined sewer overflows becomes more and more obvious.

In 1966, for the first time, substantial funds became available for research in the field of water pollution. We have witnessed an excellent start toward arriving at a rational engineering approach to reducing pollution from many sources -- including combined sewers. The work to date has not resulted in defining a solution, but rather has stressed the need for a complete engineering evaluation to determine the best solution for the physical parameters which exist.

COMBINED SEWER FACILITY INVENTORY

In 1967, at the request of the Federal Water Pollution Control Administration, the American Public Works Association undertook to make an inventory of combined sewer facilities in the United States. Every local jurisdicition with combined sewers whose population exceeded 25,000 was personally interviewed, as well as a large sampling of other jurisdictions -- including communities with a population of less than 500. In all, 641 jurisdictions were interviewed. We estimated that 46 percent of the communities with 94 percent of the population and 84 percent of the area served by combined sewers were directly interviewed.

The results of the survey indicated that 36,236,000 people, living on 3,029,000 acres were served by combined sewers. This total indicates that approximately 29 percent of the nations total sewered population is served by combined sewers.

Mere numbers do not in themselves make a problem. In the past ten to fifteen years, there has been a substantial effort to construct waste water treatment facilities. Overflows from combined sewers are gradually being identified as one of the continuing sources of pollution. The early rationale that held that since the overflow was 99 plus percent storm water it was "clean" has been disproved. Overflows are polluted.

The small flows of sanitary sewerage in large combined sewers results in low volocities. Solids are therefore settled out along the sewer line. Storm flows tend to scour out this material and carry it to the overflow. A large proportion of the sanitary sewerage also escapes in the overflow. It has been estimated that from three to five percent of the total organic load reaching the sewer leaves by the overflow.

A part of the problem of combined sewer overflows is the location of the overflow facilities and the nature of the receiving waters. Nationally, most overflows are adjacent to residential or industrially zoned land. The major receiving waters are dry water courses or waters used for limited body contact recreation or fishing.

These land and water uses are not suitable places for the discharge of sewage. Presence of the combined sewer overflows may have a serious impact upon land development and land values. For a hundred acre tract in one Michigan City, influenced by one combined sewer overflow, our appraiser estimated that a value loss of $600,000 and to the immediately adjacent area of 1,333 acres, $4,476,000. This loss of value results in a tax loss to the City alone of $70,000 per year.

The American Public Works Association, as a part of its 1967 study, was asked to estimate the cost of separating combined sewers nationwide. We analyzed figures for weeks, adjusted for prices, inflation and about everything else, and ended up with $48 billion in 1967 dollars as the answer. Of this, $30 billion was for work in the public right-of-way and $18 billion for changing the plumbing on private preperty. The complete incapability of many of our major urban areas to bear the disruption of their major commercial areas and major streets makes complete separation and unlikely goal. Therefore we also investigated alternatives and from the information available we estimated that the cost of alternate methods of treatment or control would amount to about $15 billion. Such methods include in-system and off-system holding and drainage area control.

The States, in particular, and many other agencies have enacted regulations which prohibit the construction of new combined sewer systems or the additions to existing systems. Unhappily some of the progress which is being made in metropolitan areas is in new suburban developments where separate sanitary sewers in a great many cases discharge into combined sewers and add higher concentration of sanitary sewage to the overflows.

Another major finding from our interviews was the determination that less than 20 percent of the combined sewer overflow regulators were of a true dynamic type, that is they could be adjusted to meet various flow criteria. Of the 10,025 regulators found in the jurisdictions interviewed, 40 percent were nothing more than simple weirs, many with design features which are not responsive to overflow regulation. In fact many were merely a hole in a manhole to relieve the system.

The use of improper types of regulators for the existing conditions as well as poor maintenance practices appeared to be one of the major reasons for unnecessary and prolonged overflows.

Another finding was that infiltration was recognized as being excessive in a great many systems. Although few jurisdictions had apparently surveyed their systems treatment plant records indicate the excessive wet weather flows.

Sewer personnel across the country told us of their efforts to discontinue the connection of roof gutters, area drains and foundation drains to the combined sewer system. The flow from these sources is generally credited with overloading the sewer system, causing both basement flooding, innundation of mid-city areas and more frequent and prolonged overflows.

Questions were also asked of each jurisdiction as to the number of personnel and the level of training of employees associated with the operation and maintenance of the sewer system. In jurisdictions of less than 25,000, on the average less than one-half have a full time registered engineer or an engineer in training. For the 52 jurisdictions from 10,000 to 50,000, the average was only 3.3 registered engineers in training per jurisdiction. This group also averaged 5.4 certified plant operators per jurisdiction. Thus it appears that generally, there may be an inadequate number of trained personnel available to make maximum utilization of today's technology.

The full report is available from the FWPCA as publication WP 20-11.

STUDY OF URBAN STORM WATER POLLUTION

With sewage and industrial waste treatment a reality and the water resources of the nation — or at least or major watersheds — protected; and with the overflows of combined sewers effectively regulated and minimized, in terms of the "two Q's" of quantity and quality of the spilled waste water to receiving waters, still another "sin" of the past will still stand as a challenge to the present and the future.

This will involve the evolution of a new concept of the pollutional impact of separate storm water discharges on water courses, lakes and coastal waters. Since everything is relative it is understandable that storm water has in the past been considered harmless as compared with the pollutional nature of untreated or inadequately treated sewage and industrial wastes and the nature of combined sewer overflows of admixtures of sewage and storm water runoff.

But with the elimination of minimization of these two obvious sources of pollution, it will not be surprising that attention will eventually come to bear on storm water spills. Are they a source of pollution? What are these sources? What could be done about urban runoff waste waters? What is the role of agricultoral land runoff in the total water pollution control picture and the problem of protecting the nation's water resources for use and reuse purposes?

Some of the answers to these basic questions are found in the study of Water Pollution Aspects of Urban Runoff which was carried out from 1966 to 1968 by the APWA under a contract with the FWPCA. The report on the study is published as WP-20-15.

"Clean" storm water is polluted. Rain scavenges air pollution out of the atmosphere; flows across roofs, across grass sprayed with insecticides and fertilized with nitrogen and phosphorous, pets and birds; along street gutters which may average a daily accumulation of more than a pound of debris each day per 100 ft. of curb; and finally through catch-basins where the flow displaces perhaps two cubic yards of stagnate water and carries with it some of the digested solids from the bottom of the catch-basin. By the time the storm water reaches the sewer, it may exceed the strength of sanitary sewage. When salts from snow and ice control, phenols from automobile exhausts and other contaminates are added, the storm water may have a wide range of undesirable characteristics.

TYPES OF PROBLEMS

The pollution problems which have been generally identified with combined sewers include the following:

1. Pollution of receiving waters
    a. too frequent overflows
    b. dry weather overflows
    c. prolonged overflows
    d. carryover of solids
    e. by-passing to protect waste water treatment plant facilities
2. Disruption of waste water treatment plants
    a. concentration of solids and debris in primary treatment
    b. wash-out of secondary treatment process due to low strength flows
    c. salt water instrusion

At the heart of most of these problems appears to be the combined sewer regulator and the capacity of the treatment plant.

Most jurisdictions have not attempted to assess the extent of the pollution of receiving waters. In many areas the effects of combined sewer overflows are masked by other major sources of pollution, such as untreated or poorly treated sanitary sewage, industrial waste, agricultural land run-off, feed lot run-off, and urban storm water run-off.

The disruptive aspects of combined sewer flow at the waste treatment plant are readily determined by plant operators. In many instances this has led to even further diversion or by-passing to minimize treatment plant problems.

The APWA Research Foundation, under contract with the Federal Water Pollution Control Administration and some 30 local governmental agencies, is engaged in a cooperative study of combined sewer system overflow regulator facilities and practices. This study covers design, application, construction, control and operation and maintenance procedures. The specific purpose of this Project is to analyze and evaluate the effectiveness of these practices and to establish long-needed guidelines for more efficient and dependable control of overflows and for reduction in the frequency and duration of combined sewer flows and the resultant pollution in waters receiving such spills.

The need for better practices was disclosed in the 1967 study previously described. The study resulted in a specific recommendation that an in-depth investigation of regulator practices be carried out to determine if definitive parameters of design, application, construction, control and management of such facilities could be substituted for present and past procedures and to stimulate acceptance of these improved practices in the rehabilitation of existing regulator facilities and in the planning and installation of new regulator works. The current Project is the result of this recommendation and FWPCA's belief that better practices can help resolve pollution problems resulting from combined sewer overflows.

The Function of Regulator Devices: The volume of liquid flowing in a combined sanitary and storm water sewer is greater than the carrying capacity of the interceptor sewer system, the pumping capacity of a pumping station or the capacity of a sewage treatment plant, during periods of storm and runoff. It is the function of a regulating device and the chamber in which it is installed to regulate or control the amount of the flow which is allowed to enter the interceptor system and to divert the balance to holding or treatment facilities, or to discharge this balance to a point of disposal in nearby receiving waters. The regulator, thus, has the function to transmit all dry weather flow to the interceptor and hence to sewage treatment works, and to "split" the total combined storm and sanitary flow during periods of runoff so that a portion of the flow enters the interceptor and the balance is diverted to the other points listed above.

Regulators may be of various kinds — such as stationary, movable, mechanical, hydraulic, electrical, fluidic, variable, non-variable, etc. — but their function is as described. The 1967 study of overflow problems indicated the need for improvement in regulator devices and in their operation and maintenance. Over and above today's regulator facilities, the field of combined sewer service would be benefited by the availability of other types of devices and modifications of existing equipment. Among the challenges are greater sophistication in control and actuating facilities, including onsite and remote sensing and control of intercepted flows, paced by conditions in interceptor and treatment works, and desired diversion of flows into holding and treatment processes for the effective reduction in storm water overflow pollution.

The problem of design, manufacture, application and handling of regulators is made difficult by the conditions under which these devices and regulator chambers must function. These include complex and often unpredictable hydraulic conditions imposed by dramatic changes in runoff due to storms; the heterogeneous nature of the sewage-storm water which is handled, including grit, course debris and other clogging producing wastes; the corrosive nature of the liquids; and the humid and corrosive-gaseous conditions in the regulator chambers. Further complications are created by tide water backflows and other hard-to-predict hydraulic conditions in interceptor-treatment plant networks.

Our study of combined sewer regulators has involved the interviewing of a group of jurisdictions and then in cooperation with a panel of consulting engineers preparing both a report and a manual of practice. Representatives of financially participating jurisdictions as well as the WPCF and the ASCE are serving on the steering committees for the study.

The study is well along and should be completed by early Spring of 1970.

Our detailed, extensive interviews of some seventeen jurisdiction has found only three where the operation of the regulators has been designed to minimize pollutions by assuring that the interceptor sewer is fully charged. In Seattle, this is accomplished by a hydraulically operated gate controlled by a bubbler unit downstream in the interceptor. In Minneapolis-St. Paul Sanitary Distirct, control is achieved through the use of an inflatable dam, increasing the head of the orifice discharge to the interceptor sewer. Detroit is also using a form cf "traffic" control to maximize flow in the interceptor.

An additional principle of operation to minimize pollution is to maximize in-system storage. The Seattle system in particular insures that all of the collector storage capability is utilized prior to an overflow event. This capability does much to eliminate dry-weather overflows and minimize pollution.

Engineering investigations are being made in Seattle to determine where there is justification for upgrading the facilities. The study is conducted by monitoring a facility for the length of time and quantity of flow during overflow events. From the characteristics of the contributory sewer system, a mass hydrograph is constructed to analyze the quantity and time of flow should a controlled facility be installed. One recent study indicated that for a short period of time when eight (8) events occurred which overflowed 6.4 million gallons, that had a synamic regulator been installed only one event of 2.7 million gallons would have occurred, a reduction of 85 percent in frequency and 42 percent in volume.

When information of this type is available, the value of upgrading facilities can be made. There are no magic numbers or rules of thumb — an engineering study is needed in each case.

ROLE OF INFILTRATION CONTROL

Expenditures for sanitary and combined sewers and treatment facilities amount to many millions of dollars annually and form a major part of the total amount budgeted for operations and capital improvement programs in every urban community.

Unfortunately, in most urban areas, little attention has been given to making sure that costly sanitary and combined sewers and sewage treatment facilities function properly, if at all, under wet gound conditions. So-called "separate" sanitary sewer systems often collect such large infiltration flows that they are ineffective in performing their primary function. Infiltration in sanitary sewers usually causes flows which exceed treatment plant capacity and, as a result, biological processes are either upset or raw sewage is by-passed into waterways which were intended to be protected from such contamination.

Infiltration was revealed as a major contributing factor in combined sewer overflows in a report prepared in 1967 by the APWA Research Foundation which was previously described. Thirty-four percent of the cities interviewed indicated that infiltration exceeded their specification. The increased flow in combined sewers due to infiltration decreases its in-system storage capability and results in more frequent and longer duration of overflows.

Most engineering consultants, scientists, and administrators in the field of design, operation and management of sanitary sewage collection systems have little quantitative data available to use in estimating the extent of infiltration and in making value judgements for the most effective means of prevention and control.

The APWA Research Foundation in cooperation with 35 local jurisdictions and the FWPCA has undertaken a study of economics of infiltration control, design and construction practices for new construction and remedies for existing systems where the cost benefit ratio of control indicates that such action is desirable. This study should be completed in the Summer of 1970.

In this study, the factors contributing to storm and gound water infiltration will be evaluated and analyzed to produce guidelines which will be of tangible value to designers, administrators and operators of combined and sanitary sewage collection systems and treatment plants.

The study is designed to aid in the formulation of an effective research and development program to reduce pollution resulting from combined sewer overflows and treatment plant by-passing attributable to infiltration.

A SIMULATION TECHNIQUE FOR ASSESSING STORM AND COMBINED SEWER SYSTEMS*

by

JOHN A. LAGER, P. E.**

INTRODUCTION

There are many methods under development for solving problems related to storm and combined sewer discharges to receiving waters (1). This paper describes work in progress to develop an assessment technique for comparing alternate solutions through a comprehensive computerized program capable of

> "representing urban stormwater runoff phenomena, both quantity and quality, from the onset of precipitation on the basin, through collection, conveyance (both combined and separate systems), storage, and treatment systems to points downstream from outfalls which are significantly affected by storm discharges."

This work is an 18-month cooperative project undertaken by FWPCA (Federal Water Pollution Control Administration), Metcalf & Eddy Engineers, Inc., Water Resources Engineers, Inc., and the University of Florida, with overall coordination and management provided by Metcalf & Eddy. Seven months have elapsed since the start of the work. The developed model is to be essentially complete at the end of 12 months. The concluding six months will be devoted to demonstration, testing, and finalization of the program. Demonstration cities will be selected by the FWPCA on the basis of available monitoring equipment, data, and applicability, and may include catchment areas of 50 to 5,000 acres.

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**Project Manager, FWPCA Contract No. 14-12-502, Metcalf & Eddy Engineers, Inc., Palo Alto, California.

It would be impractical to describe, in equal detail, all aspects of this comprehensive program in the time allotted; therefore, the following presentation format will be adhered to:

1. A brief description of the comprehensive program elements and basic concepts,
2. A summary of the project status as portrayed by recent demonstration of linked hydraulic subroutines and preliminary comparisons with reported hydraulic data,
3. A more detailed accounting of the quality aspects in the program development (selected on the basis of the writer's personal familiarity and the apparent lack of comparable approaches in the generally available literature), and
4. Views on the possible applications of the final program.

COMPREHENSIVE PROGRAM ELEMENTS AND CONCEPTS

The program is intended for use by municipalities, government agencies, and consultants as a tool for evaluating the pollution potential of existing systems, present and future, and for comparing alternate courses of remedial action. Although cost-effectiveness techniques will be fully utilized, the preponderance of human elements inherent in this field of work precludes, in the writer's opinion, the achievement of an optimal solution. For example, the removal of one pollution unit from a receiving water will have different values to different people at different times.

The simulation technique -- that is, the representation of the physical systems identifiable within the model -- was selected because it permits relatively easy interpretation and because it permits the location of remedial devices (such as a storage tank or relief lines) and/or denotes localized problems (such as flooding) at a great number of points in the physical system.

Since the program objectives were particularly directed toward complete time and spacial effects, as opposed to simple maxima (such as the Rational Formula approach) or gross effects (such as total pounds of pollutant discharged in a given storm), it was considered essential to work with continuous curves (magnitude vs. time), referred to as hydrographs and "pollutographs."

Because of the multitude of figures to be stored and manipulated and because the program is expected to have general application, the digital computer is the obvious operational vehicle. More specifically, the developed program will be demonstrated on the Department of the Interior's IBM 360/65.

An overview of the model structure is shown in Figure 1. Principal development responsibilities among the contractors are indicated, and the elementary sequencing through the program is presented. In simplest terms the program is built up as follows:

1. The input sources: Runoff as generated by any rainfall hyetograph, antecedent experience, and land conditions; Dry Weather Sanitary Flow as generated by land use, population density, etc.; and Infiltration as generated by available groundwater and the condition and age of the pipe elements.
2. The central core: Transport model which carries and combines the inputs from node to node in accordance with Manning's equations, and the theories of continuity, and complete mixing. All inputs are considered as occurring at nodes, and the series of linked nodes constitute the prototype collection system.
3. The correctional devices; Storage and Treatment models, which receive hydrographs and corresponding pollutographs from any selected point in the transport model, perform the designated task based upon retention time, efficiency of treatment, and other design parameters, and return the corrected

hydrographs and pollutographs to the selected point within the transport model or the receiving water.

4. The output: Receiving Water models or, in the case of a dry bed, the storm stream discharge. The receiving water may be a river, lake or estuary as identified by multi-linked nodes and operated upon by geometry, upstream flows, tides, other discharges, controls, etc. Comparison of maximum nodal values to established water quality criteria may "loop" back to the correctional devices (requiring added increments of construction) until the quality criteria are satisfied.

PROJECT STATUS - HYDRAULIC

At the September quarterly project meeting, a computer program was executed that demonstrated the feasibility of linking several functioning subprograms into a single operating unit requiring only one set of data and "one punch of the computer execute button." The test case involved six identical subcatchment areas (identical only to simplify the data takeoff) with runoff discharging to the transport model at six different points, which in turn discharged into a storage basin that overflowed into a simulated estuary receiving water. Figure 2 shows a schematic plan of the system. Figure 3 illustrates the initial rainfall hyetograph and successive hydrographs as the flow is routed through the system. Sample corresponding output data are given in Tables 1 and 2.

Individual hydraulic subprograms have been successfully tested against reported data for the Oakdale area in Chicago, the Northwood area in Baltimore, and the Selby Street area in San Francisco, but time does not permit presentation or elaboration on these results (2, 3, 4).

PROJECT STATUS - QUALITY

Whereas the literature abounds with data and theories for modeling the hydraulic aspects of rainfall-runoff and routing, very little has been found in the area of quality models and/or data with notable exceptions (5, 6, 7, 4, 8, 9, 10). Thus, having a "free hand," an approach was developed (which continues to be improved upon) of breaking down the problem into basic source elements, identified in the case of combined systems as surface runoff quality, catchbasin effects, dry weather flow quality, displacement phenomenon, and flushing; attacking each as a separate problem; then recombining the parts to determine the final effect. The source elements were further broken down by the nature of the pollutant (soluble or nonsoluble), the amount of material accumulated at the start of the storm, and the rate of removal of this material as a function of the storm where applicable or, in the case of sanitary sewage, the hour of the day.

Surface Runoff Quality

The estimate of accumulative pollutants on the ground surface at the start of the storm is based almost entirely on data presented in an FWPCA-sponsored APWA study in Chicago, which reported dust and dirt accumulations on urban streets as a function of time, land use, curb length, antecedent rainfall, and street cleaning practice (8). The study reported that this dust and dirt fraction was the best identifiable source of pollutants in urban runoff and described its soluble constituents in terms of BOD and other pollutants, all according to land use. While it is not claimed that all urban areas will accumulate dust and dirt at the specific rates measured in Chicago, these data are presently being used without modification. (Some modification could and may be systemized subsequently on the basis of a monitored air pollution index, such as dustfall.) The reported frequency-efficiency-pass relationships of street sweeping practice are also included in the model; thus, the effects of changes in practice can be indicated.

The removal of soluble pollutants from the streets to the storm inlets or catchbasins is based on the following first order equation developed by Allen J. Burdoin, Consultant to Metcalf & Eddy:

$$P_o - P = P_o\,(1-e^{-4.6rt}) \quad \text{EQ. 1}$$

Where P_o = total pounds of pollutant available on the ground at the start of the period

P = amount remaining after time t

r = rate of runoff in inches per hour

t = time in hours from condition P_o to P

4.6 = constant for the above units assuming that 90 percent of the pollution will be washed off in one hour by a runoff intensity of 0.5 inches per hour.

The removal of nonsoluble pollutants (suspended solids and grit) requires not only contact with the runoff but also physical transport by the runoff; hence an availability factor is applied to the accumulated dust and dirt before executing EQ.1.

The presently used availability factor is computed from the following equation:

$$A = 0.57 + 1.4r^{1.1} \quad \text{EQ. 2}$$

Where A = fraction of total dust and dirt available during the time increment

r = rate of runoff in inches per hour during the time increment.

Studies are in progress for a more theoretical determination based upon particle size distribution and average surface velocities computed for each time increment.

The results of an application of the Surface Runoff Quality model to a separate storm system serving a 27-acre area in Cincinnati (6) is shown in Figure 4, and sample output data are given in Table 3

Catchbasin Effects

Catchbasins traditionally have been built on inlets to combined sewer systems for the purpose of removing heavy grit which might otherwise settle in the collection system and for providing a liquid barrier to prevent sewage odors from reaching the streets. The APWA study and other studies have indicated that these basins are a significant source of pollution, reporting BOD concentrations of 60 mg/L (milligrams per liter) for a residential area in Chicago (8), and 125 mg/L in Washington, D.C. (11). The APWA study further reported the rate of removal of this soluble pollution based upon test cases using salt solutions. In these cases a catchbasin was subjected to varying inflows, and effluent salt concentrations with time were noted. An empirical equation has been fitted to these data by Burdoin. This equation further accounts for varying basin liquid volumes and volume changes during discharge:

$$R = \left(1.0 - e^{-\left[\frac{x}{1.5V}\right]}\right) \times 100 \quad \text{EQ. 3}$$

Where R = percent of catchbasin source pollution removed

x = accumulative inflow to catchbasin in gallons

V = trapped volume of liquid in basin before storm in gallons.

Dry Weather Flow Quality

This source element presents no unusual problems. Generalized aggregate values will be available by direct measurement or from sewage treatment plant operating records. The computer program takes these data when they are available, corrects them for infiltration (which is assumed to be free of pollutants), further corrects them for known industrial process flow contributions, and then distributes the balance over the study area in accordance with land use, sewage flow, family income, and the percentage of dwelling units having garbage grinders. If measured or plant data are not available, estimated average values are substituted prior to the distribution. A sample program output is presented in Table 4. Corrections to average values are included to account for the hourly flow and strength variations (also taken from treatment plant data) where the time of the start of rainfall is known, as in the verification of recorded storm data.

Displacement Phenomenon

When runoff to a combined sewer begins, a major portion of the sanitary flow then present in the system is trapped and mixed or accelerated as plug flow by the new hydraulic influx. Depending upon the size of the system, the capacity of the interceptor, and the prevailing rates of storm and dry weather flow, a substantial portion of this residual sanitary flow (as distinguished from that introduced to the system while the storm is in progress) may appear in the overflow. By starting the simulation in the model ahead of the beginning of actual runoff, thus allowing the model to establish a base sanitary flow, it is expected that this phenomenon will be properly accounted for.

Flushing

A deposition and scour model is being developed that will allow solids to accumulate in the system in areas of low velocities during dry weather flow periods and thus will provide source material for the "first flush" of a storm. Removal of deposits will be expected to follow the traditional scour equations, hence, particle size distribution and availability will be the control factors as in the Surface Runoff Quality model.

Computed results for a measured storm on the Laguna Street area of San Francisco (4) are compared with the reported results in Figure 5. The Laguna Street system, unlike that in Cincinnati, has a combined system and includes catchbasins, dry weather flow, and displacement. Since the general grades are relatively steep (the main trunk rises 300 feet from sea level in a mile and a half), no deposition or scour is accounted for.

Transport

The routing of pollutographs has been found to require much the same analysis as that required for the routing of hydrographs, although they do not behave identically. Simple time-offset routines are believed inadequate so a complete mixing (between inlets), mass balance approach has been adopted.

Treatment

Simplified models for treatment, other than direct sedimentation (which is handled in a manner similar to that for pipe deposition but with basin turbulence factors), are awaiting design criteria and operating data on methods such as those being discussed at this Seminar before being given serious consideration.

APPLICATIONS OF THE FINAL PROGRAM

As anticipated programs of relief may require expenditures of billions of dollars, it is believed that the final program will provide a worthwhile and relatively convenient tool for decision making.

Attention may be directed to the results in terms of pollution of a number of storms at various intensities, durations, and frequencies as opposed to the traditional design storm concept which deals with a single occurrence.

Different treatment and storage alternates and unit sizes may be compared in a short time space and at, perhaps, modest cost.

Results of a particular storm on a particular treatment system may be transferred to nearby catchment areas with, hopefully, confidence of the outcome.

Finally, much of the initial takeoff data which describe the existing system and receiving waters need only be collected once and stored on tape to be readily available to test new alternatives.

CONCLUSION

A program, now under development, has been described and its potential usefulness explored. This program uses the resources of the digital computer and the consortium of contractors to provide a simulation technique for modeling and assessing storm and combined sewer systems. A general overview of the program has been given, a sampling of the approach methods explained, and preliminary results shown.

ACKNOWLEDGEMENTS

The work described herein is largely the product of my working associates and our associated contractors whose efforts are hereby gratefully acknowledged.

This project is being funded by the FWPCA through Research and Demonstration Grants 14-12-502, 14-12-501, and 14-12-503.

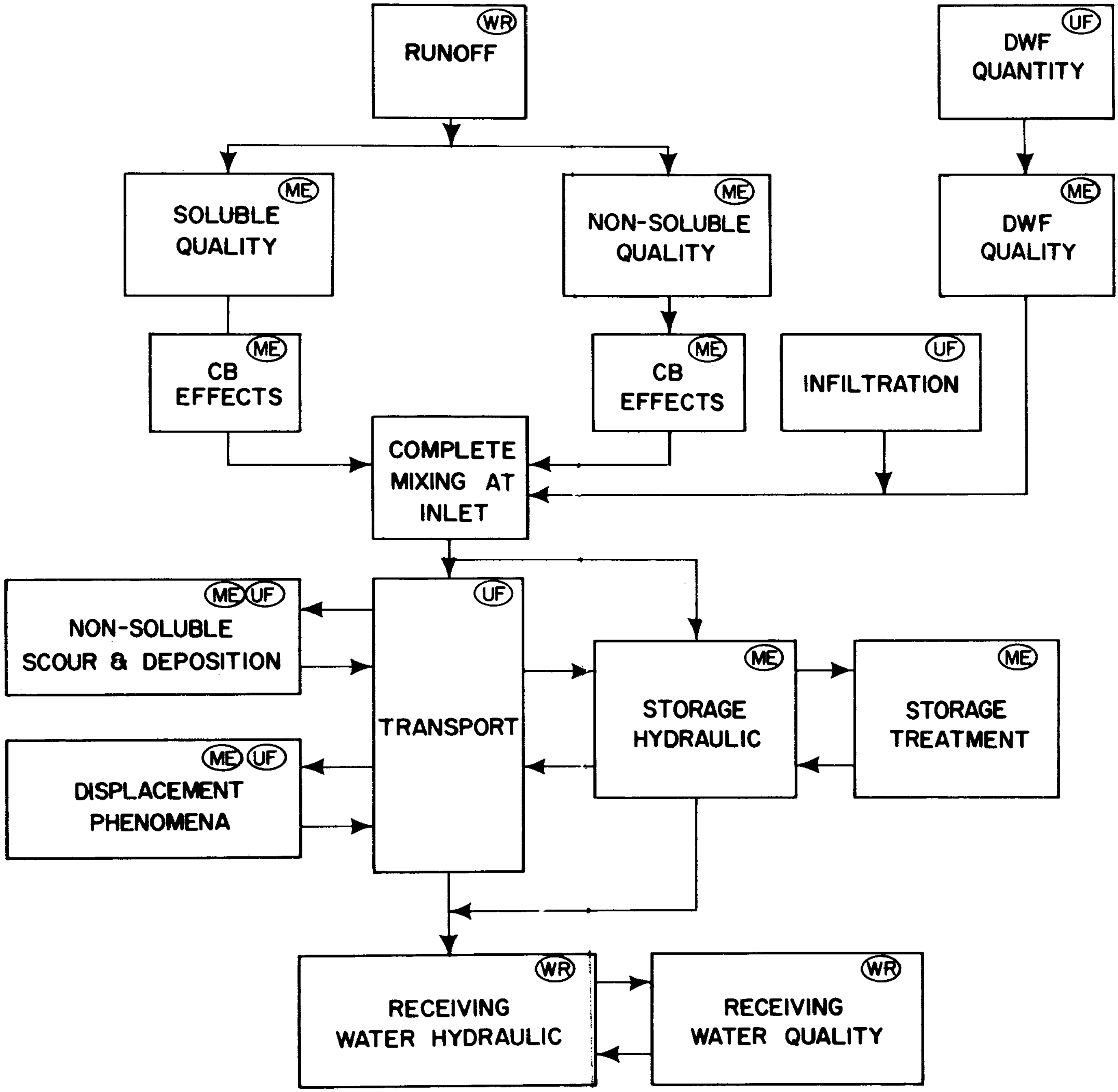


FIGURE I
OVERVIEW OF MASTER MODEL STRUCTURE

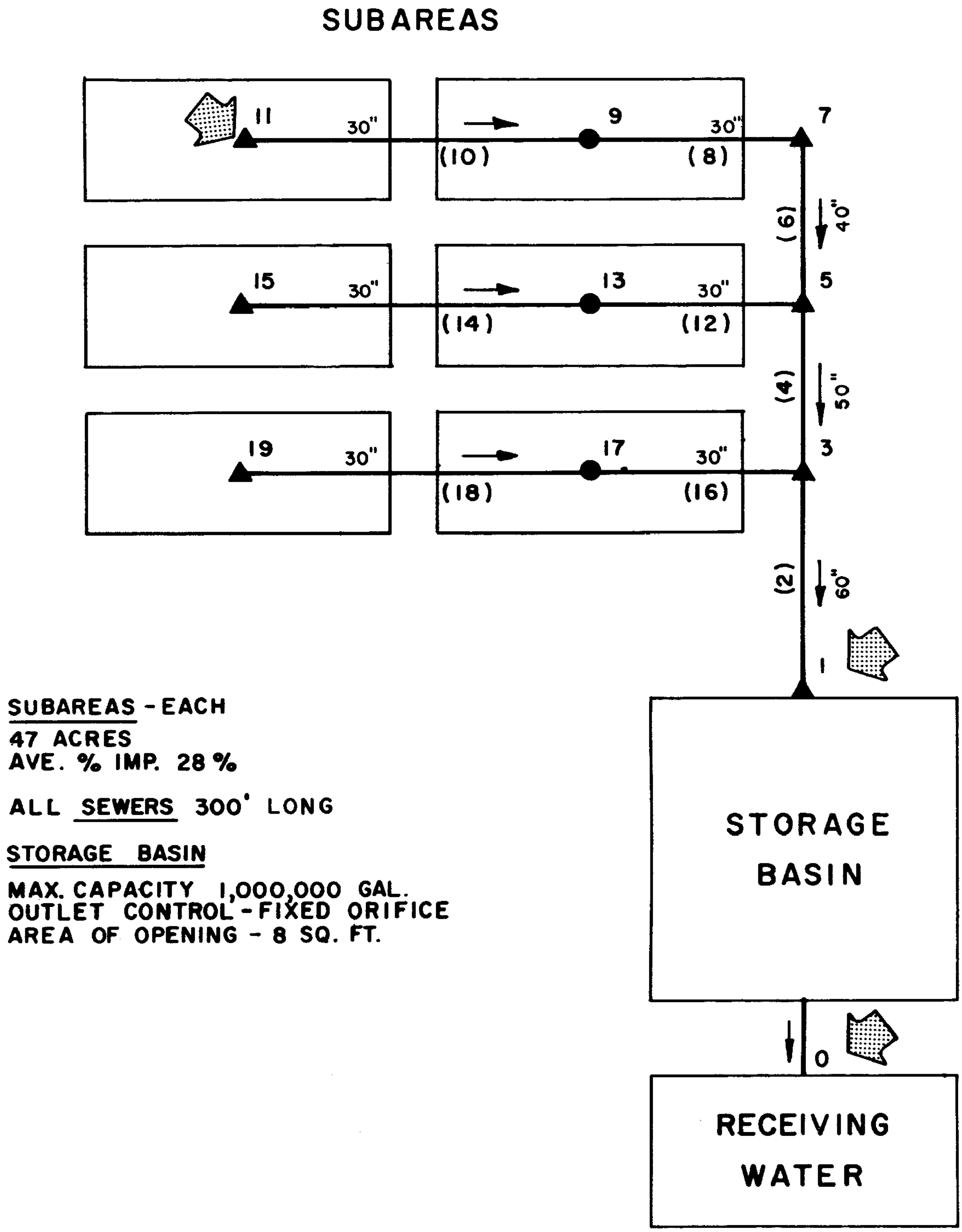
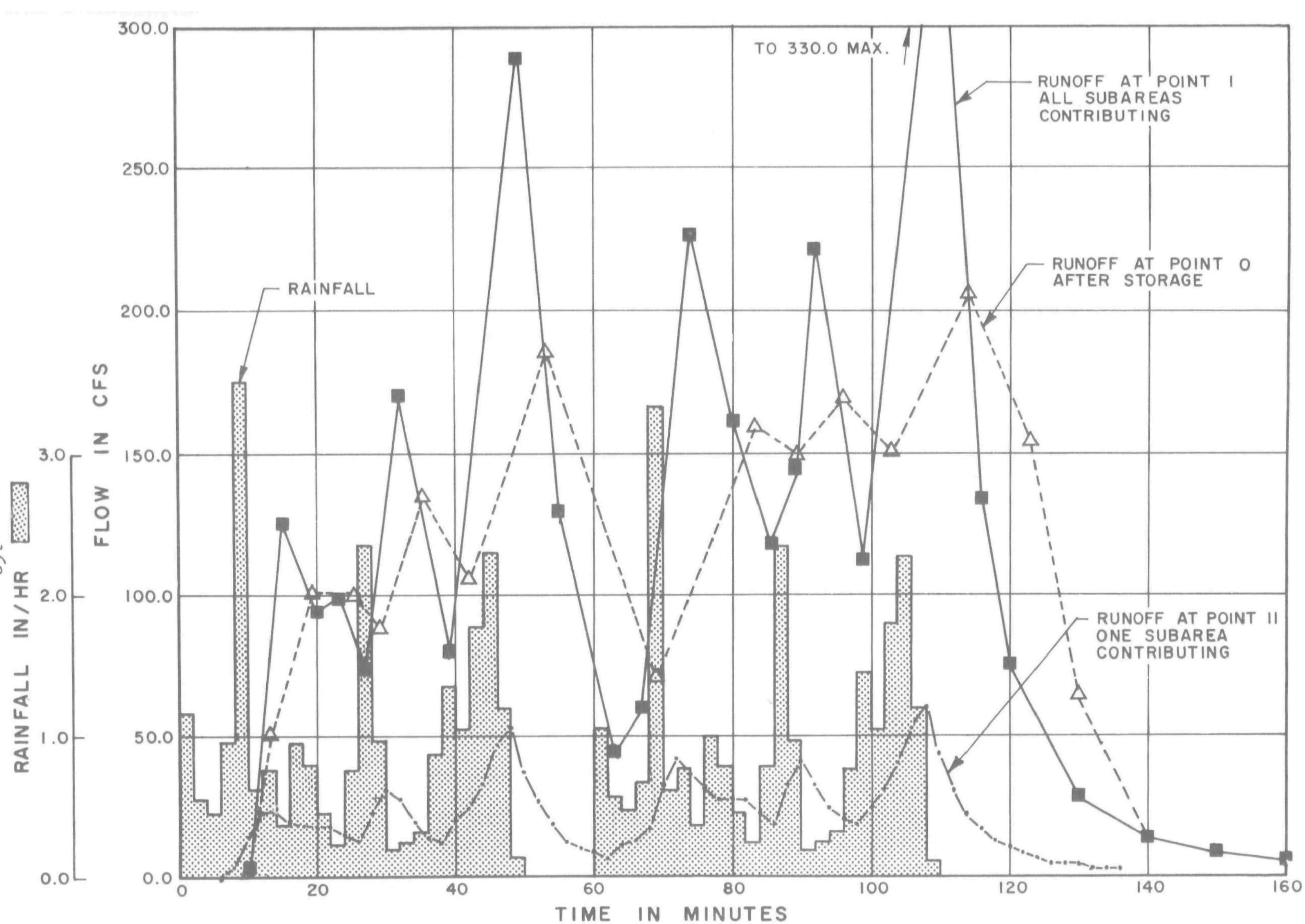


FIGURE 2
SCHEMATIC PLAN OF HYDRAULIC
DEMONSTRATION MODEL



163

FIGURE 3
COMPUTED HYDROGRAPHS
DEMONSTRATION MODEL

METCALF & EDDY

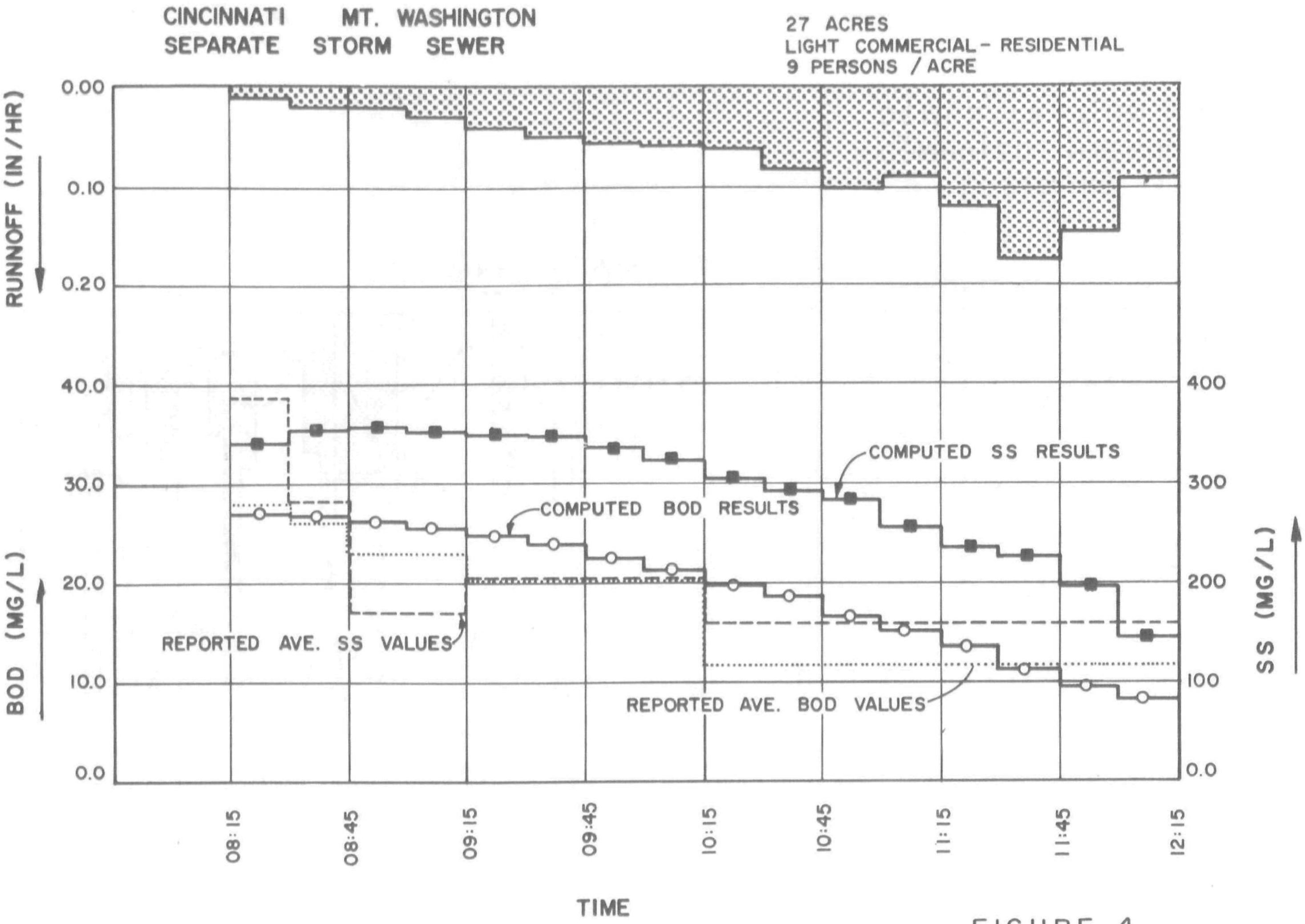


FIGURE 4
QUALITY MODEL RESULTS FOR
SEPARATE STORM SEWERS

METCALF & EDDY

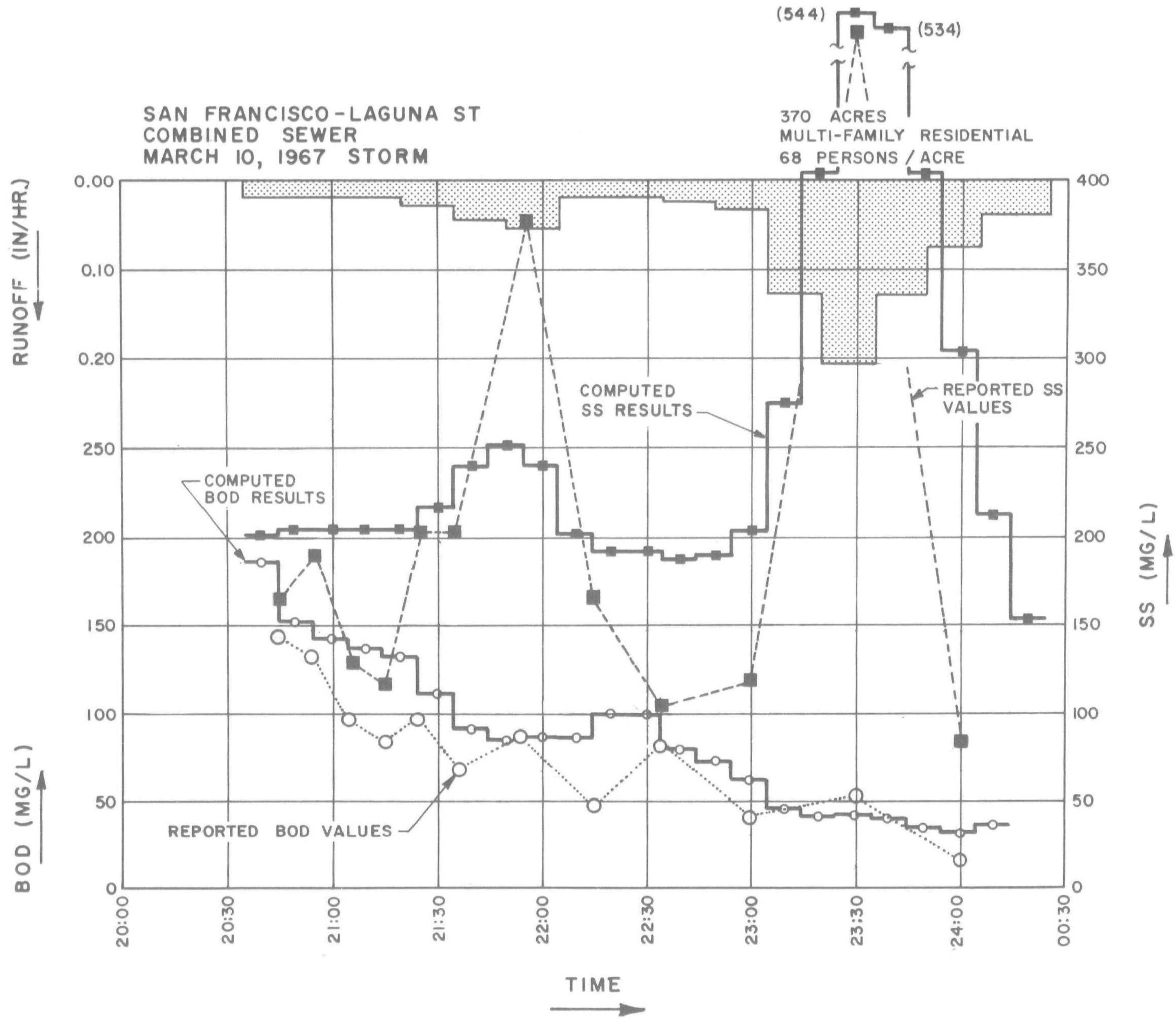


FIGURE 5
QUALITY MODEL RESULTS FOR
COMBINED SEWERS

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SELECTED OUTFLOW HYDROGRAPHS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
1	.000	.000	.164	.208	.250	.262	.280	.439	.928	2.1
	12.704	52.136	100.441	122.244	125.556	122.179	114.096	103.785	96.046	92.
	94.832	97.899	98.609	96.205	91.583	82.634	74.172	75.006	92.609	124.
	155.873	170.383	167.890	156.116	136.000	113.971	98.529	87.249	79.960	81.
	91.779	105.736	120.851	137.235	158.650	192.848	226.683	266.002	289.071	285.
	270.003	232.696	191.889	158.456	135.812	112.138	94.938	81.444	69.711	61.
	56.216	50.490	44.636	46.350	51.344	56.608	60.895	63.971	67.559	80.7
	117.576	178.325	222.159	231.205	224.862	214.029	195.118	175.777	165.429	162.
	162.628	163.334	161.891	157.518	149.286	131.095	118.603	122.356	145.755	184.
	215.084	221.860	215.283	200.119	172.463	153.045	135.603	119.914	112.641	116.
	127.775	143.472	158.745	177.600	200.556	229.431	267.090	305.169	328.351	324.
	308.420	266.942	218.615	184.651	156.321	134.732	115.792	103.548	86.114	76.
	67.320	61.181	56.492	51.340	44.336	40.318	38.674	35.479	30.135	28.
	26.355	24.632	22.865	21.030	16.334	15.608	14.527	13.627	12.861	12.1
	11.566	11.082	10.784	10.293	10.011	9.510	9.393	8.848	8.621	8.1
	8.121	7.425	7.232	5.926	5.698	5.614	5.553	5.500	5.452	5.4
	5.369	5.333	5.301	5.272	5.246	5.223	5.203	5.185	5.170	5.1
	5.147	5.137	5.129	5.122	5.116	5.110	5.105	5.100	5.095	5.0
	5.087	5.083	5.080	5.076	5.073	5.070	5.067	5.064	5.061	5.0
	5.056	5.054	5.051	5.049	5.047	5.045	5.043	5.041	5.040	5.0
3	.000	.197	.236	.261	.275	.330	.583	1.239	2.761	7.9
	28.536	72.357	116.340	128.108	126.472	119.620	109.591	99.905	94.115	93.
	96.445	93.173	98.598	95.114	87.716	78.221	72.178	78.806	103.519	139.9
	166.661	173.224	164.542	148.164	126.038	106.714	93.024	83.184	78.872	84.
	97.397	112.448	127.407	144.705	168.447	203.756	241.841	280.402	297.305	285.
	259.580	216.548	177.539	149.083	124.823	103.402	88.008	75.159	64.518	57.
	51.300	45.527	44.601	48.192	53.823	59.364	63.116	65.480	70.172	90.
	137.455	200.585	234.361	234.180	223.433	207.253	186.986	171.110	163.714	161.
	163.202	163.255	160.579	153.208	140.563	124.291	116.690	127.333	157.774	199.
	224.508	223.914	211.953	189.563	163.341	145.564	127.803	115.610	112.390	118.9
	133.641	149.947	165.013	184.141	210.047	241.779	282.272	319.923	335.642	322.
	295.028	247.308	204.490	171.070	147.309	124.379	109.430	93.547	81.321	71.
	63.538	57.689	51.996	46.669	40.735	38.403	35.013	30.757	27.699	25.7

TABLE I

SAMPLE OUTPUT DATA

HYDRO-MOD TRANSPORT

STORAGE ROUTING SOLUTION, FOR 200 TIME-STEPS THROUGH UNIT NO. 1, IS

RES NO.	KTSTEP NO.	TIME (MIN)	INFLOW (CFS)	OUTFLOW (CFS)	STORAGE (CU.FT)	DEPTH (FT)
1	0	0.0	0.0	.0	0.	0.00
1	1	1.0	.0	.0	0.	.00
1	2	2.0	.0	.0	0.	.00
1	3	3.0	.2	.0	4.	.00
1	4	4.0	.2	.1	10.	.00
1	5	5.0	.3	.2	16.	.00
1	6	6.0	.3	.2	20.	.00
1	7	7.0	.3	.2	23.	.00
1	8	8.0	.4	.3	29.	.01
1	9	9.0	.9	.5	46.	.01
1	10	10.0	2.2	1.0	94.	.02
1	11	11.0	12.7	4.1	388.	.07
1	12	12.0	52.1	17.7	1678.	.32
1	13	13.0	100.4	45.9	4344.	.83
1	14	14.0	122.2	75.9	7371.	1.41
1	15	15.0	125.6	86.2	9941.	1.86
1	16	16.0	122.2	94.4	11955.	2.21
1	17	17.0	114.1	99.5	13229.	2.44
1	18	18.0	103.8	101.5	13734.	2.53
1	19	19.0	96.0	101.2	13647.	2.51
1	20	20.0	93.0	99.7	13290.	2.45
1	21	21.0	94.8	98.5	12978.	2.39
1	22	22.0	97.9	98.0	12864.	2.37
1	23	23.0	98.6	98.1	12876.	2.38
1	24	24.0	96.8	98.0	12857.	2.37
1	25	25.0	91.6	97.2	12653.	2.34
1	26	26.0	82.6	95.0	12114.	2.24
1	27	27.0	74.2	91.4	11226.	2.09
1	28	28.0	75.0	87.8	10325.	1.93
1	29	29.0	92.6	86.9	10112.	1.89
1	30	30.0	124.4	91.6	11266.	2.09
1	31	31.0	155.9	102.1	13804.	2.55
1	32	32.0	170.4	112.5	17216.	3.10
1	33	33.0	167.9	121.2	20354.	3.60

TABLE 2
SAMPLE OUTPUT DATA
HYDRO-MOD STORAGE

TABLE 3
SAMPLE OUTPUT DATA
SURFACE RUNOFF QUALITY

TOTAL QUANTITIES REMOVED FROM ALL AREAS IN EACH TIME INCREMENT

TIME	RUNOFF CFS	RUNOFF IN/HR	BOD LBS/DT	BOD MG/L	SS LBS/DT	SS MG/L
8:15	0.10	0.00	0.15	26.7	1.92	341.6
8:30	0.35	0.01	0.52	26.5	7.03	357.5
8:45	0.50	0.02	0.73	26.0	10.08	359.1
9: 0	0.60	0.02	0.86	25.4	11.98	355.5
9:15	0.85	0.03	1.17	24.6	16.95	355.0
9:30	1.10	0.04	1.46	23.6	21.59	349.5
9:45	1.30	0.05	1.64	22.4	24.72	338.5
10: 0	1.45	0.05	1.72	21.2	26.37	323.8
10:15	1.55	0.06	1.73	19.8	26.70	306.7
10:30	1.90	0.07	1.97	18.4	31.43	294.5
10:45	2.45	0.09	2.31	16.8	38.81	282.1
11: 0	2.55	0.09	2.16	15.1	36.65	255.9
11:15	2.80	0.10	2.12	13.5	36.80	234.1
11:30	3.95	0.15	2.59	11.7	50.27	226.6
11:45	4.25	0.16	2.34	9.8	46.83	196.2
12: 0	3.10	0.11	1.46	8.4	25.07	144.0
12:15	1.20	0.04	0.52	7.7	5.53	82.0
12:30	0.00	0.00	0.00	0.0	0.00	0.0
12:45	0.00	0.00	0.00	0.0	0.00	0.0

TABLE 4
SAMPLE OUTPUT DATA
DRY WEATHER FLOW QUANITY AND QUALITY

SOLUTIONS FOR DRY WEATHER FLOW QUANTITY AND QUALITY

TIME INCREMENT = 10. MINUTES
CASE = 1

12.21	1.83	10.38	9.03	10795.31	9048.60	3561.40	9901.27

A1BOD = 1C96.67LBSPERDAY/CFS
A1SS = 1370.73 LBSPERDAY/CFS

KNUM	INPUT	CWF CFS	CQ CFS	QQDWF CFS	KLAND	DWBOD LBS/DT	DWSS LBS/DT	TOTPOP PERSONS	BODCONC MG/L	SSCONC MG/L
1	10	C.48	0.07	0.55	2	3.07	3.84			
2	11	C.26	C.04	0.30	2	2.03	2.54			
3	13	0.11	0.02	0.13	2	0.89	1.11			
4	20	C.48	C.C7	0.55	2	3.83	4.78			
5	22	0.20	C.03	0.22	2	1.54	1.93			
6	14	C.26	0.04	0.30	2	2.05	2.56			
	SUBTOTALS									
		1.79	0.27	2.06		13.41	16.76	17601.	174.	217.
7	30	0.93	0.14	1.07	2	7.36	9.20			
8	31	C.12	0.C2	0.14	2	0.94	1.17			
9	40	0.71	0.11	0.82	2	4.38	5.48			
10	5C	C.21	0.03	0.24	2	1.31	1.64			
11	16	0.40	C.06	0.46	2	3.13	3.91			
	SUBTOTALS									
		4.16	0.62	4.79		30.53	38.16	37364.	170.	213.
12	17	C.11	C.C2	0.12	2	0.83	1.04			
13	60	0.34	0.05	0.39	1	2.75	3.44			

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A few minutes might be well-spent to summarize what has taken place today and to offer a couple of comments of my own.

First of all, the matter of disposal of solids that may be removed by a treatment process or collected in a storage or sedimentation basin has not been the subject of much discussion. When looking at alternatives, consideration must be given to the solids and what are we going to do with them. Of course, there are quite a few alternatives to be considered. We have discussed today putting them back into the system with transport to waste-water treatment plant. In this regard another comment is pertinent. The Minneapolis-St. Paul and Detroit projects are using in-system control, routing, storage, etc. in the system. They are now noticing using solids concentrations in the treatment plant implement and are getting complaints from the sewage treatment plant operators related to increased solids handling problems at the treatment plant. Obviously, they are accomplishing something. The solids are not going into the river but they are increasing the solids loads on the treatment plant. This is a problem that has to be faced.

A couple of quick summary items. Alternatives must be examined in terms of both technical and economic feasibility. We are needful of designing coordinative and compatible systems. Where more than one treatment point is involved, the analyzing of alternatives and selection of the control and/or treatment sytem, the total pollution load discharged must be considered as well as the instantaneous quality of a waste stream at any particular point and time.

This is particularly true since both intra-state and inter-state enforcement actions are now including this type of consideration. The total pounds of allowable discharge are likely to be established and the water quality standards set in this vain. Use of such an approach will be increasing and it is going to place an additional burden on communities with combined sewer systems.

The seminar that we had here today has presented what ammounts to an interim report on the progress that we have made to date with the help of some 80 grantees and contractors. Much more information on performance and cost facilities is needed. Individual sewerage systems present individually unique problems requiring unique solutions.

Our research and development program is still looking for good demonstration projects to help fill these information gaps. I hope you all contribute something to it. Thank you very much for being here.

BUILDING FOR THE FUTURE - THE BOSTON DEEP TUNNEL PLAN

by

CHARLES A. PARTHUM

In early 1966, the City of Boston engaged the consulting firm of Camp, Dresser & McKee to prepare a report on improvements to the Boston Main Drainage System. This report, completed in late 1967, offered a plan of improvements which (1) correlated the work of many urban renewal projects, (2) replaced old and antiquated sewers, (3) considered the problems of an old combined sewer system, and (4) produced a course of action which will keep sanitary sewage construction in the City moving ahead with the New Boston. One of the recommendations in this report was the construction of a Deep Tunnel System to receive and dispose of all overflows of mixed sewage and storm water and surface runoff.

Early History

Boston was settled in about 1630. By the year 1701 the population had increased to about 8,000 and problems were being created by frequent digging of streets to lay or repair sewers. Until the year 1823, however, the sewers in Boston were constructed, repaired, and owned by private individuals. The purpose for which the sewers were constructed at that time was for the draining of cellars and lands and toilet and privy vault wastes were specifically excluded from the sewers.

In 1823 when the City of Boston was granted its charter, it assumed control of all existing sewers and of the construction and maintenance of new ones, but not until 1833 was it determined that the Mayor and Aldermen at their discretion might permit fecal matter to be discharged to the sewers. Between 1834 and 1870 the City conducted extensive

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Partner, Camp, Dresser & McKee, Consulting Engineers, Boston, Massachusetts. This paper was presented at the 42nd annual conference of the Water Pollution Control Federation, Dallas, Texas, October 5 - 10, 1969.

operations for reclaiming and filling tidal areas bordering the old shorelines of Boston. To meet these changing conditions, sewers were extended long distances at practically no grade to reach new points of discharge into the harbor. As a result, the deposit of sludge and debris within the sewers and upon the tidal flats around the City occurred. In 1870 the City declared that a better system of sewerage was urgent, but it was not until the period betweeen 1877 and 1884 that the City of Boston constructed what is known as the Boston Main Drainage System. This system consisted of 25 miles (39.5 km) of main and branch intercepting sewers and a pumping station and outfall sewer to Moon Island where sewage was discharged raw on the outgoing tide. The sanitary sewage collected by the Boston Main Drainage System now is discharged to the new Metropolitan District Commission sewerage system where it receives primary treatment and chlorination. Still the combined sewer overflows exist.

The Problem

At the present time, there are about 1360 miles (2150 km) of sewers in the City of Boston, many of which were built over 100 years ago. Most of these sewers, particularly in the older sections of the city, are combined and their condition is questionable. Much of the Boston Main Drainage System is surcharged and several sections have collapsed.

It is estimated that at the present time there are about 90 outlets in Boston which discharge dry weather flows of sewage frequently or mixed sewage and storm water continuously during wet weather. Of the approximately 30,500 acres (12,400 ha) of total sewered area in the City of Boston, it is estimated that about 7,000 acres (2340 ha) are served directly by combined sewers and about 10,100 additional acres(4100 ha) are now served directly by separate systems which discharge to combined sewer outlets.

Recent federal and state legislation has resulted in the classification of coastal and inland waters in the vicinity of the City of Boston. This classification, adopted by the State on June 20, 1967, and approved by the Federal Government, means that the continued discharge of untreated sewage and mixed sewage and storm water is a violation. Abatement of pollution from combined sewer system overflows presents a most formidable problem for some of our older and larger cities. Until the problem is solved, however, compliance with State and Federal water quality standards cannot be achieved. It was most important, therefore, that the City of Boston have a feasible plan to present to State and Federal authorities in its efforts to improve its sewerage system and to dispose properly of its mixed sewage and storm water discharges.

Alternative Methods to Handle Mixed Sewage and Storm Water Flows

A number of communities neighboring Boston (Brookline, Cambridge, Chelsea and Somerville) also have combined sewer systems which now discharge through outlets into Boston Harbor and adjacent waters. It was concluded that methods for handling discharges of sewage or mixed sewage and storm water from combined systems should include the applicable areas in each of these communities. The tributary area in all five communities is referred to collectively hereafter as the regional area.

To determine the most feasible method of handling mixed sewage and storm water discharges to Boston Harbor and adjacent waters, a number of alternative methods were investigated which, in addition to the Deep Tunnel Plan, included complete separation, construction of chlorination detention tanks and construction of holding tanks.

Complete Separation

Separation has been the policy of the City of Boston for about 60 years. Separation, if completely accomplished, would eliminate all discharges of overflows of mixed sewage and storm water. In Boston and neighboring municipalities where systems are now combined, separation would require the construction of a new sanitary sewer in every street where a combined sewer now exists. It would also involve new separate plumbing connnections to all of the existing buildings, and the re-plumbing of many entire buildings to separate roof drainage from the sanitary sewerage system. In many areas, yard drainage which now discharges into the combined system would also have to be repiped to the separate storm water systems. The construction of new separate sanitary sewers in the combined areas of the city would result in enormous traffic problems that would interfere with every day activities. New sanitary sewers would be required to serve 7,000 acres (2340 ha) in Boston, and an additional 5,000 acres (2020 ha) in the regional area.

It was not considered feasible or practical to completely separate existing building plumbing into separate sanitary and storm systems. The great problem of enforcement of such separation in private dwellings by owners would have to be carried out under city ordinance by teams of inspectors. The only other possible way to affect separation of building plumbing would be for the City to go into each building and accomplish the separation itself. Separation in many buildings would require extensive renovations to the buildings themselves.

Construction of Chlorination Detention Tanks

A second alternative method for handling mixed sewage and storm water discharges involves construction in the vicinity of selected outlets, of chlorination detention tanks, which would collect, detain and chlorinate discharges or overflows of dry weather flow or mixed sewage and storm water before discharging to near-by watercourses. As stated heretofore, there are about 90 outlets into Boston Harbor and adjacent waters from the combined systems in Boston alone. An equal number of such outlets exist in neighboring communities. It was estimated that about 30 tanks would be required to serve outlets in

Boston alone. Near each outlet or combination of outlets must be available sufficient land area for construction of such tanks in order to make this method feasible. It was estimated that these tanks would require a total land area of about 100 acres (40 ha) to serve 10,300 acres (4180 ha) in Boston alone and about 160 acres (65 ha) to serve 17,000 acres (6900 ha) in the regional area. The cost of taking land for this method even if it were to be made available would be prohibitive.

The enormous problem connected with the operation and maintenance of pumping, chlorination and cleaning facilities in addition to land costs and construction costs for chlorination detention tanks did not present a practical solution.

Construction of Holding Tanks

A third alternative method of handling mixed sewage and storm water discharges involves construction of holding tanks in the vicinity of the outlets, which would store the discharges or overflows until the storm subsides. The stored flows could then be released back into the dry weather interceptor system for disposal with the normal sewage flow in the sewerage system. The holding tanks would be much larger than chlorination detention tanks, more land area would be required, and the resultant costs would be higher. Therefore, holding tanks did not offer a practical solution.

Comparison of Costs

Shortly after beginning this study, the firms of Harza Engineering Company and Bauer Engineering Inc. proposed a deep tunnel storage plan for the Metropolitan Sanitary District of Greater Chicago. After a thorough review of the Chicago plan, including discussions with the engineers involved it was concluded that the basic concept of deep rock tunnels for storing overflows is most attractive and offers possibilities that other methods do not.

A comparison of costs of the above three alternative methods together with the Proposed Deep Tunnel Plan, to be described hereinafter, is as follows:

ESTIMATED COSTS OF ALTERNATIVE METHODS FOR THE BOSTON REGION

	Estimated Costs, Million Dollars		
Method	Construction	Capitalized Operation and Maintenance *	Total
Complete Separation	550.0	34.0	584.0
Chlorination Detention Tanks	400.0	133.0	533.0
Holding Tanks	715.0	99.0	814.0
Proposed Deep Tunnel Plan	430.0	66.0	496.0

*At interest rate of 4.00%

It was concluded that of the various alternative methods studied, only the method of storing overflows in deep rock storage tunnels would provide the Boston region with a positive and feasible method of completely solving the problem of combined sewers.

Proposed Deep Tunnel Plan

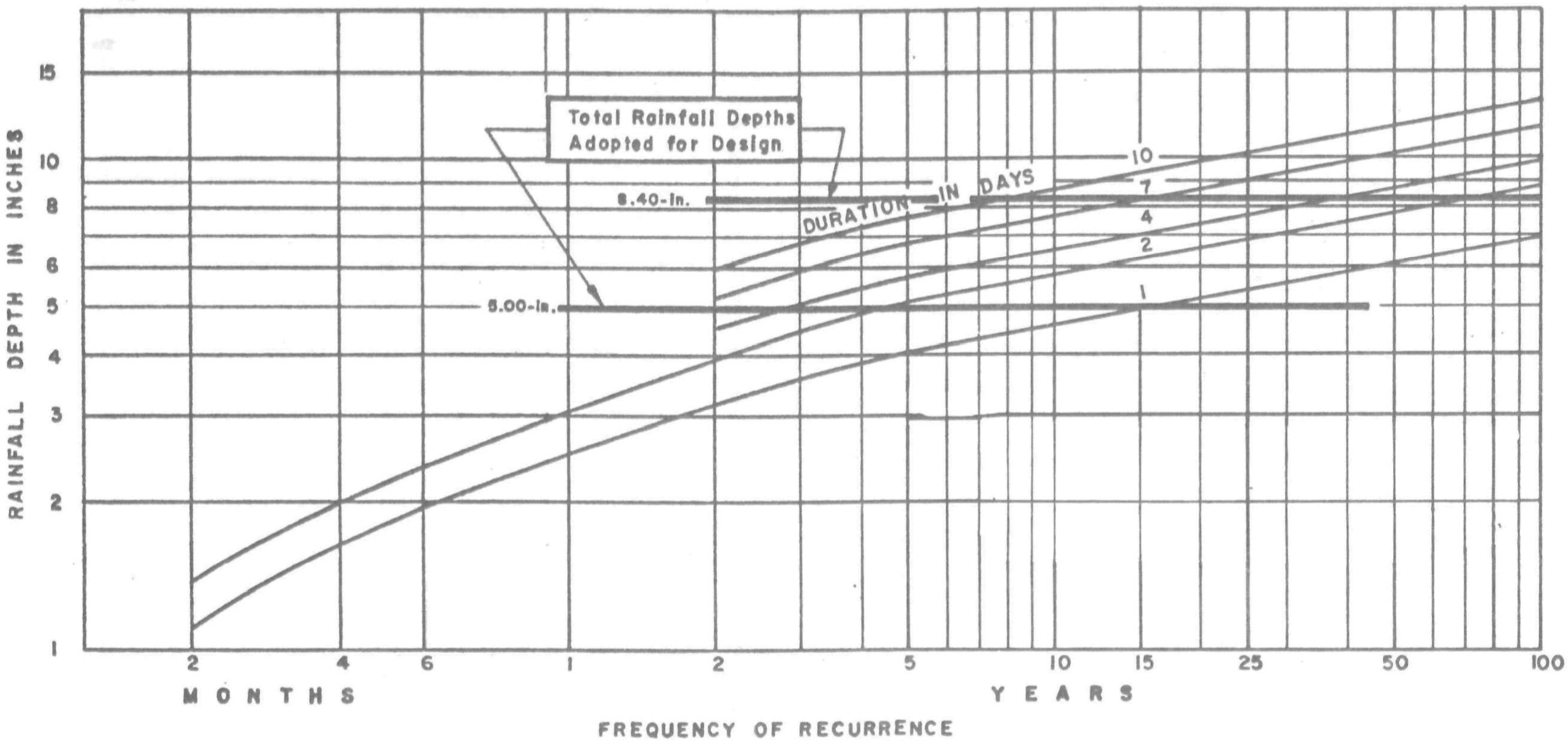
The Deep Tunnel Plan is proposed to be of sufficient size and capacity and suitably located to serve the tributary areas of Boston and the four neighboring communities which have combined sewer systems, thus solving on a regional basis the problem of water pollution abatement. The total area to be served by the proposed Deep Tunnel Plan was estimated at about 17,000 acres (6900 ha).

Rainfall

In design considerations of a tunnel storage plan, the volume and intensity of rainfall are very important factors. Two significant rainstorms were considered: (1) a 5-in (13 cm) storm in 24 hours, with a frequency of recurrent of about 15 years, which could be handled without surcharging the tunnels and (2) an 8.40-in (21 cm) storm in 24 hours which could be handled with surcharging of the tunnels. The total rainfall depth shown on Fig. 1 "Depth of Rainfall vs. Frequency for Boston, Massachusetts" for a storm of one-day duration and 100-year frequency is about 7.0-inches (18 cm). The maximum recorded 24-hour rainfall in Boston is 8.40-inches (21 cm).

Whereas, it is obvious that a tunnel system designed on the basis of 5-in (13 cm) and 8.40-in (21 cm) storms in 24 hours will not be adequate for a 24-hour rainfall in excess of 8.40-in (21 cm), such excessive rainfalls, even though not ever recorded, nevertheless were considered. The capacity of the present sewerage system is such, however, that it is unable to deliver enough flow to exceed the proposed tunnel system design capacity of 8.40-in (21 cm) in 24 hours, and the excess flow must, therefore, be stored at the surface or runoff overland to the nearest watercourse. Even when replacements are made to the surface collection system to increase its capacity, it is expected that its total capacity to deliver flows to the tunnel system will not exceed the runoff from an 8.40-in (21 cm) storm. Nevertheless, the pumps proposed have adequate capacity to pump flows from such excessive storms if the long outfall is by-passed and an alternate short outfall is employed.

Based on the set of curves shown on Fig. 1, the total volume of rainfall in one day expected for a storm frequency of about once in 15 years is about 5-in (13 cm) over the entire tributary area. From the curves, it is apparent that a depth of 5-in (13 cm) would be expected from a storm of 48-hours duration about once in 4-1/2



DEPTH OF RAINFALL VS FREQUENCY FOR BOSTON, MASSACHUSETTS

FIG. 1

years, and from a 4-day duration storm once in 3 years. Four inches (10 cm) would be expected from a one-day storm about once in 5 years. The curves in Fig. 1 were developed from data in United States Weather Bureau Technical Papers No. 40 and 49 for recurrence intervals from 2 to 100 years. Data for recurrence intervals from 2 months to 2 years were based on analysis of Boston rainfall records for the 10-year period 1955 through 1964.

As a result of these studies, it appeared reasonable that a deep tunnel storage plan could be constructed that would handle the runoff resulting from a 15-year frequency rain storm of 24-hour duration (total rainfall depth 5-in (13 cm) and dispose of this runoff within a 2-day period without surcharging the tunnels at any time. If the tunnels are permitted to surcharge, the runoff from a storm equal to the maximum which has been experienced in Boston may be handled. Essentially such a deep tunnel plan would eliminate all overflows to Boston Harbor and adjacent waters and practically all flooding of land areas and basements.

Storage vs. Pumping

There are numerous alternative arrangements possible in the development of a deep tunnel plan with relation to the volume of storage and the rate of pumping. These alternatives range from an arrangement of maximum pumping capacity with no effective storage to very large volume of storage and minimum pumping capacity. The estimated cost of a deep tunnel storage plan is dependent in large measure on the cost to excavate rock. During the course of studies to determine reasonable tunnel capacities, eight separate arrangements ranging from a pumping rate of 1370 cfs (2340 cu m/min) and 35 miles (55 km) of 33 ft (10 m) diameter storage tunnels to a pumping capacity of

2400 cfs (4100 cu m/min) and 15 miles (24 km) of 33 ft (10 m) diameter storage tunnels were investigated. It was determined from studies of relative costs that the cost increase connected with increasing the pumping capacity is far less than the cost of increasing the storage capacity. It was therefore concluded that a length of storage tunnels of about 17.2 miles (27 km) should be provided.

This analysis resulted, after some refinement, in a required minimum pumping capacity of about 2400 cfs (4100 cu m /min) to handle a 5-in (13 cm) rainstorm, together with a storage volume equivalent to a length of about 17.2 miles (27 km) of 33 ft (10m) diameter tunnels, if the tunnels are not permitted to surcharge. By permitting the tunnels to surcharge for a rainstorm of 8.40-in (21 cm) in 24 hours, a pumping capacity of about 5200 cfs (8850 cu m/min) may be obtained using the same pumps as are required for 2400 cfs (4100 cu m/min) without surcharging. Moreover, for rainstorms in excess of 8.40-in (21 cm) in 24 hours the same pumps could serve if the whole flow were discharged to the sea through a short outfall at the pumping station at Deer Island.

Fig. 2, "Deep Tunnel Storage Volume and Pumping Rates", is a mass or cumulative curve of inflow to the proposed main pumping station. It indicates that pumping at a continuous rate of 2400 cfs (4100 cu m/min) starting at about the 5th hour following the start of the 5-in (13 cm) design rainfall will empty the tunnel storage reservoir by the end of the 36th hour. For a pumping rate of about 5200 cfs (8850 cu m /min) the tunnels could be emptied in a shorter period of time.

Alternatives Considered in the Development of the Proposed Deep Tunnel Plan

As mentioned before, many alternatives were considered in developing the proposed Deep Tunnel Plan. The size and length of tunnels, the depth and length of outfalls, the size and location of chambers and the main pumping station all were variables. After much

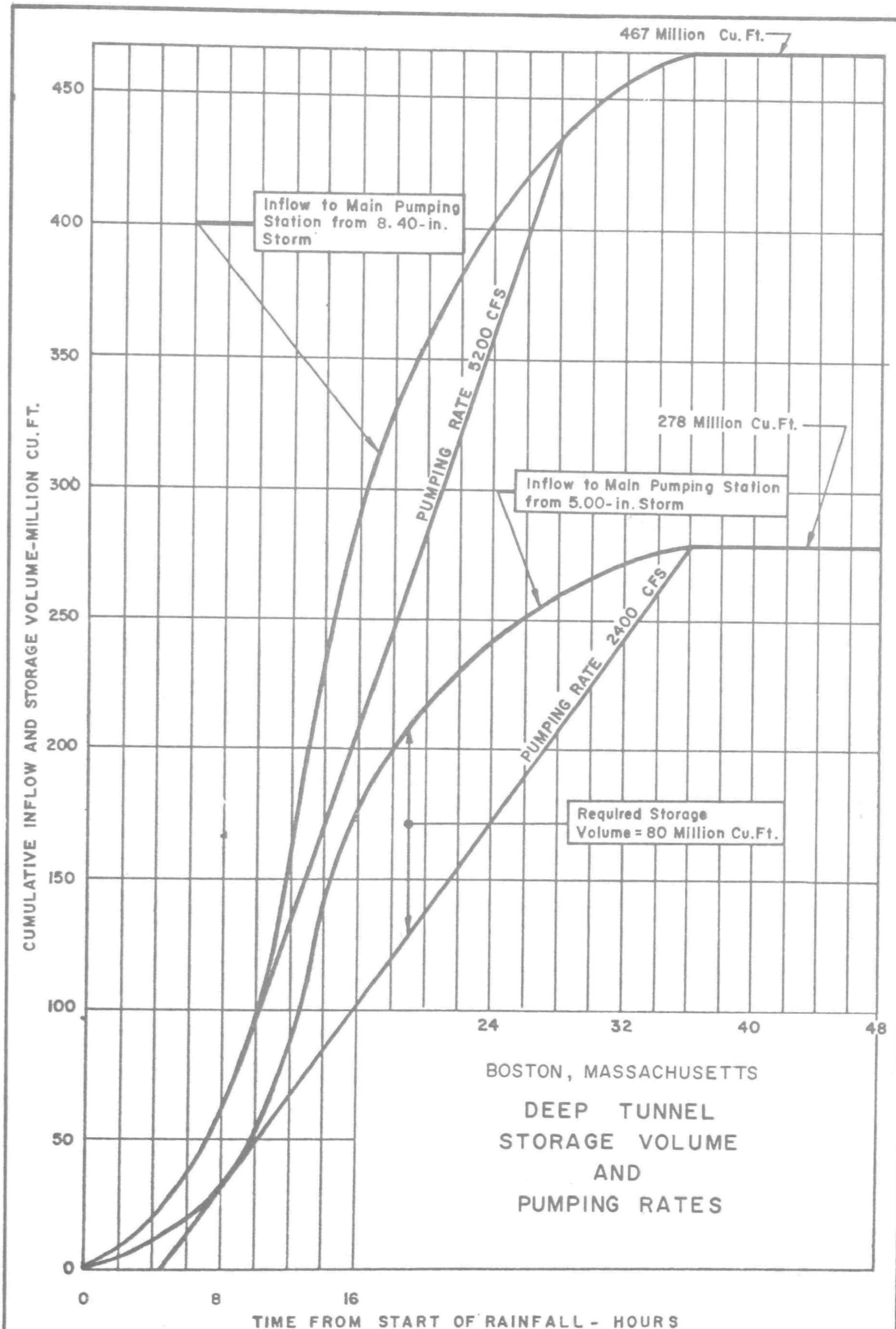

467 Million Cu. Ft.
Inflow to Main Pumping Station from 8.40-in. Storm
PUMPING RATE 5200 CFS
278 Million Cu. Ft.
Inflow to Main Pumping Station from 5.00-in. Storm
PUMPING RATE 2400 CFS
Required Storage Volume = 80 Million Cu. Ft.
CUMULATIVE INFLOW AND STORAGE VOLUME-MILLION CU. FT.
TIME FROM START OF RAINFALL - HOURS
BOSTON, MASSACHUSETTS
DEEP TUNNEL
STORAGE VOLUME
AND
PUMPING RATES

FIG. 2

consideration, the radial storage tunnel arrangement shown on the plan of Fig. 3 was adopted in order to most effectively locate the tunnels such that access to them for all parts of the area served by combined sewer outfalls would be achieved.

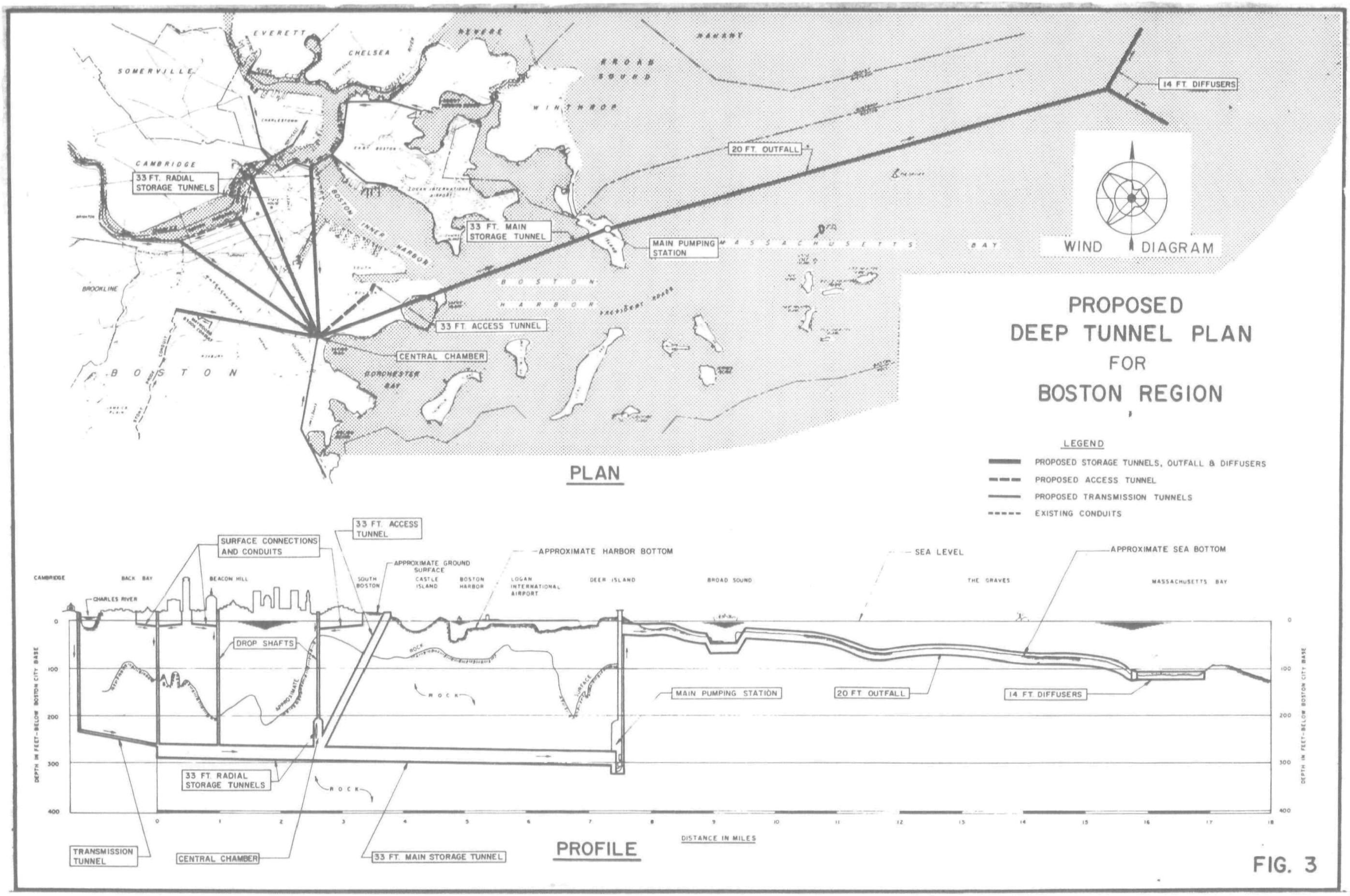
Ocean Outfall

Proper disposal of sewage effluent or mixed sewage and storm water to large bodies of water requires an effective mingling of those polluted waters within the water body to prevent the identification of the discharged wastewater, prevent odor nuisance, and reduce bacterial concentrations. It was concluded, therefore, that the disposal of all mixed sewage and storm water through a long outfall to sea would be the most effective means of abating the pollution of Boston Harbor.

During these studies and in conjunction with the design of the proposed main pumping station, seven different pumping rates through a single pipe outfall and a double pipe outfall were considered. Preliminary costs were prepared for each alternative and compared with the corresponding cost estimates for the tunnels and the pumping station.

The most economical combination included a single 9.5 mile (15 km) 20 ft (6 m) diameter outfall with twin 14 ft (4.3 m) diameter diffuser pipes. At pumping rate of 2400 cfs (4100 cu m/min) the velocity in the outfall pipe would be about 7.5 fps, (2.3 mps) and with 7-in (18 cm) diameter nozzles in the diffuser pipes 15 fps (4.6 mps) nozzle velocities would be achieved.

Our studies indicate that with a pumping rate of 2400 cfs, (4100 cu m/min) a nozzle discharge of 4 cfs (6.8 cu m/min) and a water depth of about 110 ft (33.4 m) at the diffusers, an estimated dilution ratio of about 200 parts of sea water to 1 part wastewater would be achieved in the rising plume of wastewater, if the dispersing effect of ocean currents is ignored.



PROPOSED
DEEP TUNNEL PLAN
FOR
BOSTON REGION
PLAN
EVERETT
CHELSEA
SOMERVILLE
REVERE
NAHANT
BROAD SOUND
WINTHROP
CAMBRIDGE
33 FT. RADIAL STORAGE TUNNELS
33 FT. MAIN STORAGE TUNNEL
20 FT. OUTFALL
14 FT. DIFFUSERS
MAIN PUMPING STATION
MASSACHUSETTS BAY
BOSTON INNER HARBOR
BOSTON HARBOR
33 FT. ACCESS TUNNEL
CENTRAL CHAMBER
BROOKLINE
BOSTON
DORCHESTER BAY
WIND DIAGRAM
LEGEND
PROPOSED STORAGE TUNNELS, OUTFALL & DIFFUSERS
PROPOSED ACCESS TUNNEL
PROPOSED TRANSMISSION TUNNELS
EXISTING CONDUITS
SURFACE CONNECTIONS AND CONDUITS
33 FT. ACCESS TUNNEL
APPROXIMATE GROUND SURFACE
APPROXIMATE HARBOR BOTTOM
SEA LEVEL
APPROXIMATE SEA BOTTOM
CAMBRIDGE
CHARLES RIVER
BACK BAY
BEACON HILL
SOUTH BOSTON
CASTLE ISLAND
BOSTON HARBOR
LOGAN INTERNATIONAL AIRPORT
DEER ISLAND
BROAD SOUND
THE GRAVES
MASSACHUSETTS BAY
DROP SHAFTS
ROCK
MAIN PUMPING STATION
20 FT. OUTFALL
14 FT. DIFFUSERS
DEPTH IN FEET-BELOW BOSTON CITY BASE
33 FT. RADIAL STORAGE TUNNELS
TRANSMISSION TUNNEL
CENTRAL CHAMBER
33 FT. MAIN STORAGE TUNNEL
PROFILE
DISTANCE IN MILES

FIG. 3

As a result of factors such as ocean currents, distance towards shore and composition of the wastewater, dilution ratios are expected to range from about 200 to 1 to perhaps 6,000 to 1. Comparable reductions in the concentrations of bacteria and viruses and other polluting substances would, as a result, range from about 99.50 per cent to 99.98 per cent even without chlorination. These reductions are to be compared with about 90 to 95 per cent removal of organic matter to be expected by conventional "complete" treatment. In other words, the concentration of polluting substances remaining after treatment by this method may be less than 10 per cent of that which can be expected following conventional treatment facilities. In order to provide positive kill of bacteria and viruses, it is proposed that a heavy chlorine dose be applied throughout the year for protection of recreation and shellfish taking.

The alternative of discharging the storm runoff into a surface storage reservoir at Deer Island and passing it through the existing sewage treatment plant at a controlled rate following a storm was considered but not recommended because:

1. An expenditure of 60 million dollars for a surface storage reservoir did not appear feasible because the mixed sewage and storm water volume must still be disposed of within about a 2-day period, either through the plant or through a separate outlet, in order to have the reservoir empty before the next storm.
2. The efficiency of the existing sewage treatment plant would be reduced for extended periods following storms.
3. The operation, cleaning and maintenance of such a surface reservoir will require large expenditures.

Description of Proposed Deep Tunnel Plan

The proposed Deep Tunnel Plan has been developed for collection and disposal of mixed sewage and storm water flows from 17,000 acres (6900 ha) in the Boston region and is shown on Fig. 3. The plan comprises the following principal elements:

1. Surface connections consisting of interception chambers located on outlet conduits downstream of existing or proposed control chambers which will divert dry weather flows to the existing sewerage system where it will receive primary treatment and chlorination at the existing MDC treatment works at Deer Island. Excess flows of mixed sewage and storm water flows would be discharged to the drop shafts, described below, by the surface conduits.
2. Drop shafts, either vertical or inclined, to conduct the flows from the surface connections to transmission tunnels or directly to the deep rock tunnels.
3. Transmission tunnels in rock to carry flows from drop shafts to the storage tunnels.
4. An underground reservoir consisting of a system of 33 ft. (10 m) diameter deep rock storage tunnels in a radial pattern sloping gently to a central chamber located at Columbus Park, and a 33 ft. (10 m) diameter main storage tunnel extending from the Central Chamber at Columbus Park beneath Boston Harbor to a main pumping station at Deer Island, as shown by heavy black lines on Fig. 3. The total length of five radial storage tunnels is about 12.7 miles (20 km). The main storage tunnel would be about 4.5 miles (7.1 km) in length and would be approximately parallel to and on the south side of the existing MDC sewage tunnel (not shown on Fig. 3). The total length of storage tunnels is, therefore, about 17.2 miles (27 km).
5. A central chamber located in rock, with sluice gates, tunnel ventilation and control facilities.

6. A sloping access tunnel extending from the central chamber to the vicinity of the Reserved Channel in South Boston. Its purpose would be to provide access during construction and for maintenance and inspection purposes thereafter.

7. A main pumping station located in a rock chamber at Deer Island with control building, power supply and chlorination facilities, etc., in a surface structure.

8. A 20 ft. (6 m) diameter subaqueous outfall pipe extending about 45,000 ft. (13,700 m) generally east northeast into Massachusetts Bay terminating in two 14 ft. (4.2 m) diameter diffuser pipes, each about 5800 ft. long (1770 m).

The estimated cost of the Proposed Deep Tunnel Plan is as follows:

Item	Estimated Cost million dollars
Deep Storage Tunnels (including Central Chamber and Drop Shafts)	213.0
Main Pumping Station (Deer Island	39.0
Ocean Outfall and Diffusers	54.0
Surface Connections	26.0
Transmission Tunnels (including drop shafts)	88.0
Separation (of minor areas)	10.0
TOTAL ESTIMATED CONSTRUCTION COST	430.0
Capitalized Annual Operation and Maintenance Cost (at 4% interest)	66.0
TOTAL ESTIMATED COST (for comparison with alternative schemes)	496.0

Pertinent Features of the Proposed Deep Tunnel Plan

Main Pumping Station

The location of the Main Pumping Station would be to the east of the existing MDC sewage treatment plant on Deer Island. It would consist of a circular chamber some 180 ft. (55 m) in diameter excavated in solid rock.

The station would have design capacities of 2400 cfs (4100 cu m/min) at a total head of about 350 ft (106 m) with required operating horsepower of about 110,000 and 5200 cfs (8850 cu m/min) at a total head of about 200 ft (60 m) with a maximum required horsepower of about 150,000 with the tunnels surcharged.

Storage Tunnels

The storage tunnels consisting of the five radial storage tunnels and the main storage tunnel to Deer Island are proposed to be excavated to cross sectional area equivalent to about a 33 ft (10 m) diameter circle.

The method of construction of these tunnels at the present time would appear to be by the drill and blast method. This project was discussed with contractors experienced in tunnel work. The access tunnel sloping at about 8 per cent grade from the ground surface to the central chamber at Columbus Park was proposed as an efficient means for access to the area while the tunnels are being constructed, for easy transportation of the muck to the surface for disposal either as fill in the immediate area or on barges to be disposed of elsewhere. The length of the five radial storage tunnels under Boston is such that the transportation of the muck from the tunnels to the surface should pose no unusual tunnel construction. It is thought at this time that the sides and top of the tunnels would not have to be lined except where unstable rock is encountered or where rock bolts are needed for stability. The cost of the deep storage tunnels includes 25 per cent of the tunnel length full lined, 75 per cent of the tunnel length with paved invert only and 40 per cent of the tunnel length supported by rock bolts. The bottom of the tunnels is proposed to be lined with concrete to assist in the operation and maintenance of the tunnel system and also to provide a relatively smooth surface on which the contractors' trucks may operate.

Because the primary function of these tunnels is to provide storage volume and not flow capacity for hydraulic transmission, the shape of the tunnel cross section is not critical, and a horseshoe or other section could be used instead of the circular section if it appears more advantageous and economical.

The depth of the storage tunnels is about 300 ft (91 m) below the surface. The required depth is controlled by the location of the rock surface along its profile. The invert of one radial storage tunnel and the main storage are proposed to be slightly below that of the existing MDC Boston sewage tunnel to permit dewatering the existing tunnel if required.

Considerable research and experimentation on rock boring machines (moles) with rotary cutting heads in diameters as large as 33 to 36 ft (10 to 11 m) is being done in this country and in Europe. The rock formations in Boston are hard and of varying strength. It appears likely that in the next five to ten years the excavation of hard rock by rock boring machines will become routine. If the proposed tunnels can be constructed by machines, the interior of the tunnels will be of circular cross-section and quite smooth, eliminating in general the need for concrete linings or inverts. Of great significance is the probability that the development of rock boring machines will reduce excavation costs for rock such that the costs of tunnels excavated by boring machines may within a few years be substantially less than those of tunnels excavated by drilling and blasting methods.

Ocean Outfall and Diffusers

The outfall is proposed to extend 45,000 ft. (13,700 m) generally east northeast into Massachusetts Bay. The pipe would be of reinforced concrete and be provided with special flexible joints. The pipe would be laid on the bottom of the bay in a trench sufficiently deep to prevent movement of the pipe. It would be buried where it crosses the main ship channel.

The diameter of the diffuser nozzles would be 7-in (18 cm) with a spacing on each side of the diffuser pipe of about 19.2 ft (5.8 m). The diffuser pipes would be located at approximately right angles to prevailing ocean currents in the area. The diffuser pipes would be located at about 110 ft (33 m) below mean sea level.

Advantages of the Proposed Deep Tunnel Plan for the Boston Region

1. The Deep Tunnel Plan provides the best and most practical regional solution to the problem of handling mixed sewage and storm water and assures the abatement of water pollution due to both sewage and surface runoff.
2. It is adaptable to serve any conceivable development in the region in the future and is the most economical of the methods studied for eliminating overflows to the surrounding waters. This plan may become relatively less expensive in the future as rock boring technology improves.
3. The Deep Tunnel Plan will occupy very little valuable land area, its construction will not cause interference with traffic or surface activities and it will permit the efficient draining of all areas that now flood during heavy rains and high tides.
4. The Deep Tunnel Plan provides the means for safely disposing of all polluted surface water and sewage well out to sea away from any inhabitated areas.

5. Sections of the deep storage tunnels will parallel the MDC sewerage tunnel and have lower inverts to complement the existing MDC sewerage system.

6. The large quantity of rock excavated from the tunnels will be available at low cost for fill in connection with the expansion of Logan International Airport, site development for the proposed 1976 Worlds Fair or other fill operations in and around Boston Harbor.

Conclusions

It is not reasonable to expect the City of Boston alone to effectively dispose of its mixed sewage and storm water overflows unless neighboring cities and towns having similar combined systems and overflow problems do likewise. For this reason, the Deep Tunnel Plan should be constructed as a regional operation.

Although the proposed Deep Tunnel Plan is less expensive than complete separation of the system, it nevertheless represents a major expenditure. At the present time State and Federal grants do not appear adequate, either in funds or in scope of existing grant programs to materially assist in the construction of such a proposed plan.

The City of Boston has adopted this plan and has presented it to the State and the Federal Government as its solution to the total water pollution problem.

A concerted effort by these large cities to join together and obtain substantial financial assistance from the Federal Government appears the only feasible means for correcting the mixed sewage and storm water overflow problem in many of the larger cities in the U. S.

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