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SIMULATION OF CONTROL ALTERNATIVES FOR COMBINED SEWER OVERFLOWS

Donald L. Ray

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SIMULATION OF CONTROL ALTERNATIVES
FOR COMBINED SEWER OVERFLOWS

A DISSERTATION PRESENTED

By

Donald Lee Ray

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VITA

Donald L. Ray was born October 12, 1938 in Nashville, Tennessee. He received a Bachelor of Science in Civil Engineering from Tennessee Technological University in 1962. From 1962 to 1967 he was associated with the Tennessee Highway Department, Anthony Papuchis Engineering Company, and Turner Engineering Company in Nashville as a project engineer.

In 1967 he entered the Environmental Engineering Program in the Civil Engineering Department of the University of Massachusetts. In 1968 he received a Master of Science in Civil Engineering and continued studies for a degree of Doctor of Philosophy.

In 1971 he became an assistant professor of Civil Engineering and Engineering Mechanics at The Cleveland State University, Cleveland, Ohio.

He is a registered professional engineer in Tennessee and Ohio, and a member of the American Society of Civil Engineers and the Water Pollution Control Federation.

SIMULATION OF CONTROL ALTERNATIVES FOR COMBINED SEWER OVERFLOWS (August 1972)

Donald L. Ray
B.S., Tennessee Technological University
M.S., University of Massachusetts
Directed by: Dr. Donald D. Adrian

In the past, the significance of combined sewers, occasionally discharging into streams and lakes, was not recognized or considered to be of major importance. As progress was made in the abatement of pollution from domestic, industrial and commercial sources, increased attention needed to be directed toward controlling or treating combined sewer overflows. In many locations the resulting pollution caused by the overflows prevented the attainment of the desired quality of water.

The "state of the art" concerning combined sewers was summarized. This consisted of a review of the history and development of combined sewer systems; determination of the extent and scope of these systems in the United States; a critique of basic engineering assumptions used in developing criteria dealing with the overflows; the presently used and proposed methods of control and/or treatment; and an estimate of the potential cost of alleviating the problem.

A mathematical model was developed which would describe the overflow occurrence hydraulically. This model generates synthetic hourly rainfall intensities, routes the excess rainfall over the surface to combined sewer inlets and then through the combined sewer to the point of discharge. Such a model permits the determination of the expected

frequency of occurrence of an overflow period; the total time overflows would occur; the expected duration of an overflow; the time interval between overflow occurrence; the volume discharged to the receiving stream; and the flowrate at various locations in the sewer system.

A proposed method of controlling the combined sewer overflows by injecting the excess flow into underground formations was also investigated. A laboratory simulator was constructed which would model the receiving aquifer in the vicinity of the injection well. The potential clogging of the formation by waste material, the effectiveness of reducing contaminant concentrations, and the extent of travel of the contaminant were evaluated.

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R E P O R T # 1

SCOPE, RESULTS, CONCLUSIONS AND RECOMMENDATIONS

of a

SIMULATION OF CONTROL ALTERNATIVES

FOR COMBINED SEWER OVERFLOWS

R E P O R T # 1

SCOPE, RESULTS, CONCLUSIONS AND RECOMMENDATIONS

Significance of Combined Sewer Overflows

In the past, the significance of combined sewers, occasionally discharging directly into streams and lakes, was not recognized or considered to be of major importance. This was understandable when one considers the gross pollution caused by the continuous discharge of untreated or inadequately treated waste from domestic, industrial and commercial sources. However, as progress was made in the abatement of pollution from these primary sources, increased attention needed to be directed towards controlling or treating combined sewer overflows. In many locations the resulting pollution caused by the overflows prevented the attainment of the desired quality of water even if the primary wastes were completely treated.

A combined sewer, as shown in Figure 1-1, conveys sanitary waste, consisting of domestic, industrial and commercial wastes, at all times. This flow is referred to as the dry weather flow (DWF) and includes a normal amount of infiltration which seeps into the sewers because of defective joints, pipes and manholes. During rainstorms or thaw periods, these sewers will collect and convey surface runoff in addition to the dry weather flow, and the total flow may be 10 to 100 times as great as the dry weather flow. The combined flow far exceeds the capacity of the interceptors, pump stations, and treatment facilities, which are normally

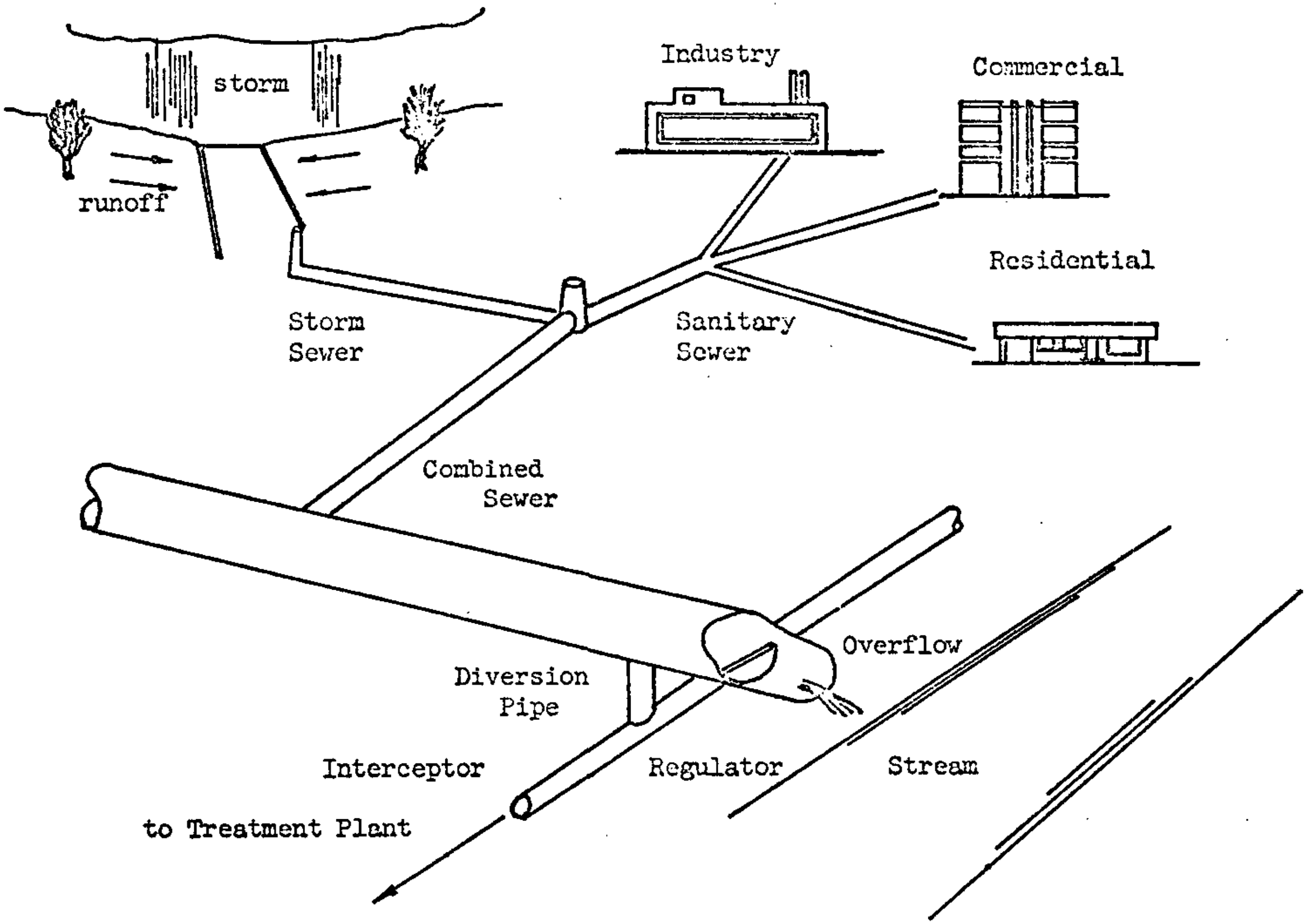

Figure 1-1 Combined Sewer System

designed to handle 3 to 5 times the dry weather flow, and consequently, local flooding or by-passing of the excess flow to the nearest water course results.

In a report (1) prepared by the U. S. Public Health Service in 1964, the importance and extent of the combined sewer overflow problem was first noted. Several generalizations as to the characteristics of these overflows were:

(a) The annual average combined sewer overflow volume represents approximately 5 per cent of the total pollutional discharges into the nation's water courses;

(b) The average overflow from a combined sewer may contain from 3 to 5 per cent raw sanitary waste; and

(c) During storm peaks, as much as 95 per cent of the sanitary waste may discharge directly to the receiving stream.

In another study (2) conducted in Northampton, England, it was revealed that the cumulative yearly biochemical oxygen demand (BOD) load in the combined sewer was approximately equal to the BOD load discharged in the effluent from the city's secondary treatment plant. Suspended solids loads were three times greater than the treatment plant effluent load.

The problems caused by these combined overflows and the different ways proposed for alleviating the problems will be reviewed in Report #2 of this study. The existence of combined sewers in the United States, and the pollutional effects of overflows from such sewers, is directly interwoven with the growth and development of urban areas. The types of sewers and waste treatment facilities which are installed during the coming years of growth, and the method in which they are operated and maintained, will to a large extent determine how effectively America will protect its vital water resources against the ravages of pollution. The large amount of urban renewal work now in progress in many cities presents an opportune situation whereby corrective measures could be made to existing sewerage problems.

Objectives of the Study

There were three basic objectives of this study. First, was to summarize the present "state of the art" concerning combined sewer overflows. This consisted of a review of the history and development of combined sewer systems; determination of the extent and scope of these systems in the United States; a critique of basic engineering assumptions used in developing criteria for dealing with the overflows; the presently used and proposed methods of control and/or treatment; and an estimate of the potential cost of alleviating the problem.

The second objective was to develop a mathematical model which would describe the overflow occurrence hydraulically. This model would use rainfall data as the input parameter and flow routing techniques to determine the resulting flow at any point in the drainage systems. Such a model would permit the determination of the expected frequency of occurrence of the overflow; the total time overflows would occur; the duration of an individual overflow; the time interval between overflow occurrences; the volume discharging to a receiving stream; and the flowrate at the various points in the sewer system. These parameters would then serve as a basis for formulating optimal methods for controlling and/or treating combined sewer overflows.

The third objective was to investigate the feasibility of injecting combined sewer overflows into underground formations. This required an evaluation of the potential clogging of the formation by waste material; the effectiveness of the underground formation in reducing contaminants; and the extent of travel of the contaminants.

Study Format

The dissertation is presented as four separate reports. This report (Report #1) contains the introductory statements, a summary of the results and conclusions of the complete study, and recommendations for further investigations. A brief review of related studies completed or underway at the University of Massachusetts is also included.

Report #2 is a summary of much of the data and information published concerning combined sewers and a list of references pertaining to the subject.

Report #3 presents a mathematical model and computer programs for generating synthetic hourly rainfall intensities and routing the surface runoff through the combined sewer system to the point of discharge into the receiving stream.

Report #4 contains a review of recharge operations, a description of a laboratory apparatus for simulating injection operations and results obtained from the simulator.

The purpose of this format was two-fold. First, the reader who is interested in only certain aspects of the study will find the material complete in one report. Secondly, the material is arranged so individual papers could be submitted for publication without a great amount of editing.

Summary of Results

In Report #3 a mathematical model was developed to generate synthetic hourly rainfall intensity, route the excess rainfall over the surface to combined sewer inlets and then through the combined sewer. Each month of the year must be considered independently and results for the month of June are given as an example of the process. The data given by the generated hydrographs were summarized on probability paper to permit the designer to reach decisions as to the likelihood of certain events occurring. The effectiveness of increasing the capacity of the interceptor sewers and treatment plant in reducing the quantity of overflow discharged to the receiving stream is also given.

The results from the laboratory simulation indicate that the injection of combined sewer overflow does not result in excessive clogging, if settleable waste are removed prior to injection. Carbonaceous waste and coliform bacteria were significantly reduced in the sixteen feet of aquifer model. The removal mechanism for carbonaceous waste was concluded to be aerobic biological decomposition and dissolved oxygen was removed from the injected water in the process. Coliform bacteria was removed by filtration and adsorption on the soil particles.

Since settleable solids were not present in the injected waste, solids removal was negligible. Colloidal solids and dissolved solid behaved as a conservative constituent.

Conclusions and Recommendations

In reviewing the problem of combined sewer overflows, it is apparent that there is a real deficiency in available information describing the quantity and quality characteristics of the combined sewer flow. A number of drainage basins with varying characteristics should be equipped with monitoring stations which would be operated for several years so as to accumulated accurate information as to the nature of the overflows.

The practice of requiring separate sanitary and storm sewerage systems is far from an ideal solution. Evidence indicates that the storm runoff is sufficiently contaminated in urban areas to require treatment. It is also concluded that installation of tertiary on advanced waste treatment processes would be illogical in municipalities where the combined sewer overflows are not controlled first.

Conclusions and recommendations resulting from the "Simulation of Flow Hydrographs from Combined Sewer Systems" are as follows.

The rainfall intensity generation model is adequate for synthesizing long term hourly rainfall data where record lengths are insufficient for an investigation of a large number of different storm events. Further work in this area is not needed.

The surface runoff model is simplified by using a modified rational method. More sophisticated models which would take into account antecedant rainfall, infiltration into the soil, surface detention, overland flow and gutter flow, should be developed. Studies such as those conducted by British Road Research Laboratory (3), the dimensionless hydrograph method proposed by Izzard (4), and the methods proposed by Tholin and Keifer (5) for use in Chicago, Illinois, could be used in the development of this model.

The implicit finite difference solution of the St. Venant equations provides a basis for determining the unsteady, gradually varying flow in circular conduits. Further work is needed in several areas, however.

(a) Modification of the model should be made to account for situations in which the sewer conduit is surcharged and flowing partially under pressure.

(b) The estimation and importance of the friction slope in obtaining stable solutions should be investigated and improved methods for including this parameter determined.

(c) Incorporation into the model of the effects of variation in roughness coefficient with depth should be studied.

(d) Additional laboratory verification of the solution should be undertaken.

(e) Finally, the entire model should be tested on a real situation.

An accurate description of a selected urban drainage basin should be determined and flow from the sewers monitored to corroborate the computer results.

The results of the laboratory feasibility study for injecting combined overflows into underground formations were favorable. Additional work should be conducted with the following emphasis and incorporating the following recommendations:

(a) A mathematical model describing the movement of pollutants in the injected water is needed. This model should take into account the combined mechanisms of convection, dispersion, and diffusion in predicting the flow of conservative pollutants, and the removal mechanisms such as filtration, adsorption and biological decomposition in predicting the concentration of non-conservative pollutants.

(b) Additional study into flow conditions at the entrance to the aquifer, where the actual piezometric curve does not correspond to the curve predicted by Darcy's equation.

(c) Improved techniques for placing the soil medium in the simulator are required to prevent washing or shifting of the soil after injection commences.

The legitimate controversy concerning the use of underground formations for disposal of waste must be noted. Although the controversy is presently focused on the injection of industrial waste into deep-wells, the arguments could be made against injecting combined overflows. One important difference, however, is the operational scheme in which injected waste will be withdrawn after each storm and treated in a normal manner.

The United States Geologic Survey is presently conducting a nationwide survey on the status of subsurface storage and other basic research in cause-effect relationships. A statement by Robert Stallman of the USGS (6) at the first national symposium on the subject entitled, "Underground Waste Management and Environmental Implications" in Houston, Texas, December, 1971, summarized the lack of knowledge presently available;

> "The available theories are either so simplified that they do not represent the real system adequately, or they are so complex that they have not been tested. I believe that the fundamental model of dispersion as related to permeability and the potential field, is now the greatest deficiency in the theory needed for prediction of effects due to waste disposal. A third area of technological weakness is the mathematical construction of predictive equations. Literally, all of our equations are based on the microscopic scale. Models based on the macroscale are needed."

Stanley Greenfield, the Environmental Protection Agency's assistant administrator for Research and Monitoring presented the EPA positions (6).

> "at the present time, we in EPA neither oppose nor promote deep-well waste injection. We regard it as an important technique with great possibility for both benefit and harm."

The argument for using the earth to dispose of waste is summed up by Ralph A. Olsen, Department of Chemistry, Montana State University (7).

> "The ability of the earth, with its extremely large adsorptive surfaces, its very diverse population of micro-organisms, to subsequently adsorb and precipitate and decompose and transform an enormous array of waste products, makes it a far more effective medium for waste disposal than the air we breathe or the water we drink."

Related Studies

Several related research projects at the University of Massachusetts have been undertaken to help answer questions raised in this investigation. Lepri (8) presented a master's thesis entitled "Physical and Chemical Characterization of Combined Sewer Overflows". In his study automatic sampling and flow recording equipment were used to monitor an overflow in Springfield, Massachusetts.

Jubboori (9) presented a doctoral dissertation to the Department of Soil Physics entitled "Injection of Sediment Containing Water Into Permeable Unconfined Formations". In his study a diluted settled waste was injected into a laboratory simulator model of an unconfined aquifer. The study would serve as a companion to the study presented in this dissertation where settled waste was injected into a simulated confined aquifer.

Martel and Misiaszek are presently conducting a study in partial fulfillment of requirements for a Master of Science degree in Environmental Engineering in which a conservative tracer is injected into the confined aquifer simulator. The resulting data will be used to obtain analytical and numerical solutions of the dispersion characteristics of the soil media in the simulator.

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R E P O R T # 2

REVIEW OF COMBINED SEWER OVERFLOW PROBLEMS

of a

SIMULATION OF CONTROL ALTERNATIVES

FOR COMBINED SEWER OVERFLOWS

R E P O R T # 2

REVIEW OF COMBINED SEWER OVERFLOW PROBLEMS

Historical Background

The development of sewerage systems in the United States generally followed European practices; the main objective was to collect, transport and dispose of stormwater. The disposal of domestic, commercial and industrial waste was the responsibility of the individual and discharge of such sanitary waste into the storm sewers was not allowed.

When authorities lifted this ban, it was only natural that the large capacity storm sewers would be used to transport sanitary waste as well. In addition, specifically designed combined sewers were constructed rather than providing separate conduits for stormwater and wastewater because of the lower investment costs.

It was much later that officials became concerned about the pollutional damage caused by untreated waste being discharged into receiving waters and began to advocate the use of separate sanitary and storm sewers and the accompanying treatment of the sanitary waste. Since it wasn't feasible to treat the entire flow from the existing combined sewers, diversion structures were designed to divert the dry weather flow to an interceptor sewer which conveyed the waste to a treatment facility. The remaining excess flow was allowed to overflow directly into the receiving stream. This was considered acceptable since the stormwater causing the overflow would tend to dilute the sanitary waste minimizing the effect of the pollution.

Extent of Problem

Combined sewers are usually located in the core of older cities which have high population densities and commecial activities. These cities and areas are most commonly found in the Northeastern and Mid-Atlantic regions of the nation and in the area around the Great Lakes.

The resulting pollution from combined sewer overflows is not solely confined to urban areas (1). In many cases, separate sewerage systems were constructed in suburban areas, which still discharged into combined sewers used in the central city. Thus, the once separated sanitary flow became combined with the storm runoff and overflowed.

In an effort to define the extent of combined sewer usage, the American Public Works Association conducted a survey of representative communities served wholly or partially by combined sewers (2). The results of this survey were published in 1967, and some of the findings are as follows.

The jurisdictions surveyed had a total population of 51 million people and an area of 6.5 million acres. Of the 51 million people included in the survey, 34 million were served directly by combined sewers. The area served by combined sewers was found to be 2.6 million acres with 2 million acres being unsewered. Projecting this data to include the entire nation, it is estimated that 18 per cent of a total population would be directly served by combined sewers.

Resulting Pollution

The effects of combined sewers overflowing into a receiving stream are similar to that of any discharge of sanitary waste for a short period of time. Oxygen is consumed by bacteria and inorganic matter; recreational areas are closed because of high coliform concentrations; aesthetic value is destroyed due to unsightly floating material and obnoxious odors; fish and other wildlife sanctuaries are ruined; and downstream use of the water resource is impaired because of the general deterioration in the quality of the water.

Little effort has been made until recently to obtain detailed information about the characteristics of the overflows. This lack of information has contributed to the uncertainty as to the extent of damage resulting in the receiving stream and the scarcity of adequate control and treatment processes. Useful information is extremely difficult to obtain because of the wide variation in flows and pollutant levels. Thus, sampling must be conducted for long periods of time and automatic recording equipment must be used.

In the past, engineers have minimized the effects of combined sewer overflows by hypothesizing that the increased stream flow, and the increased flow in the combined sewer resulting from a rainfall, would dilute any concentrations of pollutants which were present. It was also hypothesized that the first portion of any overflow would be the most polluted and consequently interception of 3 to 5 times the normal dry weather flow was provided. Neither of these hypotheses were necessarily correct.

The dilution hypothesis fails to consider that during periods of dry weather, the flow in the large diameter combined sewers would be very small, and because of the low velocity, solids would settle out in the sewer and could become anaerobic. Then when a storm occurred, the flow would increase, diluting the sanitary waste, but also resuspending any solids that had previously been deposited. This resuspension can cause higher pollutant concentrations than would otherwise be expected, if only sanitary waste was present.

Another difficulty in the hypothesis is that the river may respond slowly to the additional runoff, while the runoff which is routed through the combined sewers could reach the stream quickly. Because of this, the peak flows of the river and the overflow may occur at different times with a resulting lag period. Under these conditions, the dilution afforded the overflow by the stream would be of little benefit.

Recognition of the fact that resuspension of the deposited solids occurs during periods of high flow led to the hypothesis of a "first flush". It was reasoned that the first portion of runoff would flush the solids out of the combined sewers and also cleanse any pollutional material deposited on the land surface, and therefore, would be the most polluted portion of the overflow. Thus, if this first portion of overflow was intercepted and conveyed to the treatment plant, the remainder of the overflow would cause little damage. However, results from various tests which have been conducted, have failed to substantiate this in all cases. The occurrence of the peak pollutional loading is dependent on the hydraulics of the sewer system; the elapsed time since the previous storm runoff;

the intensity and duration of the current storm; and the characteristics of the runoff drainage area. Depending on these conditions, the peak pollution may occur anytime during the overflow or may be non-existant with high pollutional concentration throughout the overflow.

Most data that have been reported in the literature are based on composite samples, which "average" the concentrations encountered during the entire overflow period. This type of data reporting is not as useful, or meaningful, as data that gives concentrations as a function of elapsed time since overflow commenced. Knowing the concentrations of pollutants as a function of time would permit pollutographs to be developed in the same manner as hydrographs which describe variations in flows with time. These pollutographs would be very helpful in formulating methods of control and treatment which would minimize the pollutional effect of overflows.

The Federal Water Quality Administration sponsored a study, during 1965 in the Detroit-Ann Arbor area, to compare the characteristics of combined and separate storm overflows. Results were reported by Burm, Krawczyk and Harlow (3) for the chemical and physical characteristics, and by Burm and Vaughan(4) for the bacteriological characteristics. The data presented gave average values for individual storms, average concentrations as a function of elapsed time since the overflow started, average values for different seasons of the year, and average annual values. Annual average values and the maximum value observed were given as follows:

Analysis	Annual Mean	Max. Value Observed
BOD, mg/l	153	685
Sus. Solids, mg/l	274	804
Vol. S.S., mg/l	117	452
Sett. Solids, mg/l	238	656
Vol. Sett. Solids, mg/l	97	376
NH_3-N, mg/l	12.6	134
Organic N, mg/l	3.7	38.4
NO_3, N, mg/l	0.5	2.8
Total PO_4, mg/l	14.6	43.2
Soluble PO_4, mg/l	7.7	21.2
Phenols, mg/l	312	8000

BOD values were found to be higher during the spring of the year, and the mean BOD values for time periods at the start of the overflow were significantly higher than those observed during the remainder of the overflow. This would indicate a "first flush" phenomena; however, other parameters such as nitrogen, suspended solids, and volatile suspended solids did not follow this pattern.

Another study was conducted by Palmer (5) in the Detroit area, and estimates of the average quality of the overflows were:

BOD	50 mg/l
SS	250 mg/l
VSS	100 mg/l

It was stated that plus variations of as much as 100 per cent and minus variations of 50 per cent might occur.

Riis-Carstensen studied an area in Buffalo, N. Y. (6), and overflow resulting from a rainfall of 0.55 inches was characterized as follows:

BOD	100 mg/l
SS	544 mg/l
VSS	350 mg/l
Total Solids	754 mg/l

Again wide variations were observed in the strength of the overflow.

Reports of bacteriological characteristics are equally as scarce as chemical and physical values. Palmer (5) estimated that average coliform counts would be 4,300,000 organism/100 ml in the combined sewer overflow with, again, 100 per cent plus variations and 50 per cent minus variations.

The bacteriological study by Burm and Vaughan(4) reported the monthly values of total coliform, fecal coliform, and fecal streptococci as shown in Table (2-1). Several conclusions from this study were made as follows:

(1) Total coliform densities may approach those found in raw wastewater.

(2) Total coliforms in individual samples may consist entirely of fecal coliform. In general the fecal coliform should be approximately 20 per cent of the total coliform.

(3) Total coliform and fecal coliform densities increased during warm months, but fecal streptococcus counts remained relatively constant.

(4) Initial counts are about the same or slightly higher than the remaining counts. The effects of first flushing on bacteriological characteristics were therefore minimal.

From these reports it is apparent that overflows from combined sewers can have high levels of pollution and in some cases may be as polluted as raw sanitary waste.

TABLE 2-1

Monthly Average Bacteriological Concentrations of

Discharges from Combined Sewers in the Detroit Area (4).

Month	Analysis	Mean Density of Geometric Mean x 10^6	Range of Geometric Mean x 10^6
March	Total coliform	2.30	1.10 - 8.50
	Fecal coliform		
	Fecal streptoccoci		
April	Total coliform	2.40	0.57 - 8.20
	Fecal coliform	0.89	0.19 - 2.60
	Fecal streptoccoci		
May	Total coliform	4.40	1.90 - 24.00
	Fecal coliform	1.50	0.41 - 3.60
	Fecal streptoccoci	0.32	0.28 - 1.50
June	Total coliform	12.00	1.30 - 41.00
	Fecal coliform	2.70	0.47 - 8.70
	Fecal streptococci	0.74	0.66 - 0.98
July	Total coliform	37.00	12.00 - 45.00
	Fecal coliform	7.60	1.90 - 10.50
	Fecal streptoccoci	0.35	0.18 - 0.49
August	Total coliform	26.00	12.00 - 45.00
	Fecal coliform	4.40	1.40 - 16.00
	Fecal streptoccoci	0.53	0.41 - 0.79
September	Total coliform	22.00	3.20 -110.00
	Fecal coliform	5.40	3.20 - 20.00
	Fecal streptoccoci	0.60	0.20 - 0.90

Proposed Solutions

The Environmental Protection Agency, Water Quality Office (formerly the Federal Water Quality Administration) is sponsoring a wide variety of research projects directed toward finding solutions to the problem of combined sewer overflows. The solutions which have been proposed can be categorized into three basic approaches; control, treatment, and a combination of the two. Because of the nature of the problem, it is unlikely that any one method will be a panacea for all cases. The optimum solution for any given location will depend on the existing sewer hydraulics, topography, land use patterns, availability of surface storage area, rainfall and runoff characteristics, location of overflow points, capacity of municipal waste treatment facilities, water quality standards, and many other local factors (7).

Control methods relate to the physical containment of the combined waste overflow by storage, diversions, or routing techniques. The overflow is not treated and eventually must be transported to a treatment facility or discharged into the receiving water.

Separation of the sanitary flow from the stormwater runoff would be a control method. This is usually envisioned as providing two separate sewer systems. However, the use of special types of conveyance systems such as a pressure or vacuum pipes to carry sanitary waste inside of larger storm sewers have been studied (8). This will, of course, obliterate the possibility of the highly contaminated sanitary waste being mixed with the storm runoff. Recent investigations have shown however, that the stormwater may be highly polluted itself, and could pose serious pollutional problems (9).

Many public works officials have expressed doubts that complete separation can ever be obtained (2). Illicit sewer connections are difficult to locate and in some cases even harder to correct. In densely populated urban areas and in downtown commercial and business districts, the effect of tearing up streets to allow trench construction for new sewers, would be chaotic and the cost astronomical.

A useful compromise to complete separation would be to partially separate the flows as proposed in Toronto, Canada (10). A new system of storm sewers would be constructed to accommodate runoffs from streets and parking areas. No attempt would be made to intercept stormwater runoff from roofs or yard drains. It was estimated that 30 to 60 per cent of the total runoff would be diverted from the combined sewers to the new "road" sewers.

The American Society of Civil Engineers has conducted a study to determine the feasibility of using pressure sewers as a means of separating sanitary waste from stormwater (8). A pump-storage-grinder unit has been developed to pump waste from individual buildings into pressure tubing inserted into the existing combined sewer and transporting the sanitary waste to interceptors or treatment plants. Storm runoff would still be conveyed by gravity sewers.

Many different types of storage systems have been proposed to control the overflow. The storage capacity can be either on the surface or underground and may be either an integral part of the sewer system or located in a separate storage chamber.

Palmer (11) recommended that the City of Detroit provide adequate volumetric capacity in the sewers to accommodate all storms of intensity less than 1.0 in/6 hours. This would reduce the annual number of overflows from 84 to 5, and would divert 99.6 per cent of the sanitary waste to the waste treatment plant.

The Minneapolis-St. Paul Sanitary District is installing systems for maximizing control within the sewerage system (12). Regulators are operated from a central control station based on flow conditions at critical points in the system. Utilizing computer capabilities to simulate rainfall and flow levels, storm flows are diverted to the interceptor in a manner which makes maximum use of the interceptor storage capacity. Regulators consist of positive control gates and inflated rubberized-fabric dams.

Off system storage in detention tanks is widely used in Europe and to a lesser extent in the United States. Examples of this method can be found in McComb and Wayne Counties in Michigan (13); in Columbus, Ohio (14); and in Boston, Massachusetts (15). Surface lagoons and retention basins are also being utilized in Chippewa Falls, Wisconsin (16); Springfield and Shelbyville, Illinois; and in East Chicago, Indiana (17). Build up of settleable solids and algae growth have caused some operational difficulties.

The use of deep storage tunnels constructed far below the surface is receiving much attention. The most widely publicized concept is that proposed for Chicago (18). Here all of the combined sewers will discharge into the underground tunnels through drop shafts at the points of overflow. The estimated cost of 400 million dollars is only 10 per cent of the cost of the complete separation. The tunnels will be 33 to 50 feet in diameter and will be bored 200 to 400 feet below the surface, providing 360 million cubic feet of storage. The tunnels will function as a huge surge tank storing the overflows which exceed the capacity of the treatment plants. After the flow has peaked, the stored water in the underground tunnels will be pumped out and conveyed by interceptors to Chicago's waste treatment plants.

Boston, Massachusetts is also considering a deep tunnel project similar to Chicago's (19). Estimated cost of the tunnels is considerably less than the cost for complete separation. It is anticipated that rock-boring machines with rotary cutting heads of diameters as large as 33 to 36 feet will be used. As experience in the use of these machines increases, it is anticipated that solutions of this type will become more attractive.

Another type of storage container is that proposed in Washington, D. C. (20). Inflatable synthetic rubber tanks will be anchored along the bottom of rivers and overflows will be diverted into the empty tanks which will cause the tank to inflate. After the storm flow has decreased, the overflow will be pumped back into interceptors. Estimated size of the tanks would be 120 x 120 x 7 feet.

Treatment of the combined sewer overflows at individual remotely located outflows is another possible solution. Biological, physical and chemical processes which are normally used in wastewater treatment plants have been proposed. It is usually necessary that modifications to these processes be made because of the intermittent and varying nature of the flow.

Physical methods would include screening, straining, filtration, sedimentation, and other hydraulic solids removal methods. Standard sedimentation tanks require large amounts of surface area and this has resulted in efforts to use mechanical devices. One of these is a high-rate, fine-mesh screen device tested in Seattle, Washington (21). In preliminary testing it was found that the efficiency of the unit was comparable to the efficiency obtainable in a primary clarifier. Microstrainers have been tested in Philadelphia and have also proven to be as efficient in the removal of suspended solids and BOD as primary clarifiers (22).

Chemical processes using polymers to improve sedimentation rates, chemical oxidation, and new or improved methods of disinfection have also been studied. Stormwater disinfection is being attempted in New Orleans in an effort to reclaim use of bathing beaches on Lake Pontchartrain (23). Flows as high as 11,000 cfs are disinfected using a sodium hypochlorite solution. A hypochlorite batching plant has been constructed to provide the necessary quantity of disinfectant.

In an effort to increase the efficiency of existing combined sewers and to relieve constrictions which cause localized flooding, high molecular weight polymers have been found to increase the flow in sewers as much as 2 to 3 times (24). Studies are now being made to improve techniques and methods of adding the polymers when needed.

Biological treatment systems are difficult to apply to combined sewer overflows since they function best under conditions of "steady-state" flow. However, a rotating biological contactor in which a biological growth attaches itself to a series of rotating disks, which move through the wastewater, has shown some promise (7). Efficiencies comparable to that of trickling filters have been found under steady loading rates and whether or not this will be suitable for unsteady loadings is being studied.

A solution to the problem of varying flows and loadings would be to provide a holding tank prior to the biological unit which could be operated so as to even out the flows and loadings. Such a method would be an example of the combination approach, referred to previously, in which control and treatment are both required in order to obtain the desired levels of water quality. Other combinations might be: in-system or off-system storage for subsequent treatment in specially designed facilities; and temporary or short-term storage of collected stormwater followed by treatment. In Mt. Clemons, Michigan (7) small artificial lakes are used in series to store combined sewer overflows, which are treated by aeration with surface aerators. In addition to this, the effluent from the first two lakes is passed through a microstrainer before entering the third lake which should be of such quality as to permit boating and fishing.

Estimated Cost of Various Solutions

The cost estimates for the proposed solutions vary widely and are dependent on many factors. In the study made by the USPHS (25), it is estimated that the cost of complete separation of combined sewer into component storm and sanitary sewers would be approximately 30 billion dollars. Including the expense that would be incurred in making necessary plumbing changes for private residences and other buildings would increase the cost to approximately 48 billion dollars. These estimations probably are of theoretical interest only, as most officials of the highly populated municipalities consider the likelihood of being able to convert to separate sewers as remote.

Use of various alternative solutions would reduce the cost of separation by one-half to approximately 15 billion dollars. In addition the expense of separating plumbing on private property would be unnecessary. Adequate data to project cost of these alternatives are not available, however, it is likely that a combination of methods will provide the optimum solution. The feasibility of each alternative or combination will need to be evaluated recognizing local conditions.

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R E P O R T # 3

SIMULATION OF FLOW HYDROGRAPHS

FROM COMBINED SEWER SYSTEMS

of a

SIMULATION OF CONTROL ALTERNATIVES

FOR COMBINED SEWER OVERFLOWS

R E P O R T # 3

SIMULATION OF FLOW HYDROGRAPHS FROM COMBINED SEWER SYSTEMS

Need for Simulation

In order to adequately study the problem of combined sewer overflows, it was necessary to develop an improved methodology for determining the volume of the overflow and the distribution of the flow with time. Direct measurements of these overflows are seldom attempted and to initiate such programs would be of little immediate benefit, since long periods of records would be required to determine overflow characteristics. Because of this difficulty, efforts are made to relate the overflow occurrences to hourly rainfall intensity.

Predictions of flood flows in stream channels resulting from excess rainfall and runoff from the drainage basin are often accomplished by hydrologists. The unsteady, gradually varying flows are described by the St. Venant equations and various solutions have been developed which utilize computer capabilities.

Traditionally the flow in sewers was approximated to be uniform and of constant depth. The determination of flow variations with time was not attempted. Eagleson (1) proposed a solution based on the unit hydrograph concept for determining hydrographs at particular points in the sewer system. This method requires previous measurements of flows at that point and is only applicable to storms of specified duration.

In a recent comprehensive study of storm water management conducted by Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc., for the Environmental Protection Agency (2), a routing method was developed which avoided the necessity of solving the St. Venant equations. In their Transport Model, the continuity relationship was employed, but iterative techniques were used in evaluating the dynamic characteristics of the flow.

In this report, solution techniques used in streamflow routing are modified, and an implicit finite difference solution derived to solve the complete St. Venant equations for flow in sewers rather than natural channels.

Hourly Rainfall Generation

Description of rainfall process. Rainfall intensity records are available from the National Weather Service of the National Oceanic and Atmospheric Administration, which maintains recording gages throughout the United States. These gages measure total depth of rainfall at a point during each clock hour to the nearest 0.01 inch. In many cases, however, the available records are too short to permit adequate consideration of all the various combinations of rainfall sequences. The use of stochastic processes offers one method of generating much longer periods of record. The method to be used in this study was developed by Pattison (3) for use in the Stanford Watershed model.

The amount of rain which falls during consecutive clock hours can be considered as a sequence of discrete random variables, H_t. The random variables are not independent, and in fact, there is a pronounced dependency between hourly rainfall which occurs during consecutive hours.[1]

1 See appendix 3A, page 3-56

Generation of synthetic hourly rainfall data can be accomplished by using a Markov chain process (4). In this process, the probability of a system being in a given state depends only on the knowledge of the state of the system at each of the immediately preceding "n" time periods. If the present state of the system is dependent on only the preceding period, the process is referred to as a first order Markov chain. If the process depends on the two preceding periods, then the process is said to be a second order Markov chain, and so forth.

It has been observed that a first order Markov chain fails to describe the transition between a sequence of wet hours and a sequence of dry hours. The reason for this is the occurrence of an hour of zero rainfall, at the end of a wet sequence, is considered by the process to be the start of a sequence of dry hours. It is apparent from the observation of the natural rainfall process, that this is not necessarily the case, and there may be many short periods of zero rainfall occurring during a sequence of wet hours.

This characteristic is most important and results in the consideration of Markov models which are greater than first order. The conditional probability of an hour being wet given "n" immediately preceding dry hours decreases as "n" increases and approaches a constant value for any given locality. In the Springfield-Hartford area, this value of "n" is approximately 6 hours.[2] This indicates that the dependency between a dry hour and the intensity of rainfall during the next hour is dependent only on the previous 6 hours. Therefore, a sixth order Markov chain is used, when the present intensity is zero, to determine if the rainfall intensity

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2 See appendix 3B, page 3-58

during the next hour is greater than zero. For hours when there is a measureable rainfall, a first order process is adequate.

A Markov chain model can now be formulated that will assume the characteristics of a first or sixth order chain depending on which of the two following conditions are applicable. The first condition is that if the intensity of the hourly rainfall during hour (t) is greater than zero, then first order dependence characteristics are used to determine the rainfall intensity during the hour (t+1). The second condition is when the intensity of the rainfall during hour (t) is equal to zero. Then sixth order dependence characteristics are used to determine if the intensity of the rainfall during hour (t+1) is greater than or equal to zero.

Computer model. A computer program (RAINSYN)[3] was written to generate long term hourly rainfall records using the description of the process given in the previous section.

Hourly rainfall records gathered at the National Weather Service, Windsor Locks, Connecticut station, were used to estimate the transitional probabilities which control the operation of the model. The month of June was selected to illustrate the rainfall generation process, and data representing the hourly rainfalls for the month of June, from 1950 to 1969, were read into the computer. Generation of rainfall data for other months of the year could be accomplished using the same procedure illustrated in this section.

This information was then reduced to intervals of rainfall intensities (S_t) as shown in Table 3-1 to reduce the complexity of computations. These intervals formed the states of a reduced Markov chain and were selected so

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3 See appendix 3C, page 3-61for flowchart.

TABLE 3-1

Interval Grouping of Hourly Rainfalls

Reduced Hourly Rainfall State	Hourly Rainfall Interval inches/hour
1	0.00
2	0.01
3	0.02
4	0.03-0.05
5	0.06-0.10
6	0.11-0.20
7	0.21-0.30
8	0.31-0.40
9	0.41-0.50
10	0.51-0.60
11	0.61-0.70
12	0.71-0.80
13	0.81-0.90
14	0.91-1.00
15	1.01 >

that the

$$P(S_{t+1}=s \mid S_t=r) = \sum_{j \varepsilon J_s} P_{ij} \tag{3-1}$$

and would have the same values for $i \varepsilon I_r$, where

I_r = interval of values of H_t corresponding to $S_t=r$

J_s = interval of values of H_{t+1} corresponding to $S_{t+1}=s$

P_{ij} = probability that $H_{t+1}=j$ given that $H_t=i$

The transitional probabilities that describe the likelihood of H_t being included in interval r (r=2,3,...,15) and H_{t+1} being included in interval s (s=1,2,...,15) were estimated as

$$P(S_{t+1}=s \mid S_t=r) = \frac{f_{rs}}{F_r} \tag{3-2}$$

where

$$F_r = \sum_{s=1}^{15} f_{rs}$$

and, f_{rs} is the number of times that a transition from $S_t=r$ to $S_{t+1}=s$ occurs in the historical record.

Similarly the transitional probabilities that describe the likelihood of H_t being included in interval r=1 and H_{t+1} being included in interval s (s=2,3...,15) were estimated as

$$P(S_{t+1}=s \mid S_t=1) = \frac{f_{1s}}{F_1} \tag{3-3}$$

where

$$F_1 = \sum_{s=2}^{15} f_{1s}$$

The transitional probabilities that determine the likelihood of

H_{t+1} being dry (S_{t+1}=1) given that H_t is dry (S_t=1) are described by the sixth-order Markov chain. Therefore, 32 different combinations of wet and dry hours from t-1 to t-5 must be analyzed. Estimate of these probabilities are given by the following relationship.

$$P(S_{t+1}=1 \mid S_t=1) = g\left[U(S_t), \ldots, U(S_{t-5})\right] \tag{3-4}$$

where

$$g\left[U(S_t), \ldots, U(S_{t-5})\right] = P(S_{t+1}=1 \mid S_t=s_t, \ldots, S_{t-5}=s_{t-5})$$

$$U(S_{t-i}) = \begin{cases} \text{Dry for } S_{t-i}=1 \\ \text{Wet for } S_{t-i}>1 \end{cases}$$

Table 3-2 is the resulting transitional probabilities for the sixth-order chain, and Table 3-3 is for the first-order chain.

The reduced hourly rainfall sequences were next generated using the appropriate transitional matrix and Monte Carlo sampling techniques. The Monte Carlo technique involved selecting a uniformly distributed random variable with values between 0 and 1. A subfunction is available on most computer systems for this. An initial assumption was also made that the first six hours of the analysis period were dry in order to start the procedure described.

The general procedure followed was to first determine if the state of hour (t) was dry. If so, the previous five hours were considered to determine the pattern of wet and dry hours. Then the value of the random variable was selected, and a decision was made as to whether or not the rainfall at hour (t+1) was equal to zero. This was accomplished using the appropriate probabilities depending on the previous 5 hours (Table

TABLE 3-2

Sixth-Order Transitional Probabilities

(Actual Data)

State During Time						State During Time (t+1)	
t-5	t-4	t-3	t-2	t-1	t	Dry	Wet
W	W	W	W	W	D	0.72	0.28
W	W	W	W	D	D	0.91	0.09
W	W	W	D	W	D	0.88	0.13
W	W	W	D	D	D	0.89	0.11
W	W	D	W	W	D	0.63	0.38
W	W	D	W	D	D	0.84	0.16
W	W	D	D	W	D	0.71	0.29
W	W	D	D	D	D	0.94	0.06
W	D	W	W	W	D	0.86	0.14
W	D	W	W	D	D	0.86	0.14
W	D	W	D	W	D	0.33	0.67
W	D	W	D	D	D	0.95	0.05
W	D	D	W	W	D	0.60	0.40
W	D	D	W	D	D	0.88	0.13
W	D	D	D	W	D	1.00	0.00
W	D	D	D	D	D	0.95	0.05
D	W	W	W	W	D	0.82	0.18
D	W	W	W	D	D	0.91	0.09
D	W	W	D	W	D	1.00	0.00
D	W	W	D	D	D	0.93	0.07
D	W	D	W	W	D	1.00	0.00
D	W	D	W	D	D	1.00	0.00
D	W	D	D	W	D	1.00	0.00
D	W	D	D	D	D	0.94	0.06
D	D	W	W	W	D	0.62	0.38
D	D	W	W	D	D	0.96	0.04
D	D	W	D	W	D	0.60	0.40
D	D	W	D	D	D	0.89	0.11
D	D	D	W	W	D	0.84	0.16
D	D	D	W	D	D	0.89	0.11
D	D	D	D	W	D	0.86	0.14
D	D	D	D	D	D	0.99	0.01

TABLE 3-3

First-Order Transitional Probabilities

(Actual Data)

Intensity During Hour (t)	Rainfall Intensity During Hour (t+1), in/hr.														
	.00	.01	.02	.03-.05	.06-.10	.11-.20	.21-.30	.31-.40	.41-.50	.51-.60	.61-.70	.71-.80	.81-.90	.91-1.0	1.01- >
.00	*	.43	.16	.21	.10	.06	.02	.01	.01	.01	0	0	0	0	0
.01	.55	.18	.13	.08	.03	.02	.01	0	0	0	0	0	0	0	0
.02	.42	.20	.10	.19	.05	.02	.01	0	0	0	0	0	.01	0	0
.03-.05	.27	.19	.09	.25	.13	.06	.02	0	0	0	0	0	0	0	0
.06-.10	.17	.09	.07	.21	.27	.16	.02	.01	0	0	0	0	0	0	0
.11-.20	.09	.08	.08	.13	.19	.28	.09	.03	.01	.01	0	0	0	0	0
.21-.30	.17	.06	.03	.17	.20	.17	.14	.06	0	0	0	0	0	0	0
.31-.40	.18	0	0	.18	.09	.27	.09	.09	.09	0	0	0	0	0	0
.41-.50	0	.17	0	0	.33	.17	.17	0	0	.17	0	0	0	0	0
.51-.60	.75	.25	0	0	0	0	0	0	0	0	0	0	0	0	0
.61-.70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.71-.80	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.81-.90	0	0	0	0	0	0	0	0	1.0	0	0	0	0	0	0
.91-.10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1.01- >	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

* See Table 4-2 for transitional probability of going from $H_t = 0$ to $H_{t+1} = 0$. This probability is dependent on the six previous hours. The values given for the probability of going from $H_t = 0$ to $H_{t+1} > 0$ are conditional based on a prior determination that $H_{t+1} > 0$.

3-2). If the state of hour (t+1) was determined to be dry, the process was terminated for that hour and the synthesis moved on to the next hour.

However, if the state of hour (t+1) was determined to be wet, the process had to select the magnitude of S_{t+1} by making another random selection using the probabilities that define the transition from zero rainfall in hour (t) (Table 3-3). The synthesis then moves to the next hour.

If the state of hour (t) was found to be wet, ($S_t > 1$), the state of hour (t+1) was randomly selected using probabilities given in Table 3-3. After the selection of S_{t+1}, the procedure moves to the next hour.

This was repeated until the required length of record was generated. The reduced hourly data was then converted back to hourly rainfall intensities using the following relationship.

$$H_t = f(S_t) + U_{s_t} \tag{3-5}$$

where

$f(S_t)$ = lowest value of H_t in the interval of rainfall, S_t

U_{s_t} = independent uniformly distributed random variable with

$$P(U_{S_t}=k) = \frac{1}{100[f(S_{t+1})-f(S_t)]}$$

for

$$k = 0.00, 0.01, \ldots, \left[f(S_{t+1}) - f(S_t)\right]$$

In order to verify the computer model, transitional probabilities for the generated data were determined and compared with the probabilities obtained from the actual data. These matrices are shown in Tables 3-4 and 3-5, and are very close to values given in Table 3-2 and 3-3.

TABLE 3-4

Sixth-Order Transitional Probabilities

(Synthetic Data)

State During Time						State During Time (t+1)	
t-5	t-4	t-3	t-2	t-1	t	Dry	Wet
W	W	W	W	W	D	0.62	0.38
W	W	W	W	D	D	0.93	0.07
W	W	W	D	W	D	0.85	0.15
W	W	W	D	D	D	0.83	0.17
W	W	D	W	W	D	0.73	0.27
W	W	D	W	D	D	0.84	0.16
W	W	D	D	W	D	0.90	0.10
W	W	D	D	D	D	0.95	0.05
W	D	W	W	W	D	0.63	0.38
W	D	W	W	D	D	0.91	0.09
W	D	W	D	W	D	0.00	1.00
W	D	W	D	D	D	0.94	0.06
W	D	D	W	W	D	0.50	0.50
W	D	D	W	D	D	1.00	0.00
W	D	D	D	W	D	1.00	0.00
W	D	D	D	D	D	0.94	0.06
D	W	W	W	W	D	0.89	0.11
D	W	W	W	D	D	0.95	0.05
D	W	W	D	W	D	1.00	0.00
D	W	W	D	D	D	0.93	0.07
D	W	D	W	W	D	1.00	0.00
D	W	D	W	D	D	1.00	0.00
D	W	D	D	W	D	1.00	0.00
D	W	D	D	D	D	0.93	0.07
D	D	W	W	W	D	0.58	0.42
D	D	W	W	D	D	0.86	0.14
D	D	W	D	W	D	0.67	0.33
D	D	W	D	D	D	0.86	0.14
D	D	D	W	W	D	0.80	0.20
D	D	D	W	D	D	0.93	0.07
D	D	D	D	W	D	0.91	0.09
D	D	D	D	D	D	0.99	0.01

TABLE 3-5

First-Order Transitional Probabilities

(Synthetic Data)

Intensity During Hour (t)	Rainfall Intensity During Hour (t+1), in/hr														
	.00	.01	.02	.03-.05	.06-.10	.11-.20	.21-.30	.31-.40	.41-.50	.51-.60	.61-.70	.71-.80	.81-.90	.91-1.0	1.01- >
.00	*	.37	.17	.26	.11	.05	.02	.01	.02	0	0	0	0	0	0
.01	.58	.18	.11	.08	.02	.04	0	0	0	0	0	0	0	0	0
.02	.40	.17	.12	.22	.05	.01	.03	0	0	0	0	0	.01	0	0
.03-.05	.25	.16	.07	.33	.09	.05	.03	.01	.01	0	0	0	0	0	0
.06-.10	.13	.05	.10	.20	.28	.20	.03	.01	0	0	0	0	0	0	0
.11-.20	.08	.07	.06	.16	.15	.37	.06	.03	.01	.01	0	0	0	0	0
.21-.30	.14	.02	0	.24	.26	.07	.21	.05	0	0	0	0	0	0	0
.31-.40	.23	0	0	.15	.15	.23	0	.08	.15	0	0	0	0	0	0
.41-.50	0	.18	0	0	.36	.27	.18	0	0	0	0	0	0	0	0
.51-.60	.50	.50	0	0	0	0	0	0	0	0	0	0	0	0	0
.61-.70	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.71-.80	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
.81-.90	0	0	0	0	0	0	0	0	1.0	0	0	0	0	0	0
.91-1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1.01- >	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

* See Table 3-4 for transitional probability of going from $H_t = 0$ to $H_{t+1} = 0$. This probability is dependent on the six previous hours. The values given for the probability of going from $H_t = 0$ to $H_{t+1} > 0$ are conditional based on a prior determination that $H_{t+1} > 0$.

Synthesis of Surface Runoff

Description of surface runoff. The next step in the synthesis of combined sewer overflows, is to estimate the amount of total rainfall which becomes surface runoff. This amount of rainfall usually referred to as effective rainfall, is widely recognized by engineers as being less than the total rainfall. Horner (5) has shown that this decrease is the result of rain water being retained on vegetation, infiltrating into the soil, wetting the impervious surfaces, being stored in depressions, and in providing a film of water sufficiently thick to permit flow. All of these factors are normally combined in a "coefficient of runoff" similar to the coefficient "C" which is used in the Rational Method (6). This coefficient takes into account the ratio of the impervious surface area to the total area, and for densely built-up sections of a city has been estimated to be 0.7 to 0.9.

From the above factors, it is apparent that the coefficient of runoff is not constant and would increase as the duration of rainfall increased. This is due to the impervious areas becoming fully wet, surface storage depressions being filled, semi-pervious soils becoming saturated, and a flowing film of water becoming established. For high intensity rainfall lasting over 3 hours the coefficient approaches unity, but for low intensity rainfall the increase is much slower.

Tholin and Keifer (7) developed a detailed methodology for determining rainfall-runoff relationships in urban areas. The study was based on data obtained from 10 acre drainage areas with various types of uniform land use, ranging from suburban residential to industrial and commercial, in Chicago, Illinois. The runoff hydrographs from each area were then evaluated with consideration given to the infiltration capacity of pervious areas; depression storage; overland flow detention; detention in gutters; and detention in the lateral sewers system. It was concluded that for densely populated urban drainage areas, the runoff from pervious surfaces had little effect on sewer design. The runoff from roofs and paved areas caused the peak flow in the sewer to occur well before the runoff from pervious surfaces reached the sewer.

Because of the lack of data for runoff from low intensity rains and the complicated interactions which are involved, the following assumptions, based on McKee's work in Boston, Massachusetts (8), are made in this study. First, only the rainfall which falls on impervious areas is assumed to drain into the sewers. This excludes consideration of any area that is covered by grass, trees, or other vegetation. For low intensity rainfalls, and since the summer months are considered the most critical time periods, the assumption is acceptable. This illustrates the fact that the surface flow is dominated by the runoff contribution from paved areas.

This assumption is also justifiable because of the counter-balancing of the second assumption. After the initial wetting of the impervious surfaces and the filling of depression storage, the entire rain falling upon the impervious surface is considered to drain into the sewers. Hicks (9) has estimated that 10% of the rainfall on impervious surface is lost, because some of the rain on roofs and sidewalks may drain onto pervious surfaces and be absorbed in the soil. This would counter-balance the rain which falls on pervious surfaces and then drains onto paved areas.

The amount of rainfall required to initially wet impervious surfaces, fill storage depressions, and start runoff is not great. Studies by Hicks (9) indicate that the amount is in the vicinity of 0.02 or 0.03 inches. Lepri (10) calculated the amount of precipitation necessary to cause an overflow from the Worthington Street trunk sewer in Springfield, Massachusetts. Based on recorded rainfall and sewer flow data, he found the amount to be 0.0375 inches. Therefore, for this study the effective rainfall will be obtained by deducting the first 0.03 inches of rainfall from the beginning of each storm.

In Lepri's study (10), a total rainfall of 0.09 inches in 6.33 hours was found to produce 6.58 million gallons (m. gals.) of overflow from the Worthington Street drainage area. Antecedent rains had saturated the pervious surfaces and filled depression storage. Deducting the sanitary flow, which was estimated to be 4.5 m. gals. during this time period, the surface runoff was calculated to be 2.12 m. gals. The total available volume of rain water on the 1730 acre drainage area was 4.25 m. gals. Thus, only 50 per cent of the rain water actually flowed through the sewerage system and discharged into the river. This indicates that approximately half of the total area was impervious. This percentage is close to the value estimated from maps and photographs of the drainage area of 40 per cent.

In the following computer model the impervious percentages of each sub-basin were estimated individually and values ranged from 20 per cent in newer residential areas to 90 per cent in the downtown commercial areas.

Computer model. The synthetic hourly rainfall intensities (generated by the RAINSYN computer program) were used to determine the surface runoff from a selected drainage basin in Springfield, Massachusetts. Rainfall cycles were defined as shown in Figure 3-1 and begin with a measureable rainfall in hour (t) and end when six consecutive dry hours have occurred.

The total drainage basin, which contributes to the combined sewer overflow at the receiving stream, was divided into smaller sub-basins, as shown in Figure 3-2. These sub-basins were selected so that they would contribute surface runoff to a certain inlet point in the basin's sewer network. The inlet points were usually manholes which are located where characteristics of the sewers such as diameter or slope changes. All existing manholes were not represented in order to simplify the problem.

The surface runoff from each sub-basin consisted of surface flow and flow in sewers with diameters less than 18 inches. These flows were considered collectively in calculating the total surface runoff at the inlet point.

The runoff from the impervious area in each sub-basin was calculated as

$$Q_t \approx A_i H_t \tag{3-6}$$

where

Q_t = flowrate at the end of hour t, cfs

H_t = intensity of rainfall for hour t, in/hr

A_i = impervious area of the sub-basin, acres

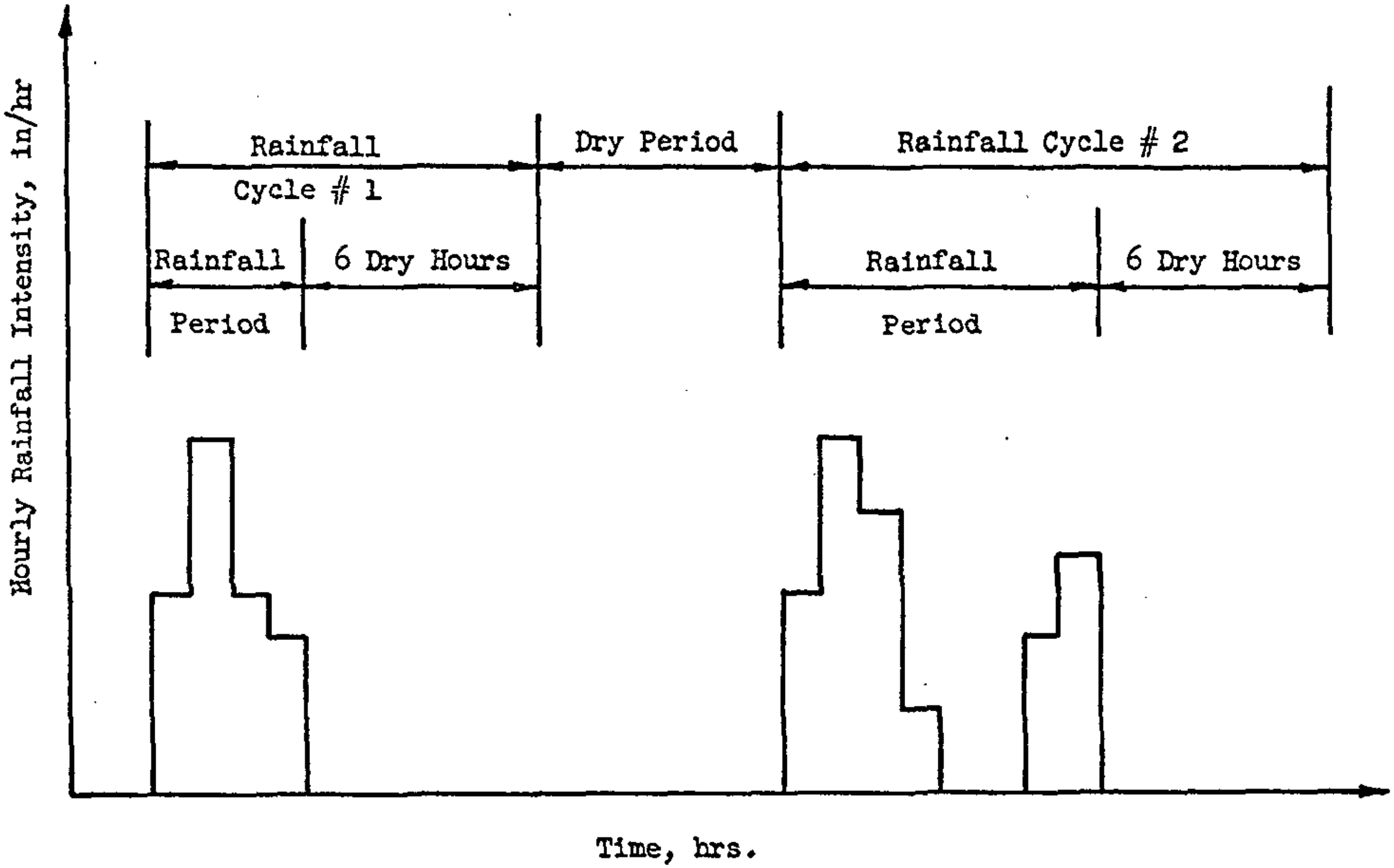
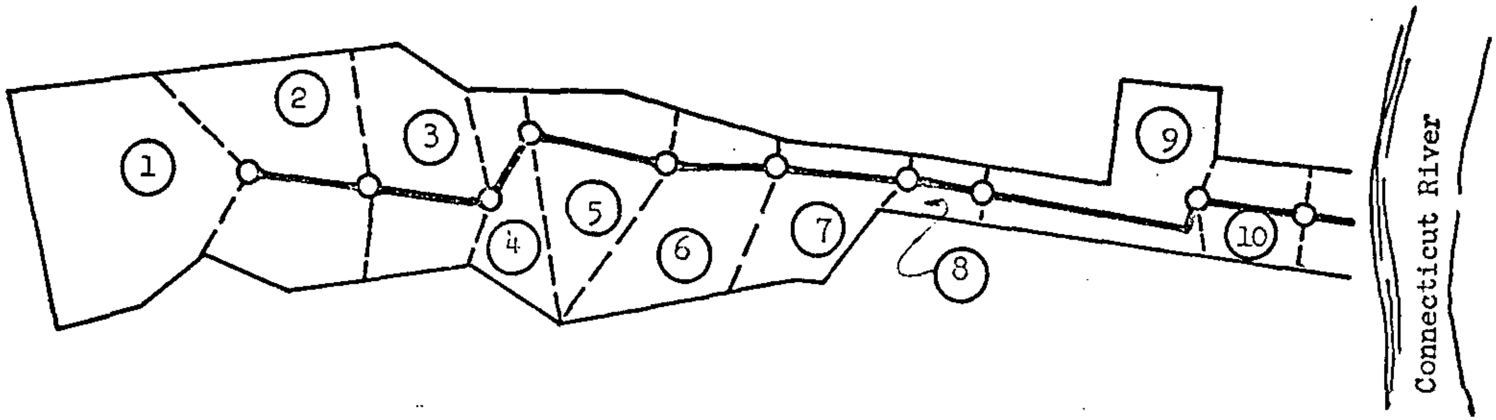


Figure 3-1 Rainfall Cycles



Sub-basin Number	Impervious Area	Length	Diameter	Slope	Roughness Factor
1	53.0	500.0	2.00	0.015	0.015
2	36.0	800.0	2.50	0.015	0.015
3	13.8	500.0	2.75	0.015	0.015
4	13.8	1400.0	3.25	0.010	0.015
5	8.6	2000.0	3.75	0.004	0.015
6	5.0	1500.0	4.25	0.020	0.015
7	5.0	900.0	5.00	0.017	0.015
8	13.2	3100.0	5.00	0.016	0.015
9	40.0	1350.0	7.50	0.010	0.015
10	38.0	1100.0	7.50	0.010	0.015

Figure 3-2 Characteristics and Schematic Layout of Combined Sewer Drainage Area

It is important to note that the values for A_i and H_t in Equation 3-6 must be in the dimensional units given since

$$1 \text{ acre} = 43{,}560 \text{ ft}^2$$

$$1 \text{ in/hr} = 1/(12)(3600) \text{ ft/sec} = 1/43{,}200 \text{ ft/sec}$$

and $(1 \text{ acre})(1 \text{ in/hr}) = (43{,}560 \text{ ft}^2)(1/43{,}200 \text{ ft/sec}) \approx 1 \text{ cfs}$

The first 0.03 inches of rainfall was deducted from each rainfall cycle to account for surface wetting and so forth. The dry-weather flow originating in the sub-basin was assumed to be equivalent to a rainfall intensity of 0.01 in/hr on the impervious area.

Combined Sewer Overflow Synthesis

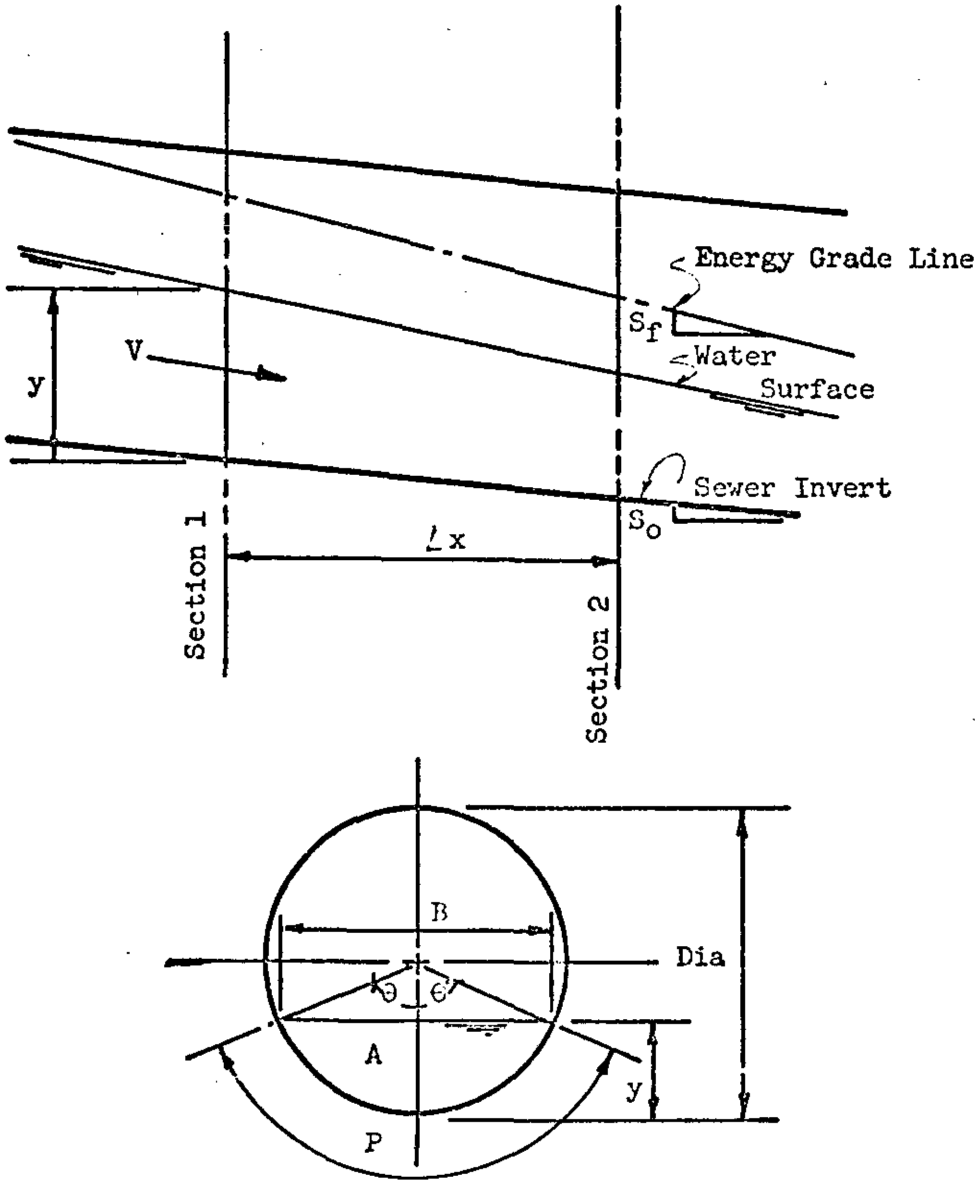
Derivation of Saint-Venant equations. The Saint-Venant equations are used to describe the unsteady, gradually varied flow which would exist when storm runoff is superimposed on the relatively steady dry weather flow in the sewers. These equations are derived from the principles of continuity and momentum and represent a one dimensional flow (11).

The continuity principle states that the rate of mass outflow must equal the rate of accumulation in any control volume. Referring to Figure 3-3, the rate of mass inflow, outflow and accumulation are described for a control volume of area A, length x, and depth y, as:

$$\text{Inflow} = \rho VA + \frac{\partial}{\partial x}(\rho VA)\left(-\frac{\Delta x}{2}\right)$$

$$\text{Outflow} = \rho VA + \frac{\partial}{\partial x}(\rho VA)\left(\frac{\Delta x}{2}\right)$$

$$\text{Accumulation} = \frac{\partial}{\partial t}(\rho A \Delta x)$$



$$\cos\theta = \frac{Dia/2 - y}{Dia/2} \quad (1-2y/Dia) \qquad B = Dia \cdot \sin\theta$$

$$A = Dia^2/4(\theta - \sin\theta\cos\theta) \qquad P = Dia \cdot \theta$$

Figure 3-3 Definition Sketch of Gradually Varying, Unsteady Flow in a Partially Filled Circular Conduit where

ρ = density of liquid, slugs/ft^3

V = average velocity, ft/sec

A = cross-sectional area of flow, ft^2

Δx = incremental length, ft.

Combining these terms according to the continuity principle yields:

$$\rho VA - \frac{\partial}{\partial x}(\rho VA)\left(\frac{\Delta x}{2}\right) - \rho VA - \frac{\partial}{\partial x}(\rho VA)\left(\frac{\Delta x}{2}\right) = \frac{\partial}{\partial t}(\rho A \Delta x) \tag{3-7}$$

Simplifying and dividing by $\rho \Delta x$, and noting from Figure 3-2 that

$$\frac{\partial A}{\partial x} = B\frac{\partial y}{\partial x} \quad \text{and} \quad \frac{\partial A}{\partial t} = \frac{B\partial y}{\partial t}$$

reduces the equation to

$$A\,\frac{\partial V}{\partial x} + V \cdot B\,\frac{\partial y}{\partial x} + B\,\frac{\partial y}{\partial t} = 0 \tag{3-8}$$

From Newton's second law, the momentum relationship is given by

$$F = ma \tag{3-9}$$

Where

F = resultant of all external forces resolved in the direction of flow x, lb

m = mass of the liquid in the control volume, slugs

a = acceleration of the liquid mass, ft/sec^2

The resultant force F is then equal to the vectoral sum of F_p, F_g, and F_f.

F_p is the resultant force due to pressure differences at section one and section two in Figure 3-2 and is equal to

$$F_p = -\rho g A \frac{\partial y}{\partial x} \Delta x \tag{3-10}$$

Where the minus sign is introduced so that a negative value of $\frac{\partial y}{\partial x}$ will produce a positive force in the x direction.

F_g is the component of the weight of liquid acting in the x direction and may be written as

$$F_g = \rho g A \Delta x S_o \tag{3-11}$$

S_o is the slope of the sewer invert, and is approximately equal to sin α for most sewer lines.

F_f is the boundary resistance force exerted in a direction opposite to the flow of liquid by the sewer on the control volume. This force is equal to

$$F_f = -\rho g A \Delta x \frac{V^2}{C^2 R} \tag{3-12}$$

where R is the hydraulic radius of the flow area. Assuming that the boundary resistance for flow at a given depth and mean velocity is the same regardless of whether the flow is uniform or non-uniform, steady or unsteady, the term V^2/C^2R may be set equal to S_f, the so called friction slope, and determined from the Manning Equation. Thus F_f is given by

$$F_f = -\rho g A \Delta x S_f \tag{3-12a}$$

The mass of the liquid in the control volume is equal to

$$m = \rho A \Delta x \tag{3-13}$$

and the acceleration of the liquid is

$$a = \frac{dV}{dt} = \frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} \tag{3-14}$$

Substituting these relationships back into the momentum equation 3-7 and dividing by $\rho A \Delta x$ and rearranging terms gives the momentum equation,

$$g \frac{\partial y}{\partial x} + \frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} = g \left(S_o - S_f\right) \qquad (3\text{-}15)$$

Equation 3-15 and Equation 3-8 are the St. Venant equations which relate the two unknowns y (the depth of flow) and V (the velocity of flow).

<u>Solutions of St. Venant equations</u>. The solution of these first order, nonlinear, partial differential equations is quite complicated. Graphical techniques were used in the past, but now with the advent of large scale electronic digital computers numerical techniques are being used. These numerical techniques can be classified as follows (12).

(a) Development of a rectangular network of characteristics (13).

(b) Explicit-differencing of characteristic equations on a curvilinear net (14).

(c) Explicit finite-differencing of characteristic equations on a rectangular network in the x-t plane (14).

(d) Direct, explicit finite-differencing of the equations of continuity and momentum in a rectangular net (15).

(e) Direct, implicit finite-differencing of the equations of continuity and momentum in a rectangular net (16).

Using the method of characteristics, the St. Venant equations can be reduced to the following differential equations (13).

$$\frac{\partial V}{\partial t} + \left(V \pm \sqrt{\frac{gA}{B}}\right) \frac{\partial V}{\partial x} \pm \sqrt{\frac{gB}{A}} \frac{\partial y}{\partial t} + \left(V \sqrt{\frac{gB}{A}} \pm g\right) \frac{\partial y}{\partial x} = g\left(S_o - S_f\right) \qquad (3\text{-}16)$$

Solving these equations by graphical techniques is a tedious and time consuming process. Stoker (14) has presented a numerical method which replaces the partial derivatives with difference quotients permitting the use of digital computers. One of the problems with this numerical method was that small errors of truncation were sometimes amplified during successive time steps and eventually the pertinent part of the solution was entirely masked. When this occurs, the solutions are referred to as unstable. The Courant-Institute group (15), which included Stoker, modified this method slightly and showed the resulting solutions to be stable, if

$$\frac{\Delta t}{\Delta x} \leq \frac{1}{|V| + \sqrt{gA/B}}$$

This restriction on the ratio of $\Delta t/\Delta x$ results in uneconomical computer use in some instances. If Δx is restricted to values small enough to accurately describe the flow profile, a large number of steps in time would be required to determine the flow profile several hours after the initiation of flow.

Numerical methods have also been used to solve the St. Venant equations directly. Depending on the way in which the partial derivatives are replaced by difference quotients, these solutions may be unstable also. The explicit finite-difference method (15) is formulated so that the partial derivatives with respect to x, and the coefficients are replaced by finite difference quotients and coefficients evaluated on time line (t_j). For example, referring to Figure 3-4 the partial derivatives with respect to t and x, are evaluated at distance line x_k as

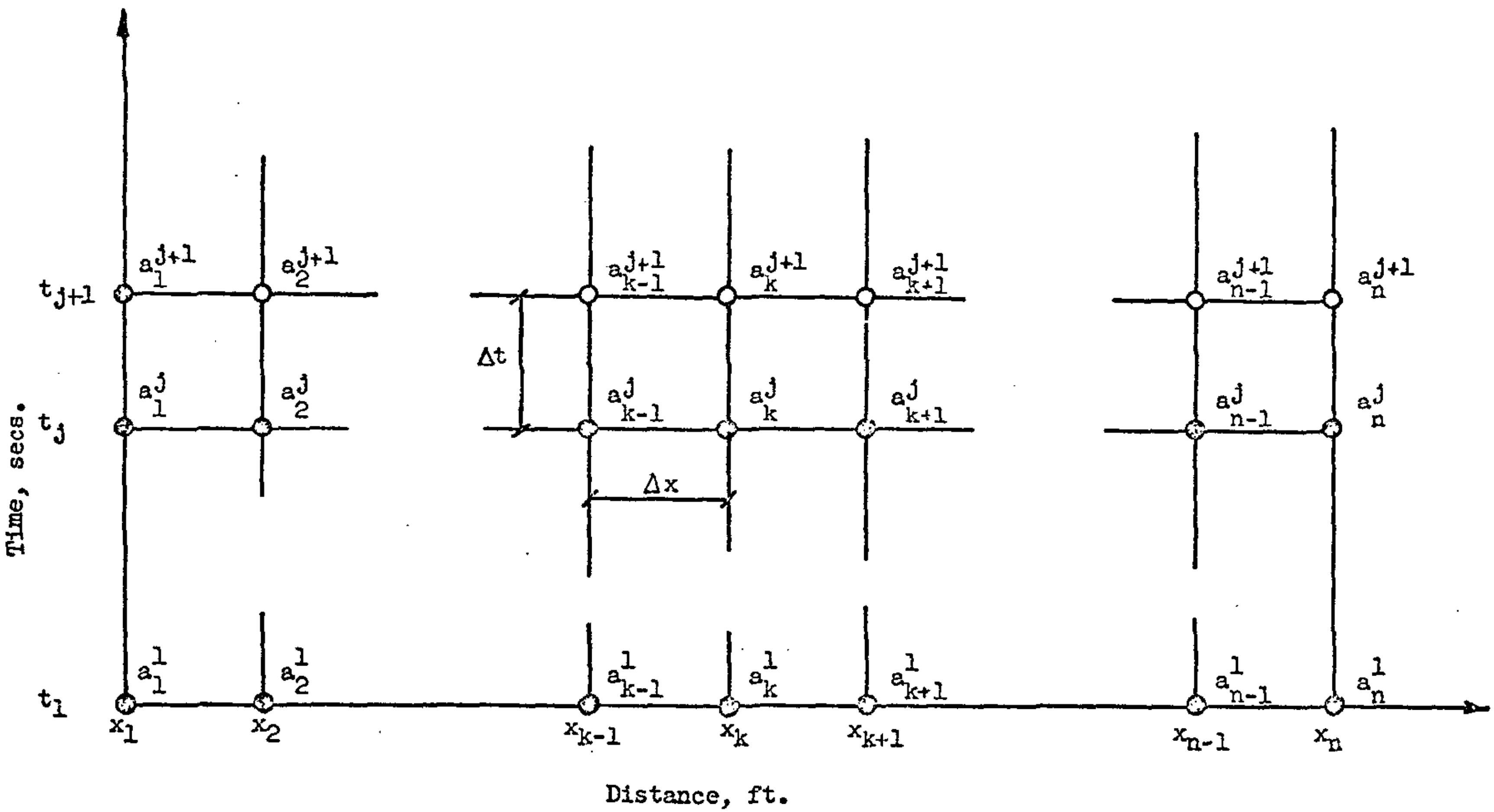


Figure 3-4 Definition Sketch for Finite Difference Method Used in Solving St. Venant Equations

$$\frac{\partial y}{\partial x} \approx \frac{y^j_{k+1} - y^j_{k-1}}{2\Delta x} \qquad \frac{\partial V}{\partial x} \approx \frac{V^j_{k+1} - V^j_{k-1}}{2\Delta x}$$

$$\frac{\partial y}{\partial t} \approx \frac{y^{j+1}_k - y^j_k}{\Delta t} \qquad \frac{\partial V}{\partial t} \approx \frac{V^{j+1}_k - V^j_k}{\Delta t}$$

This results in two linear algebraic equations with two unknowns, y^{j+1}_k and V^{j+1}_k which can easily be solved. The stability of the solution using this method would be dependent on the selected $\Delta t/\Delta x$ ratio.

When implicit finite-differencing methods (16) are used the partial derivatives with respect to x are evaluated on time line (t_{j+1}), and the coefficients, except S_f, are still evaluated on time line (t_j). The partial derivative with respect to t and x, are evaluated at distance line x_k. For example referring to Figure 3-4,

$$\frac{\partial y}{\partial x} \approx \frac{y^{j+1}_{k+1} - y^{j+1}_{k-1}}{2\Delta x} \qquad \frac{\partial V}{\partial x} \approx \frac{V^{j+1}_{k+1} - V^{j+1}_{k-1}}{2\Delta x}$$

$$\frac{\partial y}{\partial t} \approx \frac{y^{j+1}_k - y^j_k}{\Delta t} \qquad \frac{\partial V}{\partial t} \approx \frac{V^{j+1}_k - V^j_k}{\Delta t}$$

This results in more unknowns than equations. However, the successive application of this method to the remaining co-ordinates along the x-axis will produce "n" simultaneous equations with "n" unknowns which can be solved. In the implicit finite difference method, the solution is always stable and is independent of $\Delta t/\Delta x$ ratio.

The friction slope, S_f, must be evaluated along line t_{j+1} and can be approximated from the known values at line t as follows (16). Manning's equation is first rearranged and written as

$$S_f^{1/2} = \frac{V}{(\frac{1.486}{n})(A/P)^{2/3}} \tag{3-17}$$

where

V = average velocity, ft/sec

n = roughness coefficient

A = flow area, ft^2

P = wetted perimeter, ft.

If we define K as a conveyance factor equal to

$$K = \frac{1.486}{n}\left(\frac{A}{P}\right)^{2/3}$$

and square both sides of equation 3-17, we obtain

$$S_f = \frac{V|V|}{K^2} \tag{3-17a}$$

Where $|V|$ represent the absolute value of velocity. The friction slope S_f is a function of velocity (V), and conveyance (K) and S_f may therefore be expanded as follows to yield the following equation.

$$S_{f_k}^{j+1} \cong S_{f_k}^{j} + \left.\frac{\partial S_f}{\partial V}\right|_k^j (V_k^{j+1} - V_k^j) + \left.\frac{\partial S_f}{\partial K}\right|_k^j \left.\frac{dK}{dy}\right|_k^j (y_k^{j+1} - y_k^j) \tag{3-18}$$

where

$$\frac{\partial S_f}{\partial V} = \frac{2S_f}{V}$$

$$\frac{\partial S_f}{\partial K} = -\frac{2S_f}{K}$$

Since the conveyance K is a function of the flow area (A) and the wetted perimeter (P), and A and P are both functions of the depth of flow in the sewer, y, we can evaluate $\frac{dK}{dy}$ as

$$\frac{dK}{dy} = \frac{2}{3}\frac{K}{A}\left(\frac{dA}{dy} - R\frac{dP}{dy}\right) \tag{3-19}$$

Referring to Figure 3-3 which shows the geometry of circular conduit flowing partially filled the following two relationships can be established

$$\frac{dA}{dy} = B$$

and

$$\frac{dP}{dy} = \frac{1}{\sqrt{y/D - (y/D)^2}}$$

Substitution of these relationships back into equation 3-19 yields

$$\frac{dK}{dy} = \frac{2}{3}\frac{K}{A}\left(B - \frac{A}{P\sqrt{y/D - (y/D)^2}}\right) \tag{3-19a}$$

Thus, substituting appropriate coefficients, finite-difference approximation for the partial derivatives, and the approximation of the friction slope into equations 3-8 and 3-15 gives

$$\begin{aligned} &- \frac{V_k^j B_k^j}{2\Delta x}(y_{k-1}^{j+1}) + \frac{B_k^j}{\Delta t}(y_k^{j+1}) + \frac{V_k^j B_k^j}{2\Delta x}(y_{k+1}^{j+1}) - \frac{A_k^j}{2\Delta x}(V_{k-1}^{j+1}) \\ &+ \frac{A_k^j}{2\Delta x}(V_{k+1}^{j+1}) = \frac{B_k^j y_k^j}{\Delta t} \end{aligned} \tag{3-20}$$

$$\begin{aligned} &\frac{-g}{2\Delta x}(y_{k-1}^{j+1}) + g\frac{\partial S_f}{\partial K}\bigg|_k^j \frac{dK}{dy}\bigg|_k^j (y_k^{j+1}) + \frac{g}{2\Delta x}(y_{k+1}^{j+1}) - \frac{V_k^j}{2\Delta x}(V_{k-1}^{j+1}) \\ &+ \left[\frac{1}{\Delta t} + g\frac{\partial S_f}{\partial V}\bigg|_k^j\right](V_k^{j+1}) + \frac{V_k^j}{2\Delta x}(V_{k+1}^{j+1}) \\ &= g\left[S_o - S_{f_k}^{\ j} + \frac{\partial S_f}{\partial V}\bigg|_k^j V_k^j + \frac{\partial S_f}{\partial k}\bigg|_k^j \frac{dK}{dy}\bigg|_k^j y_k^j\right] + \frac{V_k^j}{\Delta t} \end{aligned} \tag{3-21}$$

These two equations are general expressions which can be applied to interior nodes on time line t_j at $x_2, x_3 \ldots x_{n-2}, x_{n-1}$.

At the left hand boundary x_1; y_1^{j+1} and V_1^{j+1} are known from boundary conditions and equations 3-20 and 3-21 may be revised to

$$\frac{B_2^j}{\Delta t}(y_2^{j+1}) + \frac{V_2^j B_2^j}{2\Delta x}(y_3^{j+1}) + \frac{A_2^j}{2\Delta x}(V_3^{j+1}) = \frac{B_2^j y_2^j}{\Delta t} + \frac{V_2^j B_2^j}{2\Delta x} y_1^{j+1} + \frac{A_2^j}{2\Delta x} V_1^{j+1} \tag{3-20a}$$

$$g\frac{\partial S_f}{\partial K}\bigg|_2^j \frac{dK}{dy}\bigg|_2^j (y_2^{j+1}) + \frac{g}{2\Delta x}(y_3^{j+1}) + \left[\frac{1}{\Delta t} + g\frac{\partial S_f}{\partial V}\bigg|_2^j\right](V_2^{j+1}) + \frac{V_2^j}{2\Delta x}(V_3^{j+1}) = g\left[S_o - S_{f_2}^{\,j} + \frac{\partial S_f}{\partial V}\bigg|_2^j V_2^j + \frac{\partial S_f}{\partial K}\bigg|_2^j \frac{dK}{dy}\bigg|_2^j y_2^j\right] + \frac{V_2^j}{\Delta t} + \frac{g}{2\Delta x} y_1^{j+1} + \frac{V_2^j V_1^{j+1}}{2\Delta x} \tag{3-21a}$$

At the right hand boundary, x_n, the finite-difference form used to evaluate the partial derivative with respect to x must be modified and a backward difference used. Therefore letting

$$\frac{\partial V}{\partial x} \approx \frac{V_n^{j+1} - V_{n-1}^{j+1}}{\Delta x} \quad \text{(backward difference)}$$

$$\frac{\partial y}{\partial x} \approx \frac{y_n^{j+1} - y_{n-1}^{j+1}}{\Delta x} \quad \text{(backward difference)}$$

and with the partial derivatives with respect to time still given by the forward difference, the following finite difference form of equation 3-8 and 3-15 can be formulated

$$-\frac{V_n^j B_n^j}{\Delta x}(y_{n-1}^{j+1}) + \left(\frac{V_n^j B_n^j}{\Delta x} + \frac{B_n^j}{\Delta t}\right)(y_n^{j+1}) - \frac{A_n^j}{\Delta x}(V_{n-1}^{j+1})$$

$$+ \frac{A_n^j}{\Delta x}(V_n^{j+1}) = \frac{B_n^j\, y_n^j}{\Delta t} \qquad (3\text{-}22)$$

and

$$-\frac{g}{\Delta x}(y_{n-1}^{j+1}) + \left(\frac{g}{\Delta x} + g\,\frac{\partial S_f}{\partial K}\bigg|_n^j\,\frac{dK}{dy}\bigg|_n^j\right)(y_n^{j+1})$$

$$-\frac{V_n^j}{\Delta x}(V_{n-1}^{j+1}) + \left[\frac{1}{\Delta t} + \frac{V_n^j}{\Delta x} + g\,\frac{\partial S_f}{\partial V}\bigg|_n^j\right](V_n^{j+1})$$

$$= g\left[S_o - S_{f_n}^{\;j} + \frac{\partial S_f}{\partial V}\bigg|_n^j V_n^j + \frac{\partial S_f}{\partial K}\bigg|_n^j \frac{dK}{dy}\bigg|_n^j y_n^j\right] + \frac{V_n^j}{\Delta t} \qquad (3\text{-}23)$$

The proper application of these equations (3-20, 3-21, 3-20a, 3-21a, 3-22, 3-23) along any time line, t_j, will yield 2(n-1) equations which contain 2(n-1) unknownswhich when solved simultaneously will give the values of depth of flow and velocity at time line t_{j+1}.

The initial boundary values at x_1, necessary to start the routing process, for each sub-basin sewer, are found by using the hydraulic elements curves for partially filled circular conduits shown in Figure 3-5. The flow rate when the conduit is full (Q_f) can be calculated using Manning's equation knowing the diameter, roughness coefficient, and slope of the sewer invert

$$Q_f = \frac{0.463}{n} D^{8/3} S_o^{\;1/2} \qquad (3\text{-}24)$$

The inlet flowrate Q_1^j is the sum of the surface runoff and the sewer outlet flow from the upstream sub-basin. Thus, the ratio of Q_1^j/Q_f is established and the ratio of y_1^j/y_f and V_1^j/V_f can be found from Figure

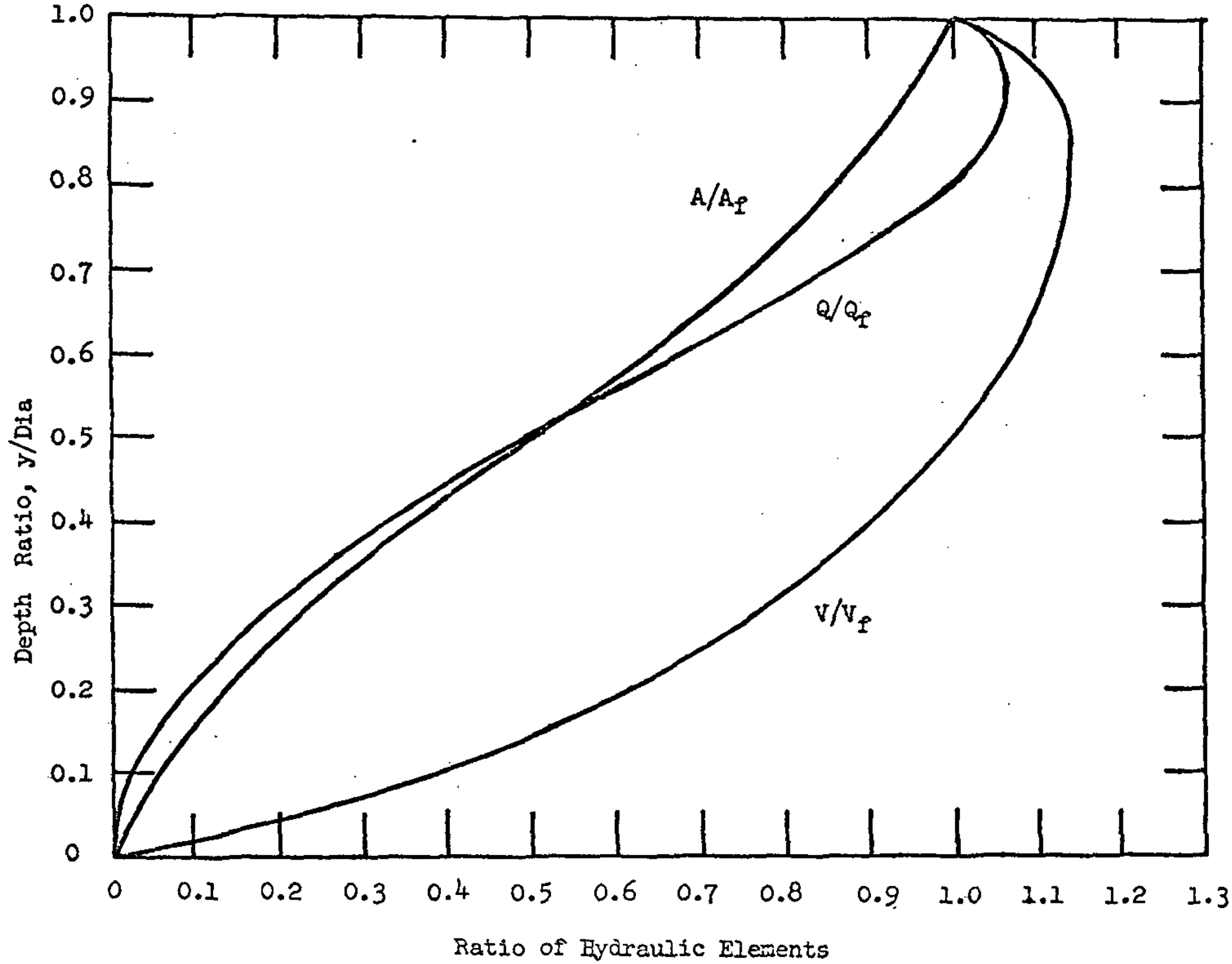


Figure 3-5 Relationship Between Hydraulic Elements of a Circular Conduit

3-5. Since the sewer diameter and the velocity when the sewer is flowing full are known the initial values of y_1^j and V_1^j are determined for any time t_j.

<u>Computer model</u>. A computer model (HYDROL)[5] using the implicit finite-difference solution to the St. Venant equations, was programmed to route the combined surface runoff and dry weather flow through the sewer network (Figure 3-3). Input into the program consisted of the characteristics of the sewer, such as diameter, coefficient of roughness, length, and invert slope, and the surface runoff hydrographs from each sub-basin.

The first part of the program calculated a rating curve for a partially filled sewer, using Manning's equation, which relates the flow rate to the depth of flow.

$$\frac{Q}{Q_f} = f(y/y_f = \text{Dia}) \qquad (3\text{-}25)$$

Using this rating curve and knowing initial values of Q, the corresponding depth can be found. Then other parameters, such as, wetted perimeter (P), area of flow (A), and the width of flow (B) can be found using the geometric relationships included in Figure 3-3. Initially the friction slope (S_f) is estimated from the known flowrate using Manning's equation.

The incremental distance (Δx) for each sewer was selected to be one-tenth of the sewer length between inlet points. This results in "n" having values from 1 to 11. The incremental time parameter (Δt) was chosen to be 1 hour to correspond with the hourly increment used in rainfall records.

The routing program starts with the inlet point farthest upstream from the receiving stream or point of discharge. The initial flow in the

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5 See Appendix 3D, page 3-62 for flowchart.

sewer discharging from this point is assumed to be only the dry weather or sanitary flow. This flow is considered to be uniform and steady, and the rating curve can be used to determine the depth of flow in the sewer and the other initial boundary values at t=0. The surface runoff hydrograph is used to establish the boundary conditions at x=0 in a similar manner.

These initial values can be used to calculate all of the coefficients required to solve the simultaneous equations. A matrix inversion technique was used in solving the 20 equations for y_k^{j+1} and V_k^{j+1} at time $t+\Delta t$. The geometric relationships were then used to calculate corresponding values of P_k^{j+1}, B_k^{j+1} and A_k^{j+1}, and new coefficients can be calculated for solving the simultaneous equations again. Therefore, each solution of the equations yields a new set of equations, which result in a stepwise progression in time. This continues until the rainfall stops and the flow in the sewer returns to a uniform and steady condition.

The occurrence of another period of surface runoff resulting from another rainfall cycle will cause the routing program to be repeated until the discharge hydrograph for the entire month is determined. The routing program for subsequent sewers was similar. The only exception was that the inlet flow would consist of the surface runoff from that particular sub-basin plus the discharge from the upstream sewer. This is repeated until the combined sewer flow hydrograph is obtained at the point of discharge to the receiving stream. Hydrographs for as long a time period as desired can be synthesized in this manner by returning to the first sub-basin and inputing the new surface runoff hydrographs.

Verification of the results of the routing model with field measurements of flows in real sewers was not attempted in this study. Instead, comparisons were made with published results of studies involving flows in partially filled circular conduits. In one recent study (2), numerical solutions using the method of characteristics were obtained for the simplified sewer layout and inlet shown in Figure 3-6. The outlet hydrograph predicted by the flow routing model compares very favorable with this solution using the incremental time and distance steps noted. Measurement of the depth of flow from a circular conduit with characteristics and inlet hydrograph shown in Figure 3-7 was performed by Yevjevich (17). Again comparison with the predicted depth of flow is very close with the predicted depth curve being slightly flatter. However, the occurrence of the peak of the two curves correspond very well on the time axis.

Figure 3-8 illustrates that the solution obtained by the implicit finite difference method is stable. Time increments of 60, 30, 15, and 7 1/2 minutes were used and the outlet hydrograph remained the same.

These comparisons indicate that the flow routing model would be an acceptable tool for determining hydrographs in a combined sewer system.

Results

The purpose of generating hydrographs, from a given drainage area served by combined sewers, was to obtain a longer "period of record" than would otherwise be available. This would significantly contribute to the accuracy needed in establishing design criteria for controlling overflows and in evaluating the importance of combined sewer overflows on the receiving

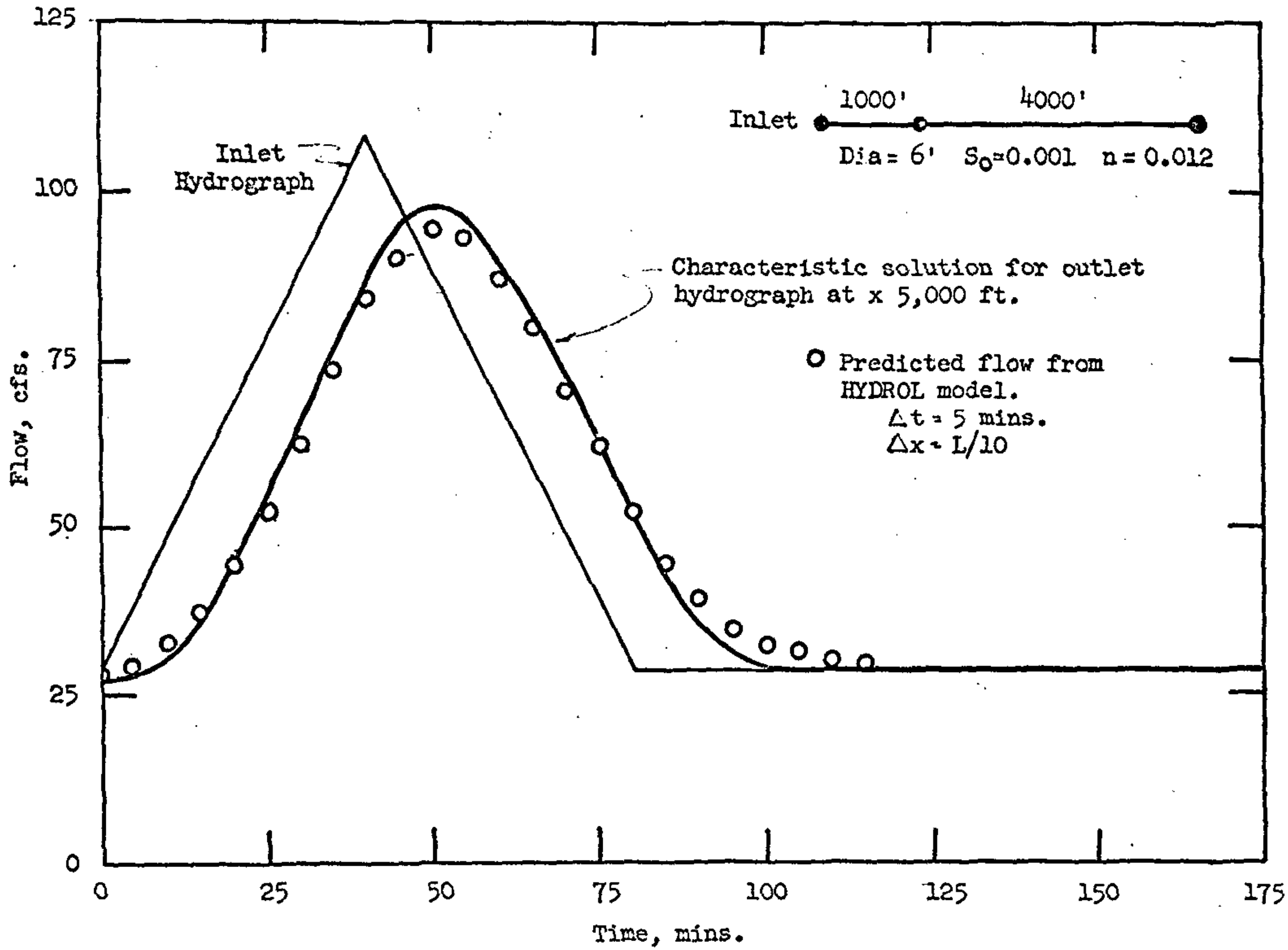

Figure 3-6 Comparison of Flow Predicted by HYDROL model and by Characteristic Solution (2)

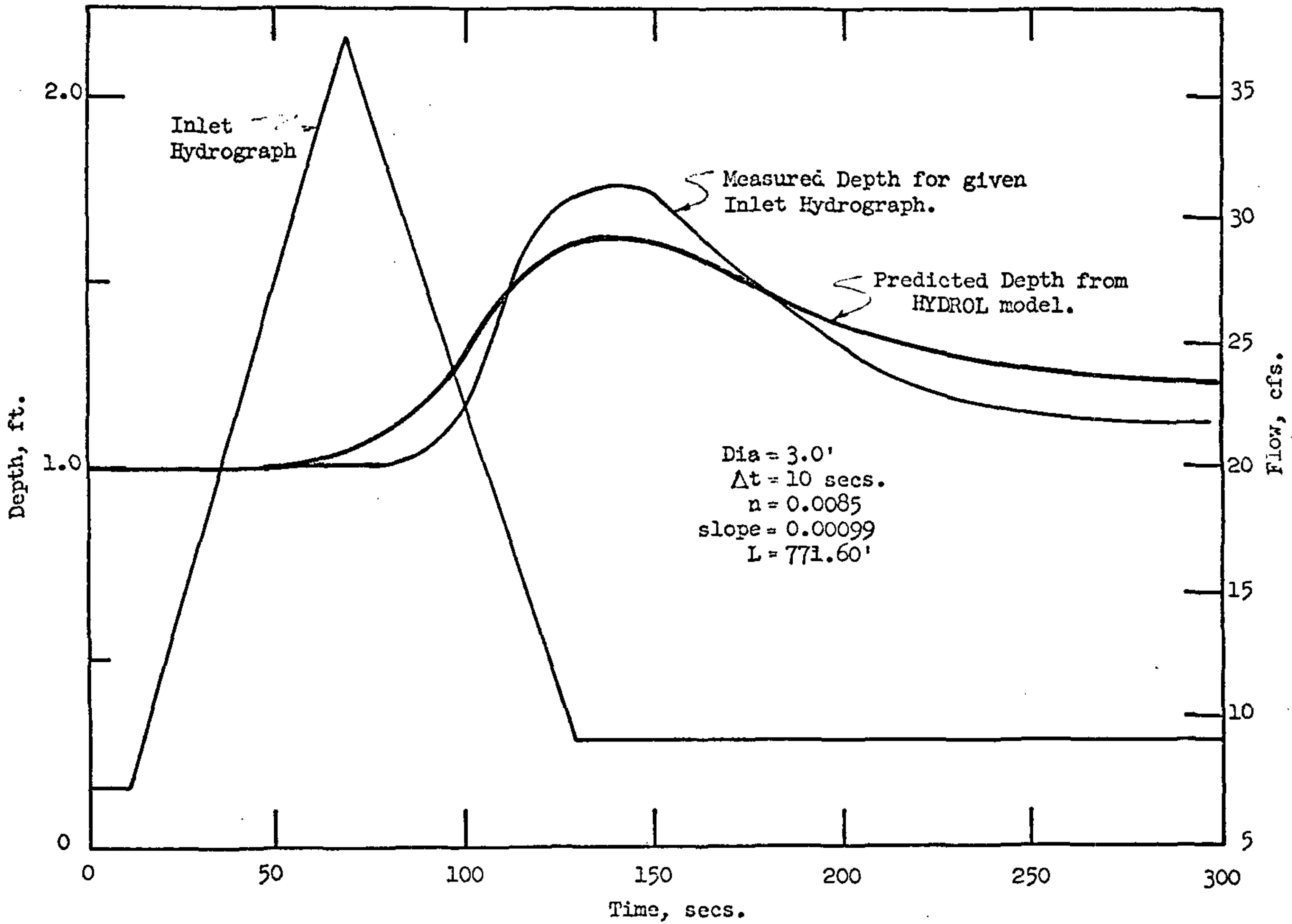

Figure 3-7 Comparison of Depth of Flow Predicted by HYDROL model and Depth Measured by Yevjevich (17)

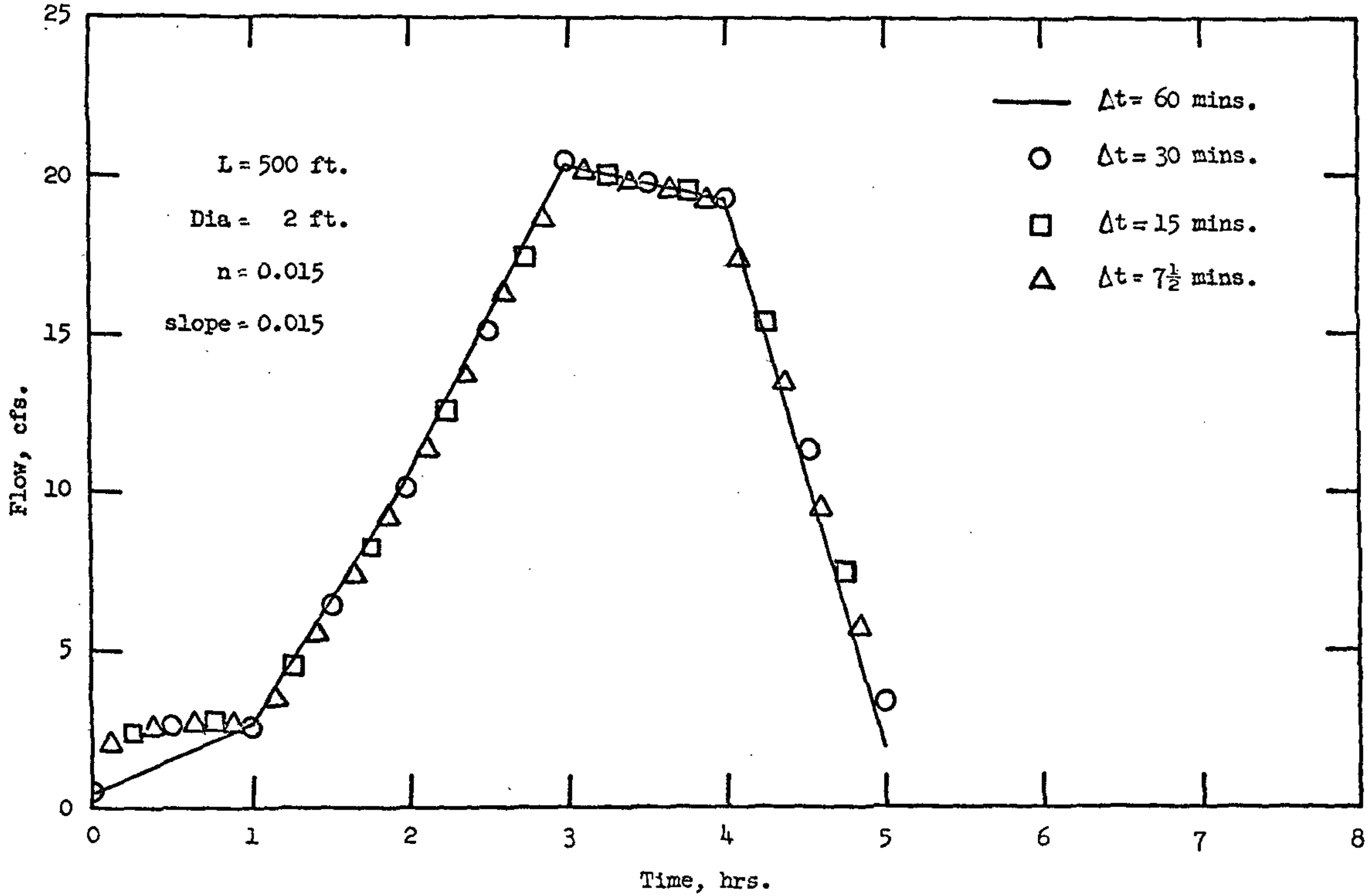

Figure 3-8 Effect of Varying Δt in HYDROL model

stream. With the simulated data this can now be accomplished by using methods normally employed, if actual data were available. One such method, which is widely used by hydrologists in analyzing streamflows, is to formulate frequency or probability curves of various parameters which are of importance.

To illustrate how this analysis would be performed, the combined sewer hydrographs for the month of June and for the study area as shown in Figure 3-2, were generated for fifteen future periods using the previously described models. Results given in this study should not be interpreted as being typical and used indiscriminately. Each drainage area would yield its own characteristic curves which would have to be analyzed independently.

Based on the generated hydrographs, the probability that the number of overflows occurring during the month of June will be less than a stated value can be determined from Figure 3-9. Fifty per cent of the months of June will be expected to have less than 4.2 overflows, if a flow equal to the dry weather flow in intercepted. In addition a range of values can be determined depending on the selected criteria. For example, there is a 10 per cent probability that the number of overflows will be either less than 1.8 or greater than 6.6.

The characteristics of the overflow are dependent on the amount of flow (expressed as a multiple of the dry weather flow) that the interceptors can accommodate. The effect of increasing this amount to 2, 3 and 4 times the DWF is also shown in Figure 3-9. In this case the average number of overflows will be reduced to 3.8, 3.4 and 2.9 occurrences.

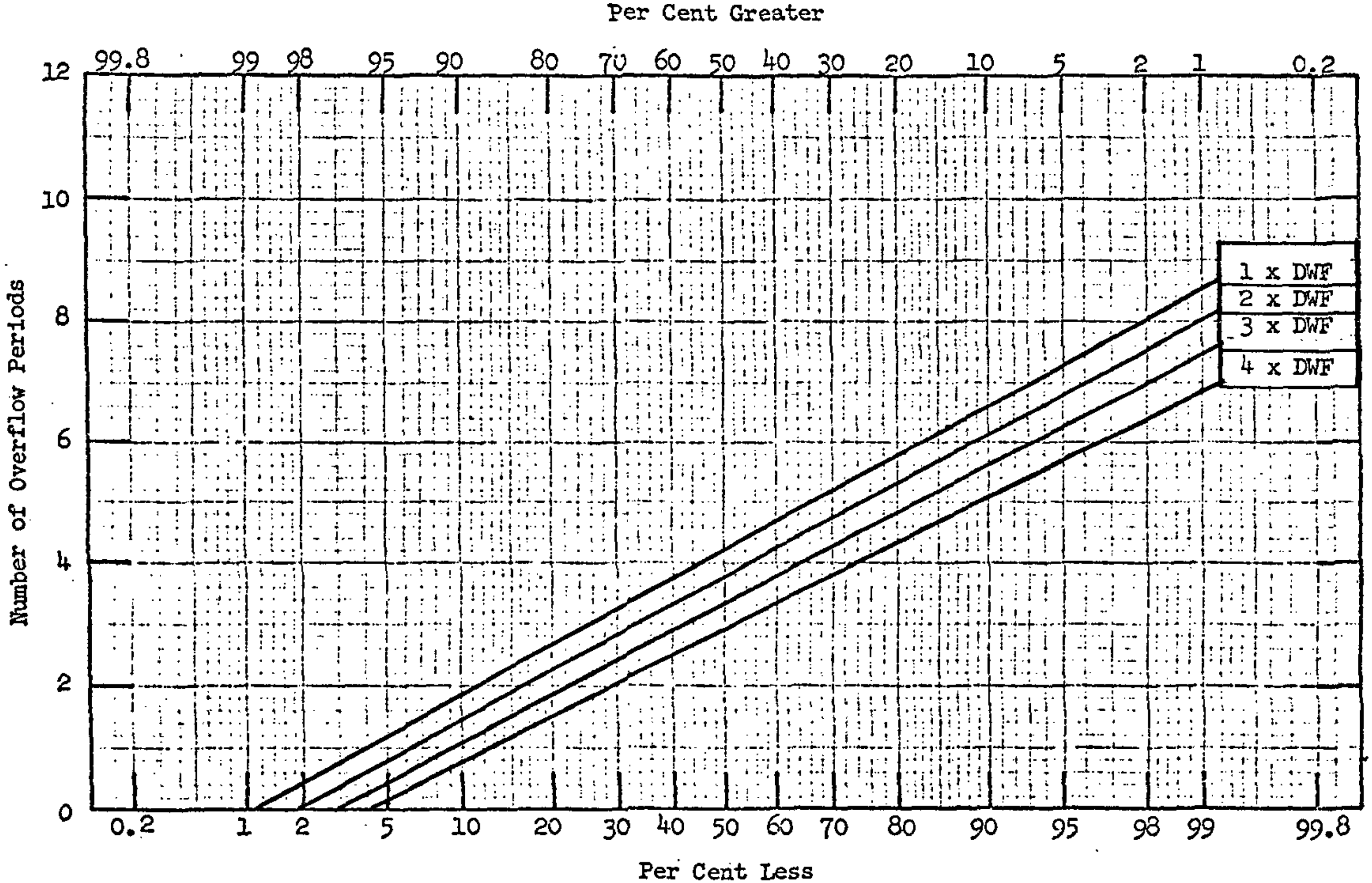

Figure 3-9 Probability that the Number of Overflows during the Month of June Will Be Less Than a Stated Value.

Although this information does not reveal how long the overflows will last, it would be of importance in situations where manual maintenance was required after each overflow period. This would be the case if diversion gates had to be reset or if cleaning of screens or other equipment was required. A large number of overflow periods would indicate that automatic or self-cleansing devices should be installed.

In Figure 3-10 the probability that the total time of all overflows occurring during the month of June will be less than a stated number of hours is given. If the interceptor capacity is equal to the dry weather flow, the median number of hours the combined sewer will overflow is 46 hours or 5.5 per cent of the month. The range of values for this case would be that for 10 per cent of the months of June the combined sewer would be expected to overflow either less than 23 or more than 70 hours (3.1 and 9.4 per cent).

Doubling the capacity of the interceptor significantly decreases the number of hours during which overflows will occur. The median time of an overflow is reduced to 24 hours or 3.2 per cent of the month. Ten per cent of the months would have either less than 7 or more than 41 hours (0.94 and 5.5%) of overflows. Further increase in the capacity of the interceptor is not as significant. Providing for three times the dry weather flow results in a median total overflow time of 19 1/2 hours or 2.62 per cent of the month and for four times the dry weather flow, the median total overflow time is 15 hours or 2.01 per cent of the month.

It is interesting to note that even if four times the dry weather flow is intercepted, approximately 90 per cent of the months of June would

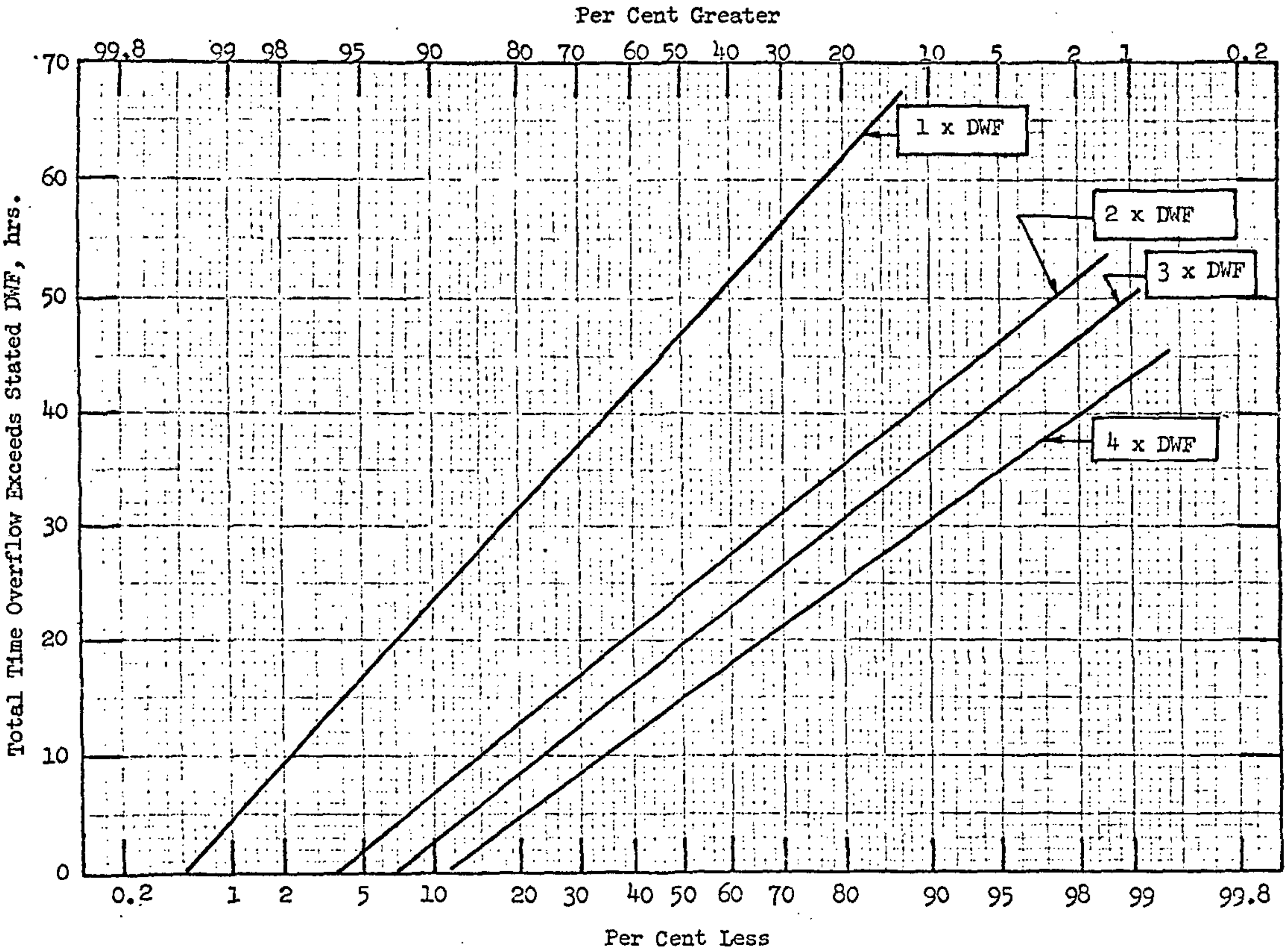

Figure 3-10 Probability that the Total Time of Overflow during the Month of June Will Be Less Than a Stated Value.

still have periods when the combined sewer would overflow to the receiving stream.

The total time that overflows occur would be significant in evaluating the effect of the overflow on the receiving stream. Decision could be made as to the effectiveness of intercepting larger amounts of the combined sewer flow. Estimates could also be made as to the total time the control of treatment equipment would be in use and the total manhours required, if manual supervision was necessary.

The probability that the duration, or length of time, of a single overflow period will be less than a stated value is given in Figure 3-11. Fifty per cent of the overflow periods will be of less than 8 1/2 hours duration. Ten per cent of the overflows will have durations either less than 3 or greater than 20 hours.

Interception of 2, 3 and 4 times the dry weather flow reduces the average duration of an overflow to 4.1, 2.1 and 1 hour, respectively. The substantial reduction in length of overflow when twice the dry weather flow is intercepted illustrates that a large percentage of the overflow is the result of low intensity rainfalls of long duration.

The duration of an overflow period would be important in many types of treatment methods used in controlling combined sewer overflows, for instance, the time that pumps and other mechanical equipment would have to be continuously operated or manually controlled. Decisions as to the types of monitoring and sampling programs to be implemented would also be influenced by the duration of the overflows.

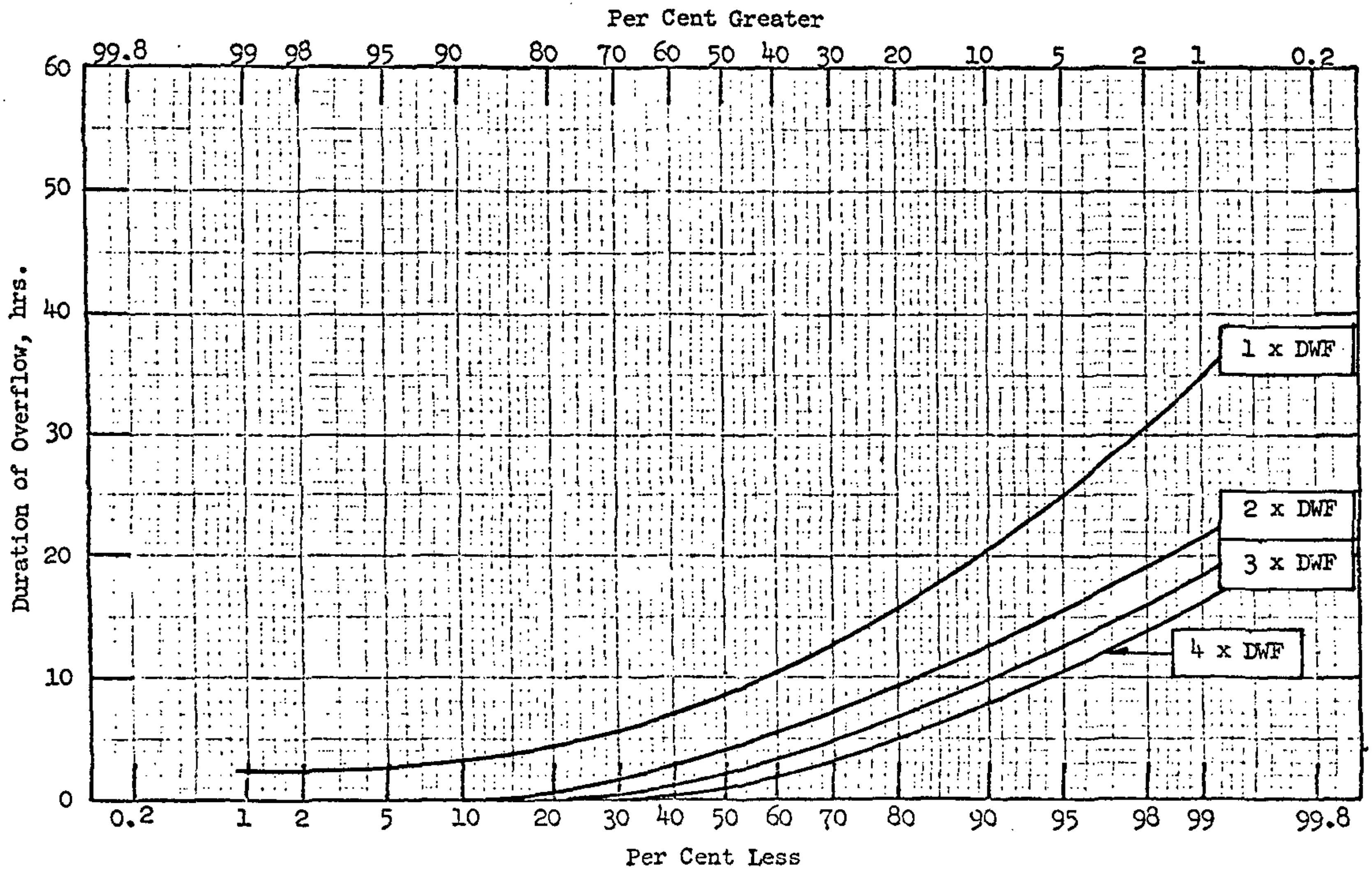

Figure 3-11 Probability that the Duration of an Overflow Will Be Less Than a Stated Value.

The probability that the time between successive overflow periods is less than a given value is shown in Figure 3-12. From this curve it is noted that 50 per cent of the intervals will be less than 90 hours. A wide range of values are noted and 10 per cent of the intervals will be either less than 20 or greater than 340 hours.

Increasing the interceptor capacity will result in longer intervals between the overflow periods. Providing for the interception of four times the DWF will mean that 90% of the time intervals will be greater than 40 hours rather than the 20 hour interval resulting from intercepting one times the DWF.

This curve would aid in determining the time available between overflows for performing maintenance work and emptying storage containers. For example, if temporaty storage was used to prevent discharges to the receiving stream and the stored waste was diverted to the treatment facility during normal flow periods, the time between overflow periods would be of importance. In this case providing for emptying of the storage container in 20 hours would mean 90% of the time the container would be empty before the next overflow occurred, if only the dry weather flow was intercepted.

Figure 3-13 gives the probability that the total volume of overflow occurring during the month of June will be less than a stated value. The median volume of overflow during the month of June is 12.5 million gallons if only the dry weather flow is intercepted. This represents 27.2% of the total dry weather flow. The volume of overflow would be less than 3 or

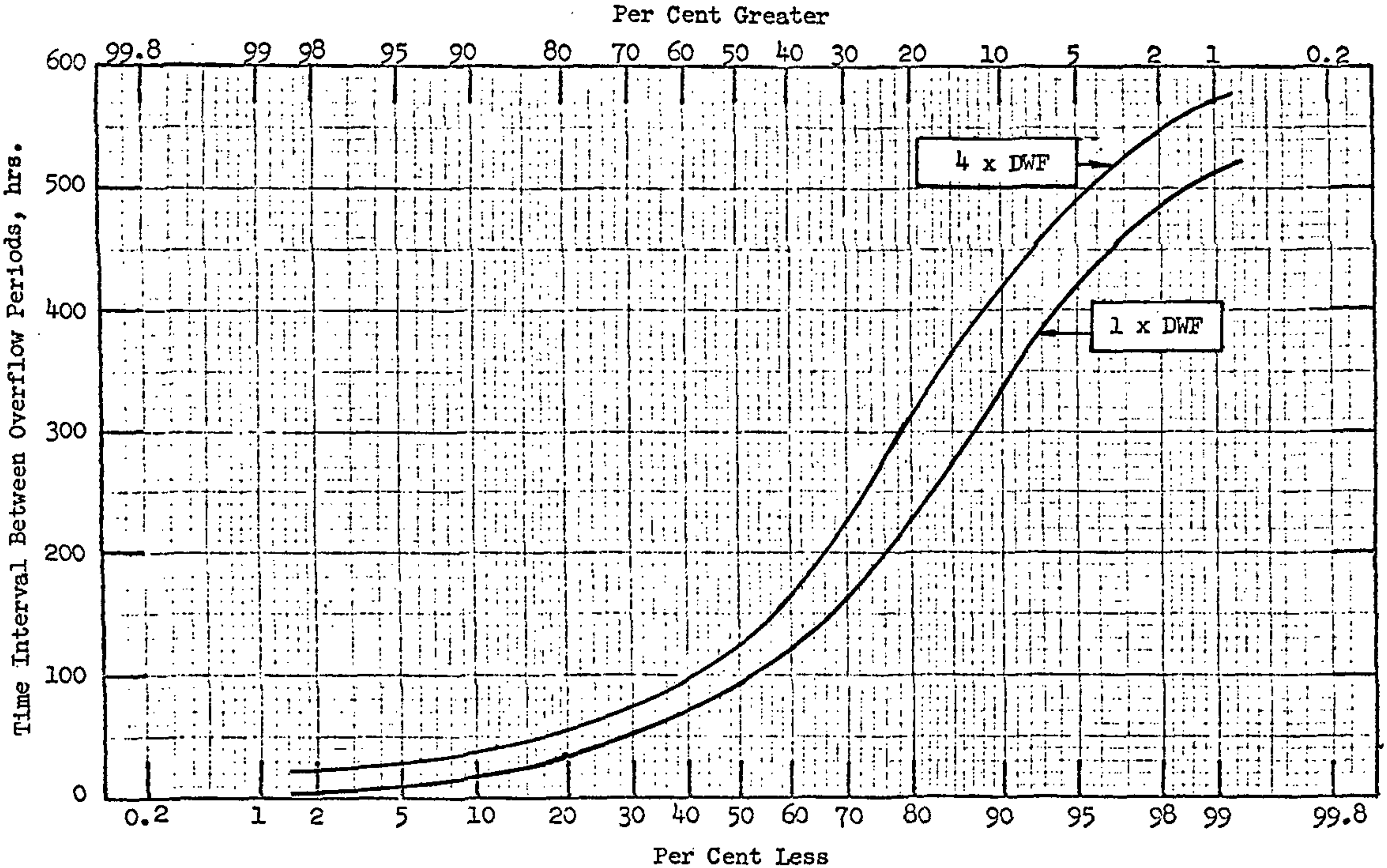

Figure 3-12 Probability That the Time Interval Between Successive Overflows Will Be Less Than a Stated Value.

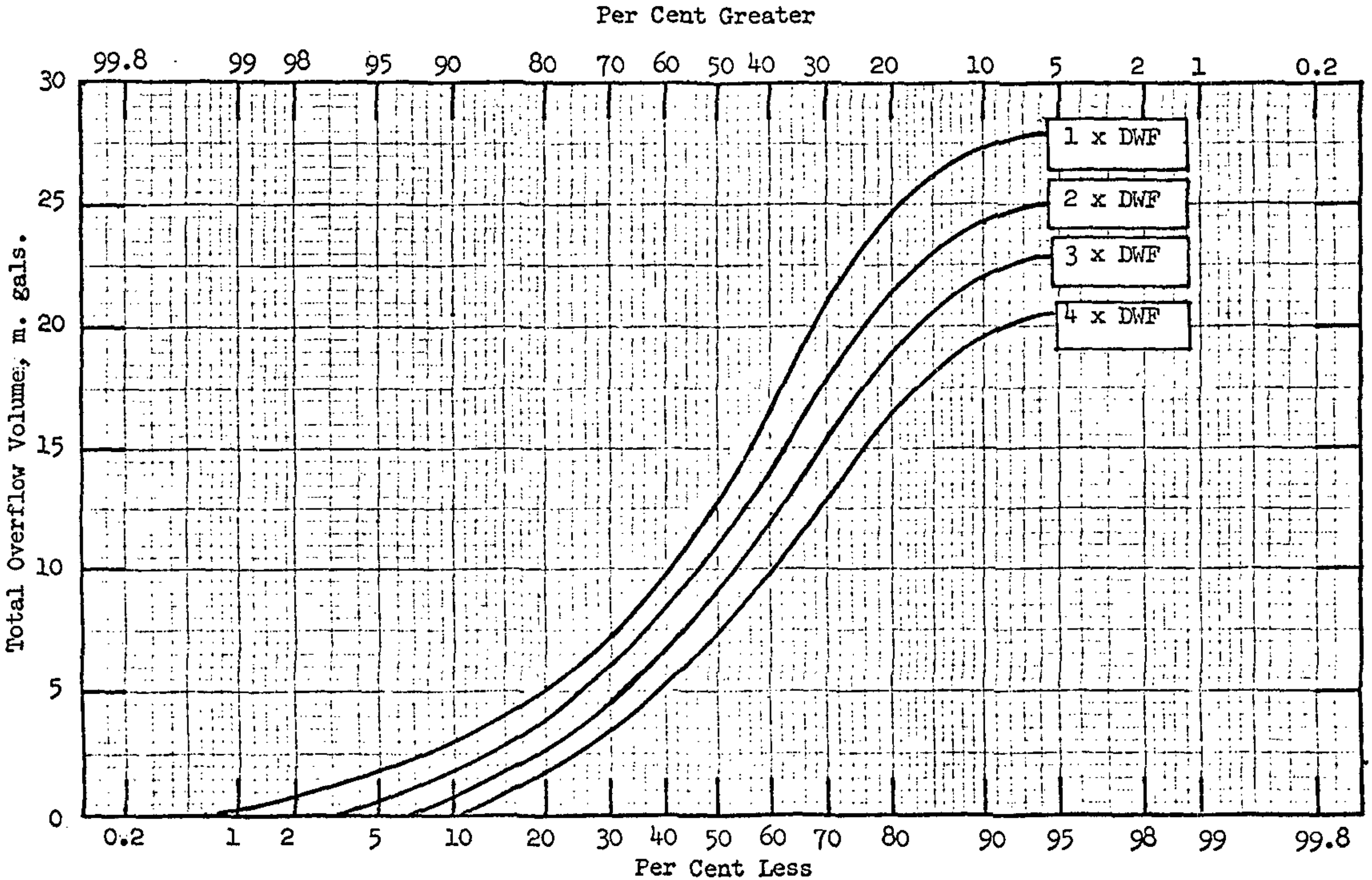

Figure 3-13 Probability That the Total Overflow During the Month of June Will Be Less Than a Stated Value.

greater than 27 m. gals. for 10 per cent of the months of June.

Increasing the capacity of the intercepting sewers to 2, 3 and 4 times the dry weather flow reduces the median volume of overflow to 11, 9.1 and 7.5 m. gals., respectively. Thus, increasing the interceptor capacity four times will only reduce the median total volume of overflow to be expected by 40 per cent. Again, it is also noted that with this four fold increase in capacity, a volume of overflow greater than zero will occur in 89 per cent or approximately 9 out of 10 months of June

The total volume of combined waste discharging to the stream would also be meaningful in conjunction with the total time of overflows in evaluating the efficiency of any proposed control measures. The additional amount of flow above the dry weather flow needed to be treated during the month could also be determined and the cost estimated.

The next curve Figure 3-14 shows the probability that the discharge from any separate overflow occurrences will be less than a stated volume. Fifty per cent of the occurrences in which the flow exceeds the dry weather flow will have a volume less than 1.25 million gallons. Ten per cent of the overflows will have volumes either less than 0.2 or greater than 8.7 m. gals. If the interceptor capacity is increased, the average overflow volume would be reduced to 0.9 m. gals. for two times the DWF, or 0.8 m. gals. for three times the DWF, and 0.4 for four times the DWF.

Decisions as to the size of storage basins could be based on this information. In order to prevent 90 per cent of the overflow events from discharging any combined waste to the stream, 8.7 million gallons of

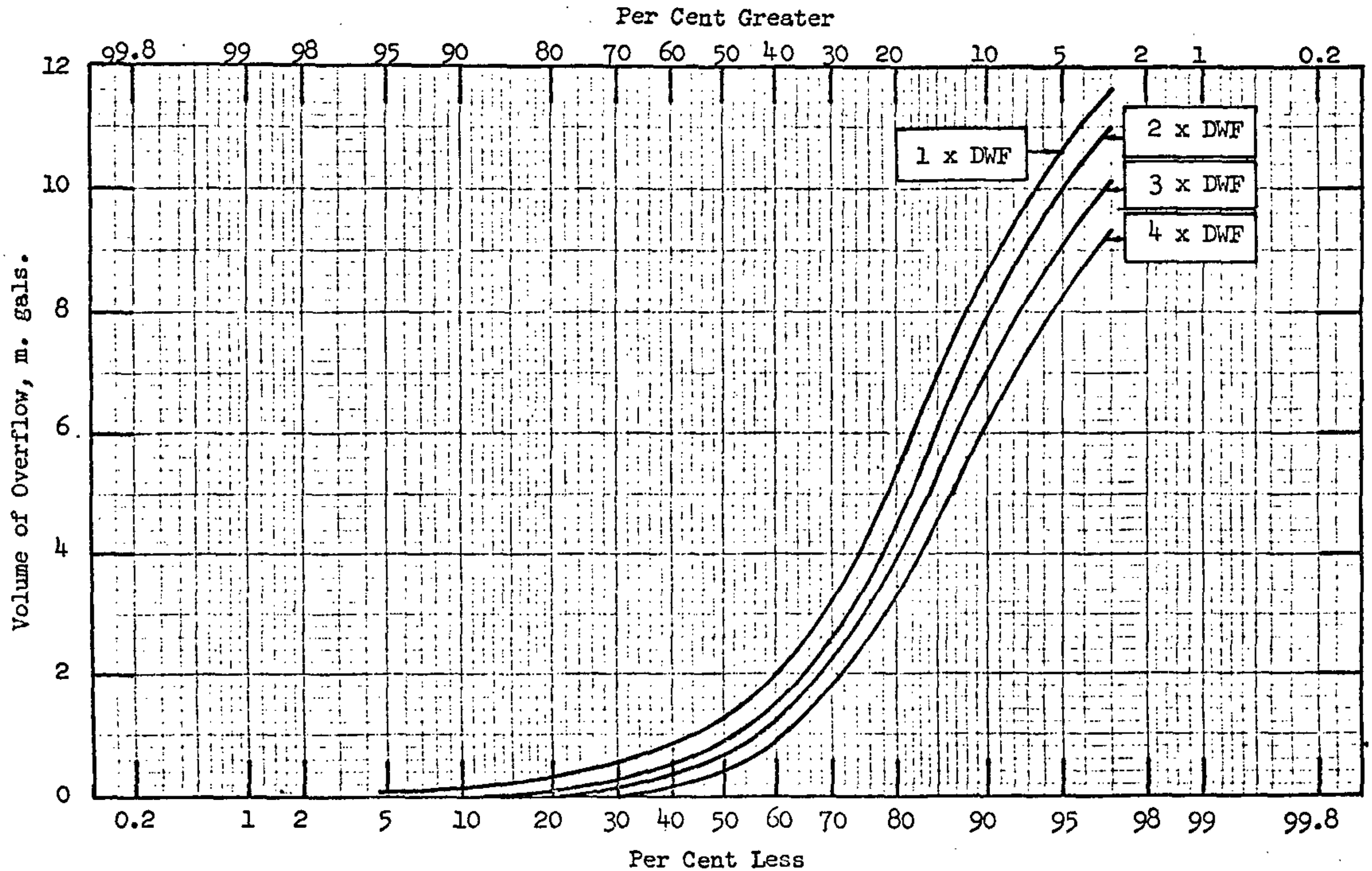

Figure 3-14 Probability That the Volume of an Overflow Will Be Less Than a Stated Value.

storage would be required. If the interceptor capacity was increased to four times the DWF, the required storage could be reduced to 6.1 m. gals. and still capture 90 per cent of the overflows.

The final curve, shown in Figure 3-15, gives the probability that the flowrate is less than a stated value. The median flowrate is found to be 3 cfs with 10 per cent of the flowrates either less than 2.5 or greater than 32 cfs.

The flowrate would be important in determining pump capacities and treatment rates. A treatment unit capable of treating two times the dry weather flow would be adequate approximately 55 per cent of the time. If the treatment unit was designed to accommodate 3 or 4 times the dry weather flow, it would be adequate to treat 62 per cent and 66 per cent of the overflows.

Conclusions

The mathematical and computer models presented in this report provide a methodology for overcoming one of the most vexing problems in managing or controlling combined sewer overflows. As stated previously these overflows are seldom monitored and the scarcity of data available to describe the characteristics of the combined flow makes selection of design criteria risky to say the least. Using this flow routing methodology, data could be generated for a sufficiently long period of time to provide the designer with the basic information necessary in making sound judgements as to the best means of controlling or treating combined sewer flows.

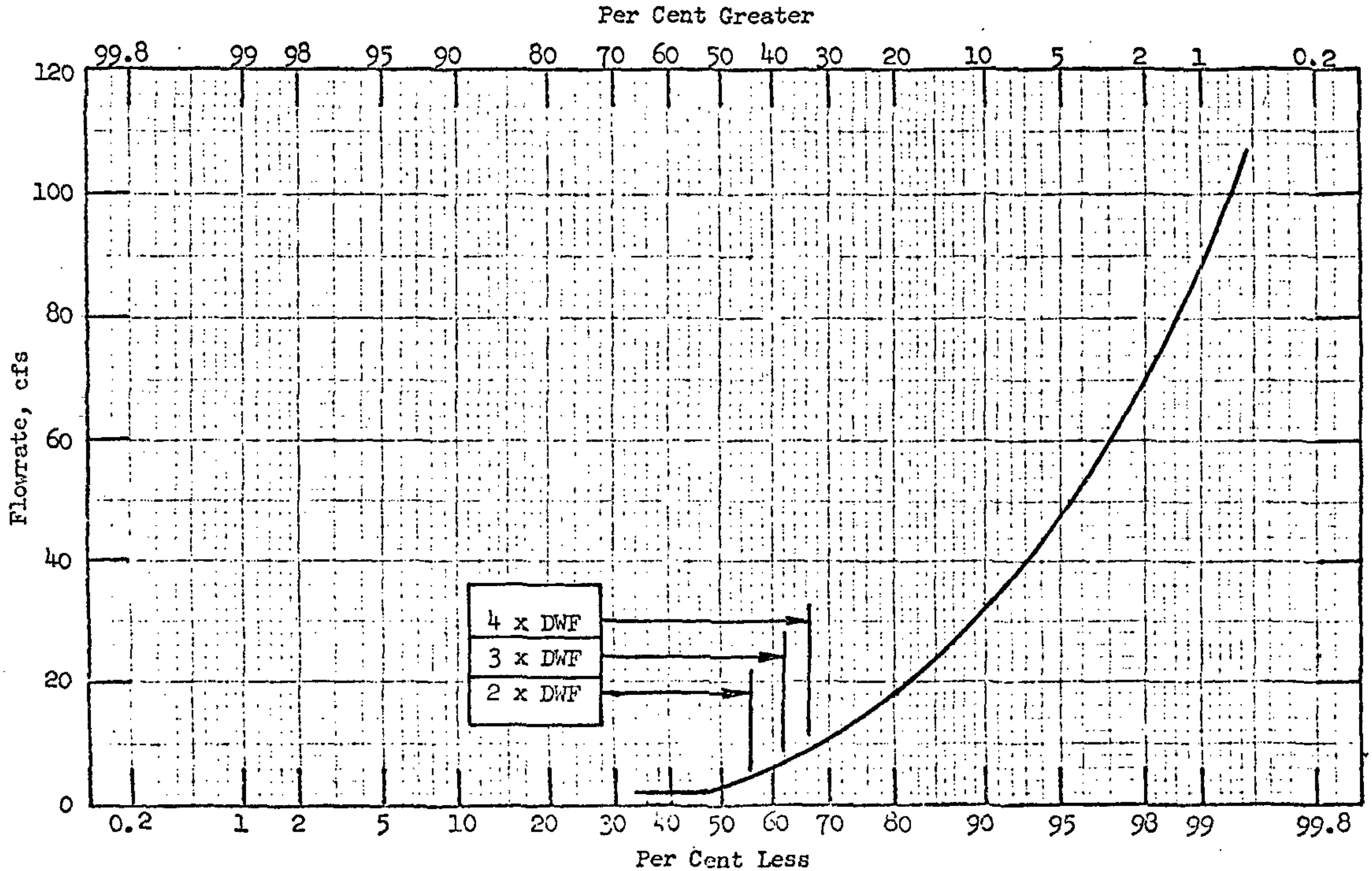

Figure 3-15 Probability That the Flowrate Will Be Less Than a Stated Value.

Several larger cities, such as Minneapolis-St. Paul and Cleveland (18), are presently attempting to optimize their existing sewerage systems by controlling the flow using real-time computer models. Recording rain gauges are located in numerous drainage basins, and the actual rainfall measurements are transmitted directly to a computer center and serve as input into the routing model. Control gates and dams are installed in the sewers and can be adjusted from the computer center. The operator at the computer center then automatically adjusts the gates and dams based on the simulated flow results given by the computer to minimize the discharge to the stream. The routing methodology described in this report could also be used for this purpose by inputing the actual measured rainfall intensity as it occurs rather than the synthesized rainfall data.

Another use of this routing model would eliminate the necessity of simulating the resulting flow every time a storm occurs. After once generating the synthesized hydrographs, all of the data necessary to construct unit hydrographs at selected points throughout the sewer system are available. These unit hydrographs could then be used when future storms occur to predict total hydrographs. This would significantly reduce the computer time required in the complete flow routing simulation and would probably be easier to implement in practice since the techniques for applying unit hydrograph theory are well established.

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NOMENCLATURE

Symbol	Definition
A	Cross-sectional area of flow, ft^2
A_i	Impervious area of the sub-basin, acres
a	Acceleration of the liquid mass in the direction of flow x, ft/sec^2
B	Width of flow at depth y, ft.
D	Diameter of sewer, ft.
F	Resultant of all external forces resolved in the direction of flow x, lb.
F_f	Boundary resistance force, lb.
F_g	Weight of liquid in control volume, lb.
F_p	Resultant force due to pressure differences, lb.
f_{rs}	Number of times a transition from $S_t=r$ to $S_{t+1}=s$ occurs
$f(S_t)$	Lowest value of H_t in the interval of rainfall, S_t
g	Gravitational acceleration, ft/sec^2
H_t	Intensity of rainfall for hour t, in/hr
I_r	Interval of values of H_t corresponding to $S_t=r$
J_s	Interval of values of H_{t+1} corresponding to $S_{t+1}=s$
K	Conveyance factor, ft/sec
m	Mass of liquid, slugs
n	Manning's coefficient of roughness
P	Wetted perimeter, ft.
P_{ij}	Probability $H_{t+1}=j$ given $H_t=i$
Q	Flowrate, cfs

Q_t	Flowrate at the end of hour t, cfs
R	Hydraulic radius, ft.
S_f	Friction slope
S_o	Slope of sewer invert
S_t	State of rainfall intensity during hour (t)
U_{s_t}	Independent uniformly distributed random variable
V	Average velocity, ft/sec
y	Depth of flow, ft.
Δt	Incremental time, sec.
Δx	Incremental length, ft.
ρ	Density of liquid, slugs/ft^3

APPENDIX 3A

Dependency of Hourly Rainfall Intensities Occurring in Consecutive Hours

The conditional probabilities of the rainfall intensities during hour (t+1) being equal to 0.00, 0.01, and 0.02, given that the rainfall intensity during hour (t) is equal to 0.00, 0.01, and 0.02, based on rainfall measurements recorded at Bradley Field International Airport, near Hartford, Connecticut, are as follows:

$H_{t+1} = 0.00$

$$P(H_{t+1} = 0.00 \mid H_t = 0.00) = 0.98$$

$$P(H_{t+1} = 0.00 \mid H_t = 0.01) = 0.55$$

$$P(H_{t+1} = 0.00 \mid H_t = 0.02) = 0.42$$

$H_{t+1} = 0.01$

$$P(H_{t+1} = 0.01 \mid H_t = 0.00) = 0.01$$

$$P(H_{t+1} = 0.01 \mid H_t = 0.01) = 0.18$$

$$P(H_{t+1} = 0.01 \mid H_t = 0.02) = 0.20$$

$H_{t+1} = 0.02$

$$P(H_{t+1} = 0.02 \mid H_t = 0.00) = 0.004$$

$$P(H_{t+1} = 0.02 \mid H_t = 0.01) = 0.13$$

$$P(H_{t+1} = 0.02 \mid H_t = 0.02) = 0.10$$

Since the conditional probability of a particular rainfall intensity occurring during the hour (t+1) is different when different rainfall intensities occur during hour (t), the hourly rainfall intensities in successive hours are not independent. Similar dependence can be shown for other values of H_{t+1}.

APPENDIX 3B

Basis for Using Sixth-Order Markov Chain Model When the Rainfall Intensity During Hour (t) is Equal to Zero.

The conditional probability that the rainfall intensity during hour (t+1) is greater than 0.00, given "n" preceding dry hours, based on rainfall measurements recorded at Bradley Field are calculated as follows:

n=1

$$P(H_{t+1} \geq 0.01 \mid H_t = 0.00) = \frac{P(H_{t+1} \geq 0.01 \cap H_t = 0.00)}{P(H_t = 0.0)} = 0.02295$$

n=2

$$P(H_{t+1} \geq 0.01 \mid H_t = H_{t-1} = 0.0) = \frac{P(H_{t+1} \geq 0.01 \cap H_t = H_{t-1} = 0.00)}{P(H_t = H_{t-1} = 0.00)}$$

$$= 0.0194$$

n=3

$$P(H_{t+1} \geq 0.01 \mid H_t = H_{t-1} = H_{t-2} = 0.0) = \frac{P(H_{t+1} \geq 0.01 \cap H_t = H_{t-1} = H_{t-2} = 0.0)}{P(H_t = H_{t-1} = H_{t-2} = 0.00)}$$

$$= 0.0175$$

n=4

$$P(H_{t+1} \geq 0.01 | H_t = H_{t-1} = H_{t-2} = H_{t-3} = 0.00)$$

$$= \frac{P(H_{t+1} \geq 0.01 \cap H_t = H_{t-1} = H_{t-2} = H_{t-3} = 0.00)}{P(H_t = H_{t-1} = H_{t-2} = H_{t-3} = 0.00)}$$

$$= 0.01612$$

n=5

$$P(H_{t+1} \geq 0.01 | H_t = H_{t-1} = H_{t-2} = H_{t-3} = H_{t-4} = 0.00)$$

$$= \frac{P(H_{t+1} \geq 0.01 \cap H_t = H_{t-1} = H_{t-2} = H_{t-3} = H_{t-4} = 0.00)}{P(H_t = H_{t-1} = H_{t-2} = H_{t-3} = H_{t-4} = 0.00)}$$

$$= 0.01535$$

n=6

$$P(H_{t+1} \geq 0.01 | H_t = H_{t-1} = H_{t-2} = H_{t-3} = H_{t-4} = H_{t-5} = 0.00)$$

$$= \frac{P(H_{t+1} \geq 0.01 \cap H_t = H_{t-1} = H_{t-2} = H_{t-3} = H_{t-4} = H_{t-5} = 0.00)}{P(H_t = H_{t-1} = H_{t-2} = H_{t-3} = H_{t-4} = H_{t-5} = 0.00)}$$

$$= 0.01478$$

These results when plotted as shown in Figure 3B-1, show that the conditional probabilities become essentially constant after 6 hours. Thus, it is concluded that a sixth-order Markov chain model would adequately predict the occurrence of a wet hour at (t+1) if hour (t) was dry.

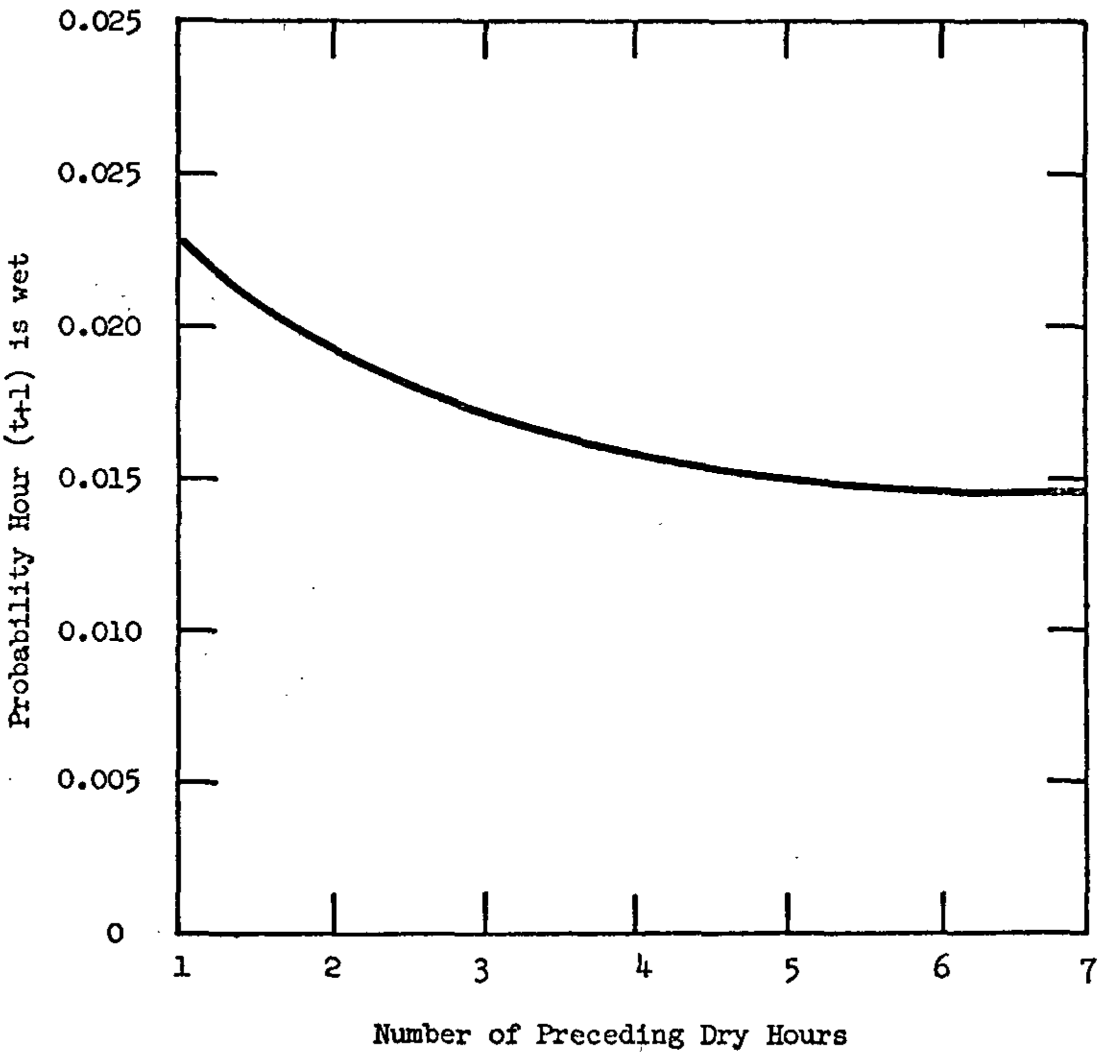
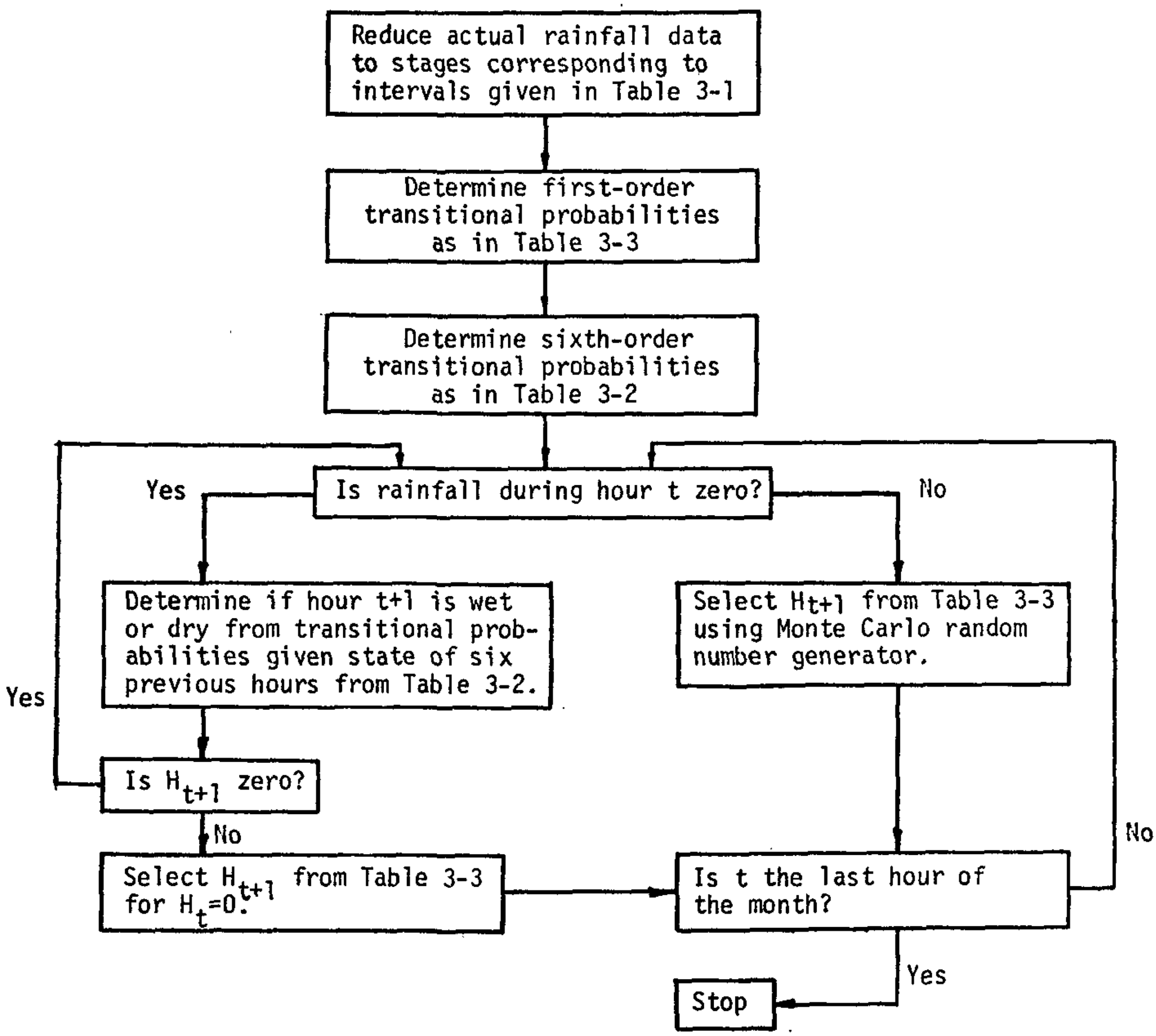


Figure 3B-1 Transitional Probabilities of Hour (t+1) Being Wet Following a Dry Period.

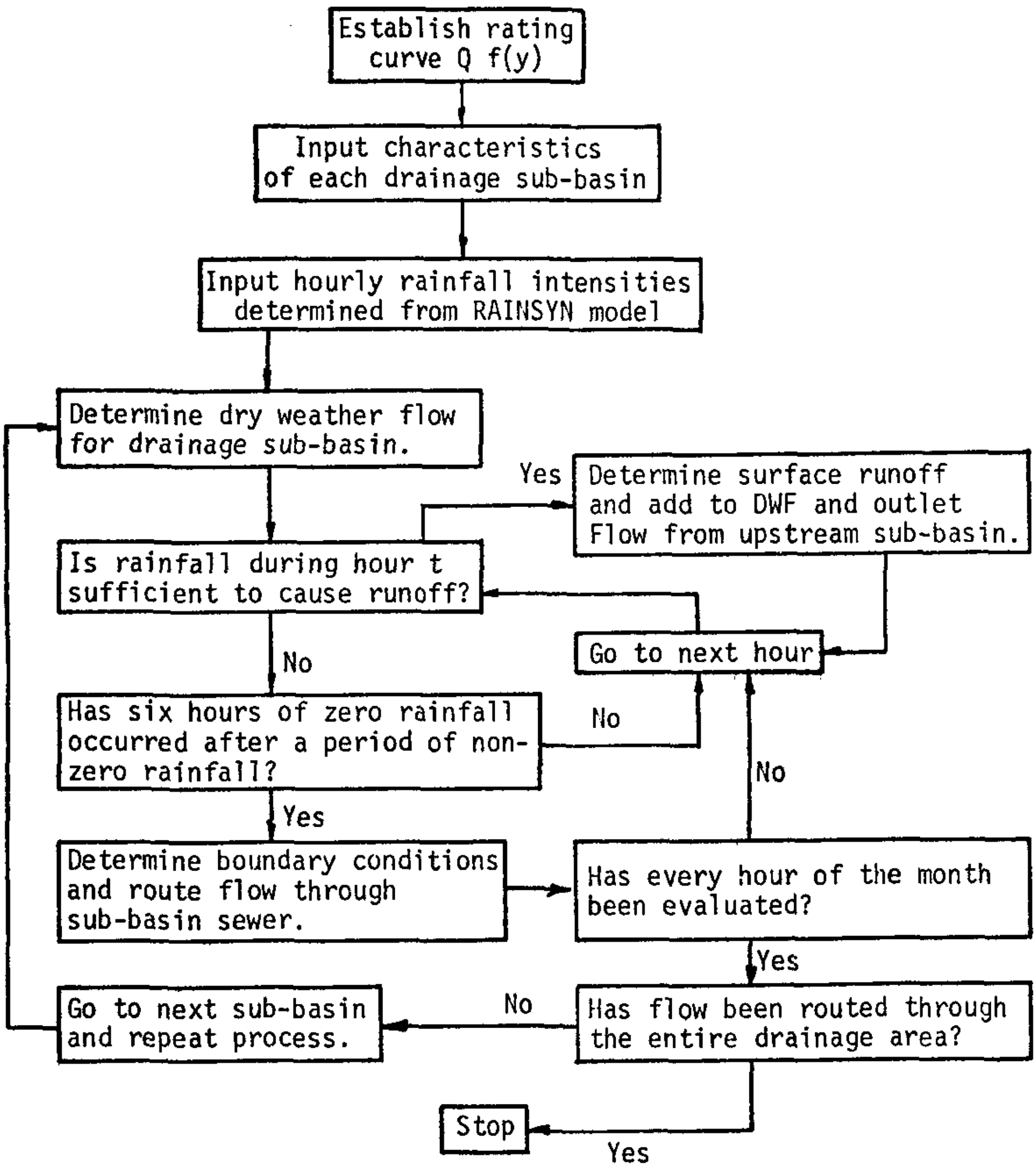
APPENDIX 3C

FLOW DIAGRAM FOR RAINSYN COMPUTER MODEL



APPENDIX 3D

FLOW DIAGRAM FOR HYDROL MODEL



R E P O R T # 4

SUBSURFACE DISPOSAL OF COMBINED SEWER OVERFLOWS

of a

SIMULATION OF CONTROL ALTERNATIVES

FOR COMBINED SEWER OVERFLOWS

R E P O R T # 4

SUBSURFACE DISPOSAL OF COMBINED SEWER OVERFLOWS

Description of Subsurface Disposal System

A possible solution to the problem of combined sewer overflows is to inject the storm runoff into underground formations. The determinations as to whether or not suitable formations exist where injection is proposed would have to be made on an individual, localized basis.

The description of the basic process would be as follows with appropriate modification made when warranted.

During the periods of high storm runoff, the quantity of waste normally by-passed and discharged directly into the receiving stream would first flow through a screen to remove large objects. The flow would then proceed through a high capacity sedimentation unit, such as an inclined tube settler, which would require less space than a standard sedimentation tank. After removal of settleable material, the overflow would be injected into an underground aquifer by injection pumps. After the storm had passed and the storm runoff reduced to a quantity capable of being conveyed by the interceptor sewer and treated, the pumps on the injection well would be reversed. The injected waste, now stored in the aquifer, would be pumped back into the sewerage system at a rate which would not overload the system and would be treated in the normal manner.

This reversal of flow is intended to accomplish two objectives. First, it is reasonable to assume that overflows could not be injected into the underground formation indefinitely since the available storage capacity of the aquifer would be exceeded. By pumping the water out of the aquifer and into the sewerage system, the aquifer capacity could be maintained.

Secondly, the reversal of flow in the underground aquifer should serve as a backwash mechanism to unclog the pores which have been filled with solid material. This will redevelop the well so that when the next overflow occurred, the flow could be readily injected.

Because of the wide variations in flow, it is proposed that several sedimentation units and injection wells would be utilized in a parallel manner capable of operating separately or together.

Advantages of this system would be: (a) The naturally occurring storage capacity present in the pores of the earth's material would be utilized rather than having to excavate the required volume; (b) Valuable surface areas would not be used for storage, freeing such areas for other development; (c) The entire overflow could be injected preventing any discharge to the stream; (d) Injection facilities could be decentralized and located throughout the drainage area as required.

The system has many potential disadvantages also. Some of these are: (a) Suitable underground formations may not be located where injection is required or desirable; (b) Formations which furnish water for domestic or industrial usage may be destroyed by the injection of wastewater; (c) Public health may be endangered unless chemical and biological contaminants are removed by filtration or adsorption as the wastewater flows through the soil formation; (d) Contaminants such as suspended solids may clog the pores of the aquifer to such an extent that injection is no longer possible; and (e) The cost of providing the required injection facility may be so large that the system would be economically infeasible.

Review of Subsurface Disposed Technology

Disposal methods. Use of underground formations for the disposal of waste is not new. Private on-site disposal systems such as cess pools and septic tanks have been used for years in unsewered residential areas. Deep well disposal of highly toxic and non-degradable industrial waste is becoming increasingly popular because of the success of the petroleum industry in disposing of brines into underground formations.

The methods which are commonly used to dispose of the wastewater into underground formations are similar to methods used to artificially recharge groundwater supplies. There are three general methods used for this: surface spreading, recharge pits, and injection wells (1). The choice of a particular method is primarily dependent on the geological conditions present.

Surface spreading involves spreading wastewater over a permeable surface and allowing the water to percolate through the soil to the groundwater. Shallow ditches or low dikes are sometimes used to contain the wastewater in particular areas and to enable various areas to be flooded intermittently. The retention of high percolation rates is a major problem in most areas. Scarifying the surface periodically or maintaining a growth of vegetation on the surface is also reported to be helpful (2).

The feasibility of renovating secondary sewage effluent by use of surface infiltration basins was evaluated by Bouwer, et al (3) as part of the Salt River Project in Phoenix, Arizona. Infiltration rates ranged between 2 and 3 ft/day at a water depth of one foot. The rates were highest for grass-covered basins and lowest for gravel-covered basins. Intermittent flooding periods of 2 to 3 weeks and 10 days of drying in the summer and 20 days in the winter resulted in the maximum infiltration of approximately 400 ft/year. Suspended solids, BOD, and fecal coliform were almost completely removed as the water seeped through the soil. Nitrogen was partially removed in grass-covered basins as were phosphates and fluorides. Recovery of the renovated wastewater was accomplished by pumping wells located in the basin. The quality of the water was suitable for unrestricted irrigation and primary-contact recreation.

Where shallow impermeable formations form a barrier impeding the direct downward percolation of water, an excavated shaft, trench or pit may be used. This method would offer advantages where land costs are high, since the amount of surface area required would be small. Skodje (4) has conducted laboratory tests of gravity shafts which would be used to recharge waters from the Sheyenne River to water bearing deposits located beneath 50 to 150 feet of clay and silt. The shafts would be bored through the impervious cover penetrating the total depth of the aquifer and then backfilled with granular material to provide a passage for the recharge water to the aquifer. Since most of the clogging would occur in the upper layers of the shaft, it would be easy to maintain the recharge rate by removing the clogged material periodically and replacing the material with clean gravel.

Injection wells are normally used when the impervious layers are very thick and in areas where surface area is limited. The physical construction details of an injection well are similar to a producing well (5).

Existing injection facilities and projects. In order for injection wells to be justifiable economically, they are normally used for multiple purposes. There are several types of recharge wells; those screened above the water table are considered to be diffusion wells and those screened below the water table may be either shallow or deep injection wells. The capacity of injection wells is usually less than their specific yield as pump wells. The major advantage of recharge wells is the small amount of land usage required, and the fact that they may be placed on the right-of-ways of public roads. The major disadvantage is the degree of advanced treatment which may have to be provided to prevent clogging.

Butler, et al, has reported on tests conducted for the University of California, Richmond Project (6). These tests were directed primarily toward determining the rate and extent of travel of bacteria and determining if treated domestic sewage would have any adverse effect upon a normal aquifer. The injection well penetrated about 100 feet into the ground, including a 5 foot thick artesian aquifer. The permeability coefficient was found to be approximately 1400 gal/day-ft^2 and the flow velocity in the aquifer was estimated to be 26 ft/day at the injection well. Fresh water containing 10, 20, or 27 per cent settled sewage was injected into the aquifer and samples were taken from 23 surrounding monitoring wells.

It was found that bacteria traveled a maximum distance of about 100 feet in the direction of normal groundwater movement. Biochemical Oxygen Demand (BOD) was used to measure the travel of organic material and it was found that the BOD dropped sharply in the first 10 feet from the injection well. Ammonia was apparently rapidly adsorbed on the aquifer material while the limits of travel of nitrates, phosphates and other anions and cations were not determined.

Clogging of the well was not due to the organic solids carried by the domestic sewage, but primarily due to suspended and colloidal solids. Clogging was heaviest in the direction of flow in the aquifer, and a bacterial slime growth was also experienced. Intermittent pumping for short periods of time was effective in redeveloping the well.

Injection of reclaimed wastewater into an underground aquifer constituted the second phase of a reclamation study conducted by the Los Angeles County Flood Control District at the Hyperion Waste Treatment Plant (7). Reclaimed water was supplied to the recharge well at an injection rate of 0.3 cfs. Prior to the recharge, the effluent was chlorinated to reduce the concentration of slime-producing bacteria.

During operation of the injection well a characteristic rapid increase in the injection head usually occurred even with water. After a few days, the head became more or less constant, and subsequent increase was due to clogging. It was determined that the clogging was caused by deposition of suspended solids. However, effluent from the Hyperion plant containing as much as 6 per cent solids could be injected, if the solids were of suitable particle size, fully oxidized and contained a minimum of inert clay or silt.

A zone of biological activity was believed to exist within 50 to 150 feet of the injection wells which accounted for most of the observed reduction in organic material and nitrogen compounds. Odors were found to increase while color decreased with distance from the injection well.

The Orange County Water District in California (8) studied the feasibility of constructing a barrier to sea water intrusion with one element of the barrier to be a pressure ridge created by a line of injection wells about 4 miles inland. One source of water for the injection wells was trickling filter effluent. After 7 months of injection, the following conclusions were reached: (a) The treated trickling filter effluent is injectable and does not cause excessive clogging; (b) Coliform bacteria have appeared sporadically 100 feet from the injection well, but human intestinal viruses were not observed; (c) Many of the chemical constituents undergo rapid changes as the injected water travels in the confined aquifer; (d) Odor and taste persist in the injected water and could be a serious detriment to utilizing reclaimed wastewater as a source of injection water.

Wastewater reclamation by artificial recharge has been practiced for many years in Long Island, N. Y. by using recharge basins and injection wells (9,10,11). This has provided a means of supplementing the naturally available water supply, preventing salt water intrusion into the fresh water supply, and also eliminating the need to install expensive trunk and outfall sewers to the ocean.

At present, there are more than 1,000 recharge wells returning used groundwater back to the underground reservoirs. This is mainly due to state regulations which prohibit the construction of new industrial wells with capacities exceeding 69.4 gpm unless the water is returned in an uncontaminated condition to the ground. Most of the water is used for cooling, and the temperature of the groundwater has increased.

Tests are now being conducted to determine the feasibility of constructing a line of "barrier-injection" wells across the entire south shore of Nassau County. Initially 27 mgd of highly treated wastewater will be injected into a series of carefully spaced wells that will be screened at various levels in the underground aquifer. The purpose of these tests will be to obtain information regarding the physical and chemical factors that control the rates and injection pressure at which the treated effluent can be injected.

In Israel underground water storage is an integral part of the country's national water supply system (12). Part of the artificial recharge of these supplies is connected with flood water control and wastewater reclamation. However, the majority of the recharge operations were directed toward replenishing over-exploited aquifers, preventing sea-water intrusion, and providing seasonal and perennial water storage in the system.

In 1966-1967 over 100 million cubic meters of water were recharged by approximately 100 wells. The water is recharged by specially constructed recharge wells, and also, by dual purpose wells which serve as recharge wells during the winter period and as production wells during the summer. The water used originated in Lake Galilee and is chlorinated and settled before recharge. It is of drinking water quality at the recharge well, but gradual clogging is observed as recharge operations continue. Subsequent use of the water has shown a deterioration in the water quality as indicated by high coliform bacteria counts.

It was concluded that the suspended organic matter was probably the principal cause of the clogging. During recharge this material is filtered out in a relatively small section of aquifer in the vicinity of the well and accumulates there. This causes a considerable decrease in permeability and the specific discharge of the well. During the period between recharge and pumping, the accumulated organic material undergoes various biochemical processes in which oxygen is depleted, ammonia appears, and an objectionable odor occurs when the water is removed.

The Grand Prairie Region of Arkansas was selected by the U. S. Geological Survey (13) for an investigation of the fundamental principles pertaining to artificial recharge of groundwater reservoirs in alluvial deposits by injection wells. The subsurface geology of the region consists of Quaternary sand and gravel which had been partially dewatered by irrigation pumping. This had resulted in an acute water supply problem for the rice farmers and others living in the area.

Most of the problems encountered during this study involved clogging of the wells and aquifer. The causes of the clogging were air entrainment; suspended solids and microorganisms in the recharge water; precipitates caused by chemical reactions between the groundwater, aquifer, and recharge water; and temperature differences between the injected water and the warmer native groundwater and weather conditions.

Geohydrological aspects of injection wells. A study conducted by the American Water Works Association (2) concluded that in determining the technical feasibility of a recharge operation, it was important to study the geological features of the area, such as the depth and size of the aquifer, the permeability and transmissibility of the soil, the direction of groundwater flow, and the location of existing wells. Obvious geological features, such as alluvial fans or wide river valleys were found to be useful in determining areas to investigate. In general, where the recharge water could flow in a radial direction from a recharge source, higher rates of recharge were obtained than if the groundwater flowed in only one direction.

The vertical and horizontal permeability of the aquifer determined the maximum recharge rate of a particular site. Where the geological structure of the aquifers contained several formations, the formation with the lowest permeability determined the maximum recharge rate, Estimates of overall permeability could be calculated from the results of pumping tests on existing water wells in the vicinity. However, the permeability varies depending on the temperature and viscosity of the recharge water, the ion exchange occurring within the soil, and the extent of clogging occurring at the soil-water interface.

The principal difference in the hydraulics of groundwater flow as compared with the unconfined water of surface supplies is the resistance of the aquifer to the flow of water. Whereas, water in a lake or other surface supply assumes a uniform level, the top surface of the underground water may exhibit marked gradients for long periods of time. When water is pumped from an aquifer, a cone of depression is formed and this cone fills very slowly when pumping is stopped. The reverse is true when water is recharged into an aquifer, and an injection mound or cone of elevation is formed and this mound may persist for a long time after the recharge is stopped.

The term "injection head" is used to describe an injection well's ability to distribute water into underground formations. The term is somewhat analogous to a pumping well's drawdown. The injection head may be defined as the height of the column of water within the injection well casing required to overcome friction losses encountered as the water moves into the aquifer.

The term "clogging rate" is used to describe the rate of change of the injection head. This term is a good quantitative measure of the clogging mechanisms in an injection well. Clogging is a major problem and is due to the filtering action at the water-soil interface of the well. The reasons for the clogging are numerous and many of the factors may occur simultaneously. These reasons were summarized as follows by a task group of the American Water Works Association reporting on experiences with injection wells (5).

Inorganic or organic dissolved materials could result in a biological growth either inside or outside the casing and perforation, at the aquifer face, or in pores of the aquifer some distance from the well. A slime growth was found to be the most troublesome and was caused by new organisms injected into the aquifer or by stimulation of dormant microorganisms within the formation. Clogging was also due to the chemical products resulting from bacterial activity or chemical action. Examples of this type of a chemical product were sulfate reduction or the precipitation of iron salts.

Mechanical clogging was caused by improper well design and construction methods. Drilling mud had to be completely cleared, and precautions taken in the installation of perforations and in packing the gravel. The possibility existed for erosion of layers of fine-grained materials in the vicinity of the casing, if excessive redevelopment was used.

Suspended solids in the injected water could result in blocking the pores in the gravel packing at the interface of the gravel pack and the natural aquifer, or blocking aquifer interstices themselves. Where clogging occurred, it was dependent on the size of the particles, the gradation of the formation, and the flow velocity. Solids causing clogging may be the result of erosion of residual layers of drilling mud or from within the formation itself. Soil particles may reorient themselves into a denser, less permeable pattern.

A third form of clogging was chemical in nature. Air bubbles were released from the injected water or gases could be released when the temperature of the injected water was less than the groundwater.

Other chemical aspects of clogging usually involve some type of chemical reaction. In the presence of high concentrations of dissolved oxygen or chlorine, metabolic products of autotrophic bacteria may be precipitated. This could include hydroxides of iron, ferrous bicarbonate and metal sulfides of sulfur, or calcium carbonate. There may also be a chemical interaction between the dissolved chemicals in the injected water and the soil itself, as well as reaction between the injected water and the native groundwater of different chemical characteristics.

After the recharge well had clogged, it was necessary to redevelop the well by flushing the clogged material from the pore spaces. Several different methods were available for this depending on the type of well construction used. High rate pumping was used for moderate redevelopment. Redevelopment should be practiced as a preventive maintenance process periodically rather than waiting for severe clogging to build up requiring more drastic measures such as the use of acids, compressed air, or explosives.

Movement of pollutants. It is very important in any recharge operation to know the extent that bacteria may travel. A large number of bacteria can be effectively removed as the water moves through a few feet of sand. The removal process involves mechanical straining, settling in fine passages between grains, and chemical precipitation by electrical charges or temperature changes. While water-bearing sand formations provide full opportunity for effective mechanical straining, fractured and creviced rock formations may not afford conditions for bacteria removal.

Systematic investigation of pollution travel in underground formation was initiated in California (7). Results have shown that in all cases bacteria behaved as particulate matter, penetrating to a maximum distance in the soil initially and regressing as clogging developed and organisms died.

The behavior of viruses in groundwater is still not well understood. Significant data on the removal of viruses have been produced at the Santee Project in California (14). Activated sludge effluent which was 100 per cent virus positive was reduced to 100 per cent negative after being spread on the surface and allowed to percolate through the soil to groundwater. This removal is probably the result of an adsorption phenomenon, and the fact that survival time of most viruses is thought to be relatively short.

Chemical contaminants frequently travel 2 to 30 times further than bacterial pollutants. There is a tendency for many contaminants to be retained on earth materials by chemical and physical sorption (15). The degree to which contaminants are retained depends on the character of the contaminant and of its host water, and of the earth materials. As a general rule, clays have a greater sorptive capacity than sands, resulting in an inverse relation between the factors of sorption and permeability.

Aquifers have a considerable capacity to adsorb ammonium ions and phosphorous compounds. The Chemical Oxygen Demand (COD) is also reduced by underground travel. Carbonaceous and nitrogenous materials are the two main groups of substances that exert an oxygen demand in the injected water. This oxygen demand usually far exceeds the supply of oxidizing agents such as dissolved oxygen and chlorine present. From this, it can be concluded that most of the reductant removal is by mechanisms other than oxidation.

Laboratory Simulation of Injection Process

Purpose of laboratory simulation. The purpose of simulating the proposed injection system in the laboratory was to obtain preliminary data which would aid in evaluating the technical practicality of the proposed system in solving combined sewer overflow problems. The major factors which were of importance in the evaluation were the effects of clogging which increased the injection pressure head, the movement of chemical and biological pollutants through the aquifer, the characteristics of the injected waste, and the characteristics of the aquifer materials.

Description of simulator. A laboratory model was designed and constructed to simulate actual recharge conditions which would exist in the field as closely as possible. A simplified definitive sketch of these field conditions is given in Figure 4-1. A flow rate of "Q" is injected under pressure at the well, and flows radially away from the point of injection with a decreasing radial velocity. Flow in the confined aquifer is assumed to parallel to the confining layers and uniform over the depth of the aquifer. In addition, the assumption is made that the flow net is symmetrical about the injection well.

For the flow described above, it was not necessary to model the entire flow area. Instead, sectors of the circle of influence were selected as shown in Figure 4-2. The sides of these sectors coincide with radial streamlines emitting from the injection well, and as the radial distance from the well increased, different streamlines with smaller intersecting arc angles were selected which prevented the sectors from becoming too

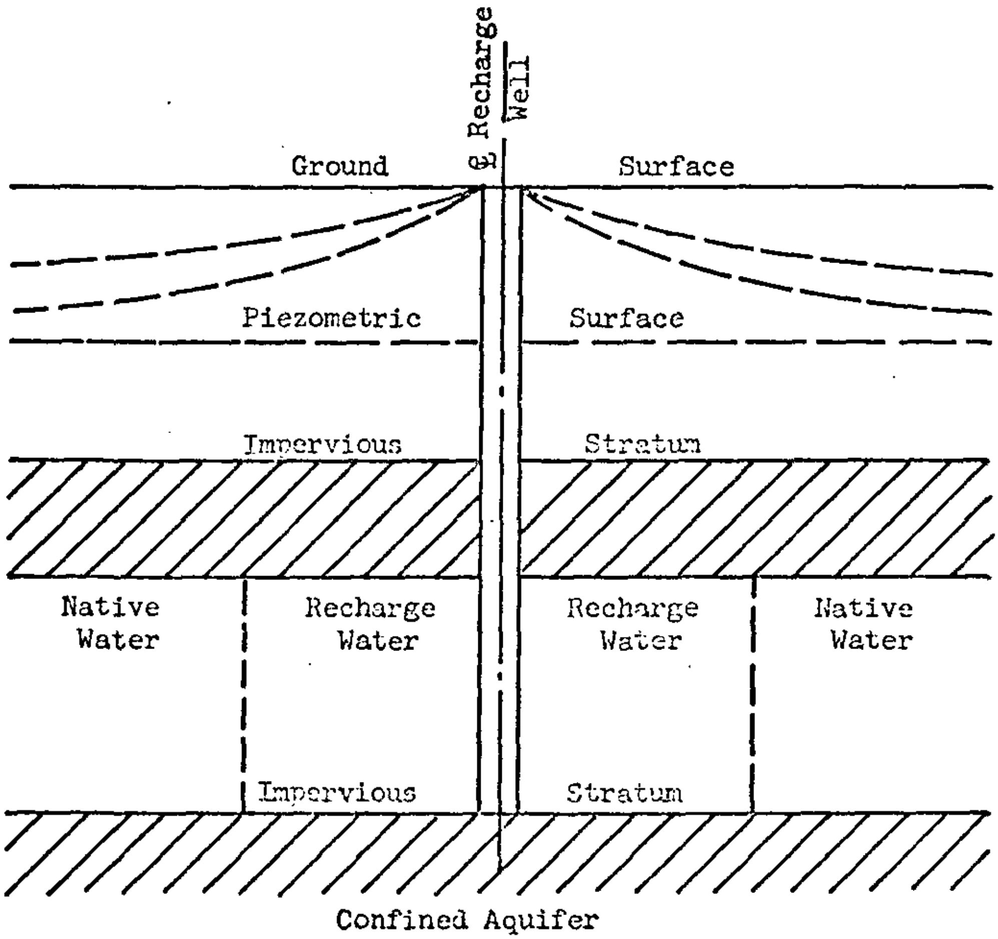


Figure 4-1 Definition Sketch of Recharge or Injection Well

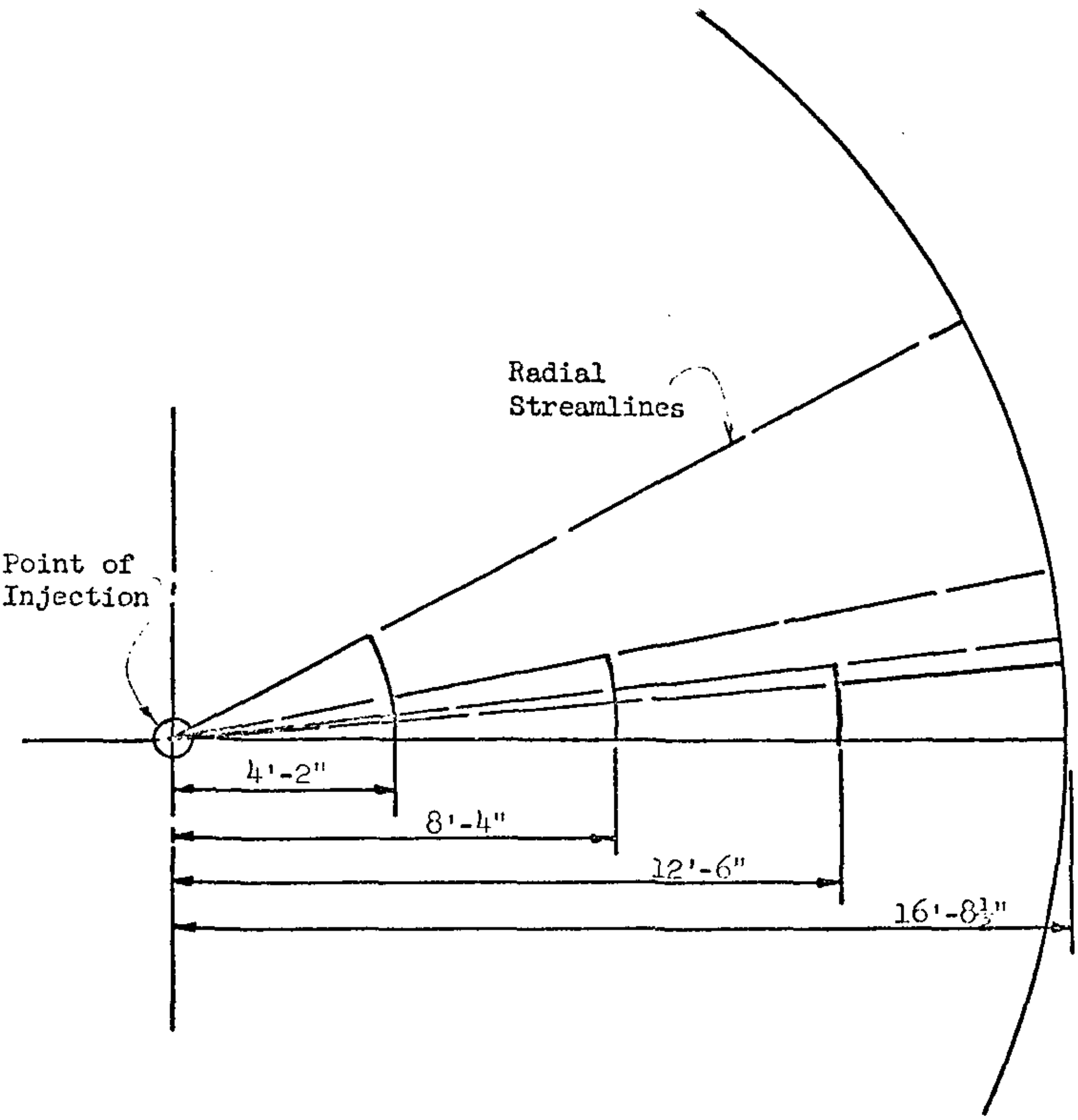


Figure 4-2 Plan View of Sections in the Injection Well Circle of Influence

large. The flow had to be adjusted from one sector to the next since for steady-state conditions the flow between any two streamlines remains constant and is proportional to the intersecting arc angle. To accomplish this the discharge from each sector was divided, with only the portion of the flow equal to the ratio between the two arc angles being allowed to flow into the next sector. For example the flow into sector 2 (Q_2) would be determined by the following equation.

$$Q_2 = Q_1 \frac{\theta_2}{\theta_1} \tag{4-1}$$

Where

Q_1 = flow from sector 1, gpm

θ_1 = arc angle of sector 1, degrees

θ_2 = arc angle of sector 2, degrees

The sectors were fabricated from 5 ft x 3 ft x 1/2 inch acrylic plastic sheets. The arc angle for the first sector was chosen to be 27.9°; the second sector 10.81°; the third sector 6.66°; and the fourth and final sector 4.82°. This permitted the modeling of flow for a radial distance of 16.7 feet. The inside depth of each sector was 6 inches.

The bottom, sides, and ends of each sector were cemented together using a special acrylic adhesive to insure water tight joints. Since the tops had to be removed periodically, they were screw connected to the sides and ends and a leak proof compound placed in the joints before tightening. Angle iron bracing was then placed across the tops of the sectors and bolted to the support frame to add strength to the plastic sectors.

The separate sectors were then arranged in a manner which would minimize the amount of laboratory floor space required. Figure 4-3 shows a plan view of the final arrangement used. Photographs of the sectors are also included in Figure 4-4.

Two 55 gallon plastic drums were used as feed tanks in order to maintain an uninterrupted injection flow and are shown in Figure 4-5. One tank was filled with a particular concentration of waste and the injection process started. While this tank was being emptied, the second tank was being filled with a different concentration of waste. At the proper time the flow could be switched from one tank to the other by simultaneously opening and closing the gate valves from the two tanks.

A variable flow rate pump shown in Figure 4-6 was used to inject the waste into the simulator. This pump had a flow range of 1-5 gpm and was easily adjusted when necessary to maintain a constant flow rate into the simulator. In addition a globe valve, placed ahead of the pump, provided added control of the injected flow rate.

Plumbing connections from one sector to the next were fabricated from 3/4 in. cast iron pipe or 3/4 in. flexible plastic pipe (Figure 4-7). Two discharge pipes from each sector were used and by adjusting a globe valve in either pipe the proper division of flow was accomplished. Rotameters were used in each of the discharge pipes from the first sector in order to measure the total flowrate being injected and also to obtain the ratio of the flow rates in the two lines. A flowmeter was necessary only

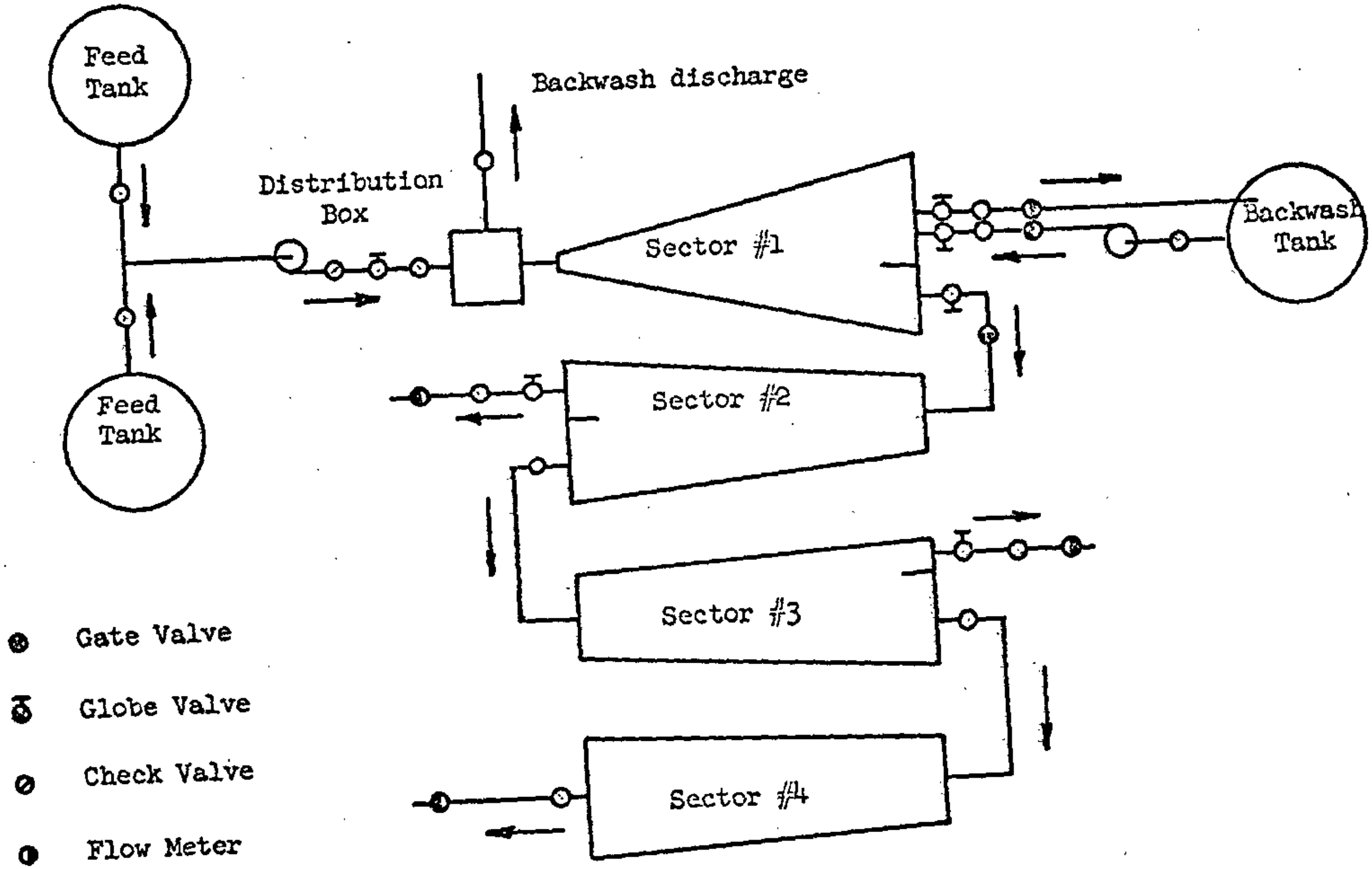


Figure 4-3 Plan View of Laboratory Simulator

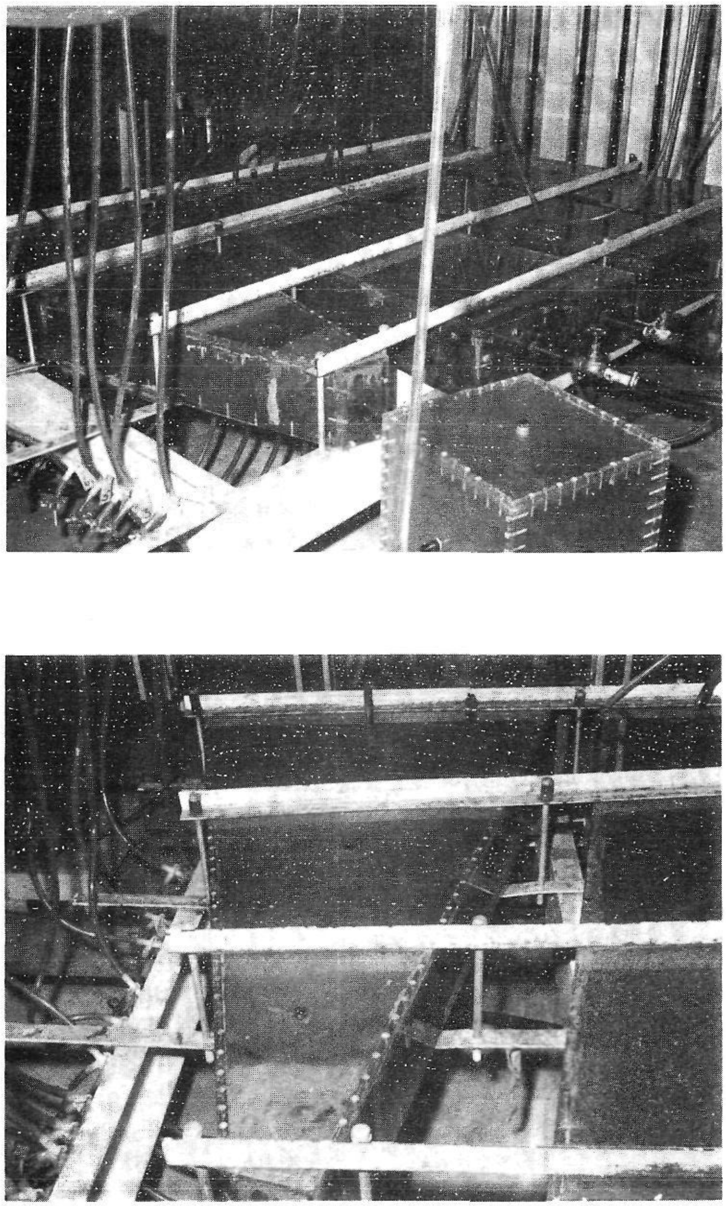
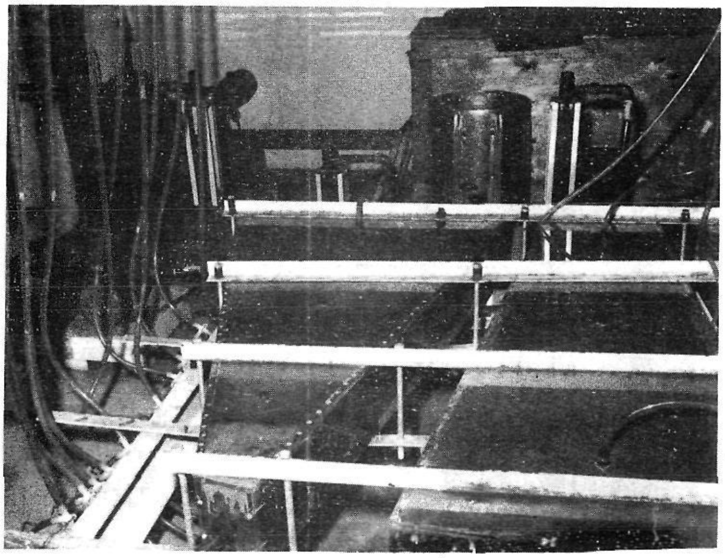


Figure 4-4. Photographs of Laboratory Simulator-Sectors.



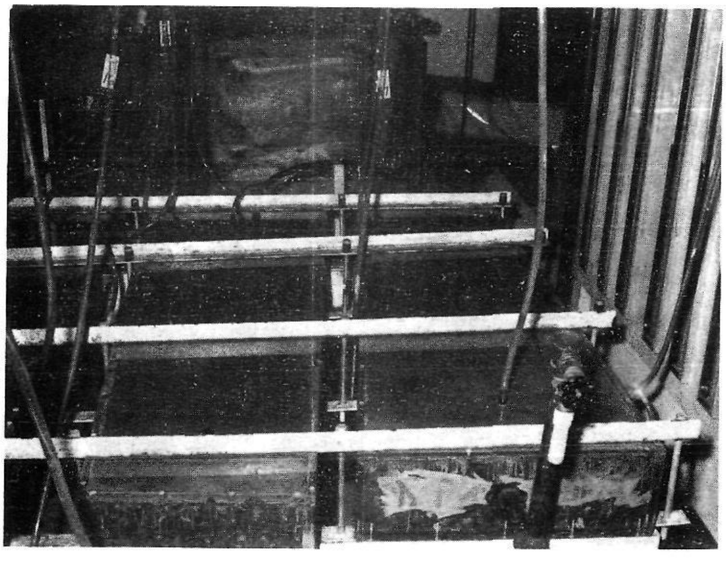


Figure 4-4 (cont.)

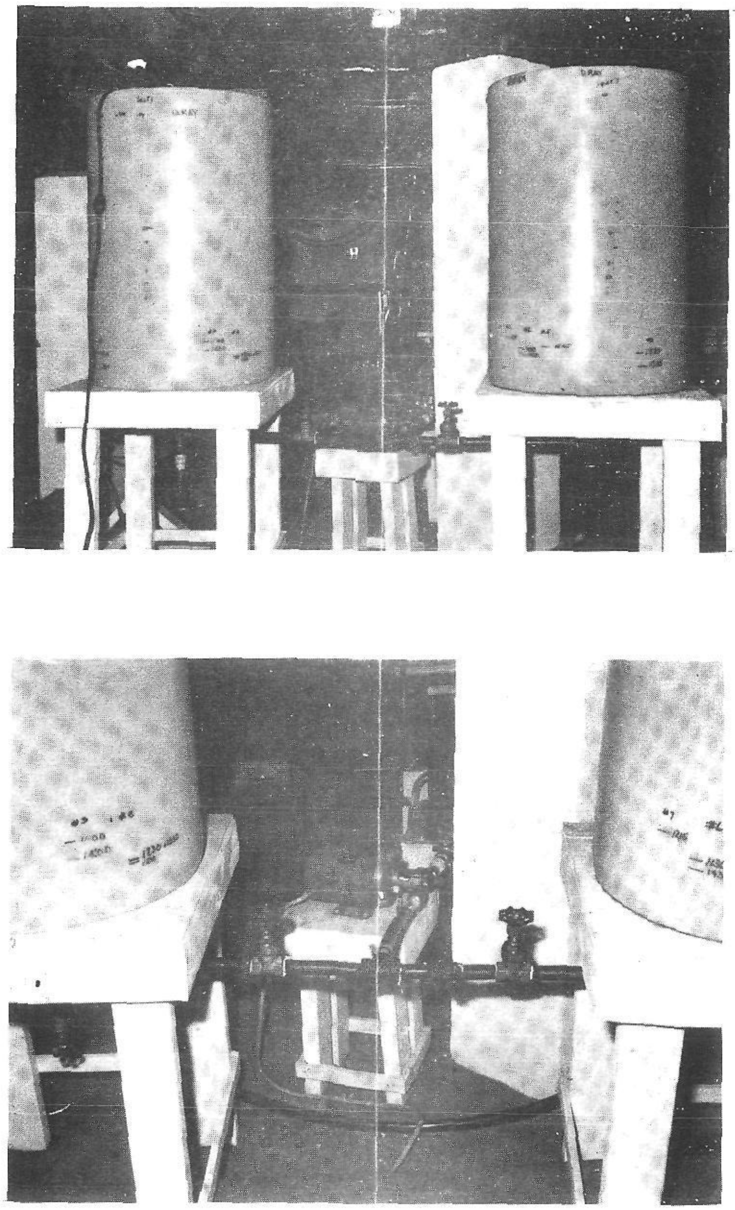


Figure 4-5. Photographs of mixing tanks containing injection water and control valves.

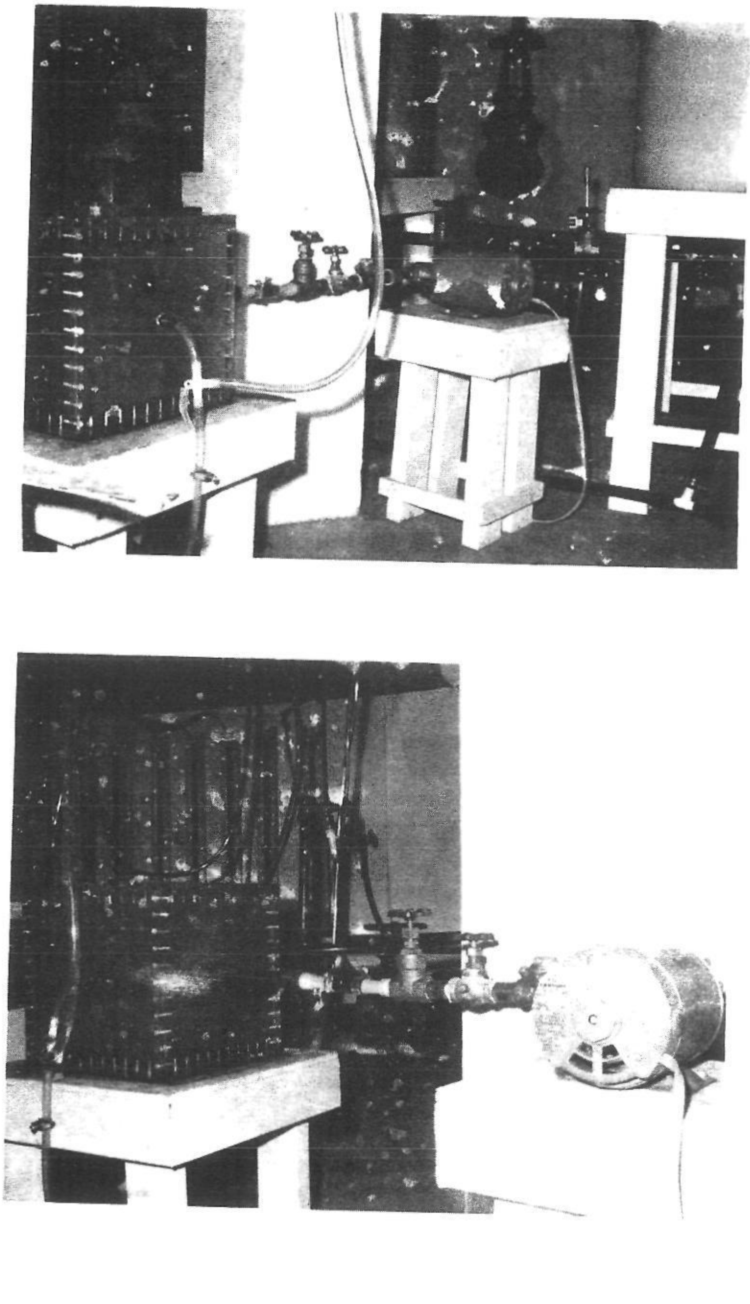
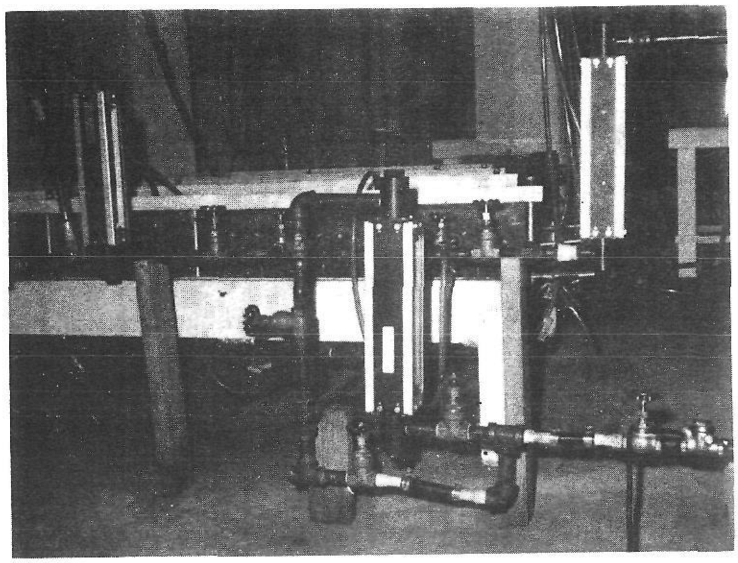


Figure 4-6. Photographs of injection pump and distribution chamber.



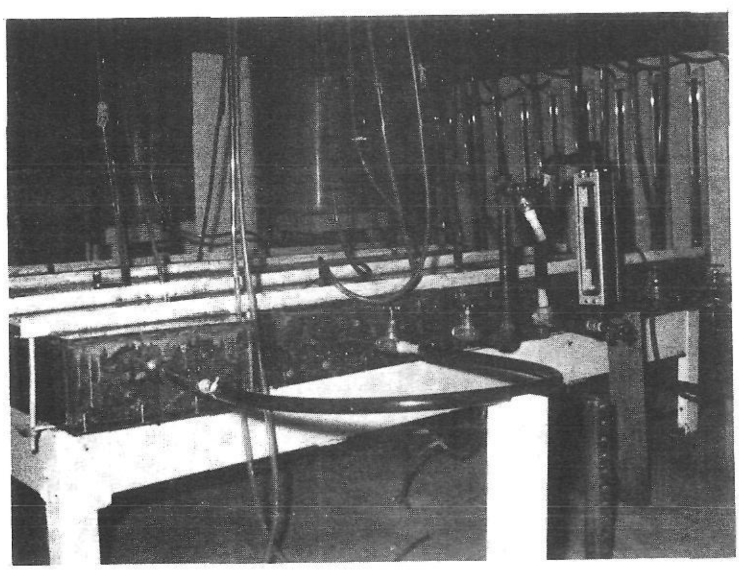


Figure 4-7. Photographs of plumbing connections.

in the waste discharge lines of the remaining three sectors, since the flowrate being injected into each of these sectors was known and the flowrate in the other discharge line could be calculated from the principles of conservation of mass. Globe valves were used on these sectors also to obtain the proper ratio of flow in the two discharge pipes.

The sectors included sampling and pressure ports located through the bottom along the radial center line at distances as shown in Table 4-1. The ports were spaced so that an accurate representation of the changes in pressure head and characteristics of the flow could be determined. Plastic tubing, 3/8" I.D., was connected from the ports to mercury manometers for pressure determination and included a bleed-off valve for sampling the flow at that point as shown in Figure 4-8.

The waste discharge stream from the first sector was collected in a third 55 gallon plastic tank to be used in backwashing the first sector. After completion of a run, this water was pumped back through sector one with the flowrate being measured by a rotameter and controlled by a globe valve located between the backwash pump and rotameter. This arrangement is shown in Figure 4-9.

Each sector was filled with a soil medium and at each end approximately 6 inches of coarse gravel was added to distribute the flow evenly over the flow area. The gravel had an average diameter of approximately 1 1/2 inch and did not appreciably affect the characteristics of the waste.

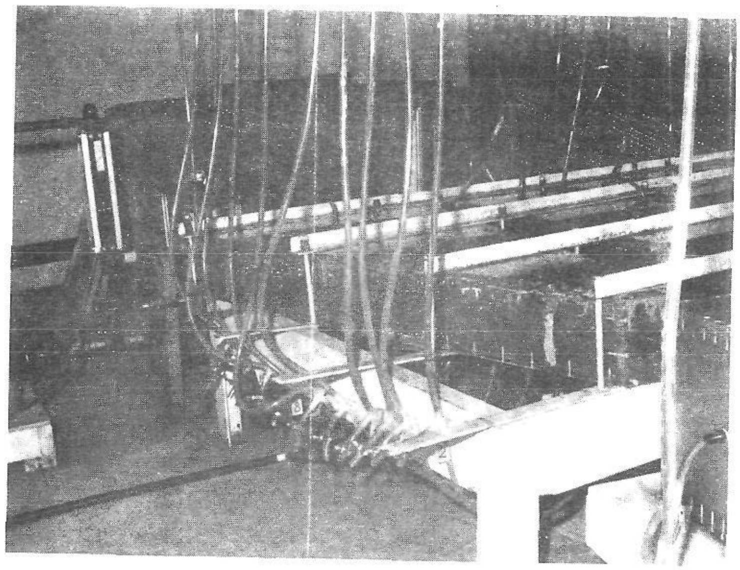
Complete runs were made with the two different soil mediums in the simulator. Gradation curves for these soils are shown in Figure 4-10. The first soil had an effective size (maximum diameter of the smallest 10

TABLE 4-1

Distances of Sampling and Pressure Ports

from the Center of the Well

Port Number	Distance in Feet
1	0.0
2	0.2
3	0.5
4	0.7
5	0.95
6	1.40
7	1.90
8	2.4
9	2.9
10	3.4
11	3.8
12	4.0
13	6.1
14	8.2
15	10.2
16	12.45
17	14.62
18	16.70



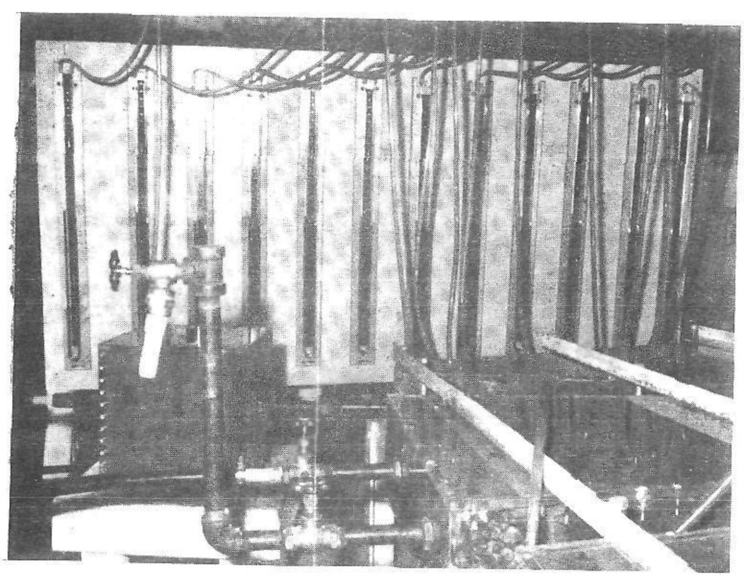
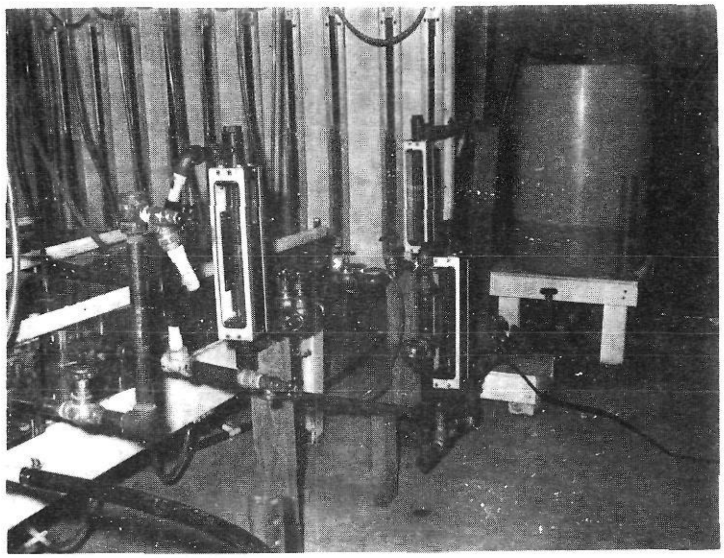


Figure 4-8. Photographs of pressure and sampling ports, and manometer board.



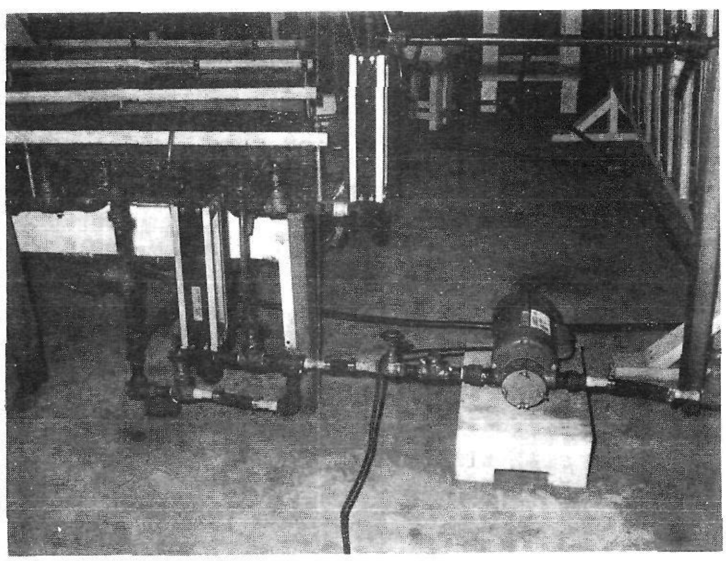
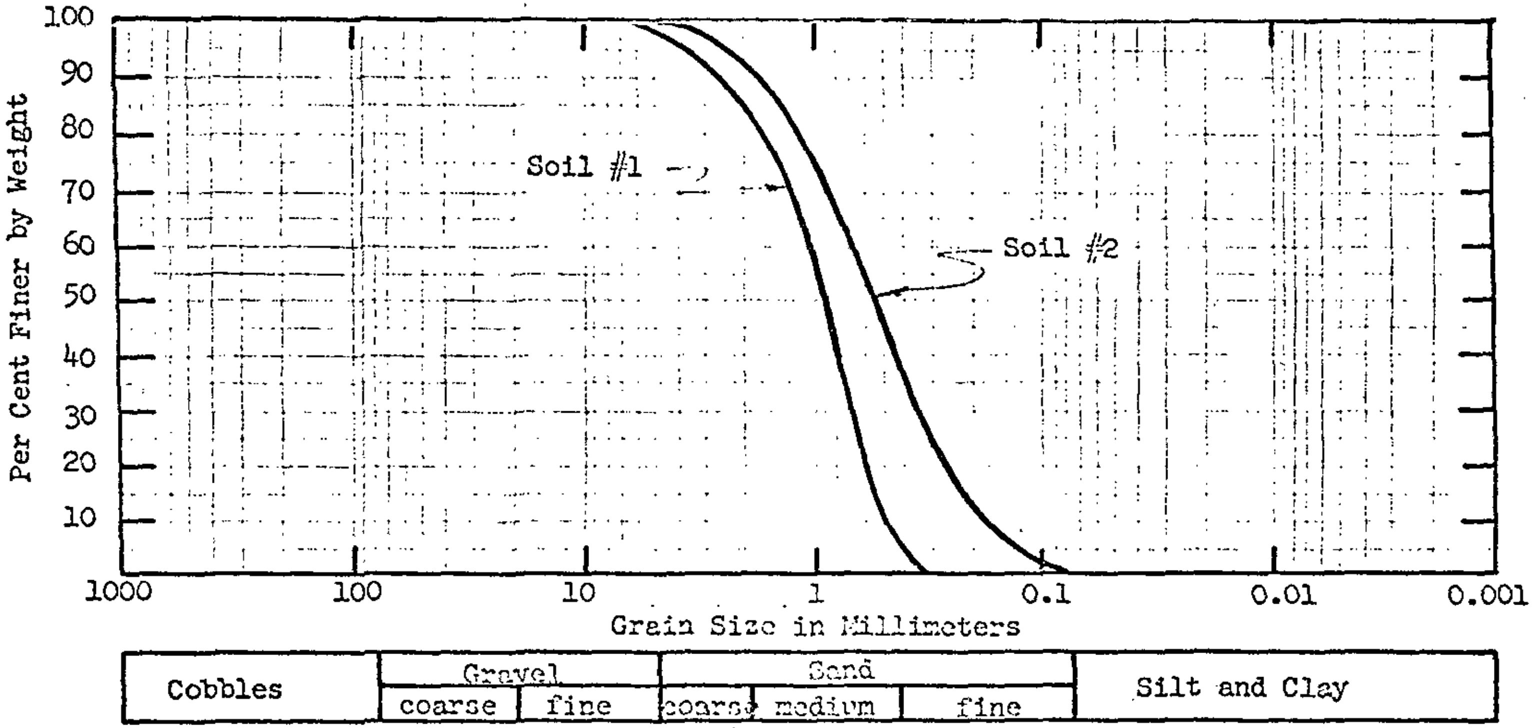


Figure 4-9. Photographs of backwash pump and control valves.



UNIFIED SOIL CLASSIFICATION SYSTEM, CORP OF ENGINEERS

Figure 4-10 Classification of Soils Used in Laboratory Simulator per cent) of 0.5 mm, and an uniformity coefficient (maximum diameter of the smallest 60 per cent divided by the effective size) of 2. This soil is classified as a coarse-medium sand and had a high coefficient of permeability. Because of the low uniformity coefficient difficulty was encountered in preventing this soil from washing while under injection pressure flow.

The second soil had a higher uniformity coefficient of approximately 4, and an effective size of 0.17 mm, which indicated a well graded sand with more fines than the first soil. This reduced the coefficient of permeability but provided a much more stable soil which did not wash as much as the first soil.

Extreme care was necessary in placing the soil in the sectors to insure uniform compactness. Dampened soil was placed in approximately 1 1/2 to 2 inch layers and thoroughly compacted before the next layer was placed. Difficulty was still encountered in getting the sector filled completely so that there would not be a gap between the top of the sand and the top of the sector. This was overcome by hydraulically injecting additional soil through openings in the top after it had been securely fastened to the sides and ends.

Operation of the simulator. The sectors were filled with soil and completely saturated by injecting tap water until a constant piezometric head was established. This corresponded to the initial equilibrium conditions of the actual aquifer which would be saturated and under pressure. To accomplish this the free discharge lines from sectors 2, 3, and 4 were raised approximately 8 inches above the top of the sectors.

excess of that which was required to go through sector 2, into storage tank 3 to be used for subsequent backwashing. The remaining flow was piped into sector 2. The flow in the waste stream from sector 2 was discharged into a floor drain since sector 2 was not backwashed as was sector 1.

The operation of sectors 3 and 4 was the same as used in sector 2.

After the injection process was completed, sector 1 was backwashed. This was accomplished by closing valves in lines going from sector 1 to sector 2, from sector 1 to storage tank 3, and from the distribution box to tanks 1 and 2. The valve from the distribution box to the discharge line was opened and the stored water in tank 3 pumped back through sector 1. The backwash rate was greater than the injection rate and of shorter duration. A rotameter was located in the line from tank 3 to sector 1 to record the actual rate of backwashing.

Collection of data. Samples were taken and piezometric water levels determined immediately before injection began, 45 minutes later and then at 90 minute intervals until the conclusion of the injection cycle. This was performed at each of the port openings given in Table 4-1.

The piezometric water levels, determined while the waste was being injected, were compared with the piezometric levels, determined when water without any waste was being injected. The difference between the two determinations at a particular point could be analyzed as discussed in the next section to yield a direct indication of clogging which had occurred.

Sodium chloride was added to the injection water and the chloride ions used as a tracer element. Since the chloride ion is not adsorbed or

After the initial equilibrium conditions were established a bla
run was made. This consisted of plain tap water, without any sewage
being injected through the simulator. Pressures were recorded as a
tion of time and distance until a steady state condition was achieve
that is the piezometric heads no longer changed with time. From thi
information, the permeability of the soil material as it actually ex
in the sectors was determined. In addition, pressure curves were co
structed and used as a basis in comparing the changes in pressure wi
time because of the clogging which occurs when a fluid containing se
is injected.

The waste which was injected consisted of a mixture of settled
tary waste and tap water. Three basic types of pollutographs (vari
of concentration of pollutants with time) were used to simulate the
tions in pollutant concentration in the combined sewer overflow. Th
first pollutograph had a concentration of 20% settled sewage initia
was decreased 5% every 90 minutes to a minimum value of 5%. The sec
pollutograph had an initial concentration of 5% and was increased 5
90 minutes to a maximum concentration of 20%. The third pollutograp
started with a concentration of 5% and was increased to 15% and ther
and was then decreased to 15% and 5% in 90 minute increments. The f
pollutograph would represent an overflow where "first flushing" occu

The waste water which was to be injected was pumped from tank 1
at a constant rate of 1 gpm into a distribution box which correspond
the well casement. Valve #3, shown in Figure 4-3, was closed and th
from the distribution box went through sector 1, and was divided at
wide end of sector 1 into two streams. One stream discharged the fl

altered by flow through soil formations, it can be used to establish a basis for evaluating the adsorption and reduction of other chemical constituents of the injected waste. This procedure is also explained in the next section of this report. The concentration of chloride ions was determined by the Argentometric Method given in Standard Methods (17).

Other characteristics of the injected waste which were analyzed included total carbon, dissolved oxygen, total solids, and total coliform.

Total carbon was determined according to procedures given in Methods for Chemical Analysis of Water and Waste, 1971, published by the Environmental Protection Agency (18). A micro sample of the injection waste was injected into a Beckman Organic Analyzer which included a catalytic combustion tube which is enclosed in an electric furnace thermostated at 950°C, which converts the carbonaceous material to CO_2 and steam.

Dissolved oxygen concentrations were determined by the Probe Method given in reference (18). A Yellow Spring Instrument (YSI) Model 54 was used.

Total solids which consist of homogenous suspended and dissolved solids in the samples were analyzed by the method given in reference (18). Determination of the suspended and filterable solids were performed on certain selected samples.

Total coliform was analyzed by the Membrane Filter Standard Test - A, given in Standard Methods (17). No enrichment was used and prepared filter was placed directly on an M-Endo agar medium and incubated for 24 hours.

Results and Analysis

Clogging. The clogging of the interstices of the soil by the solids contained in the injection waste was evaluated by observing the changes in the piezometric pressure levels at different radial distances and times throughout the injection process. The equation relating the piezometric pressure, quantity of flow and the radial distance from the point of injection, can be developed as follows.

Referring to Figure 4-1 and using Darcy's Law for flow through a porous medium

$$V_r = -K\ dh/dr \tag{4-2}$$

where

V_r = velocity at radial distance r, ft/sec

h = piezometric pressure, ft

K = coefficient of permeability, ft/sec or gal/day-ft^2

and the flow equation

$$Q = A \cdot V_r \tag{4-3}$$

where

A = $2\pi rb$

b = height of the confined aquifer

the following equation is derived

$$Q = -2\pi rbK\ dh/dr \tag{4-4}$$

Separating variables and integrating between the limits r_w and r,

$$H/Q = h_w/Q - \frac{1}{2\pi bK} \ln r/r_w \tag{4-5}$$

Dividing the piezometric head (H) by the flowrate (Q) normalized the effect of variable flowrates and permits direct comparison of the H/Q terms regardless of the flowrate.

Equation 4-5 can be linearized into a straight line equation

$$y_i = a + sx_i \tag{4-6}$$

if we let

$y_i = H_i/Q$

$a = h_w/Q$

$s = -1/2\pi bK$

$x_i = \ln r_i/r_w$

and plotting H/Q versus r/r_w on semi-log graph paper will yield a straight line relationship.

Using the method of least squares, a regression (or predicting) equation is obtained which will minimize the vertical deviation between the observed values of H/Q and the predicted values of H/Q at $\ln r/r_w$. This regression equation should also be a straight line and would be

$$\hat{y} = \hat{a} + \hat{s}x_i \tag{4-7}$$

where

$\hat{y}$ = predicted value of y at x_i

$\hat{a}$ = estimated value of h_w/Q

$\hat{s}$ = estimated value of $-1/2\pi bK$

$x_i = \ln r_i/r_w$

The estimated values of $\hat{a}$ and $\hat{s}$ are obtained by substituting the observed data into the following equations

$$\hat{s} = \frac{\sum_{i=1}^{n} x_i y_i - n\overline{xy}}{\sum_{i=1}^{n} x_i^2 - n\overline{x}^2} \tag{4-8}$$

and

$$\hat{a} = \overline{y} + \hat{s}\overline{x} \tag{4-9}$$

where

$\overline{x}$ = mean value of x_i

$\overline{y}$ = mean value of y_i

n = number of observations

If the observed data obtained for t=0 in each injection experiment is used in equations 4-8 and 4-9, an estimate of the initial soil characteristics can be found, since the injected water at this time would be free of any waste. The coefficient of permeability is found from the estimate of $\hat{s}$ as

$$K = -1/2\pi b\hat{s} \tag{4-10}$$

and the initial piezometric head h_w/Q would be equal to $\hat{a}$.

Assuming that the deviations between the observed values and the predicted values of H/Q were solely a function of errors, the mean square of the deviations about the regression equation could be used to estimate σ_E^2 or

$$S_E^2 = \frac{\sum_{i=1}^{n} y_i^2 - \hat{a}\sum_{i=1}^{n} y_i + \hat{s}\sum_{i=1}^{n} x_i y_i}{n-2} \tag{4-11}$$

The predicted value of y given by the regression equation (4-7) could also be used to predict a range of individual y values associated with a given x value (19). When this was used a 100 γ per cent prediction interval was provided by

$$\left.\begin{matrix} L' \\ U' \end{matrix}\right\} = \hat{y} \pm t_{\gamma/2,\ n-2}\, s_{\hat{y}} \tag{4-12}$$

where

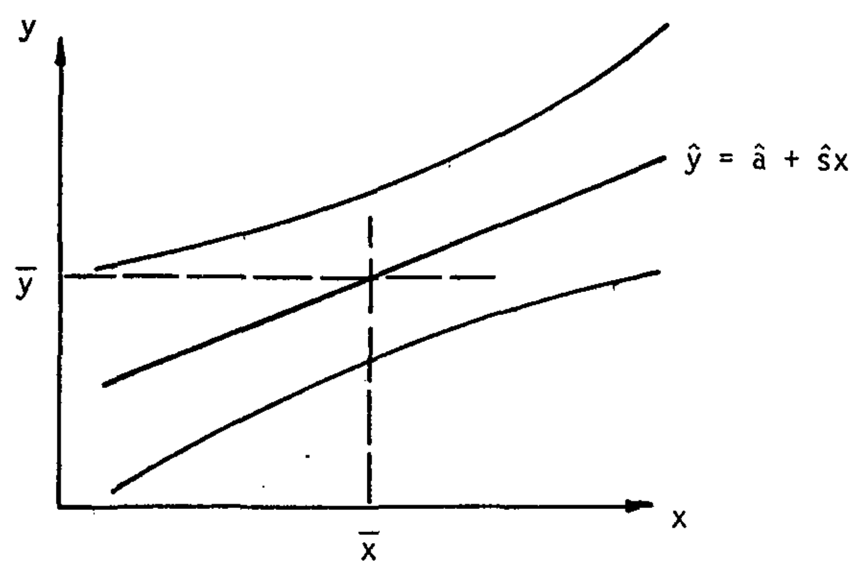
$$s_{\hat{y}}^{2} = S_E^{2}\left[1 + \frac{1}{n} + \frac{(x_i - \overline{x})^2}{\sum_{i=1}^{n} x_i^2}\right]$$

$t_{\gamma/2,\ n-2}$ = percentage points of the student t distribution with n-2 degrees of freedom.

Equation 4-12 can be rewritten after substitution of $\hat{y}$ and $s_{\hat{y}}$ as

$$\left.\begin{matrix} L' \\ U' \end{matrix}\right\} = \hat{a} + \hat{s}x_i \pm t_{\gamma/2,\ n-2}\, s_E \sqrt{1 + \frac{1}{n} + \frac{(x_i - \overline{x})^2}{\sum_{i+1}^{n} x_i^2}} \tag{4-13}$$

A graphical representation of the prediction interval specified by equation 4-13 would be as follows:



The 100 γ per cent prediction intervals for the deviation of observed values of H/Q from the regression equation were used to determine whether or not subsequent values of H/Q, observed when a waste is injected, were within the range of deviations to be expected because of random errors or were the results of other influences. If the values of H/Q were above the upper predicting interval (U'), it can be stated with 100 γ per cent confidence that the aquifer was beginning to clog at that point causing the H/Q values to increase.

The results of this analysis with γ = 0.95 for the first soil, which was classified as a coarse to medium sand were inconclusive, because of the instability of this soil. It does not appear, however, that any of the experiments, which included duplicate injection of the three different pollutographs discussed previously, resulted in clogging of the aquifer at any point. In fact, the H/Q values dropped below the lower prediction interval in some cases which must be due to excessive short circuiting caused by shifting of the soil.

The results obtained when the combined water was injected into the second soil which contained a larger percentage of fines were much more conclusive. These results are shown in Appendix 4A, Figures 4A-1 through 4A-6.

For a pollutograph in which the percentage of waste increased with time, the results are given in Figures 4A-1 and 4A-2. The H/Q values at r/r_w = 1 were observed to decrease during the first stages of the injection when a low concentration.of waste was present in the injected water.

However, when the concentration of waste reached a percentage of 15 per cent after 3 hours of injection, the H/Q value began to increase and after 6 hours and 45 minutes the value was slightly above the upper prediction interval. This then indicated that clogging was beginning to occur.

In the case in which the concentration of waste is highest at the beginning of the injection period and decreased thereafter, the aquifer becomes clogged much more severely. The results of this type of polluto-graph are given in Figures 4A-3 and 4A-4. The H/Q values begin to increase almost immediately and after 2 hours and 15 minutes exceed the upper prediction interval. After 5 hours and 15 minutes the H/Q values at $r/r_w = 2$ are also above the upper prediction interval, which indicates that the clogging of the interstices is occurring further into the aquifer. However, it is significant that the clogging is still confined to the first 12 inches of the aquifer. This means that perhaps the aquifer could be redeveloped by reversing the flow in the aquifer.

In the third case in which the concentration of waste was increased initially to a maximum value and then decreased, clogging was indicated after the concentration reached 15 per cent and continued even after the waste in the injected water was decreased. These results are shown in Figures 4A-5 and 4A-6. This is reasonable to expect since the clogging of the interstices of the aquifer would accumulate and trap more of the waste solids even though the percentage decreased. This also indicated that recovery of the flow capacity of the aquifer is not possible due to decomposition of the trapped solids while a waste is still being injected even though the waste is diluted. Other means must be employed to restore the flow capacity.

Again the extent of the clogging was confined to the first 12 inches of the aquifer.

Chemical and Biological Reduction. The analysis of the effectiveness of the soil medium in reducing chemical and biological pollutants included in the injected waste water was complicated due to several factors. Changes in the observed concentration of the pollutants may have been the result of any or all of the following: variation in the concentration of waste added to the injected water in order to simulate actual overflow conditions; deviation in the flow through the simulator from ideal plug flow; and adsorption, filtration and microbial decomposition of the pollutant.

The purpose of this simulation study was to evaluate only the last effect. To accomplish this, the effects of the first two had to be isolated. This was achieved by using chlorides as a tracer to determine concentration versus time curves (pollutograph) for a conservative pollutant at different radial distances. Although some chlorides were present in the settled sewage, additional amounts were needed to accurately detect changes in concentration. Sodium chloride was, therefore, added to the injection water so that the resulting chloride concentration would be approximately proportional to the percentage of settled waste as follows:

% settled waste	mg/l of Cl^-
20	100
15	75
10	50
5	25

Since the concentration of chloride ions is not altered by the flow through the soil, any changes would have to be the result of either variations in the concentration being injected or flow patterns in the simulator. It was assumed that the effects of these factors would be the same on the other pollutants as on chlorides. For example, if the concentration of chlorides decreased 20% then the concentration of other pollutants, such as total carbon, would also be 20% less.

The concentration versus time curves resulting from injecting a conservative pollutant for equal periods of time would have the same time base, and the ordinates would be proportional to the concentration being injected. This is similar to the principle proposed by Sherman (20) in his development of the unit hydrograph theory. Referring to Figure 4-11, if a constant concentration is injected into the simulator, for a duration of time (t_D), the resulting pollutograph would have a time base (t_B) which is the time required for all of the pollutant to pass a given location. The ordinates at times t_1, t_2, t_3, and t_4 would be c_1, c_2, c_3, and c_4, respectively. If the concentration being injected was reduced by 50%, then the time base would remain the same, and the ordinates would be reduced 50%.

The observed concentration at time t_i can be expressed as U_iC, where U_i is a proportionality constant relating the injection concentration (C) to the observed concentration (c_i). The value of the proportionality constant can be obtained by dividing the observed concentration at t_i by the known concentration being injected.

$$U_i = \frac{c_i}{C} \tag{4-14}$$

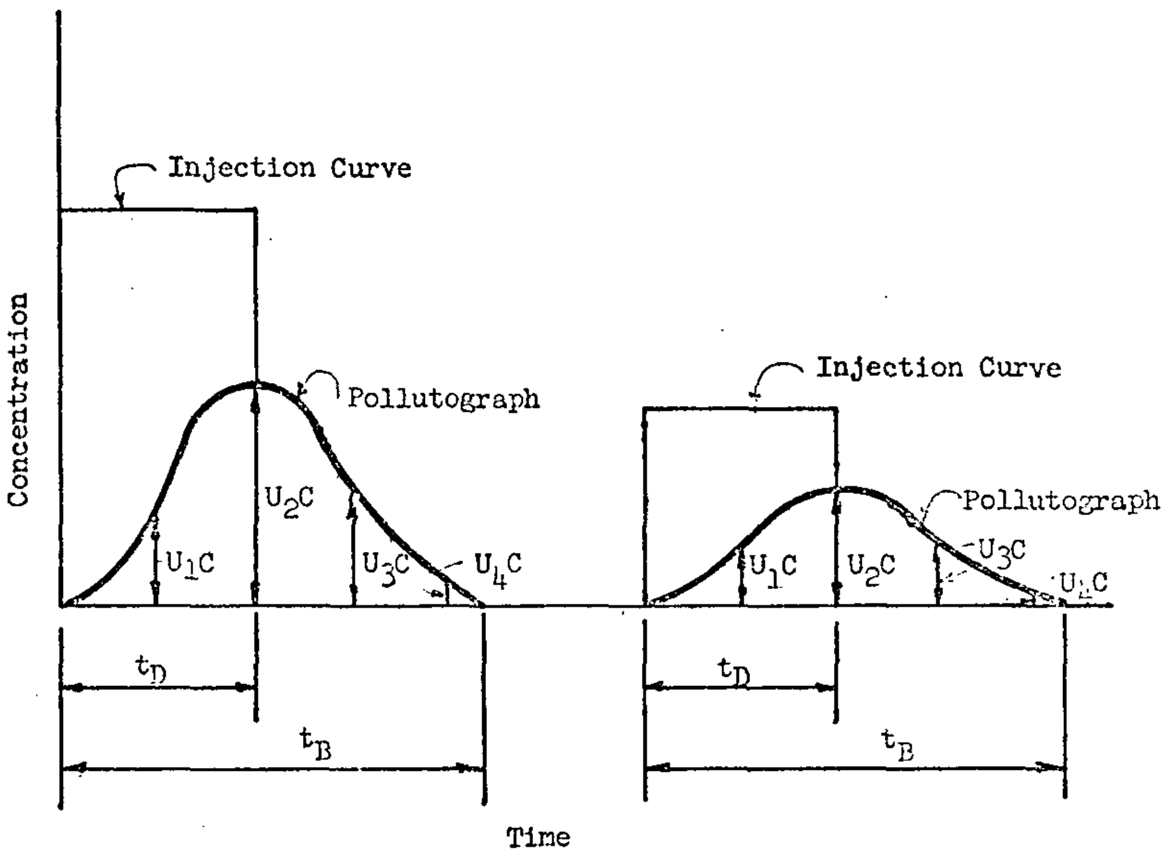


Figure 4-11 Pollutograph at a Given Location Resulting from Injecting a Constant Concentration of a Conservative Pollutant

The analysis of more complex pollutographs resulting from a series of injections of equal duration, but varying concentrations, can be accomplished in a manner similar to the analysis of runoff from complex storms (21). Each injection period, shown in Figure 4-12, will produce a pollutograph with ordinates proportional to the injected concentration, and having equal time bases. The ordinates of the total pollutograph would be the sum of the ordinates of the individual pollutographs at any time. Figure 4-13 graphically illustrates this addition. These total ordinates would be as follows:

$$c_1 = U_1C_1 \qquad (4\text{-}15a)$$

$$c_2 = U_2C_1 + U_1C_2 \qquad (4\text{-}15b)$$

$$c_3 = U_3C_1 + U_2C_2 + U_1C_3 \qquad (4\text{-}15c)$$

With the injection concentration (C_i) and the observed concentration (c_i) known, the coefficients U_i can be found. Beginning with equation 4-15a U_1 is determined and its value substituted into equation 4-15b and U_2 determined. This could be continued until all of the U_i values are found. The values of U_i would represent the ordinates of a pollutograph resulting from injecting a unit concentration and may be referred to as a "unit pollutograph".

Improved accuracy in the values of U_i may be obtained by considering a number of different injection series. A least square analysis could then be performed which will minimize the squared deviations between the observed value (c_i) and a predicted value of the pollutograph ordinate

$$(\hat{c}_i = U_iC_1 + U_{i-1}C_2 + \ldots + U_2C_{i-1} + U_1C_i)$$

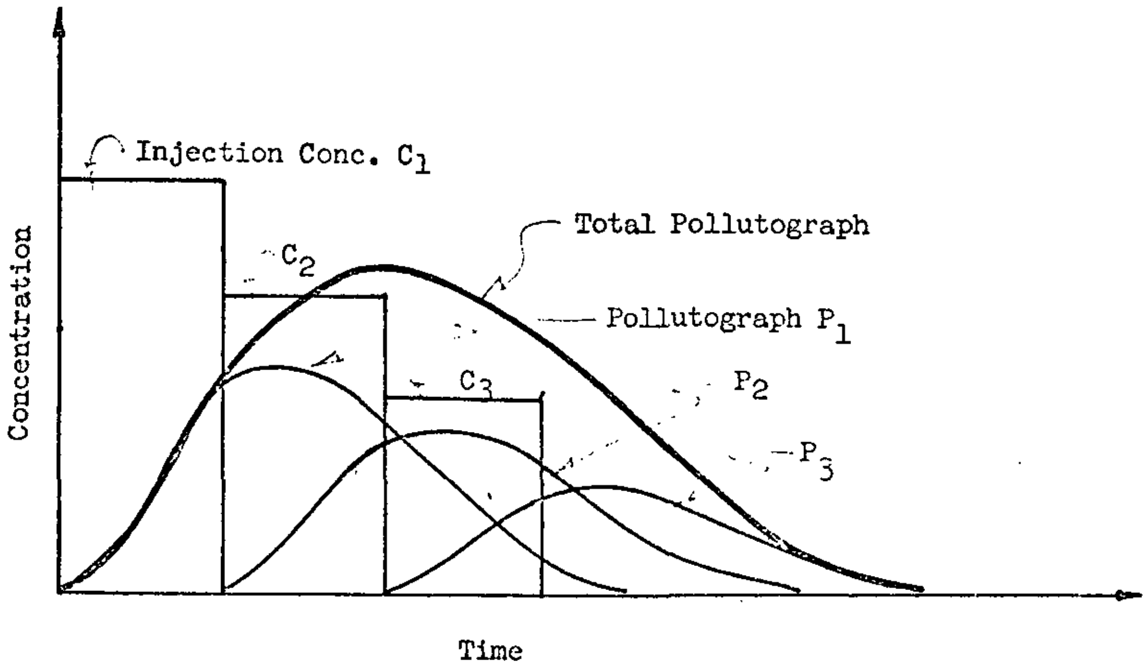


Figure 4-12 Pollutographs Resulting When a Series of Varying Concentrations of Conservative Pollutant Is Injected

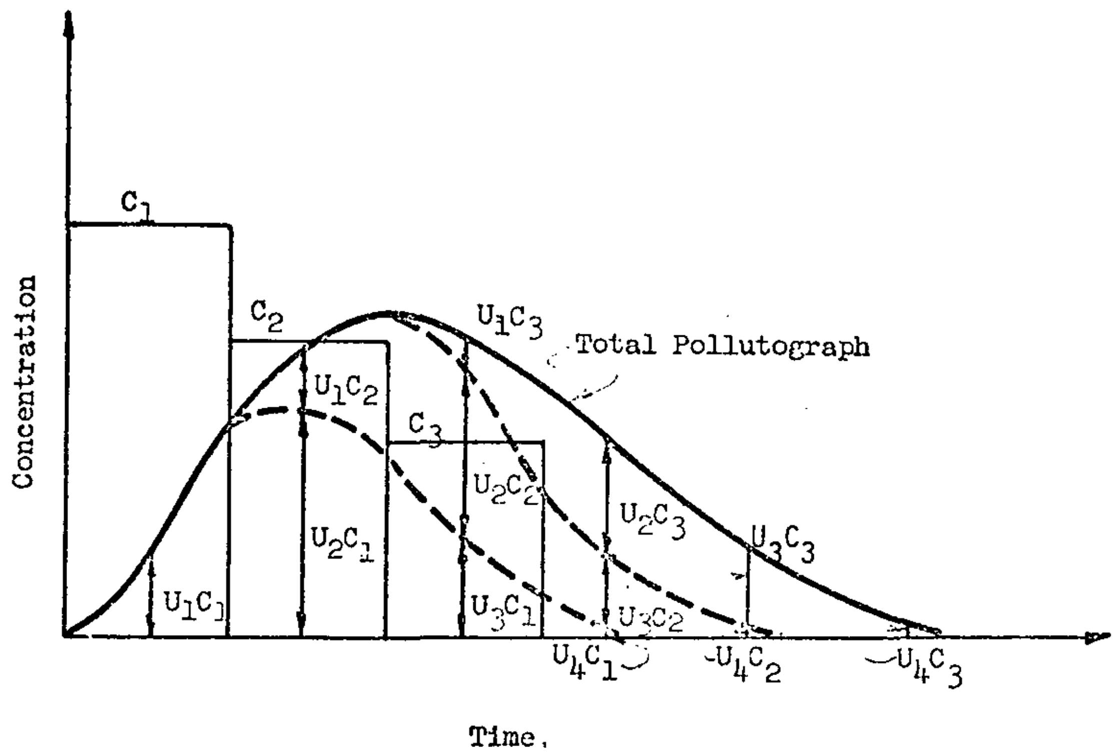


Figure 4-13 Summation of Pollutographs to Yield a Total Pollutograph

This analysis yields the following equations:

$$U_1 = \frac{\sum_{k=1}^{n} c_1^k \, C_1^k}{\sum_{k=1}^{n} (c_1^k)^2} \tag{4-16}$$

and for $i > 1$

$$U_i = \frac{\sum_{k=1}^{n} c_i^k \, C_1^k - U_{i-1} \sum_{k=1}^{n} c_2^k \, C_1^k - \ldots - U_1 \sum_{k=1}^{n} c_i^k \, C_1^k}{\sum_{k=1}^{n} (c_1^k)^2} \tag{4-16a}$$

where

n = number of runs used in the analysis.

Evaluation of the above equations using the observed concentration of chloride at various times and radial distances yields the following values of U as a function of time and radial distance.

Radius	Time, mins.			
	45	135	225	315
2'	0.984	0	0	0
4'	0.746	0.246	0	0
6'	0.491	0.393	0.090	0.030
8'	0.204	0.557	0.214	0.049
12'	0.160	0.308	0.289	0.079
16'	0.105	0.175	0.185	0.168

Total pollutographs, which result from injecting known concentrations of other chemical and biological pollutants, were then calculated according to the relationships given in Equation 4-15, using these unit pollutograph ordinates. The calculated pollutographs represent the concentration versus time curve which would be expected if the pollutant concentration was not affected by the soil medium. Comparison of the calculated pollutograph with the observed pollutograph at different radial distances illustrates the effectiveness of the soil in reducing the pollutant concentration.

Graphs showing the calculated pollutographs and the observed pollutograph at radial distances of 2, 4, 6, 8, 12 and 16 feet, for total carbon, dissolved oxygen, total coliform and total solids, are given in Appendix 4B. The solid line is the calculated pollutograph and the dashed line represents the observed pollutograph. The area under each of these curves, multiplied by the flow, would represent the total amount of pollutant expected to pass the given radial point after a certain time and the actual amount of pollutant passing, respectively. The difference in the two amounts would be the amount removed and the difference divided by the expected amount would yield the per cent reduction.

The "step" curve shown represents the injection concentration of the pollutant at the well. Since the initial concentration of total carbon, total solids and total coliform were approximately proportional to the percentage of settled waste added to the injection water, the shape of these curves follows the pattern of the three basic types of injection pollutographs described in the section entitled Operation of the simulator.

The initial concentration of dissolved oxygen was not proportional to the settled waste added. The extent of stirring used in mixing the settled waste into the injection water was the major factor affecting the initial concentration. The amount of waste added did have some effect on the concentration since high percentages of waste produced lower dissolved oxygen concentrations as would be expected.

The results of this analysis when applied to the total carbon constituent of the injected waste for the three basic injection pollutographs are shown in Figure 4B-1 through 4B-6 of the appendix. The per cent reduction of the calculated amount of total carbon at various radial distances was calculated and are given in Figure 4-14.

In the first 2 feet of the simulated aquifer, the per cent reduction was higher for pollutographs which have a low initial concentration of carbon (Type II and III) being approximately 28% as opposed to 21% reduction for the pollutograph which had a high initial concentration of carbon (Type I). If the reduction was basically due to biological decomposition, this would result from organisms having a chance to become established and acclimated to the waste as a gradually increasing concentration of waste passed the 2 foot section. As the radial distance increased, this advantage is minimized and the reduction percentage becomes higher for the initially high concentration of total carbon. At distances farther into the aquifer, the reduction percentage is affected more by the total amount of waste which could "theoretically" have reached that distance in the given time period. This amount is greater for Types I and III pollutographs, and thus the reduction mechanism would be more efficient. The larger amounts of total carbon calculated to pass a given distance for a Type III pollutograph would also explain why the percentage reduction was consistently higher for this pollutograph than the other two.

The decreasing reduction percentage which occurs at radial distances of 12 and 16 feet into the aquifer also indicates a biological decomposition mechanism. The biological organisms are significantly reduced at these

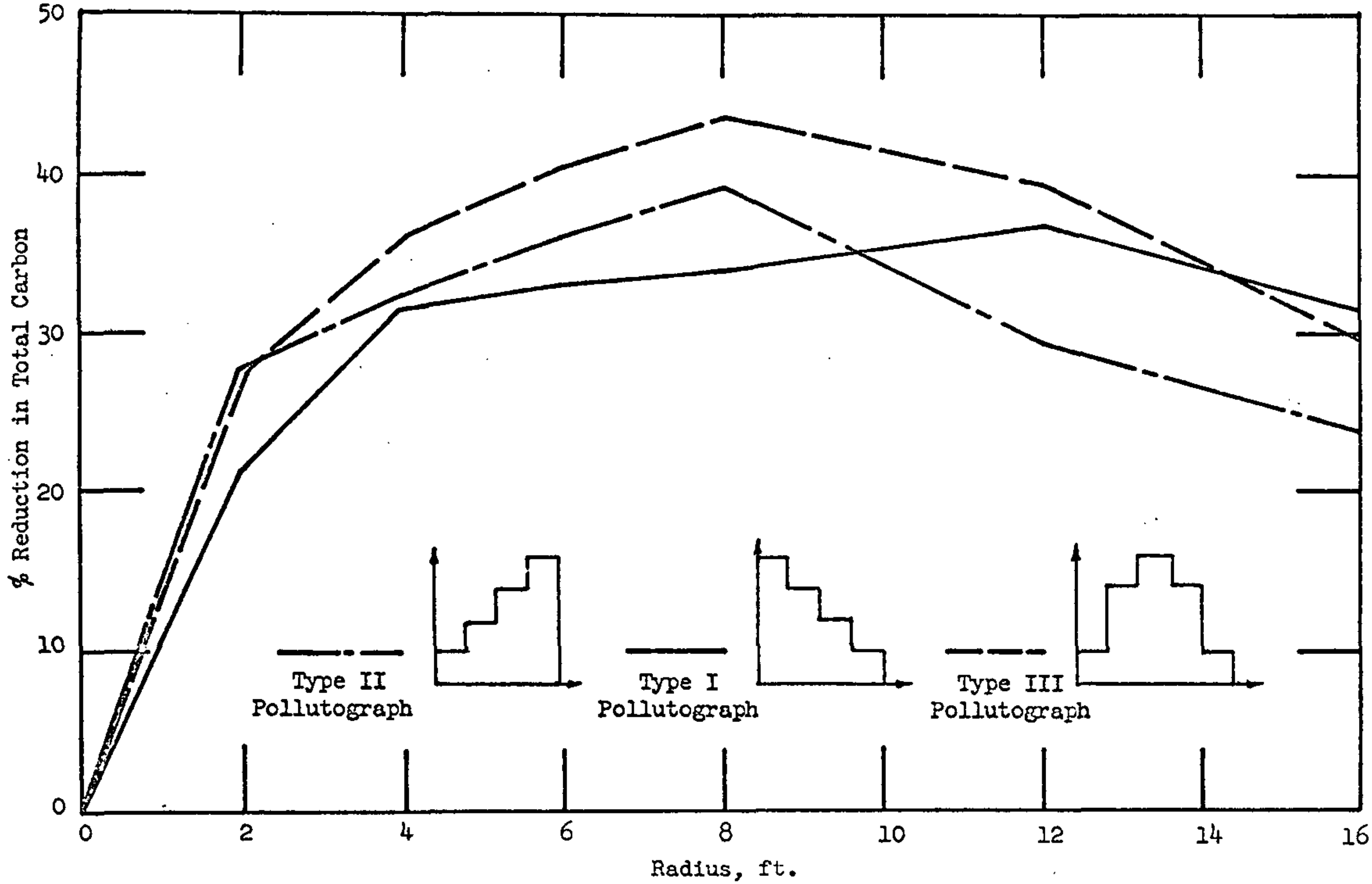


Figure 4-14 Total Carbon Reduction vs. Radius after Six Hours of Injection

distances, as can be seen later, and the reduction in organic waste would not be as efficient in these regions as in the area closer to the point of injection.

Curves showing the calculated concentrations and the observed concentrations of dissolved oxygen as a function of time and distance are shown in Figures 4B-7 through 4B-12 of the appendix. As stated previously, the initial concentration of dissolved oxygen in the injection waste did not follow any discernible pattern as did other constituents. However, the unit pollutograph theory would still be applicable since the injection concentration was known.

The results substantiate the conclusion that the total carbon reduction was due to an aerobic biological decomposition. This is illustrated by comparing the observed concentration curves of total carbon with the observed dissolved oxygen concentration curves. It is noted that these curves in general have opposite slopes at any given point and high concentrations of total carbon coincide with low dissolved oxygen concentrations. This indicates that the dissolved oxygen was being used in the decomposition process in proportion to the amount of total carbon present.

The percentage reduction in the calculated amount of dissolved oxygen at various distances could not be related to the percentage reduction of total carbon. This is reasonable to expect since the biological decomposition of waste requires a certain amount of oxygen regardless of the amount of oxygen available.

Because of this, efforts were made to correlate the total amount of dissolved oxygen used in the decomposition or reduction of an amount of total carbon. Figure 4-15 shows the amount of total carbon and dissolved oxygen removed at given radial distances after 6 hours of injection. The amount of dissolved oxygen utilized is relatively constant regardless of the amount of total carbon removed. This results in a variable ratio between the amount of oxygen required to reduce a unit of total carbon. This relationship is shown in Figure 4-16, and reveals that amount of oxygen required per unit of total carbon removed decreases as the amount of total carbon removed increases.

The effectiveness of the soil medium in reducing the total coliform concentration after 6 hours of injection at various radial distances is shown in Figure 4-17. Pollutographs illustrating the differences in the calculated concentration and the observed concentration of total coliform are shown in Figures 4B-13 and 4B-18 of the appendix.

The reduction percentages for all three initial pollutographs attain a level of approximately 96% in the first 6 feet of aquifer and remains virtually constant thereafter. This indicates that the coliform organisms are removed very rapidly close to the point of injection. This also corresponds to the finding that the biological decomposition of the organic waste was most effective in the first 8 feet of the simulator aquifer.

Total solids pollutograph curves showing the calculated concentration and the observed concentration for the 6 hour injection period are shown in Figures 4B-19 through 4B-23 of the appendix.

Inspection of the curves leads to the conclusion that there was no significant reduction in the total solids. Given the slight amount of clogging which was discussed previously, this is not unexpected.

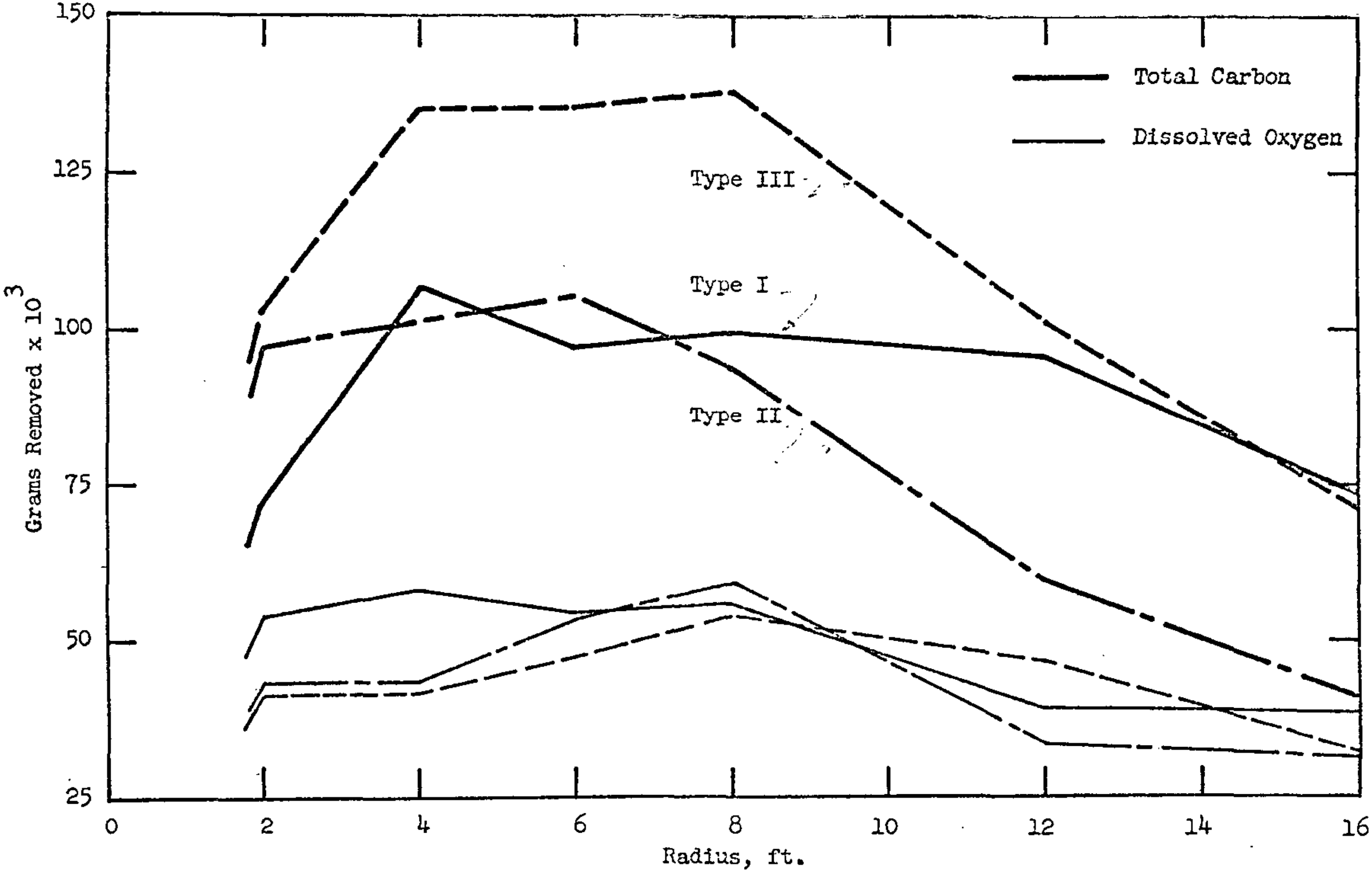


Figure 4-15 Comparison of Total Carbon Removal with Dissolved Oxygen Removal vs. Radius after Six Hours of Injection

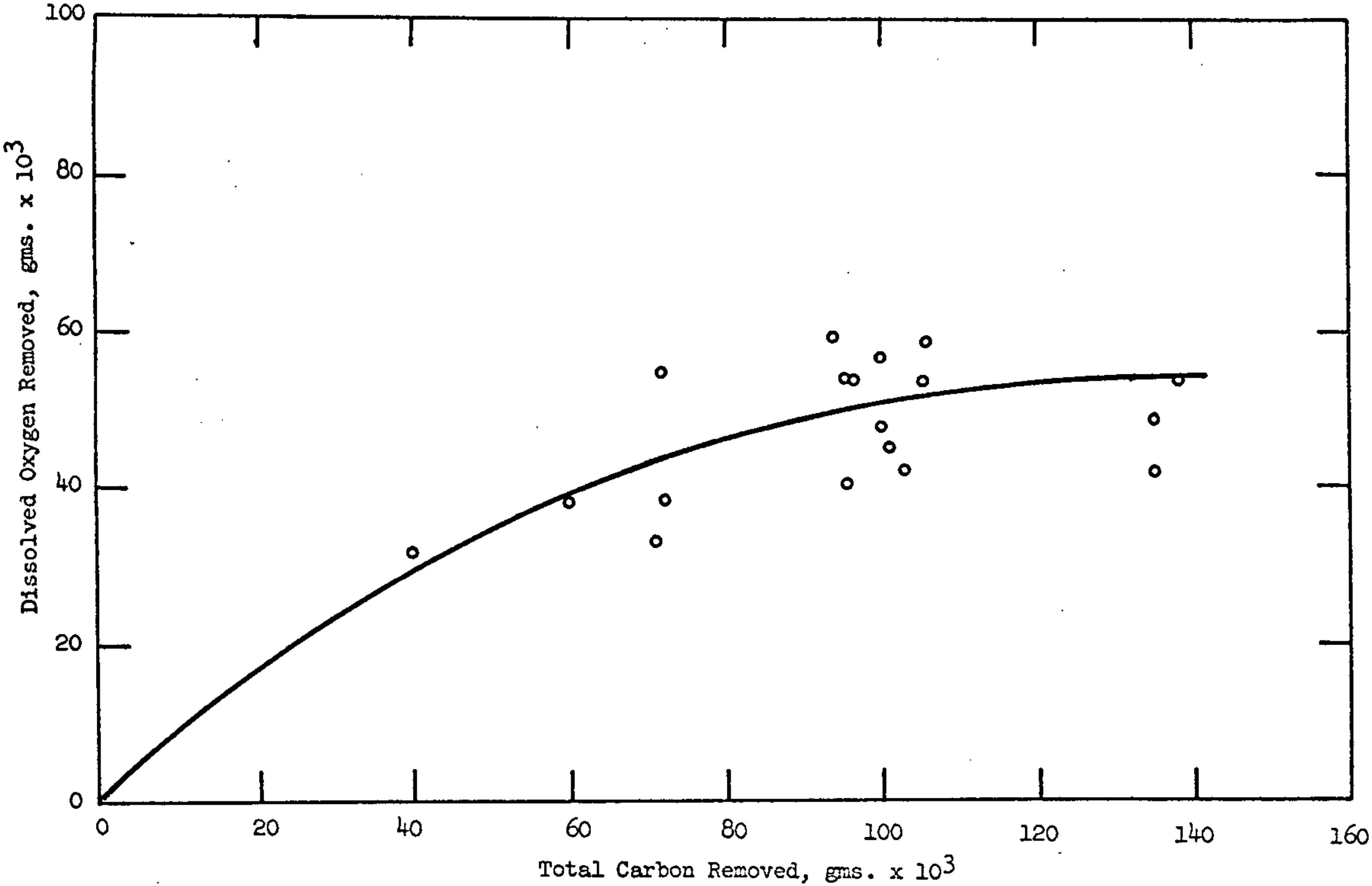


Figure 4-16 Dissolved Oxygen Removal vs. Total Carbon Removed

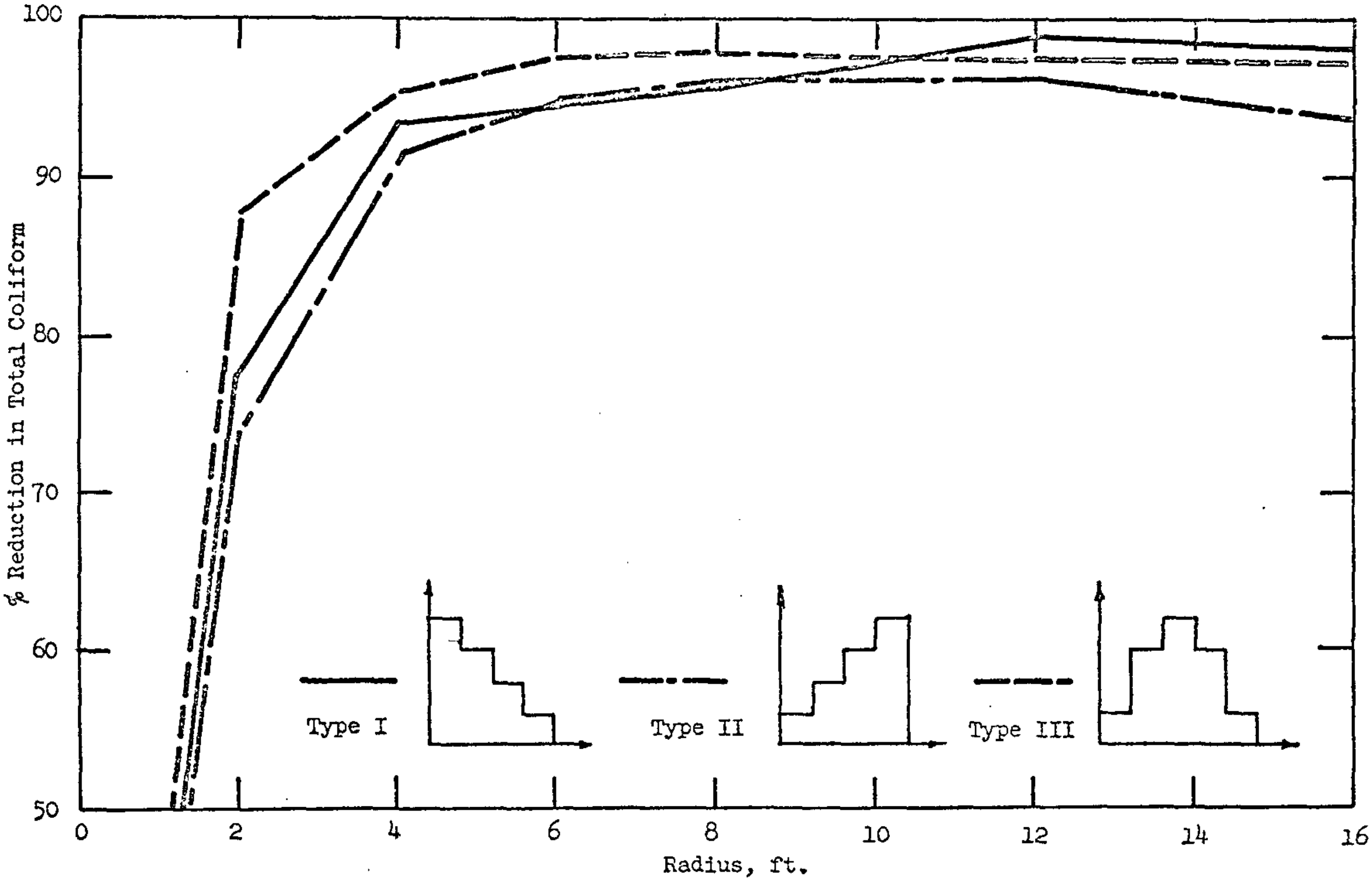


Figure 4-17 Total Coliform Reduction vs. Radius after Six Hours of Injection

The injected waste was visually observed to be quite turbid. However, all of the samples from the simulator were clear, including samples taken at the ports within inches of the point of injection.

Conclusions

The laboratory simulator described in this report provides an economical method for evaluating the preliminary feasibility of injecting combined sewer overflows into underground formations. If the soil medium is selected so as to be representative of the actual aquifer characteristics and carefully placed in the simulator, the flow regime in the aquifer can be duplicated. Results obtained from injecting waste with varying concentrations of pollutants would then determine whether or not further field studies are justifiable. If not, the expense of actually constructing injecting wells in the field may be avoided.

The results from the laboratory simulation study confirms conclusions, as to the advantages of using underground water basins for the reclamation of wastewater, reached by Ludwig (22) as early as 1950 and recently summarized by Coe and Laverty (23). The advantages were cited as: (a) Use of existing underground water storage and transmissive capabilities permits intermittent operation of treatment facilities; (b) The sands and gravel of the aquifer provide an excellent natural treatment medium; and (c) Waste water loses its identity to prospective users.

Based on this study it is concluded that additional investigation is warranted for certain areas of the United States. The most favorable areas would be along the sea coast where salt water aquifers exist. In these locations, the possibility of contaminating fresh water aquifers would be minimal. Other areas such as the southwestern arid regions could also be considered where salinity of the aquifers has increased because of recycling irrigation water.

This study indicates that the concentration of contaminants in the injected waste, such as carbonaceous material and coliform bacteria are rapidly reduced within the first few feet of the injection well. Contaminants did not travel very far from the source of injection in the time injection was underway. It is estimated that within a maximum of 100 feet from the well waste concentration would be practically nonexistent for an intermittent operation such as this in which the flow was reversed during dry periods.

The reduction of carbonaceous material corresponds to findings in other investigations (3, 6, 7, 8, 15). The mechanism resulting in the reduction was reported as being biological decomposition in two of the studies (6,7). When the organic material remained in the aquifer for an extended period of time it was reported that biochemical processes in which dissolved oxygen is depleted may occur (12). Physical and chemical sorption resulted in a reduction of organic chemicals in one of the studies (15).

Based on the results of the simulation study and observed reductions reported by others of actual field test it is concluded that the mechanism for removing carbonaceous material was aerobic biological decomposition. Dissolved oxygen present in the injected waste is used in the process.

The effectiveness of a soil medium in removing bacteria also corresponded with reported results (3, 6, 7, 8). The removal is generally believed to be the result of mechanical straining or sedimentation in fine passages between the sand grains (5). If the waste remains in the aquifer for a long period, as was the case in Israel (12), high coliform concentrations may reappear.

Since the injection waste consisted of settled sewage mixed with tap water, larger non-filterable particles were not present in high contrations. The colloidal and filterable solids which were present, were not reduced and behaved as a conservative constituent. If the injected waste has been previously settled as in this study and investigations such as those in California (7, 8) the removal of solids is not significant.

Clogging of the simulator was not as severe as that reported as occurring at existing injection facilities (4, 5, 6, 12, 13). Results from these facilities indicate that the main cause of clogging was suspended solids. As stated previously these solids were not present in high concentrations in the waste used for the simulator study. Clogging caused by precipitation of metabolic products of autotrophic bacteria or slime growths due to injection of organisms into the aquifer, which were observed at several existing facilities, was not experienced in the laboratory study.

Long term effects of injecting wastewater were not evaluated in this study. However, the intermittent operation of the process during periods of runoff and the withdrawal of injected waste during dry periods should mitigate the occurrence of biological growth and chemical reactions within the aquifer. This would also minimize the possibility of injected waste traveling far distances into the aquifer and developing high subsurface pressures which may result in earthquakes. The length of time continuous injection would be required is a critical factor in the proposed injection process. Determination of the duration of overflows requiring injection, can be accomplished by methods developed in Report #3.

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NOMENCLATURE

Symbol	Definition
A	Flow area of confined aquifer, ft.
a	Piezometric pressure at the well divided by flowrate, ft/gpm
$\hat{a}$	Estimated value of h_w/Q, ft/gpm
b	Height of the confined aquifer, ft.
C_i	Injected concentration of pollutant, mg/l
c_i	Observed concentration of pollutant, mg/l
h	Piezometric pressure, ft.
h_w	Piezometric pressure at the well, ft.
K	Coefficient of permeability, gal/day-ft^2
L'	Lower limit of prediction interval in equation 4-12
n	Number of observations used in equation 4-8
Q_i	Flow from sector i, gpm
r	Radial distance from point of injection, ft.
r_w	Radius of well, ft.
s	Slope of linearized equation 4-6
S_E^2	Mean square of the deviations about the regression equation 4-6
$\hat{s}$	Estimated value of $-1/2\pi bK$
U_i	Unit pollutograph ordinate
U'	Upper limit of prediction interval in equation 4-12
V_r	Radial velocity, ft/day
x_i	Natural log of r_i/r_w
$\bar{x}$	Mean value of x_i

Appendix 4A

GRAPHS SHOWING THE EXTENT OF CLOGGING FOR DIFFERENT TYPES OF INJECTION POLLUTOGRAPHS

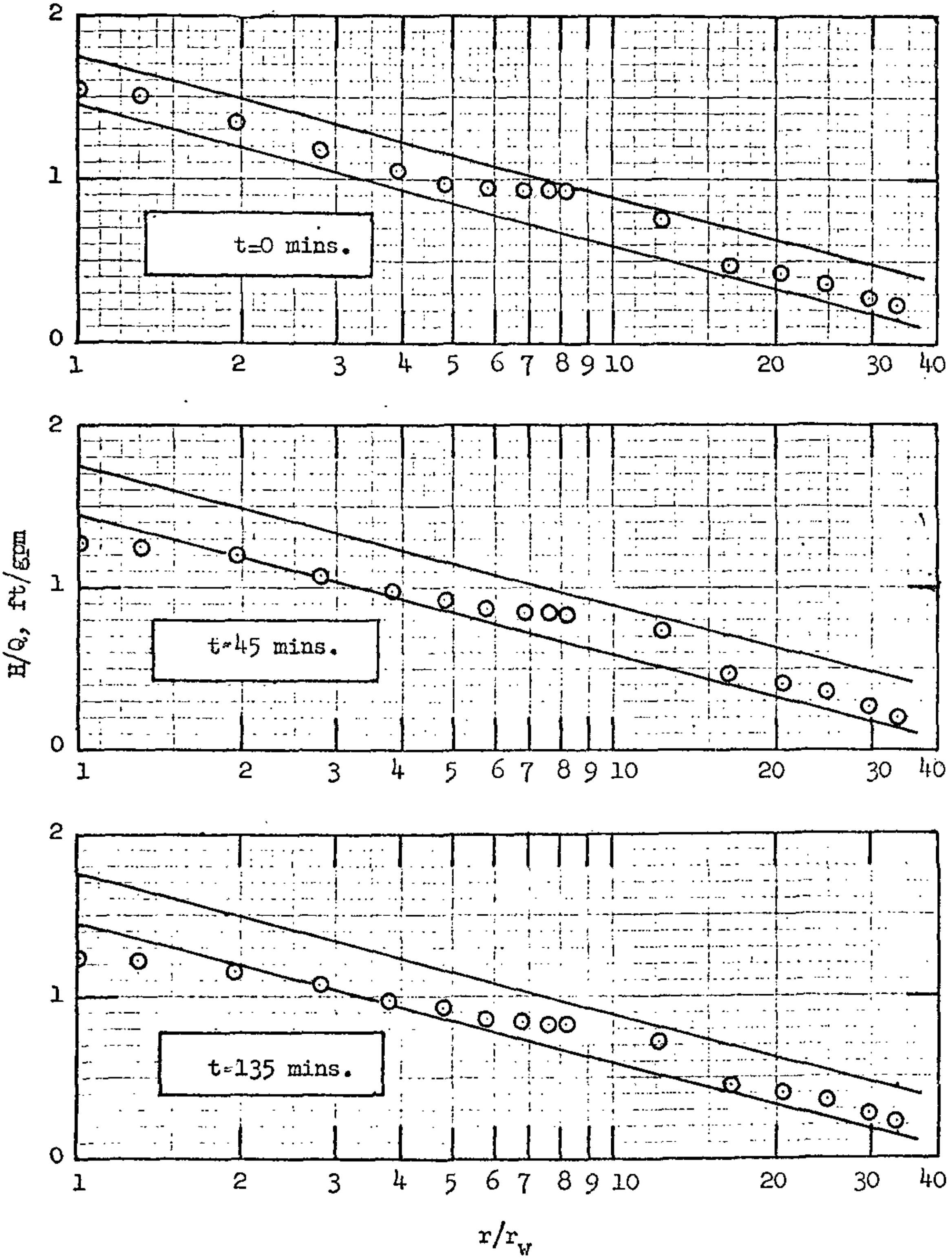


Figure 4A-1 Normalized Piezometric Surface Curve Showing the Extent of Clogging (Run #1 - Type II Injection Pollutograph)

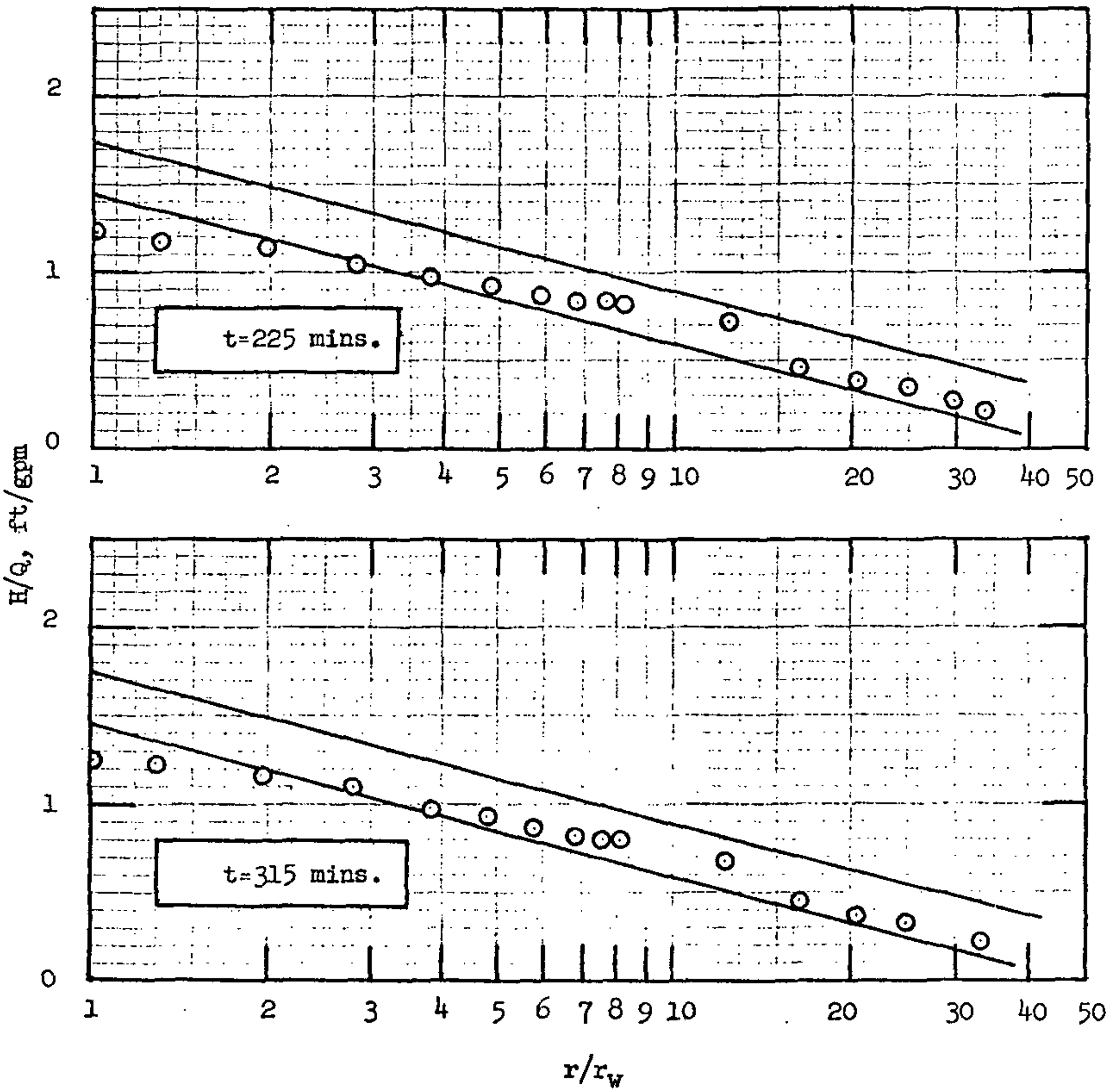


Figure 4A-1 (cont.)

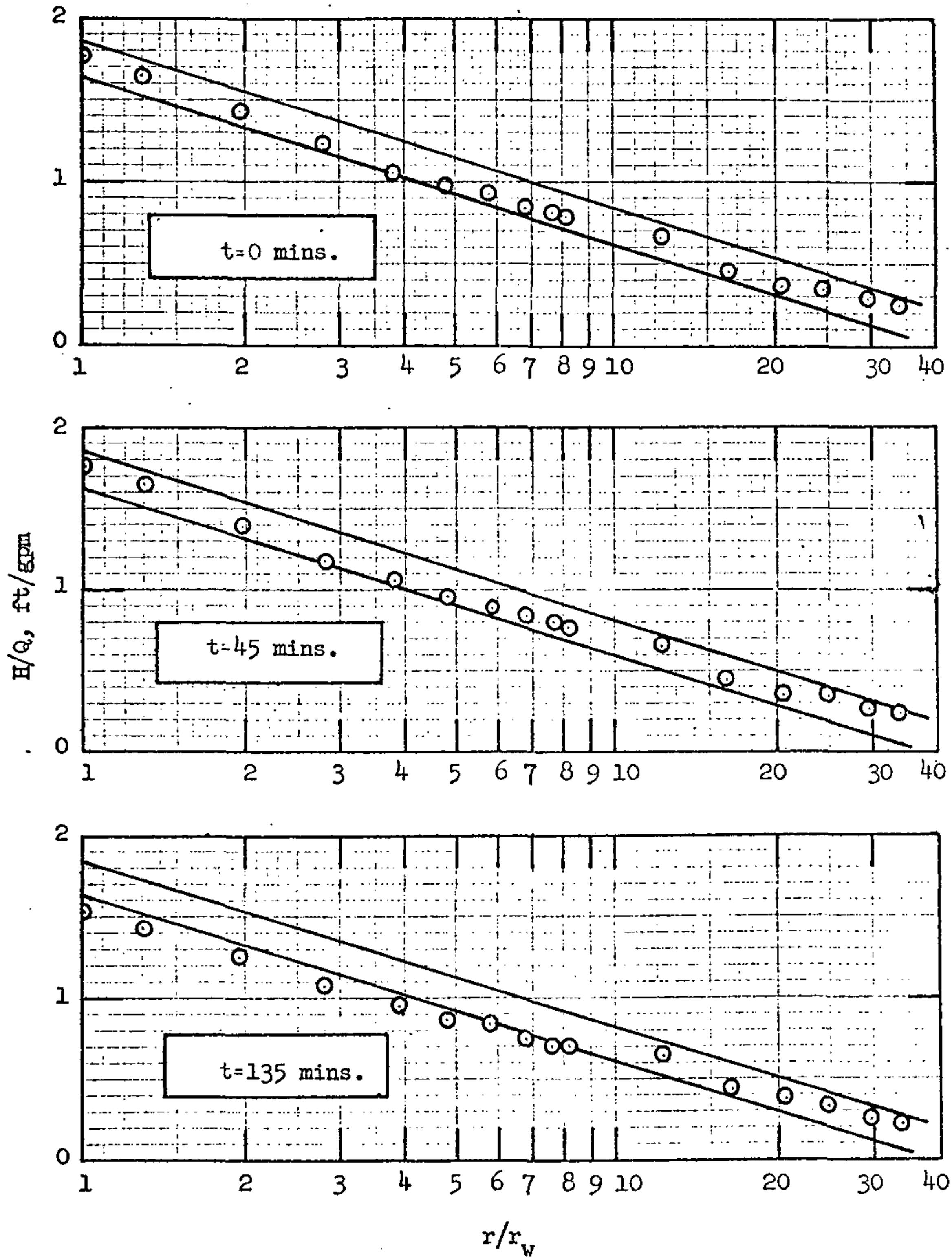


Figure 4A-2 Normalized Piezometric Surface Curve Showing the Extent of Clogging (Run #2 - Type II Injection Pollutograph)

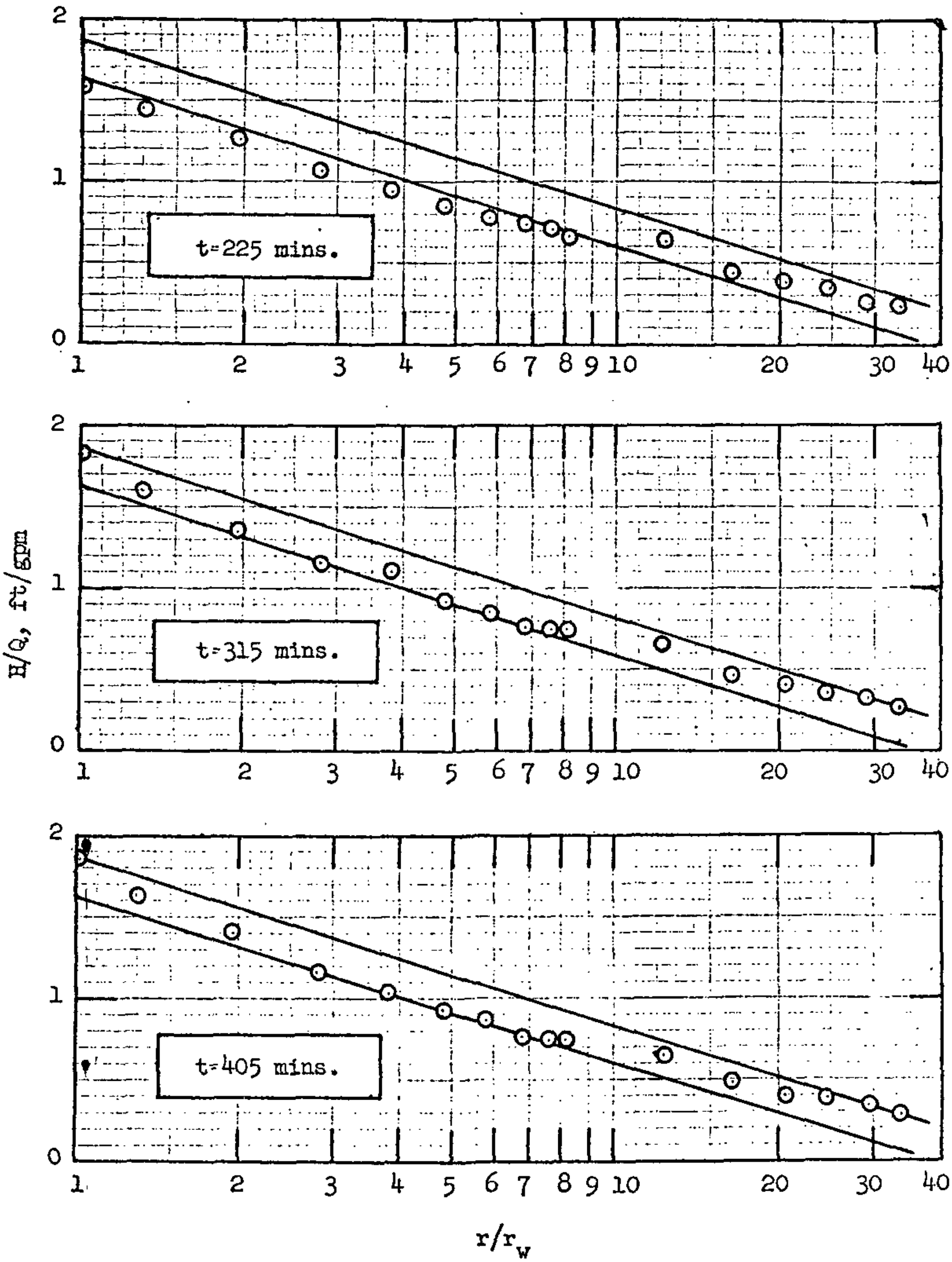


Figure 4A-2 (cont.)

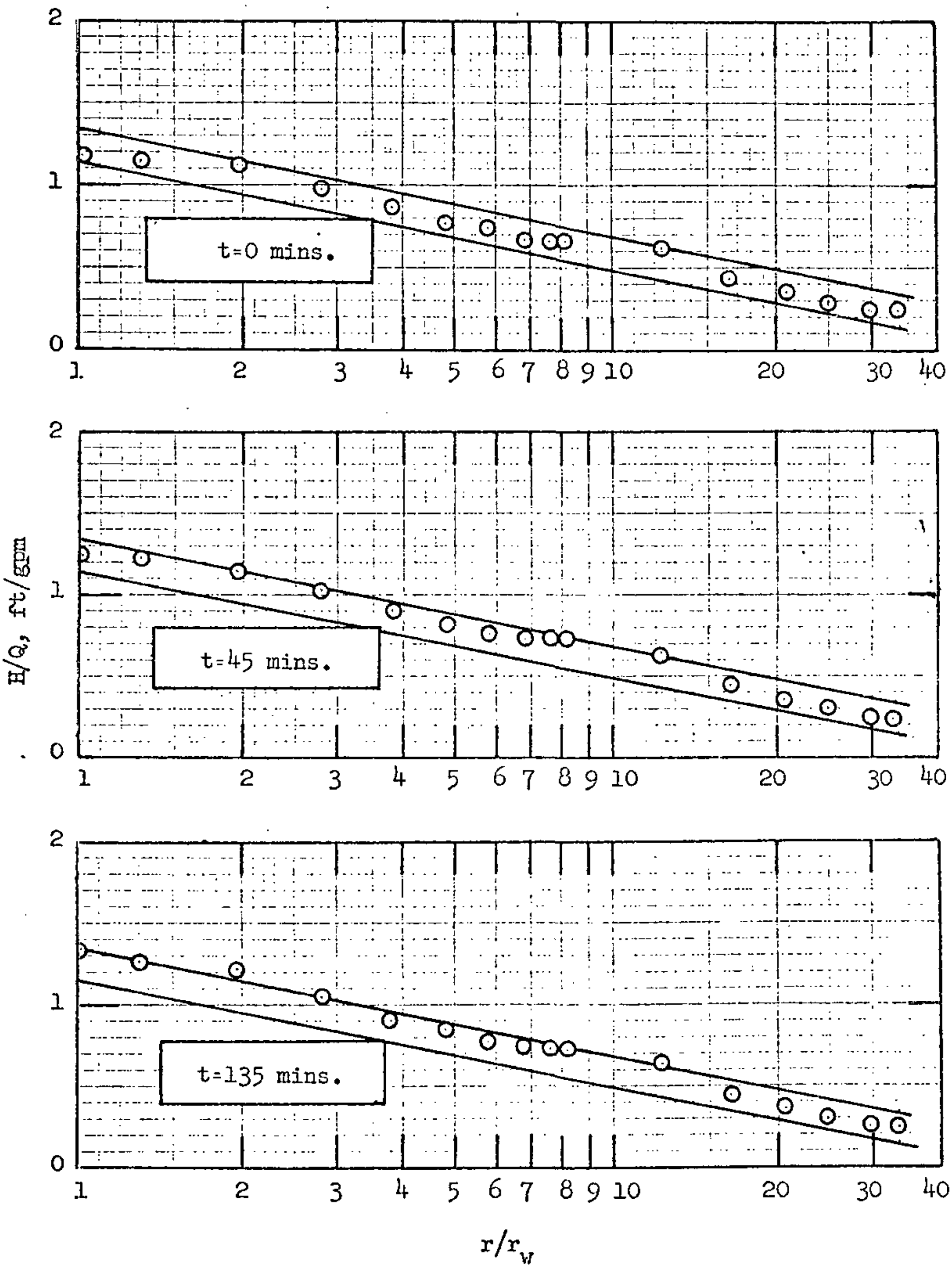


Figure 4A-3 Normalized Piezometric Surface Curve Showing the Extent of Clogging (Run #3 - Type I Injection Pollutograph)

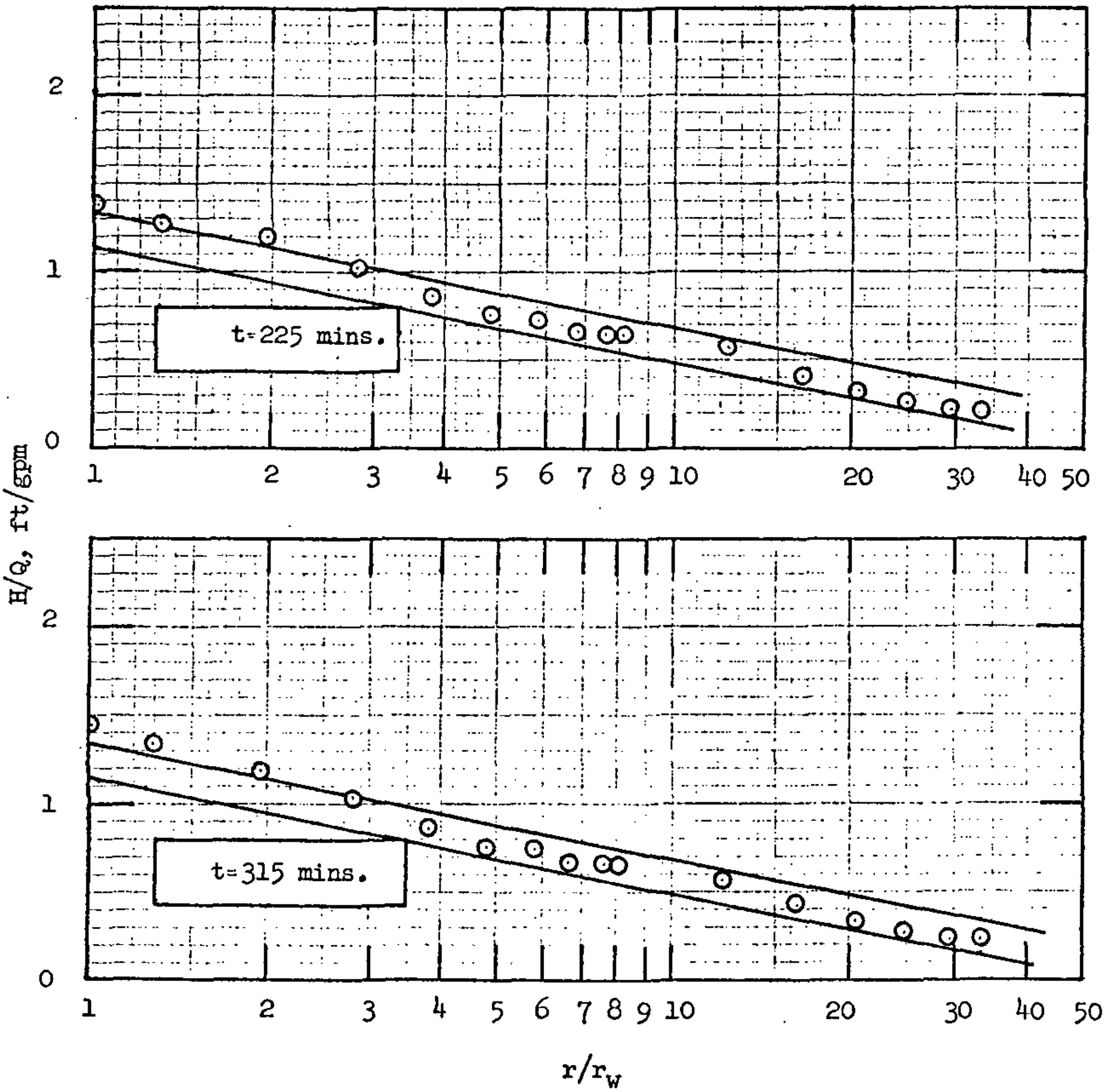


Figure 4A-3 (cont.)

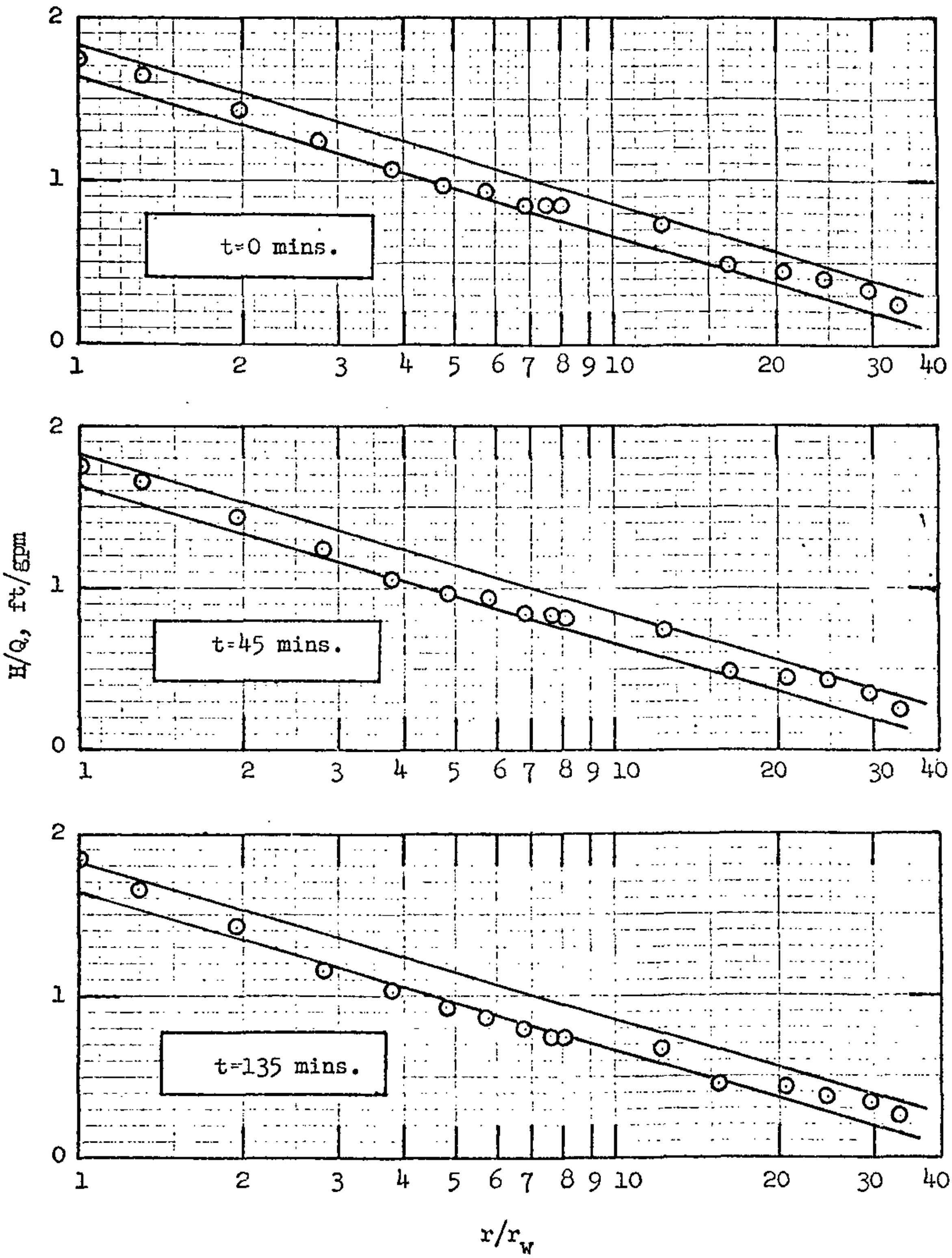


Figure 4A-4 Normalized Piezometric Surface Curve Showing the Extent of Clogging (Run #4 - Type I Injection Pollutograph)

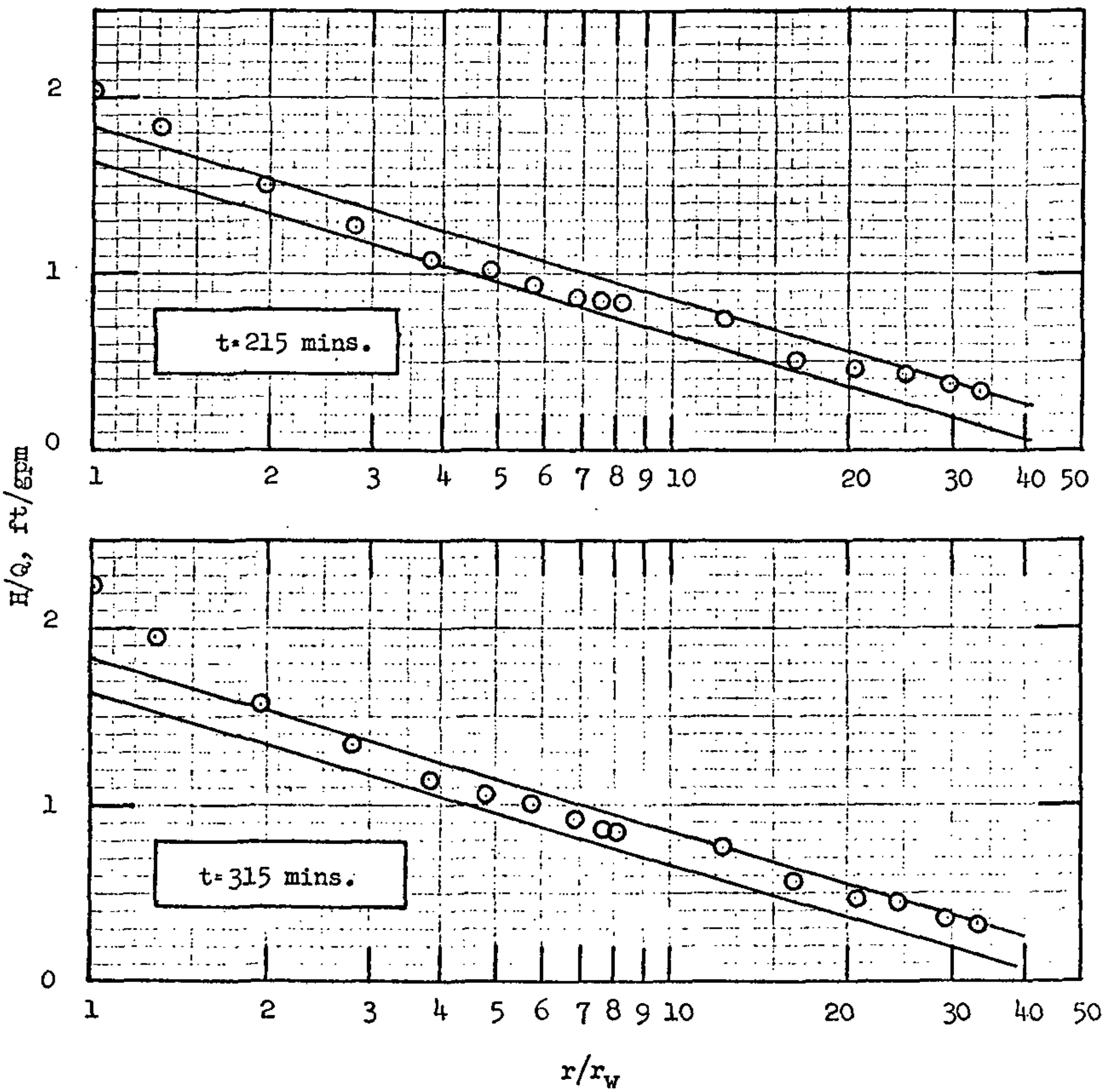


Figure 4A-4 (cont)

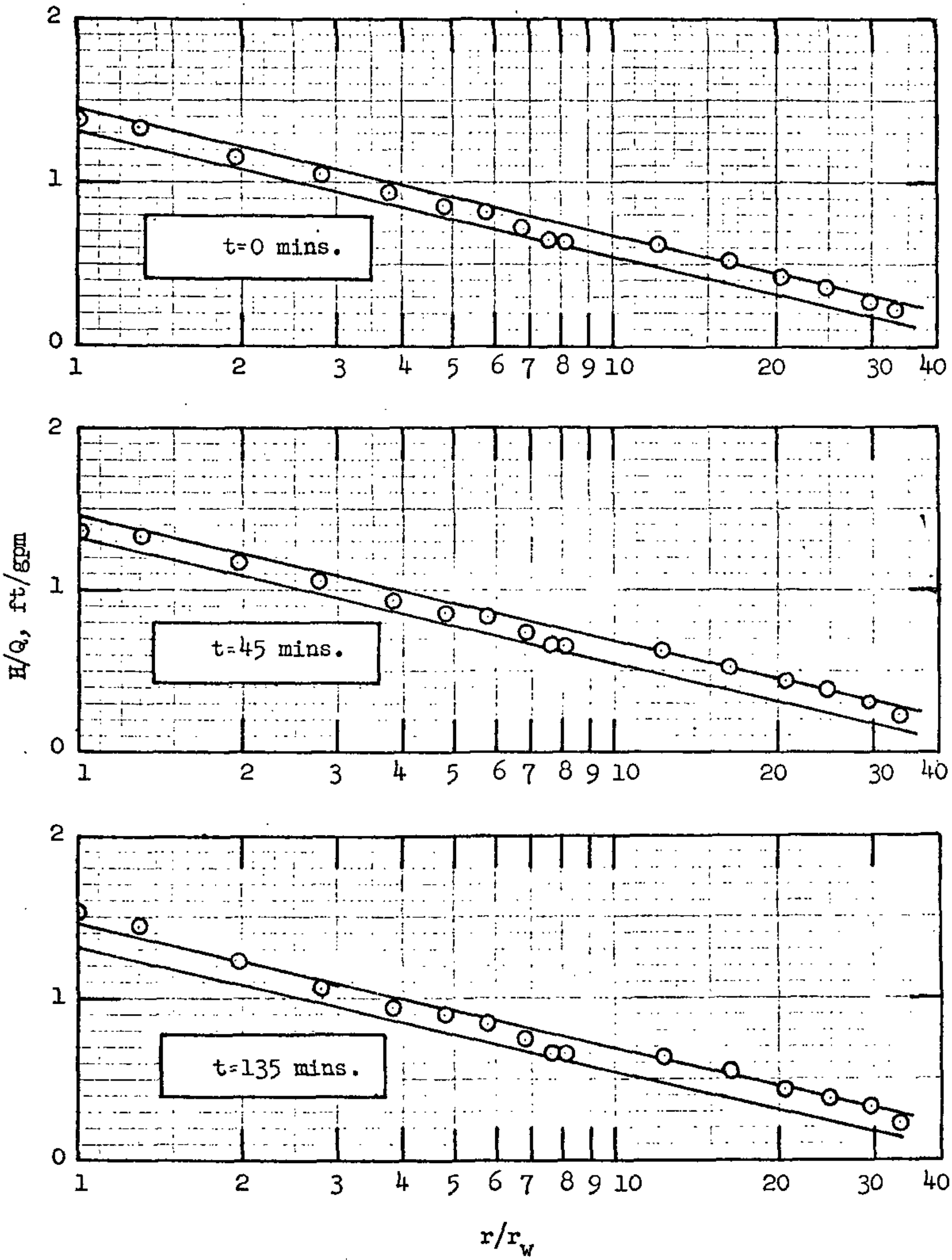


Figure 4A-5 Normalized Piezometric Surface Curve Showing the Extent of Clogging (Run #5 - Type III Injection Pollutograph)

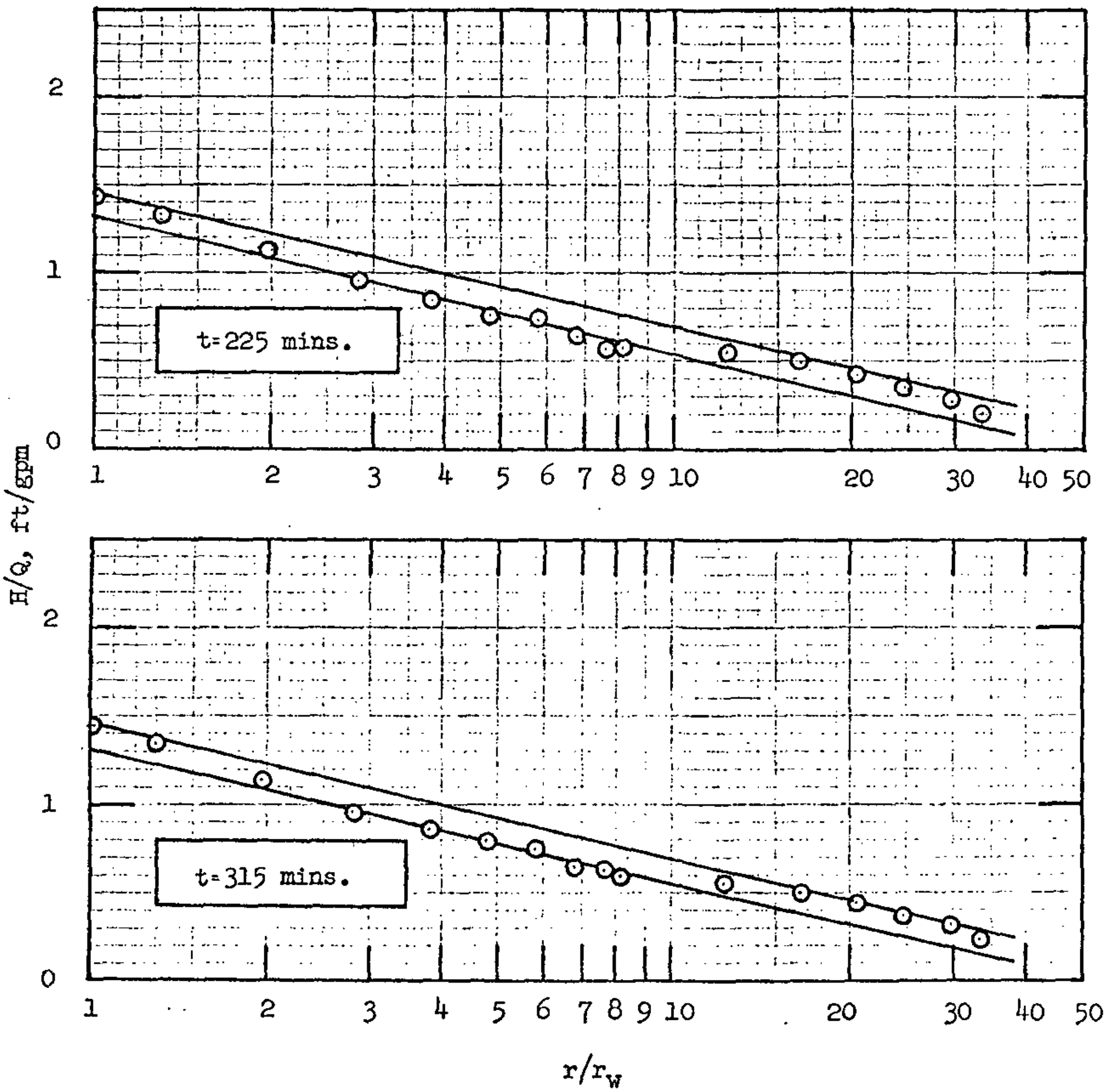


Figure 4A-5 (cont.)

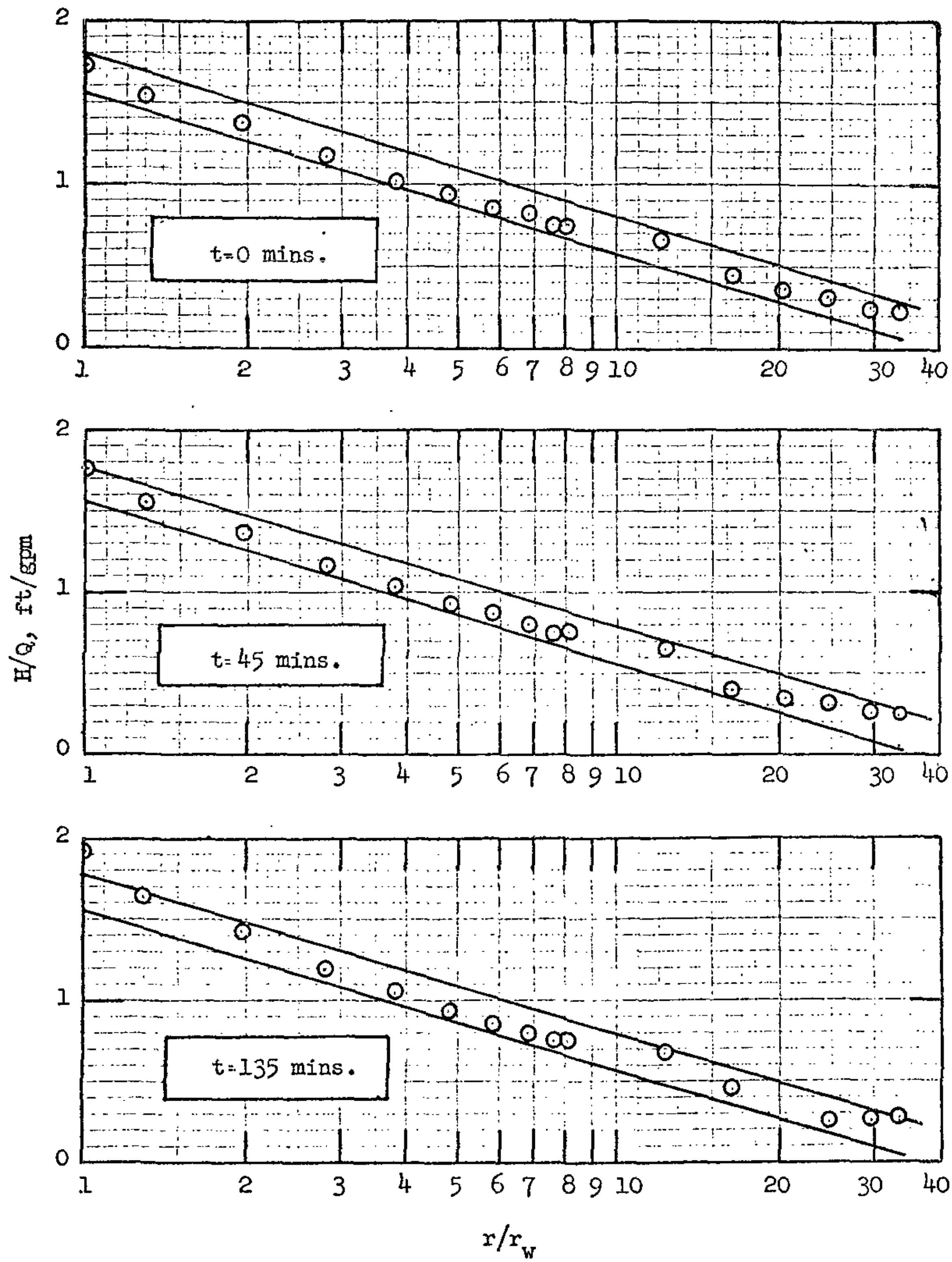


Figure 4A-6 Normalized Piezometric Surface Curve Showing the Extent of Clogging (Run #6 - Type III Injection Pollutograph)

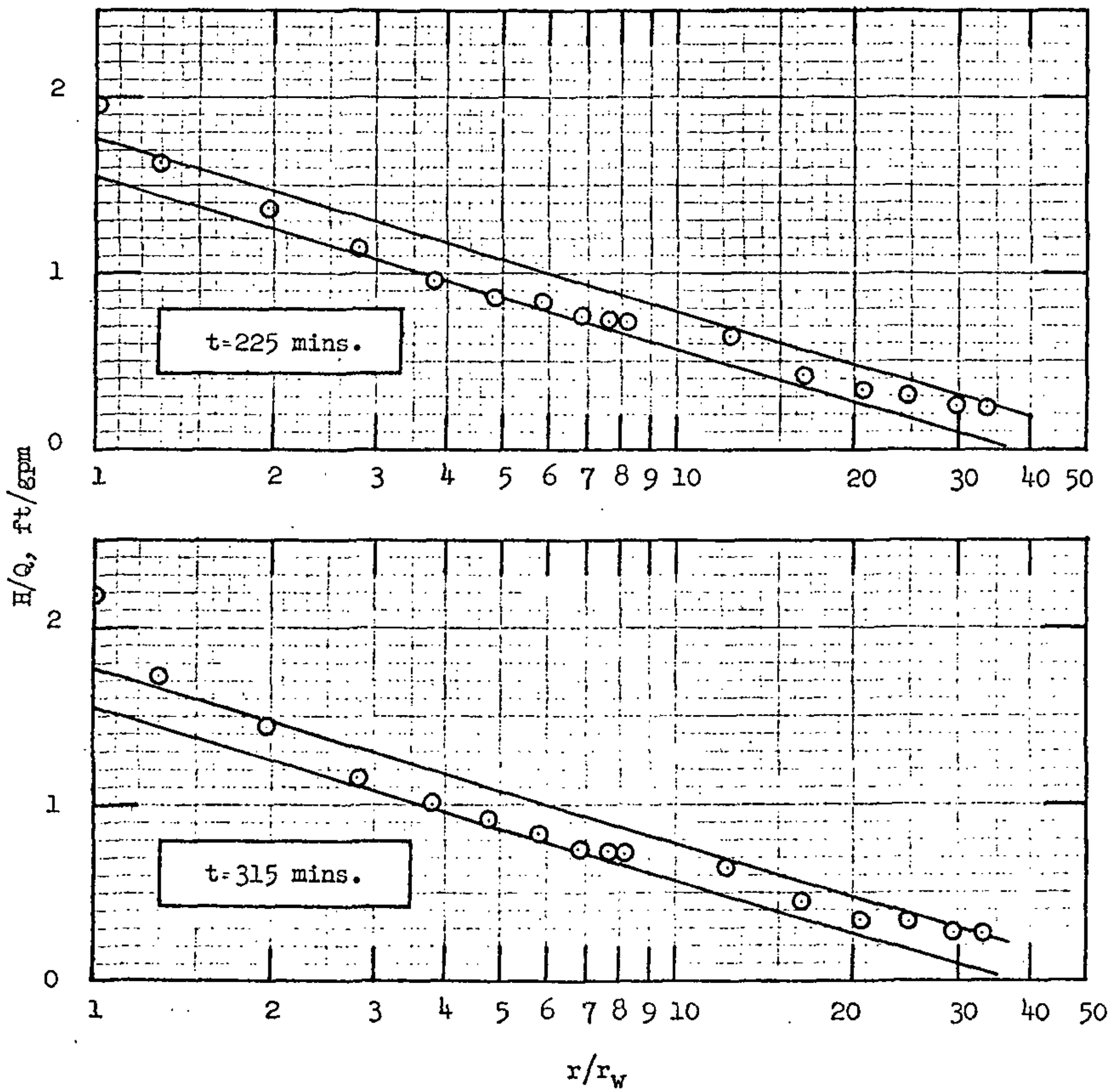


Figure 4A-6 (cont.)

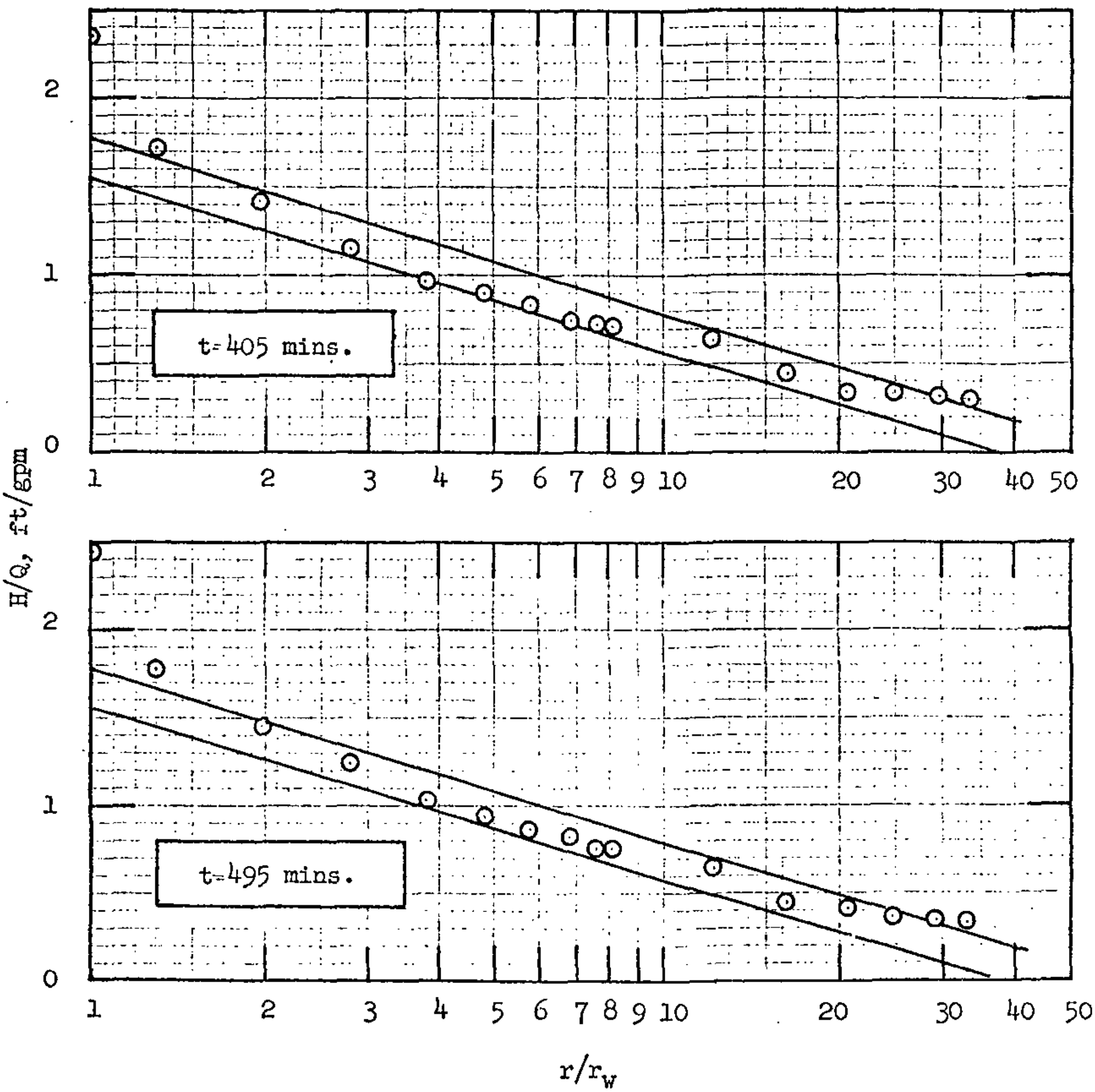


Figure 4A-6 (cont.)

APPENDIX 4B

GRAPHS SHOWING CHANGE IN CHEMICAL AND BIOLOGICAL CONSTITUENTS FOR DIFFERENT TYPES OF INJECTION POLLUTOGRAPHS

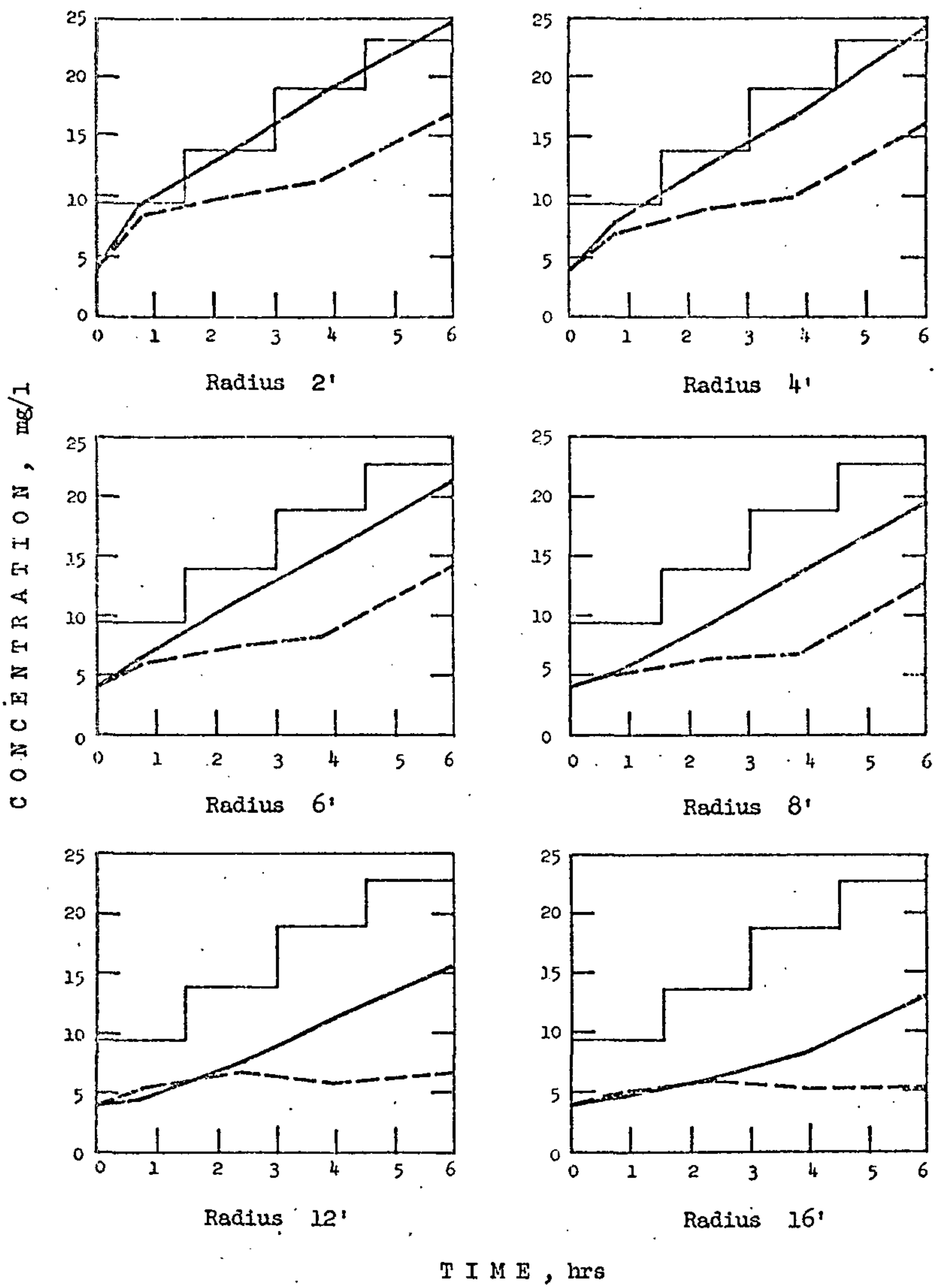


Figure 4B-1 Total Carbon Concentration vs. Time.
(Run #1 - Type II Injection Pollutograph)

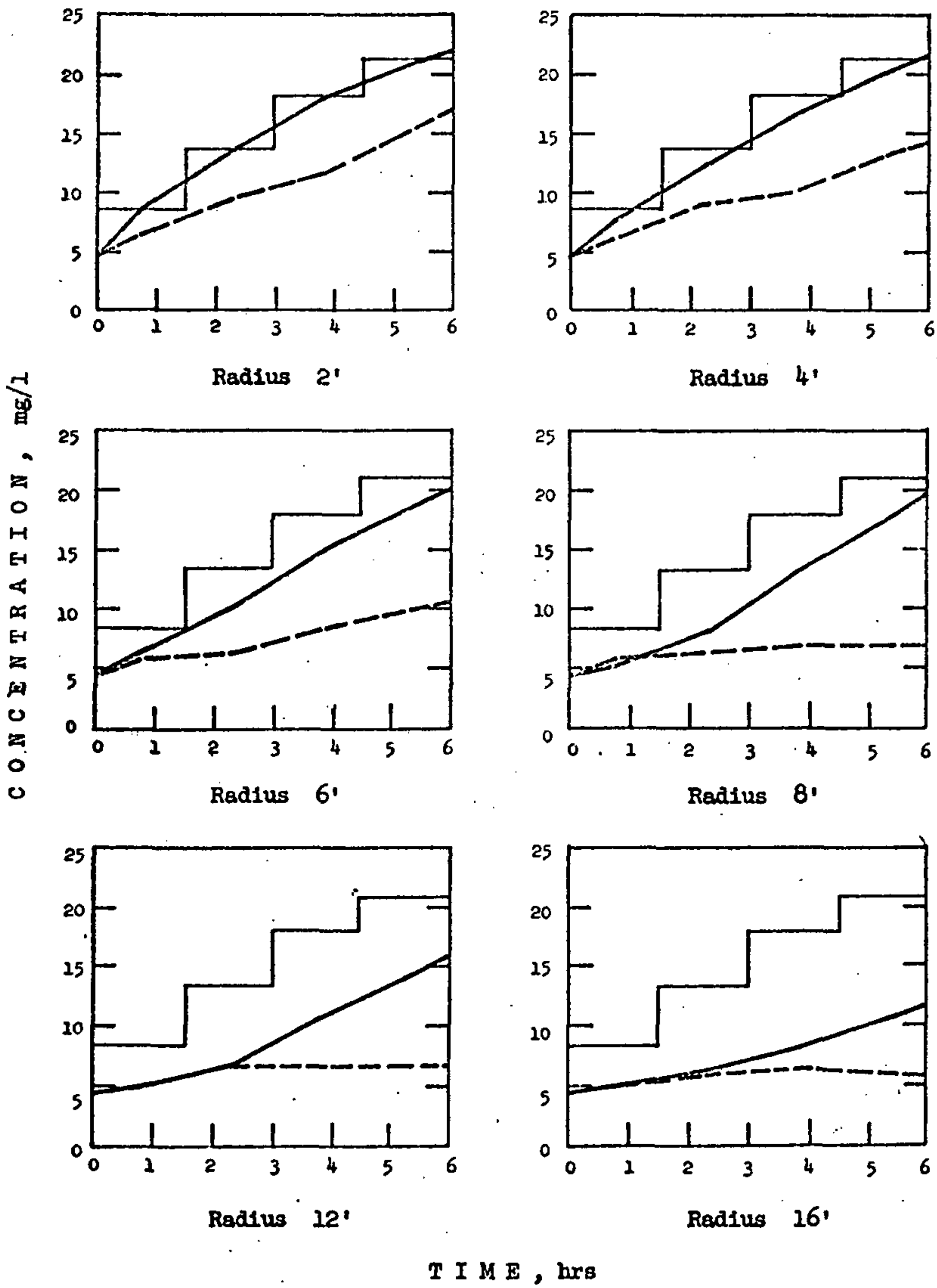


Figure 4B-2 Total Carbon Concentration vs. Time
(Run #2 - Type II Injection Pollutograph)

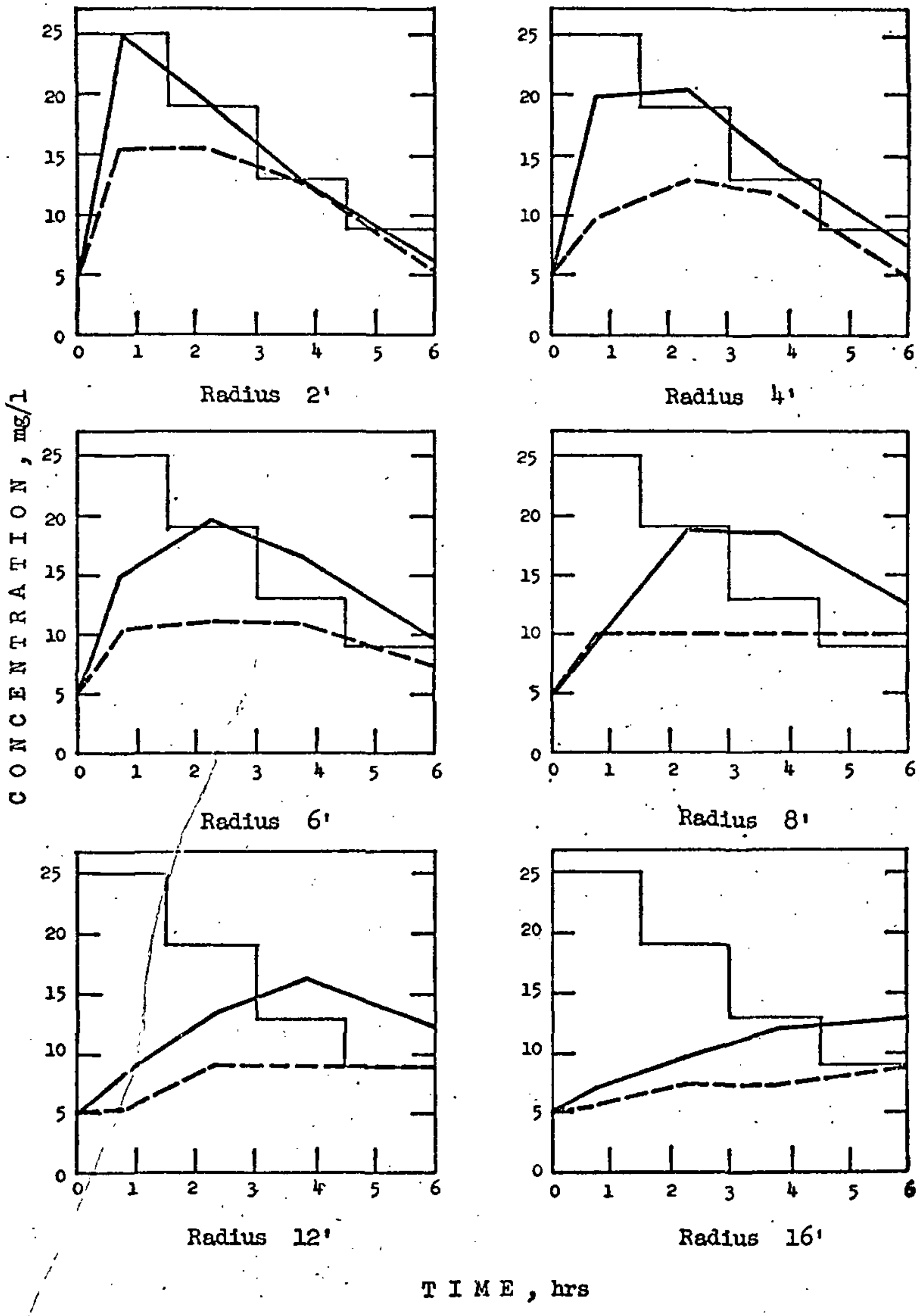


Figure 4B-3 Total Carbon Concentration vs. Time
(Run #3 - Type I Injection Pollutograph)

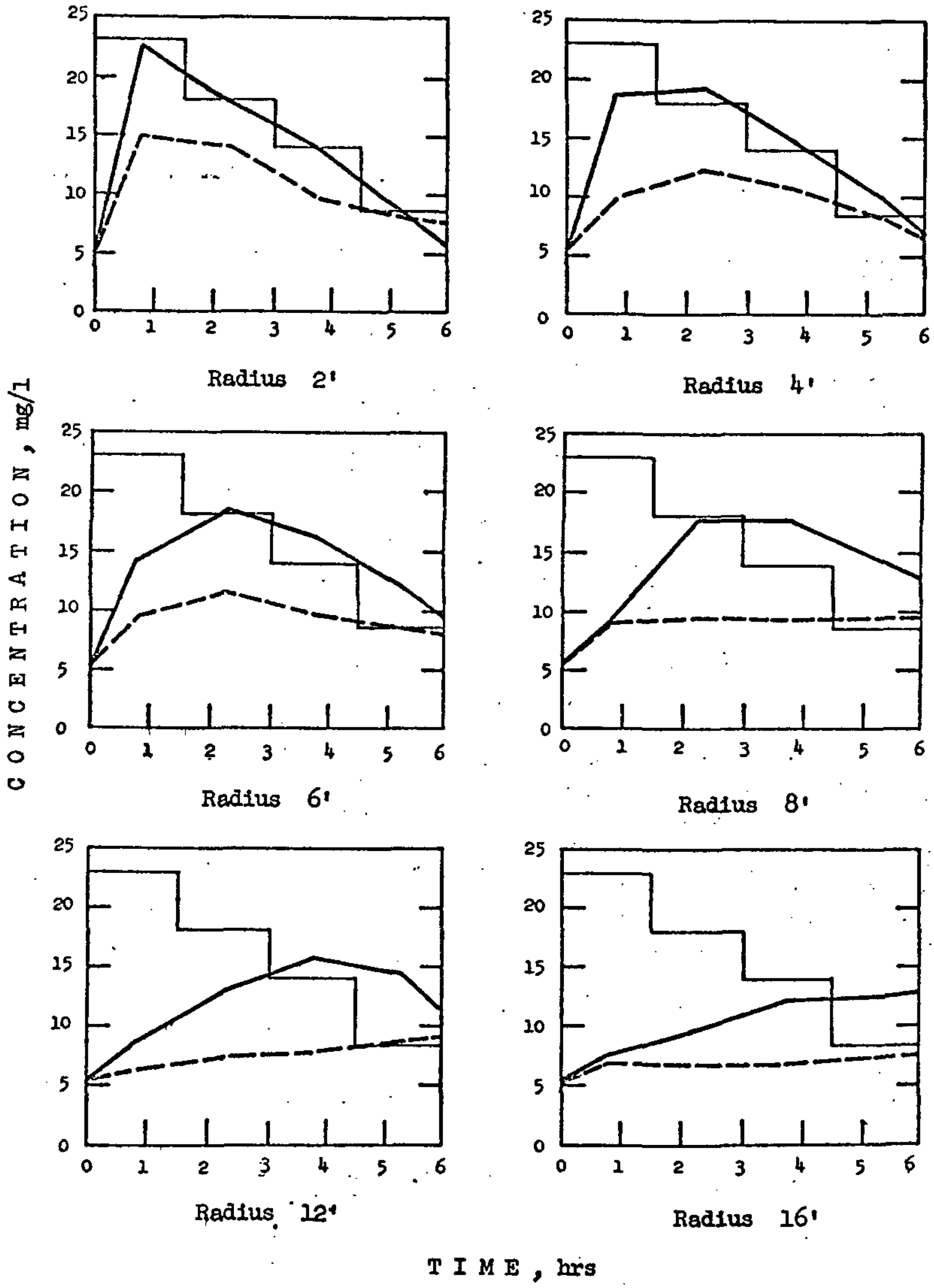


Figure 4B-4 Total Carbon Concentration vs. Time
(Run #4 - Type I Injection Pollutograph)

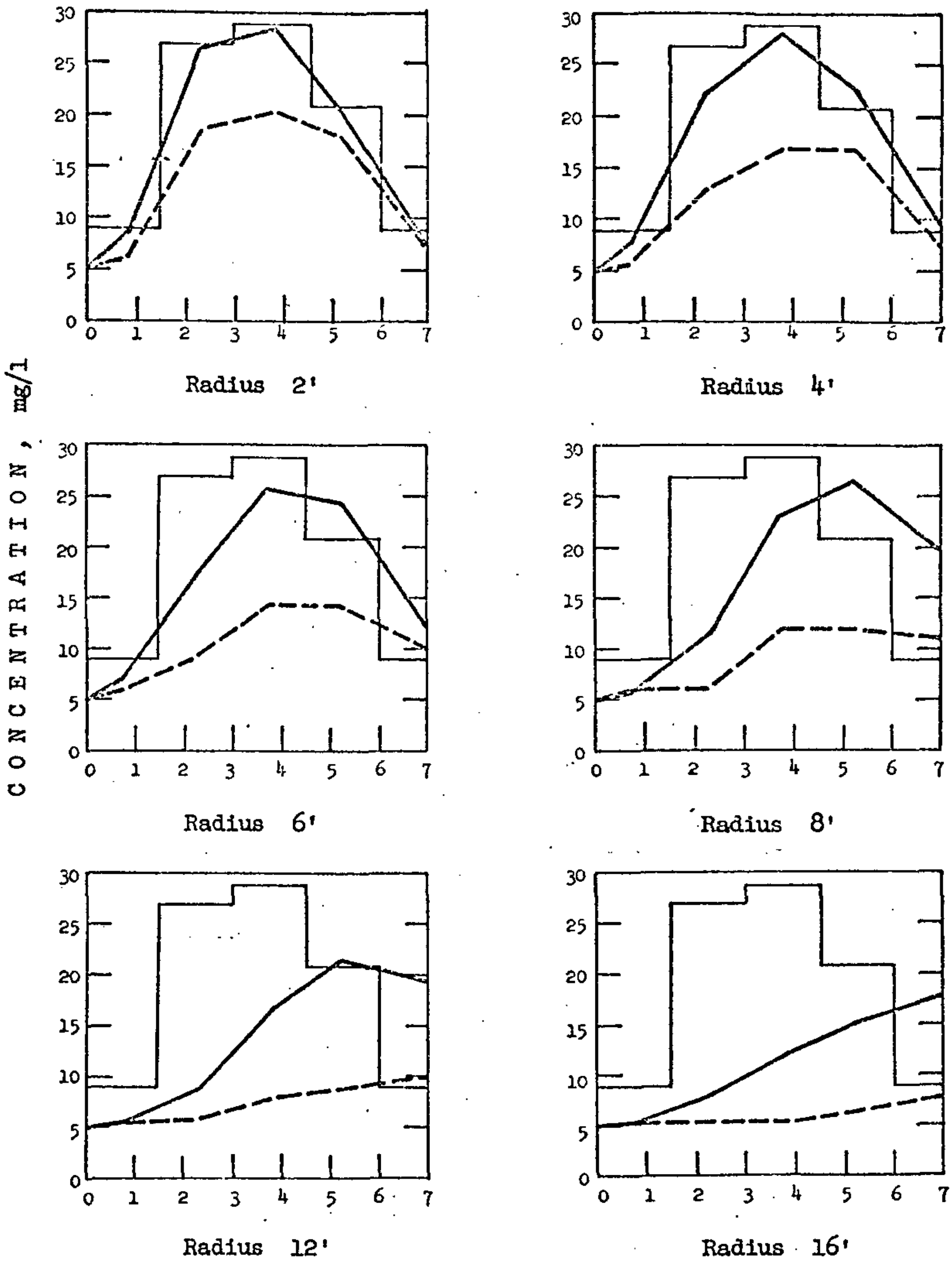


Figure 4B-5 Total Carbon Concentration vs. Time
(Run #5 - Type III Injection Pollutograph)

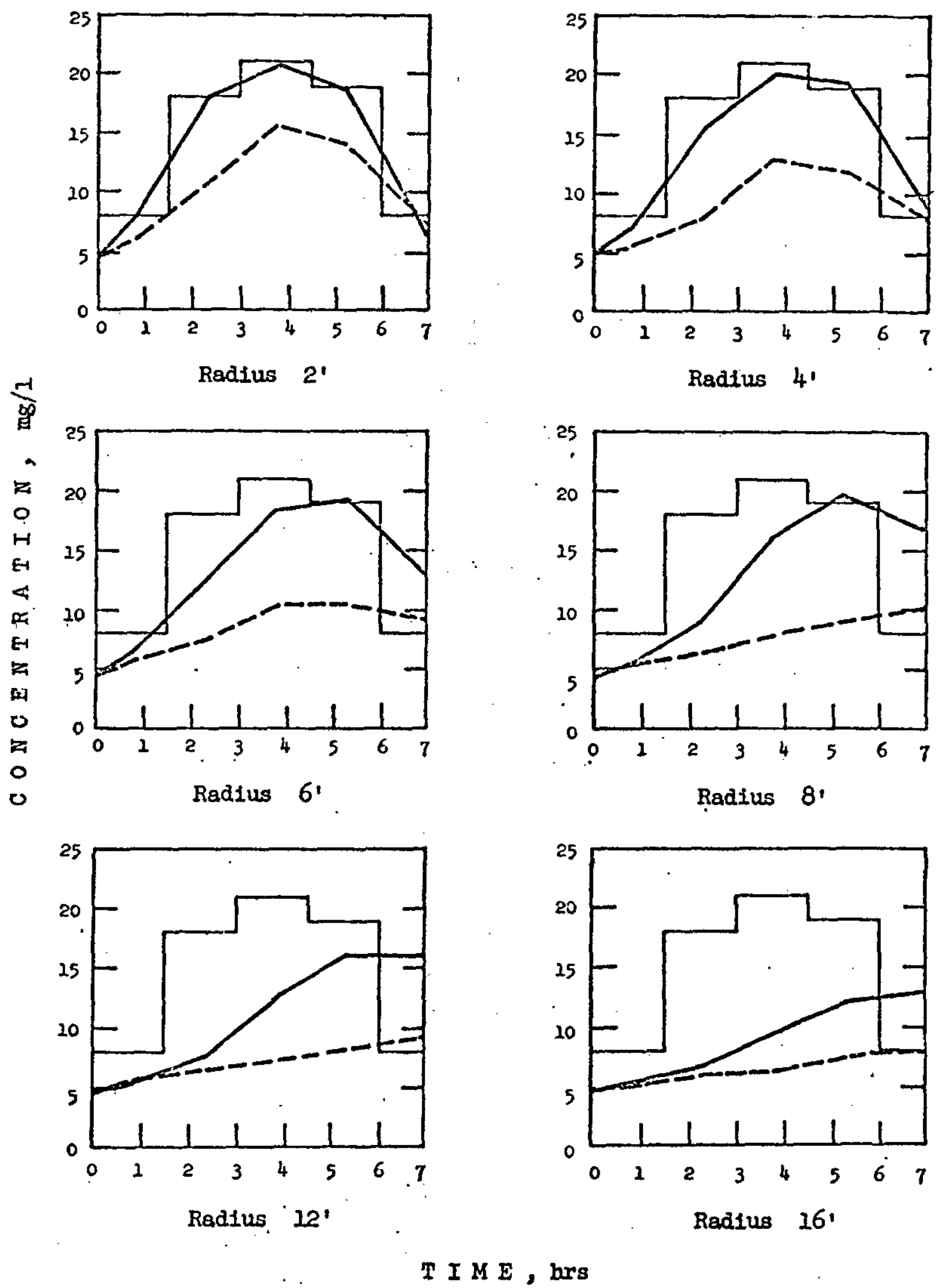


Figure 4B-6 Total Carbon Concentration vs. Time
(Run #6 - Type III Injection Pollutograph)

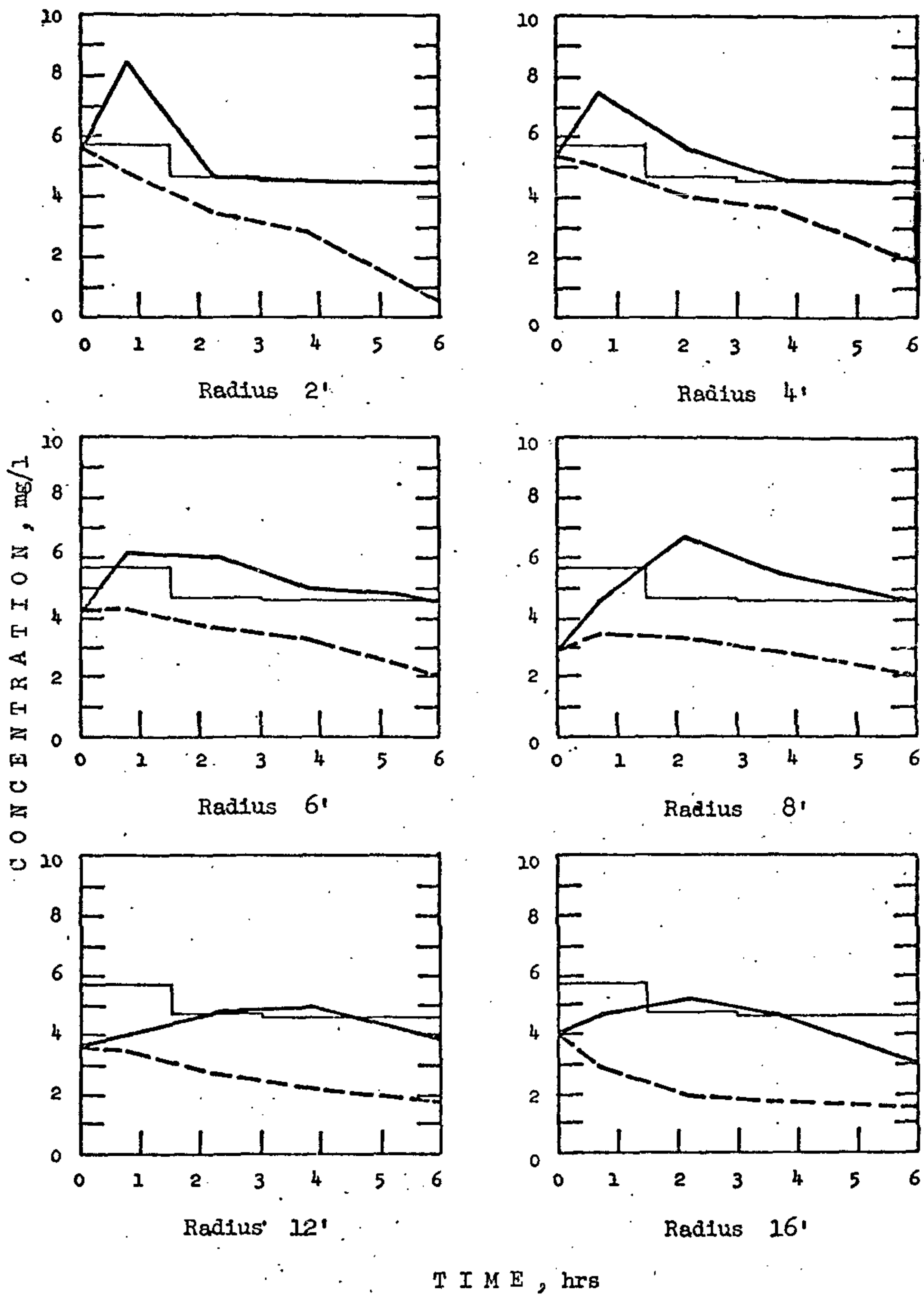


Figure 4B-7 Dissolved Oxygen Concentration vs. Time
(Run #1 - Type II Injection Pollutograph)

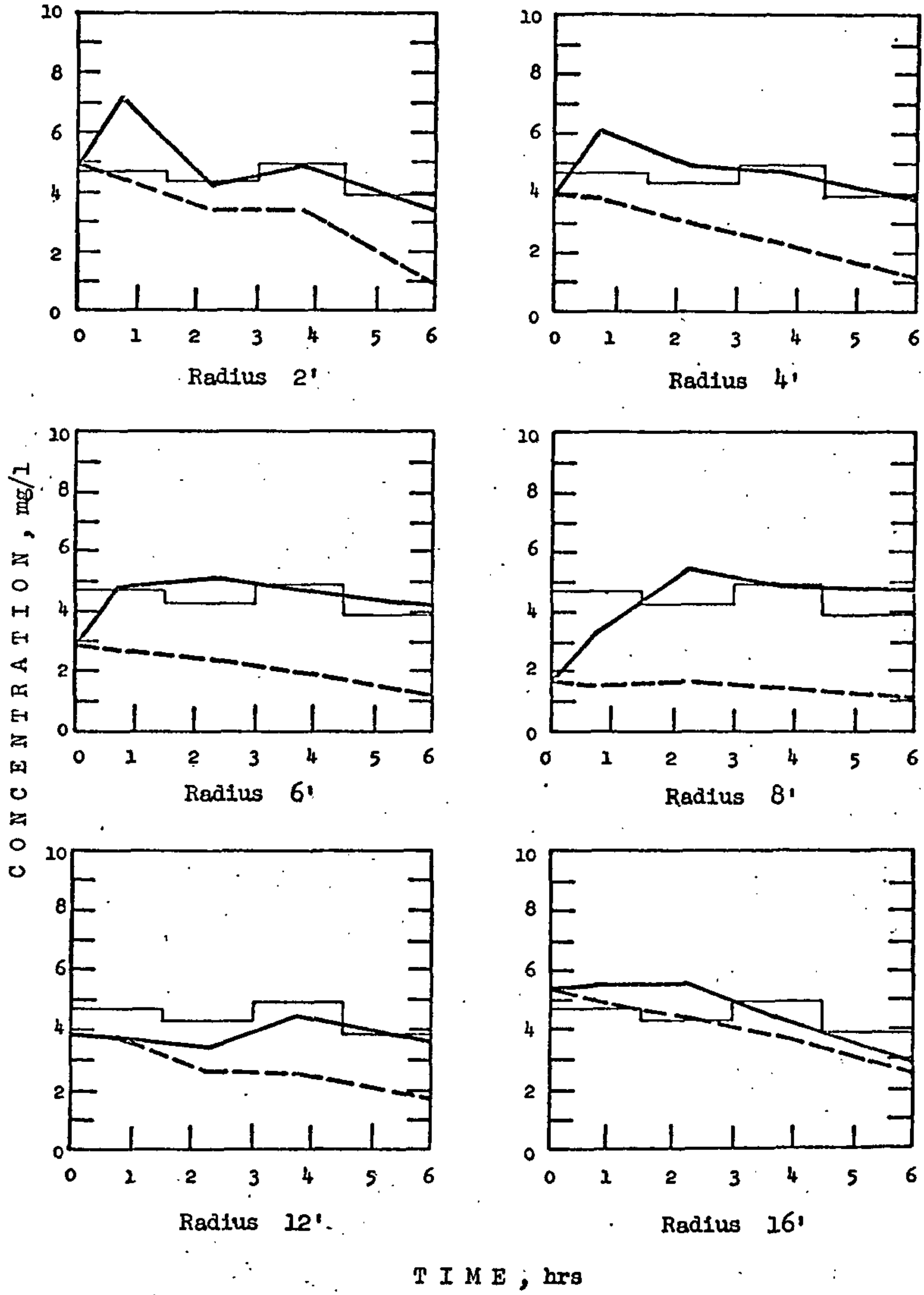


Figure 4B-8 Dissolved Oxygen Concentration vs. Time
(Run #2 - Type II Injection Pollutograph)

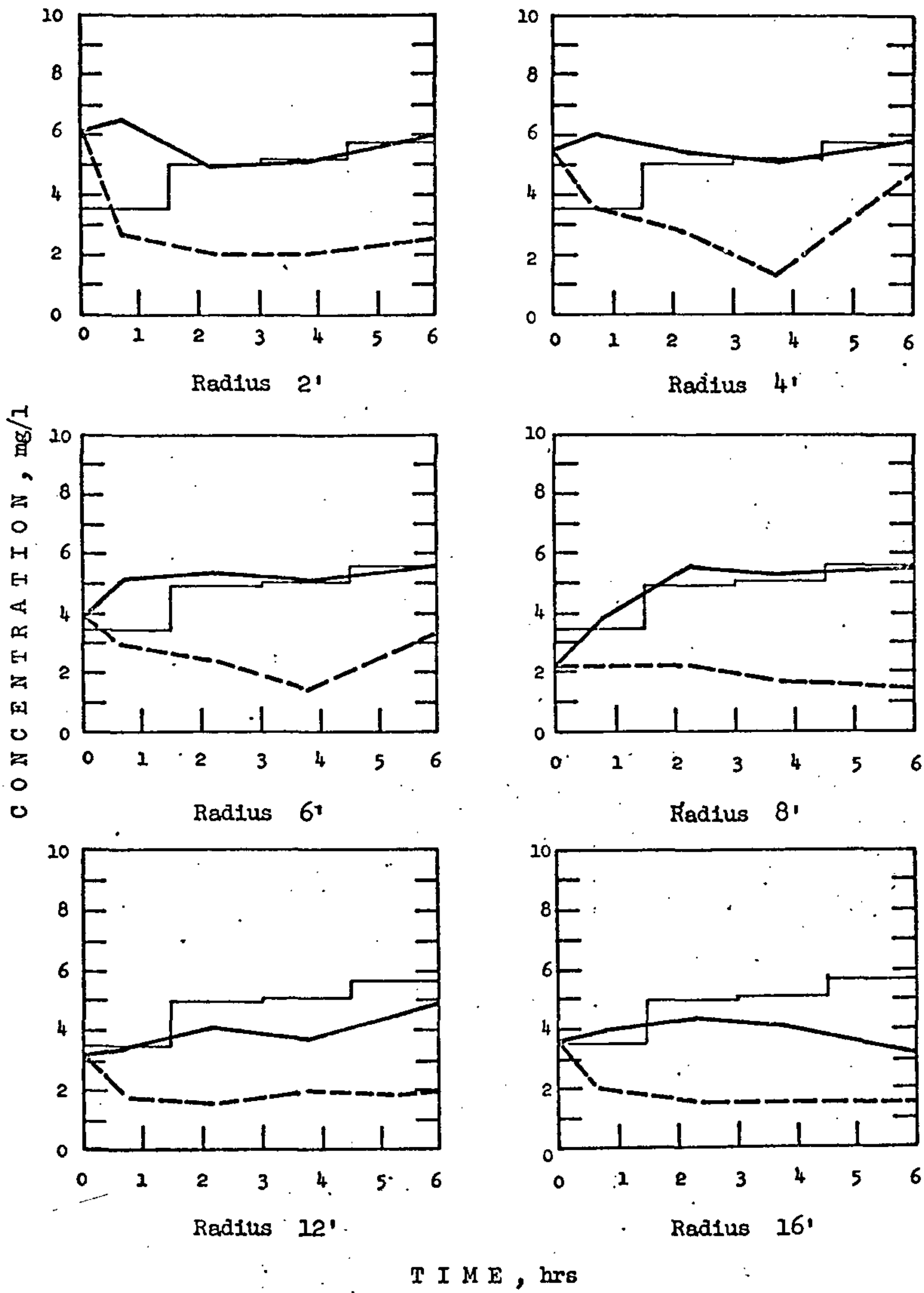


Figure 4B-9 Dissolved Oxygen Concentration vs. Time
(Run #3 - Type I Injection Pollutograph)

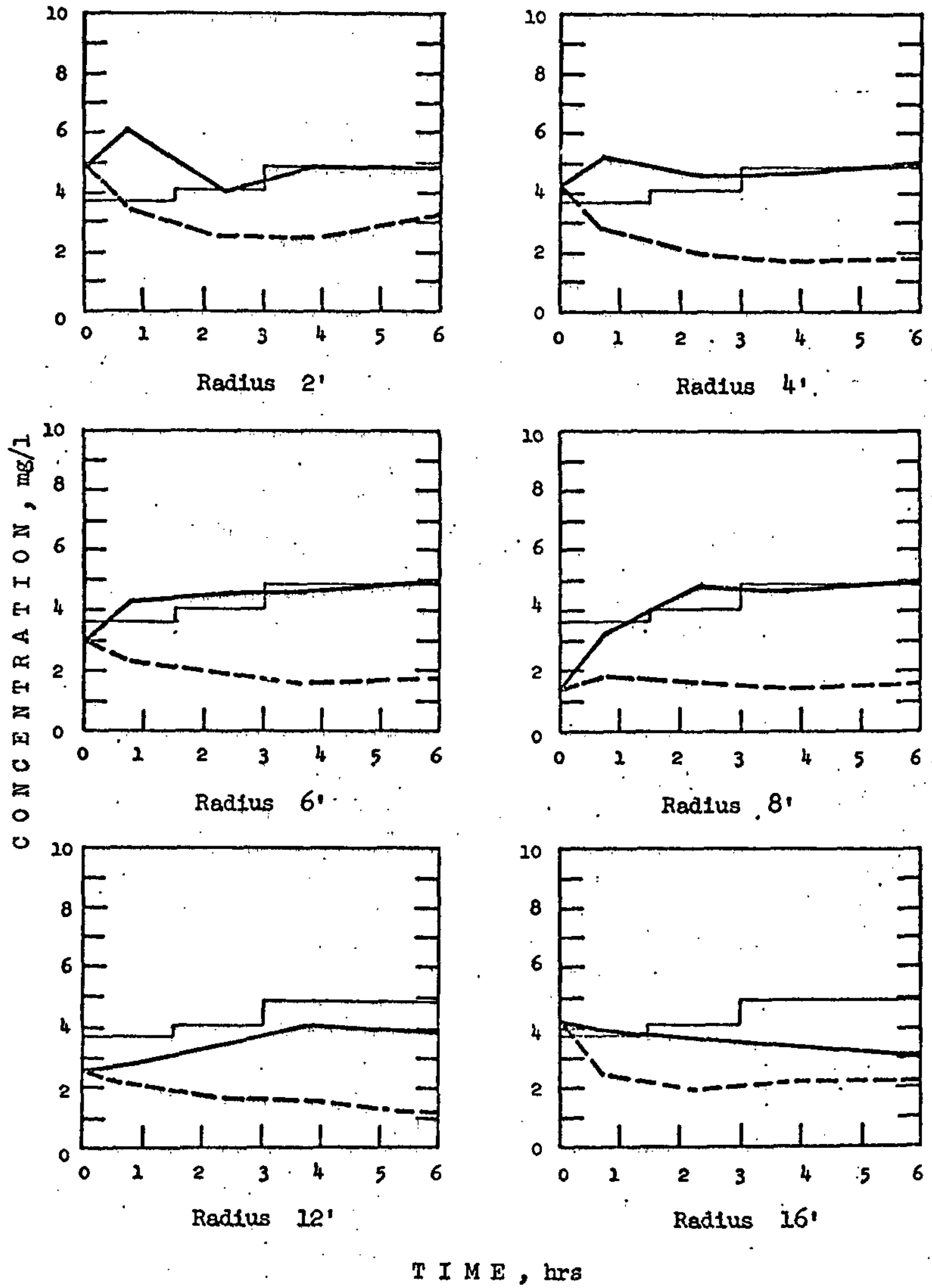


Figure 4B-10 Dissolved Oxygen Concentration vs. Time

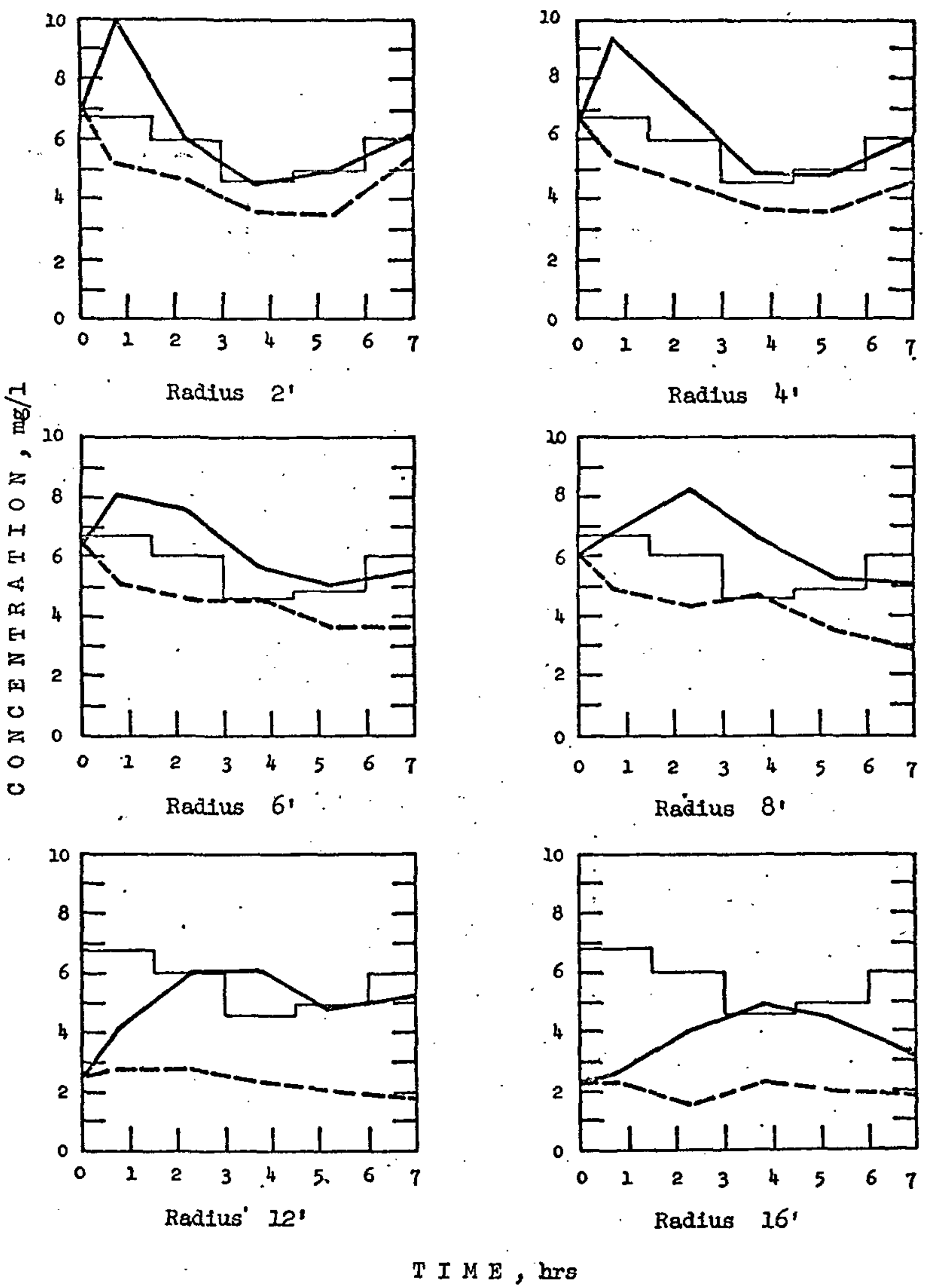


Figure 4B-11 Dissolved Oxygen Concentration vs. Time
(Run #5 - Type III Injection Pollutograph)

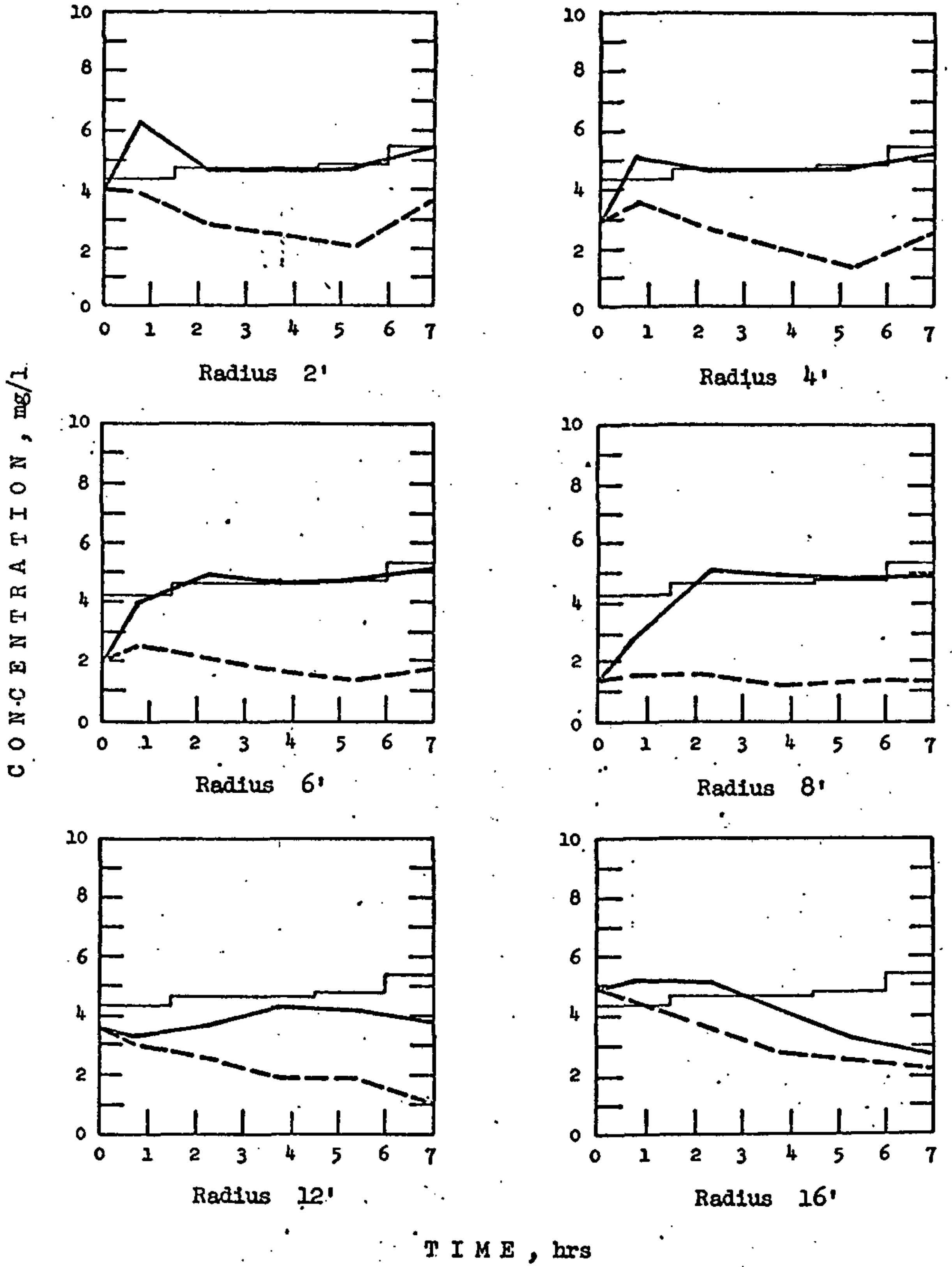


Figure 4B-12 Dissolved Oxygen Concentration vs. Time
(Run #6 - Type III Injection Pollutograph)

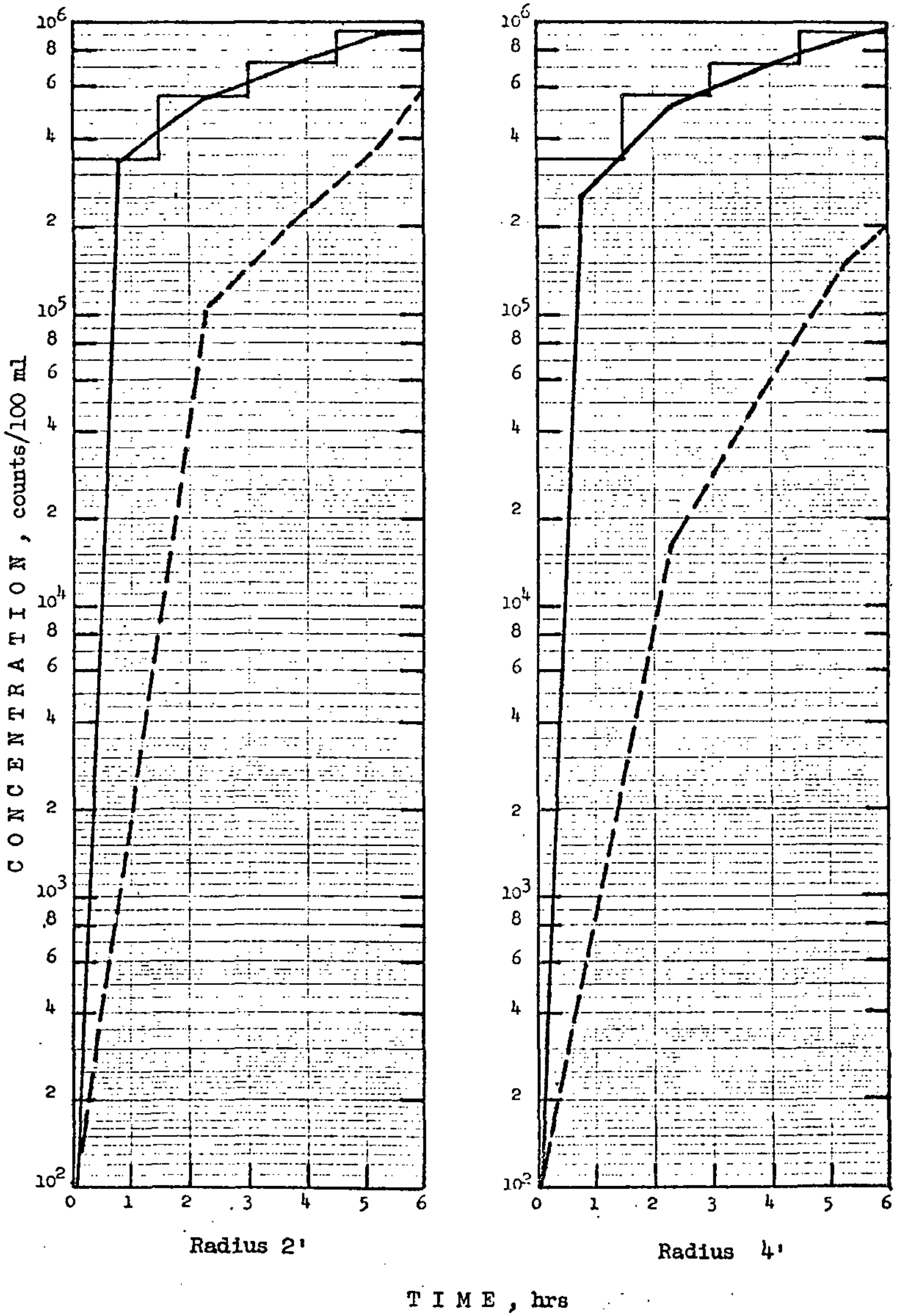


Figure 4B-13 Total Coliform Concentration vs. Time
(Run #1 - Type II Injection Pollutograph)

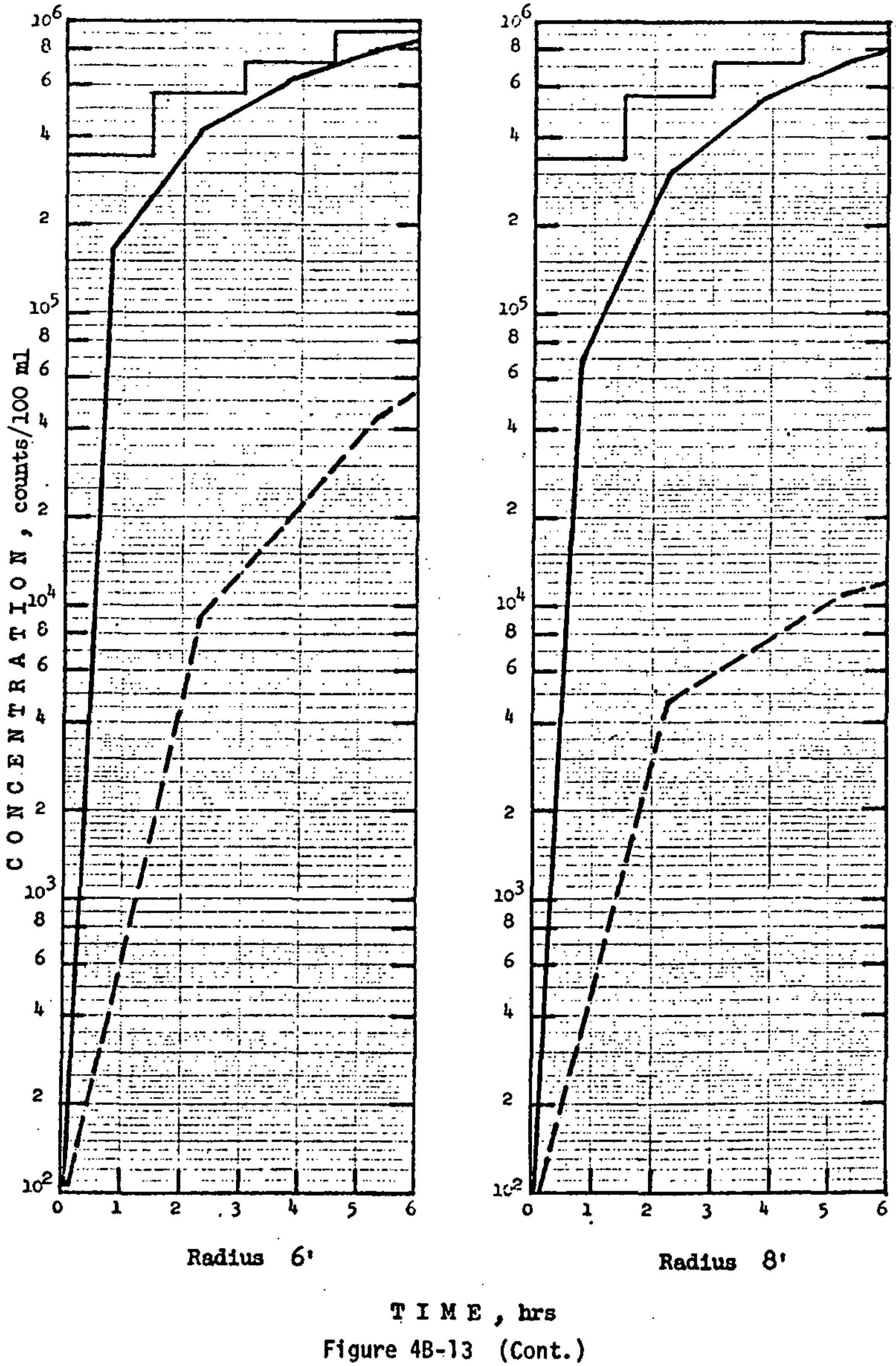


Figure 4B-13 (Cont.)

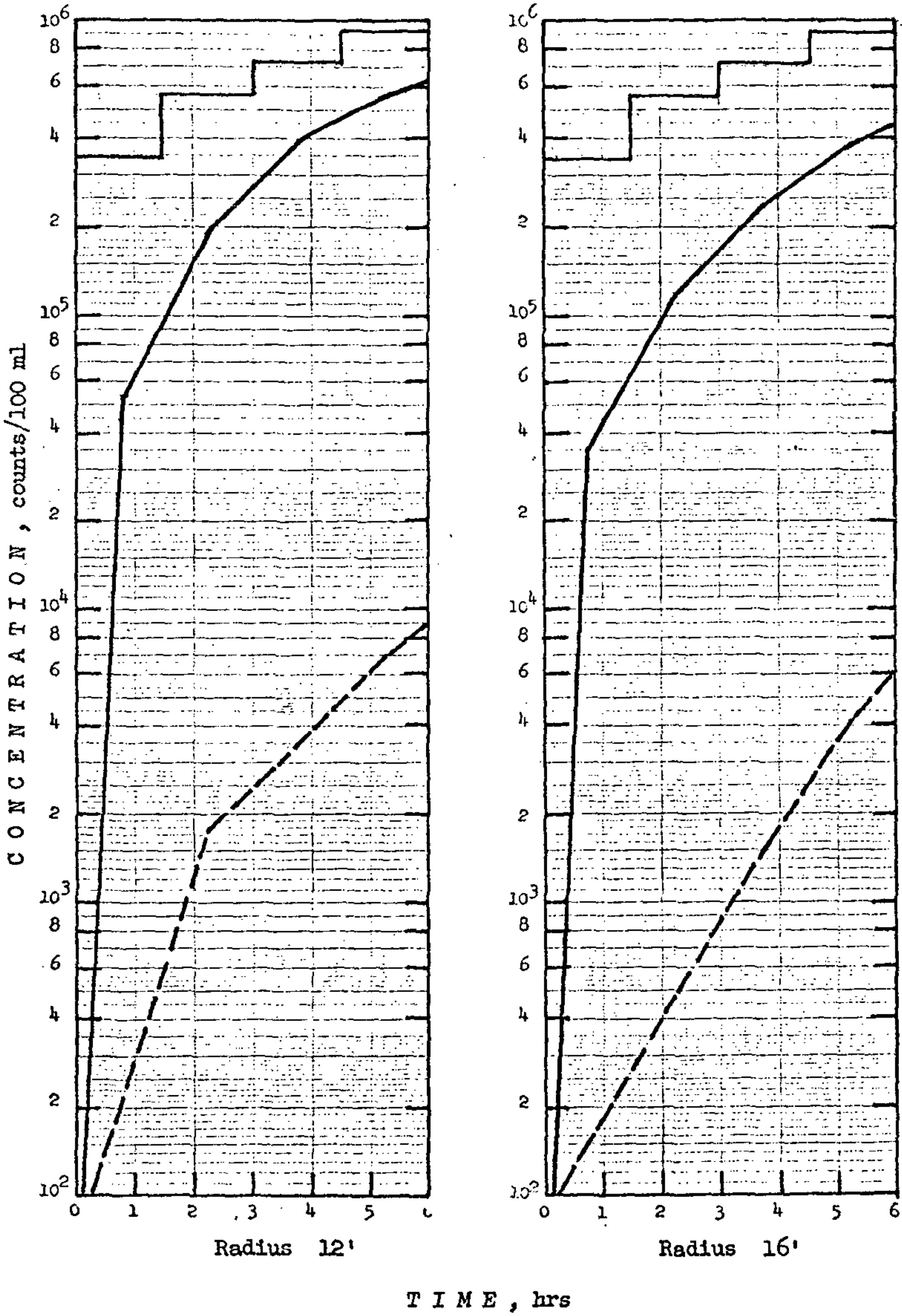


Figure 4B-13 (Cont.)

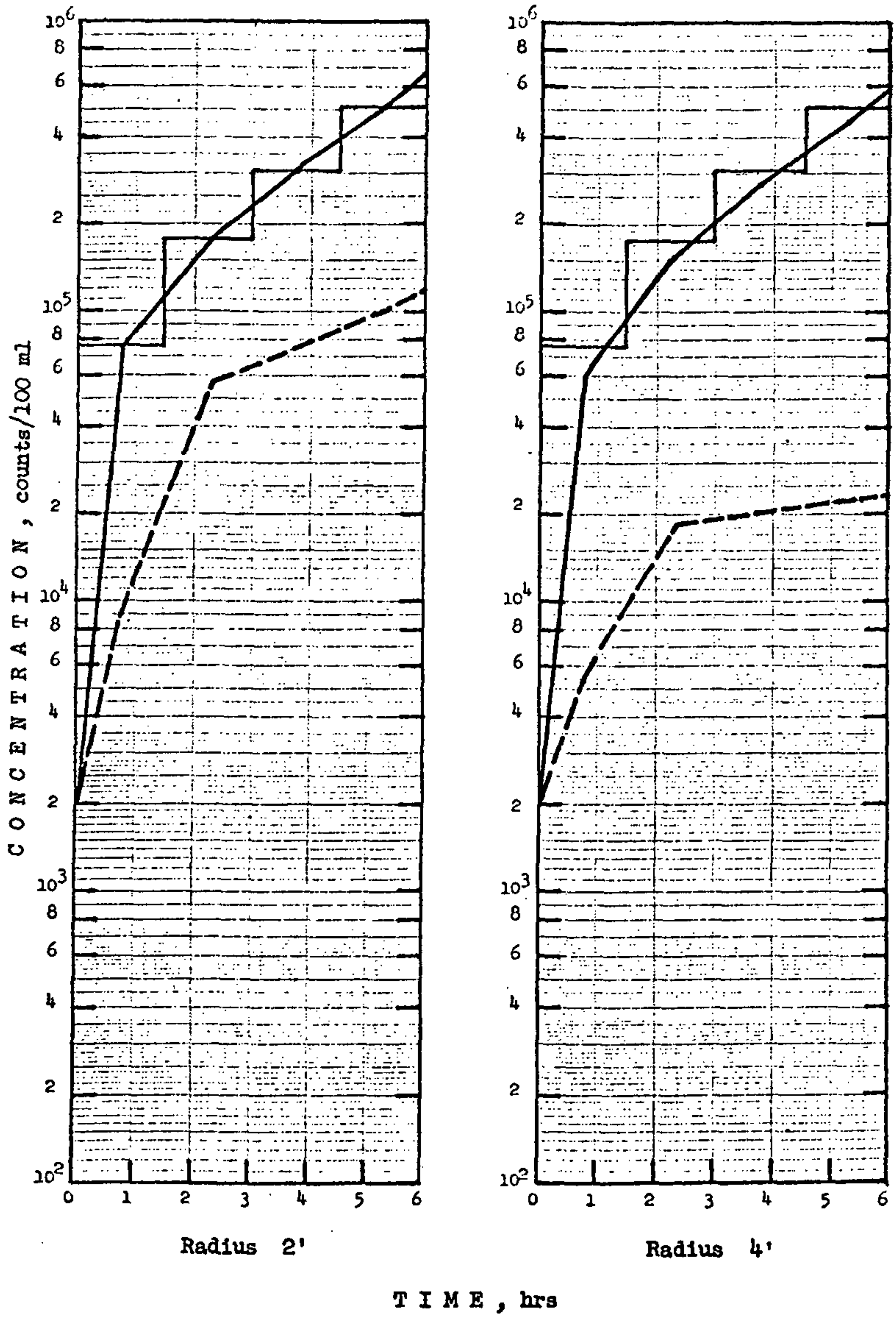
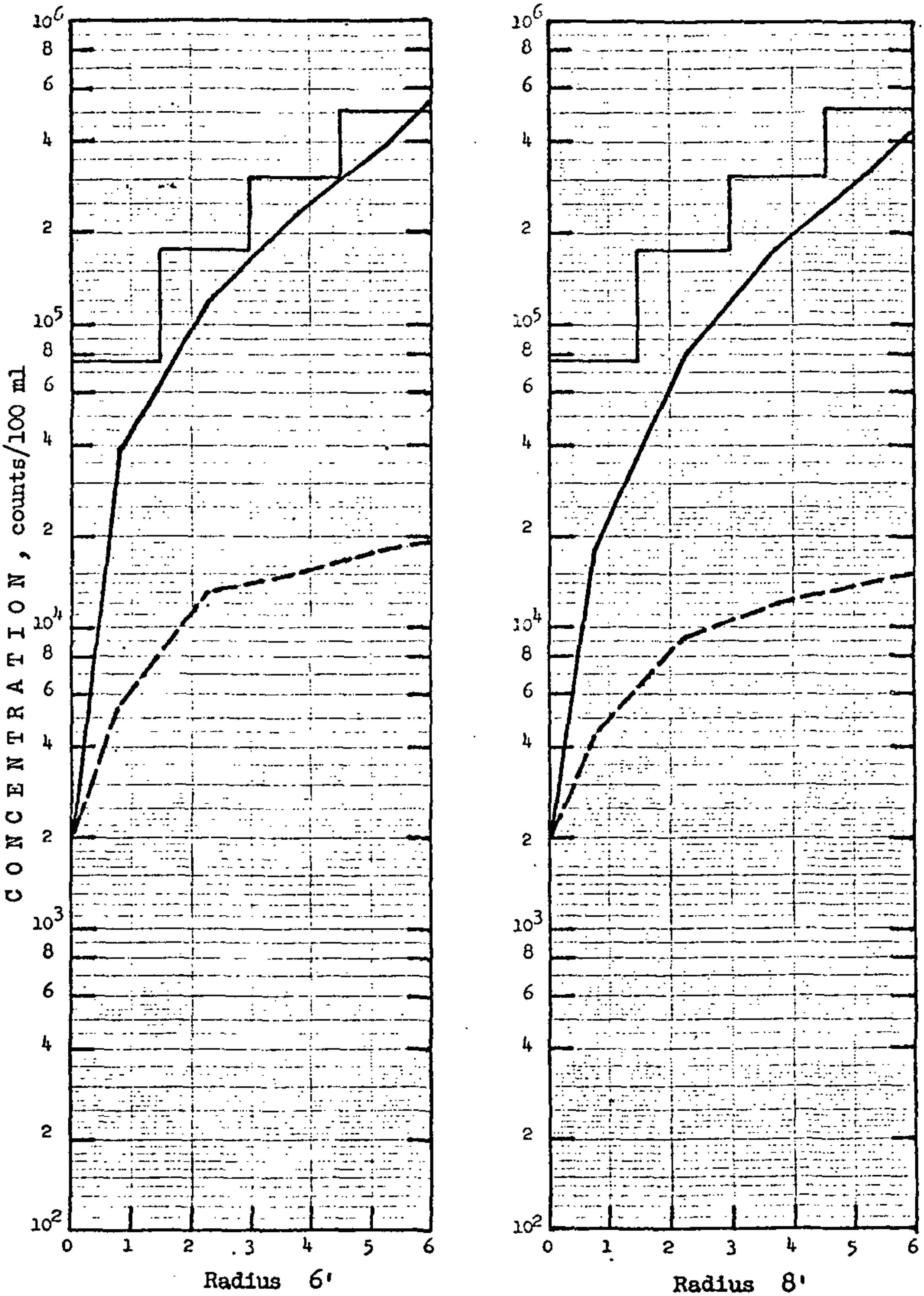
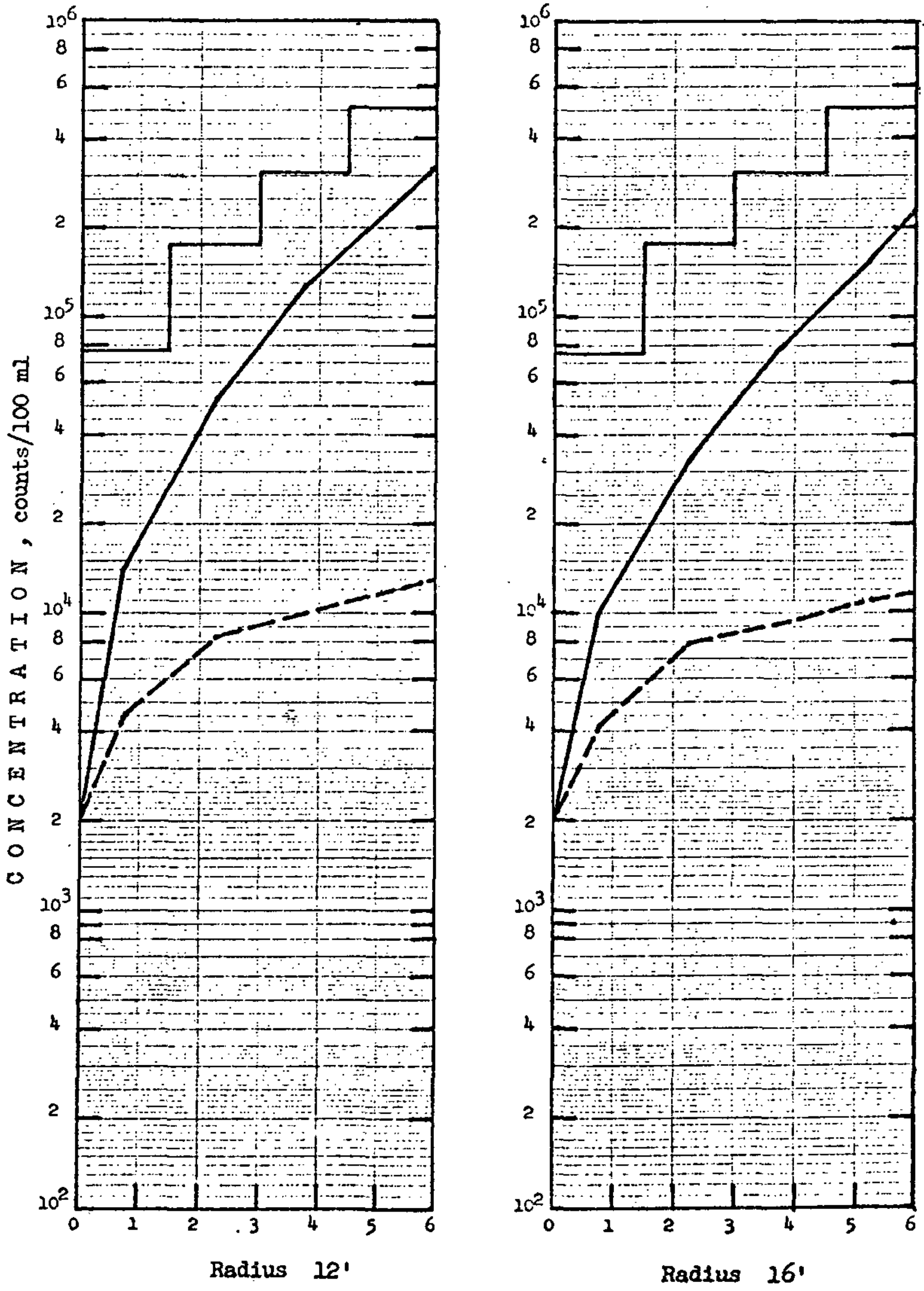


Figure 4B-14 Total Coliform Concentration vs. Time
(Run #2 - Type II Injection Pollutograph)



T I M E , hrs

Figure 4B-14 (Cont.)



T I M E , hrs

Figure 4B-14 (Cont.)

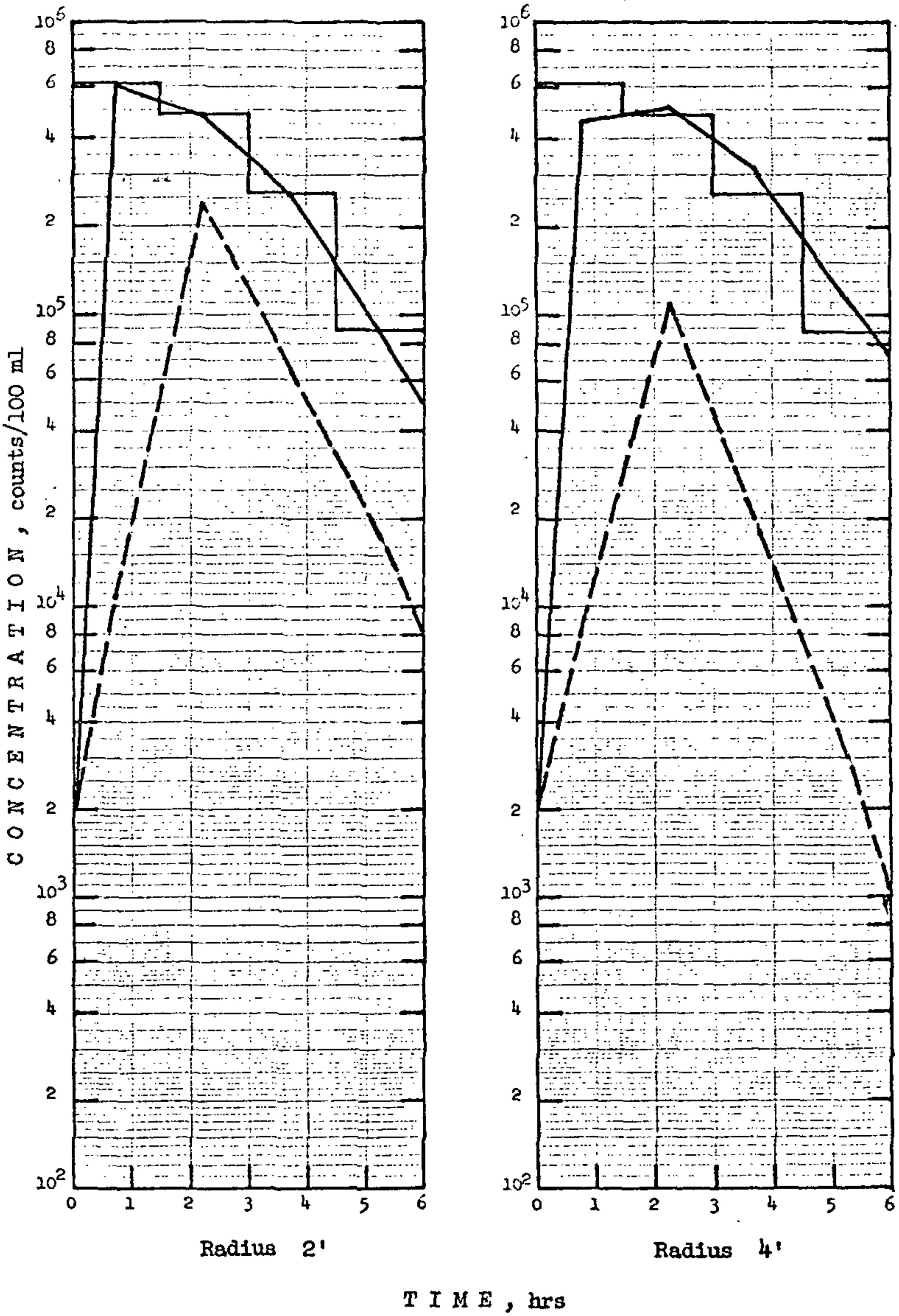


Figure 4B-15 Total Coliform Concentration vs. Time
(Run #3 - Type I Injection Pollutograph)

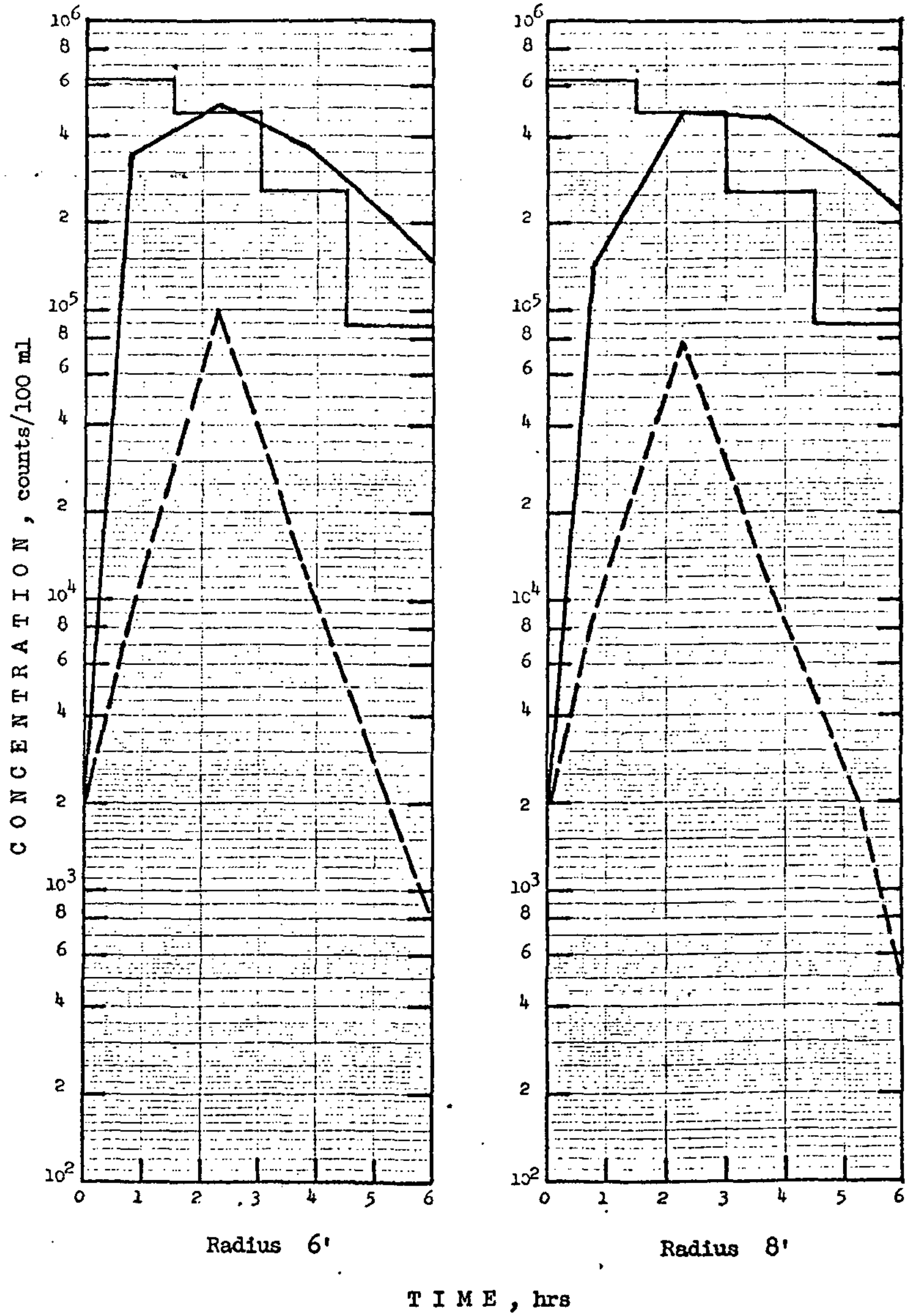


Figure 4B-15 (Cont.)

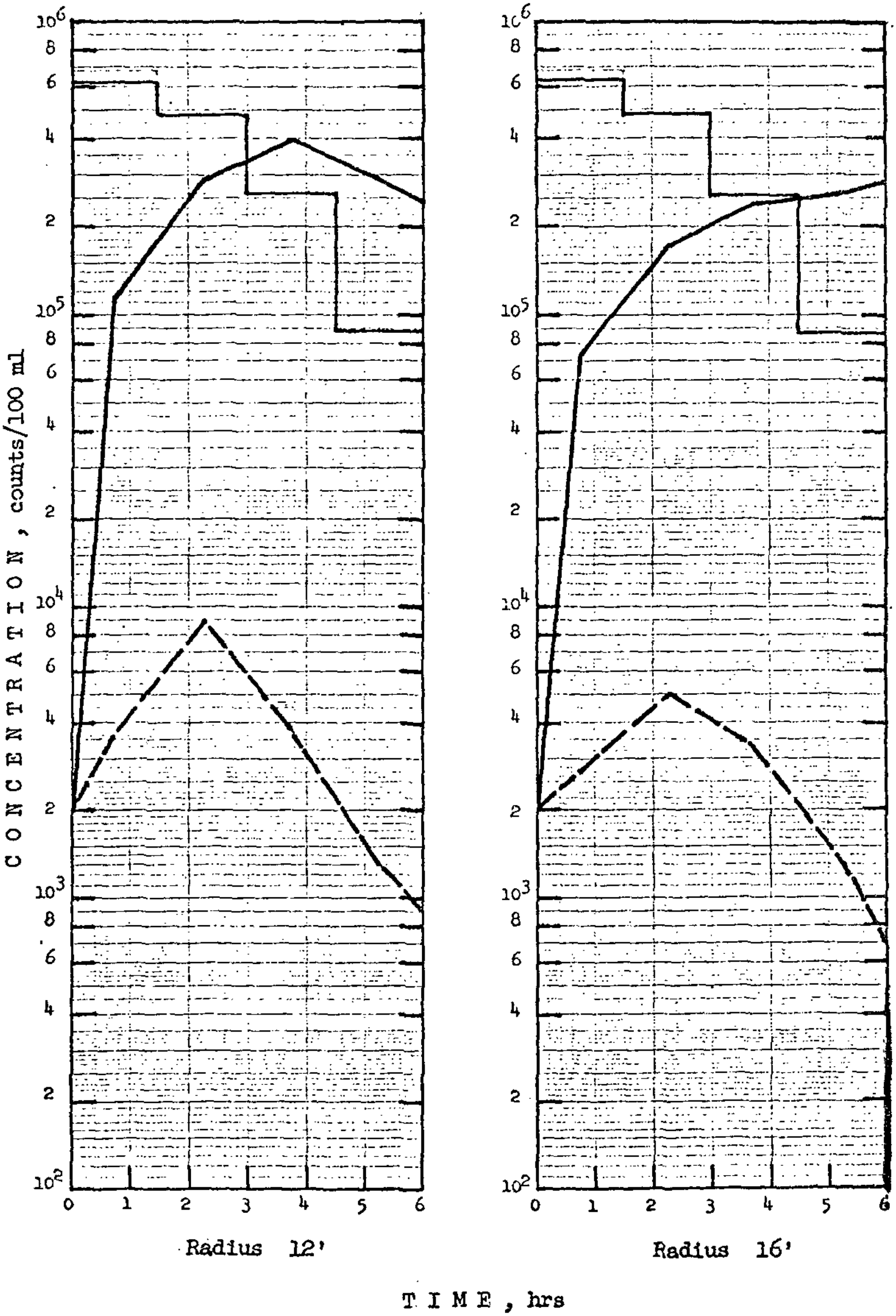


Figure 4B-15 (Cont.)

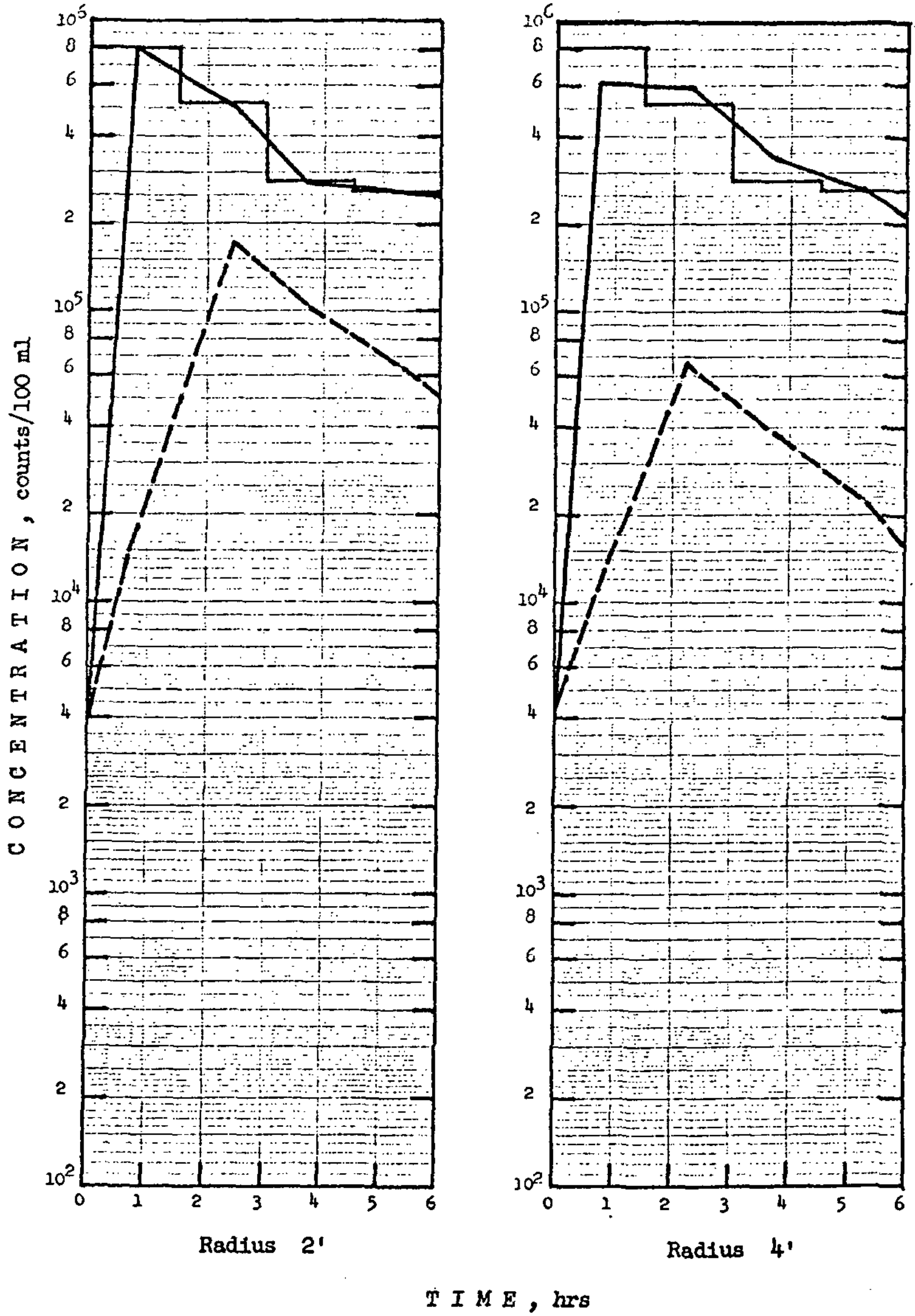


Figure 4B-16 - Total Coliform Concentration vs. Time
(Run #4 - Type I Injection Pollutograph)

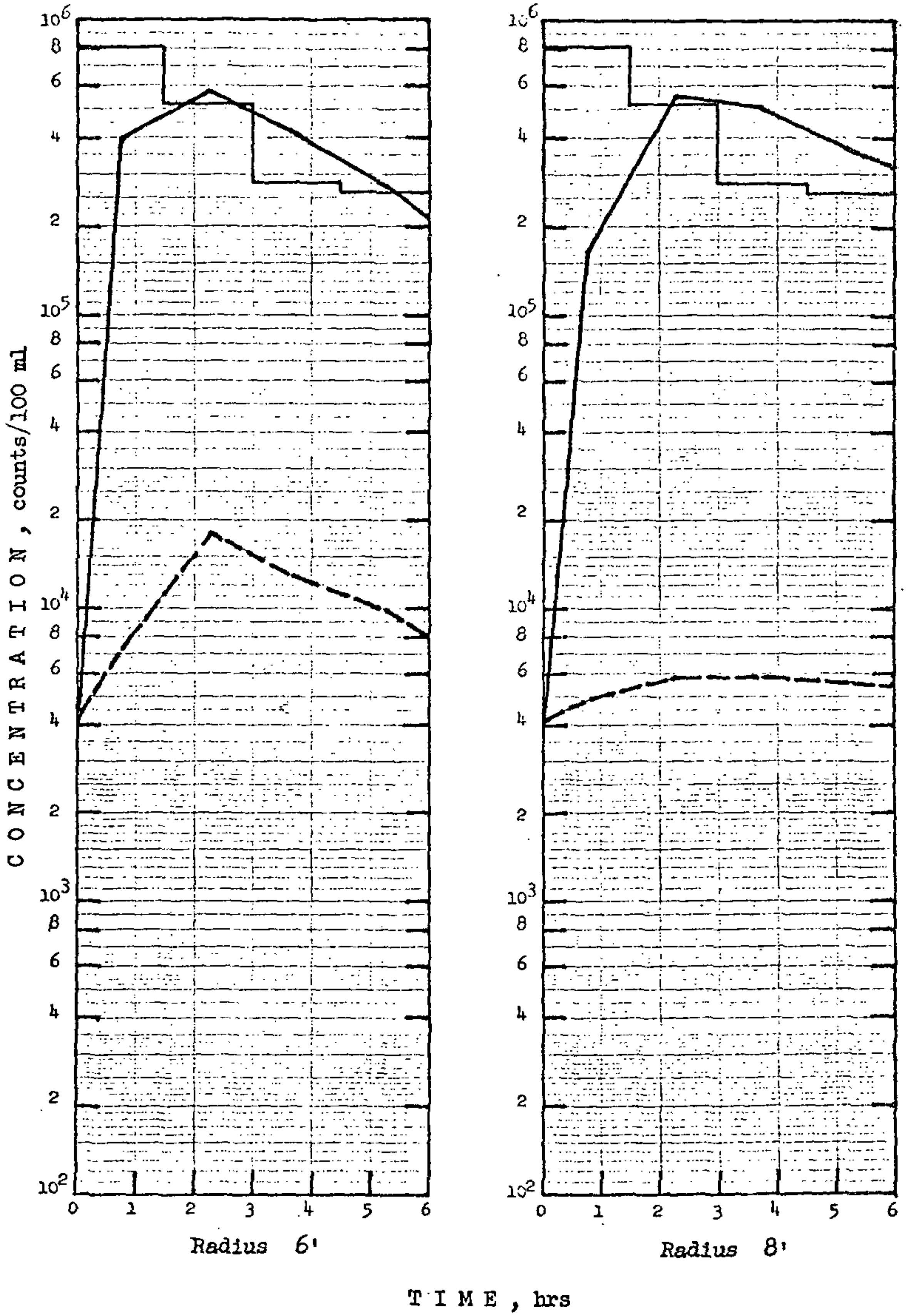


Figure 4B-16 (Cont.)

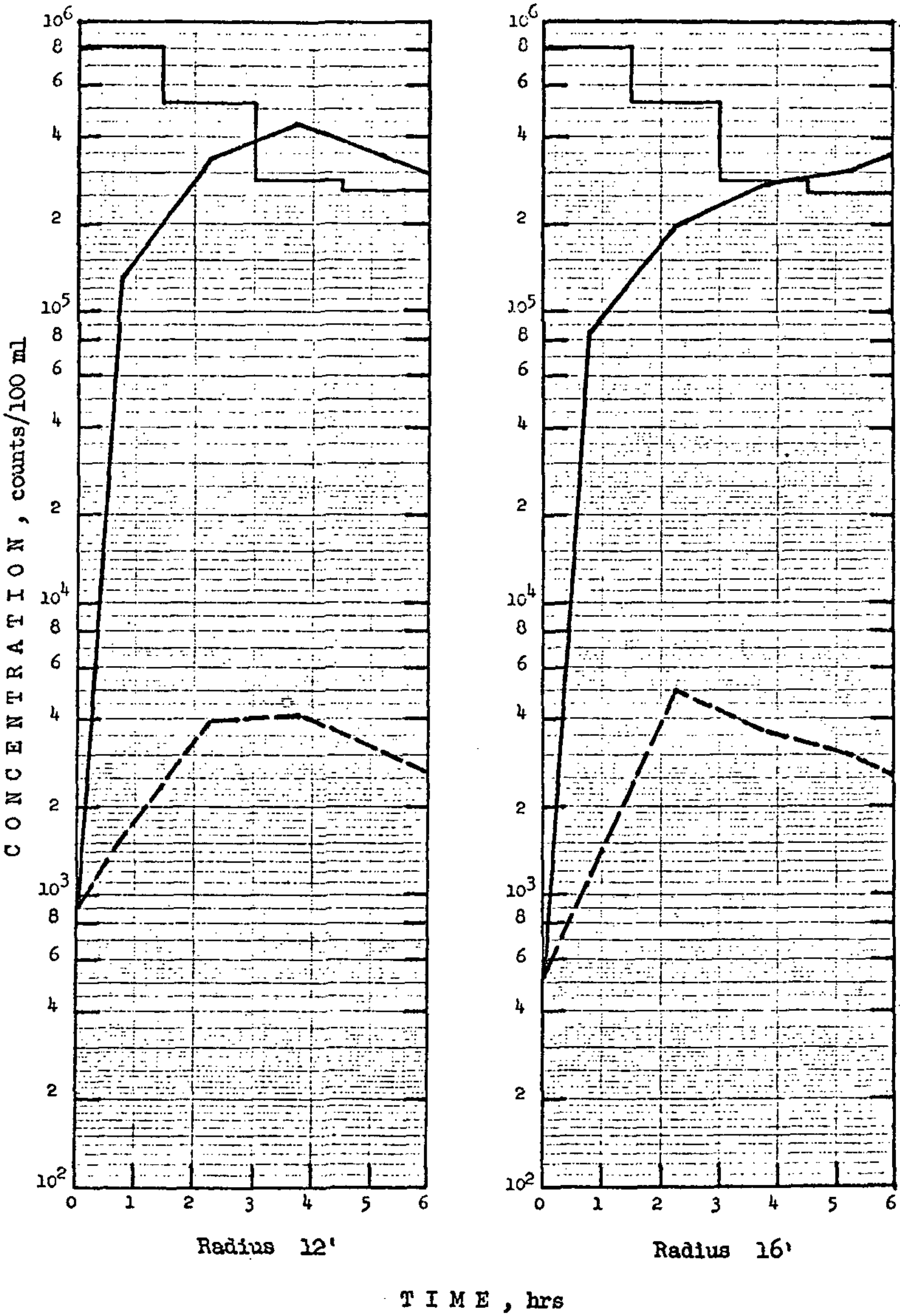


Figure 4B-16 (Cont.)

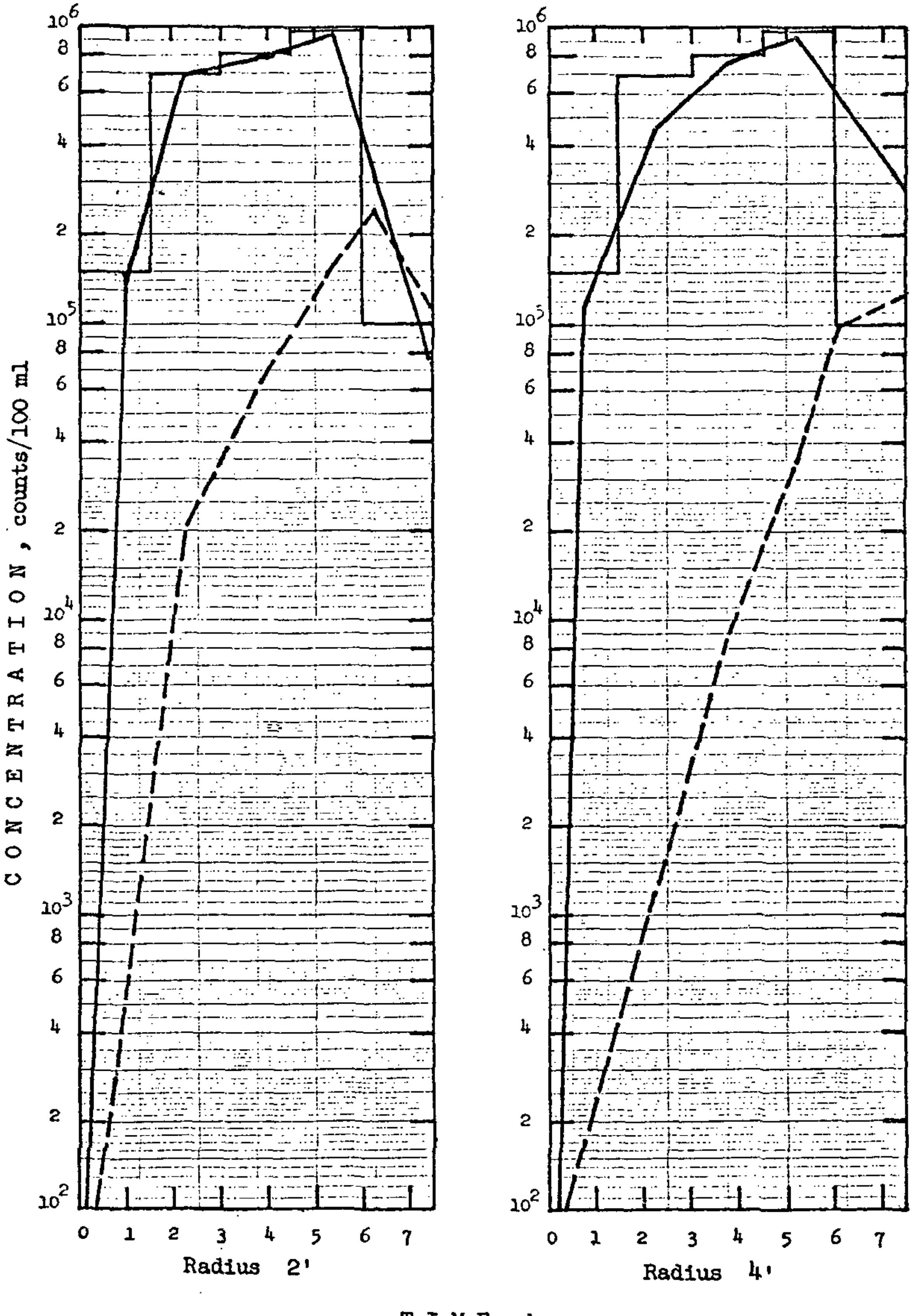


Figure 4B-17 Total Coliform Concentration vs. Time
(Run #5 - Type III Injection Pollutograph)

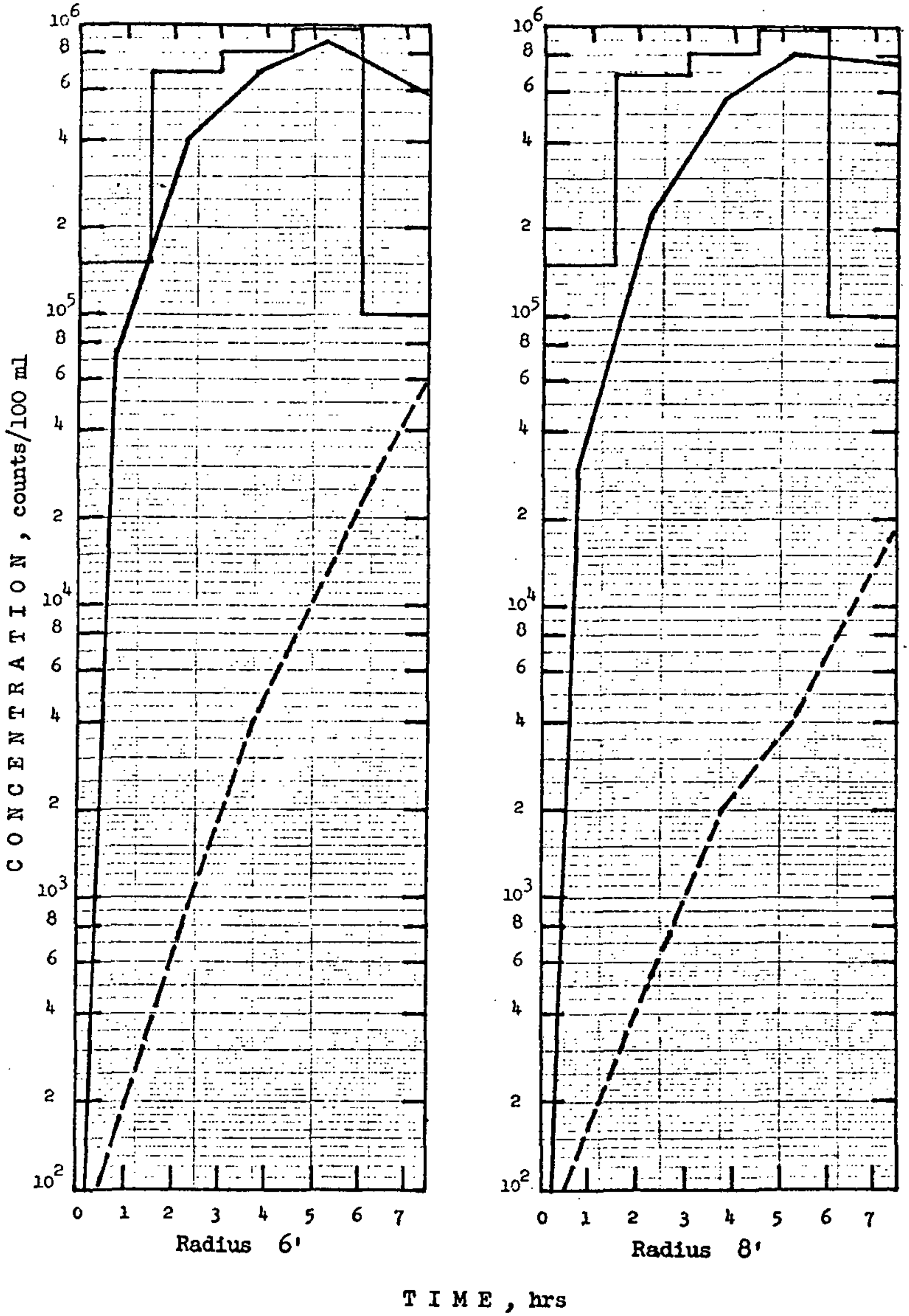


Figure 4B-17 (Cont.)

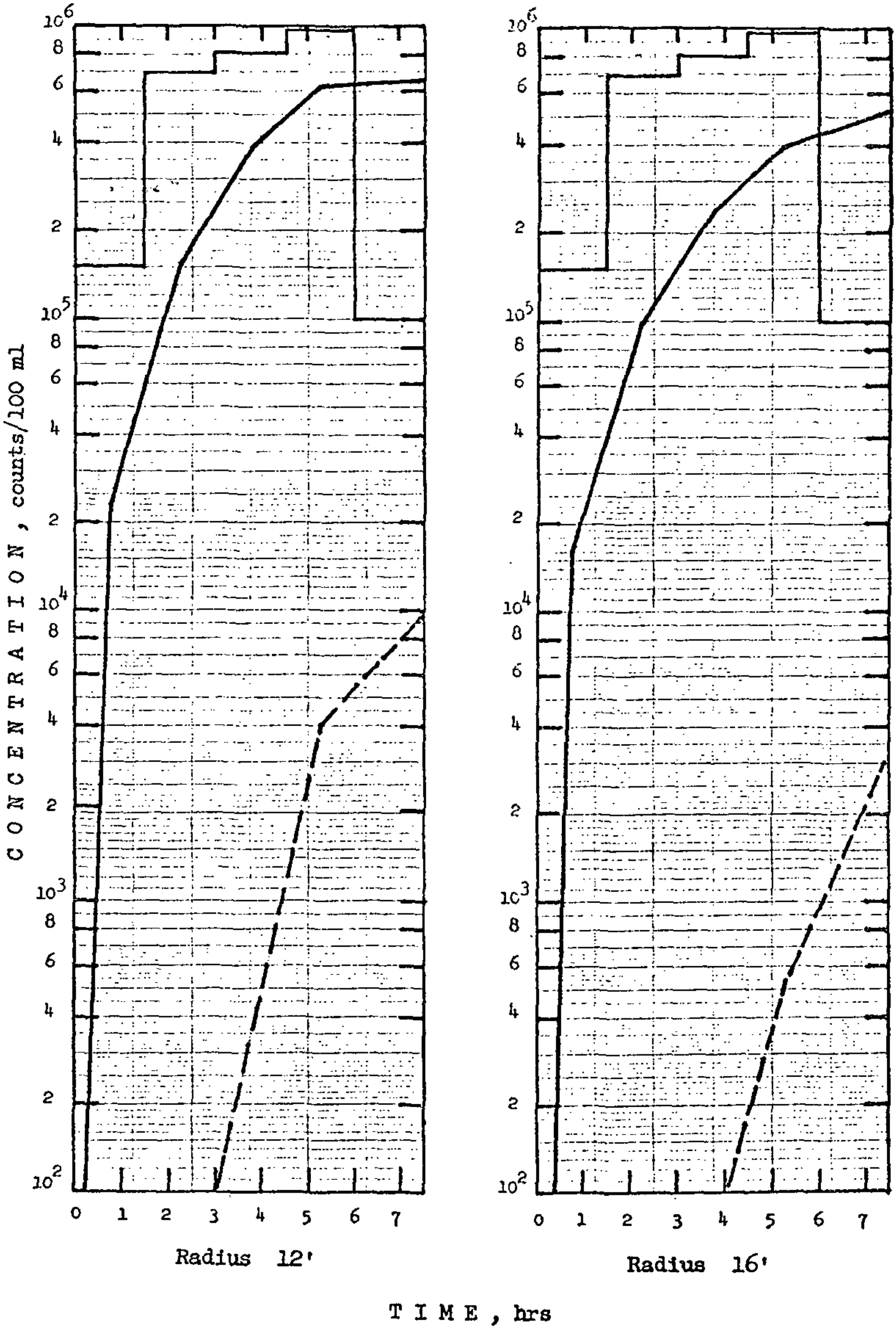


Figure 4B-17 (Cont.)

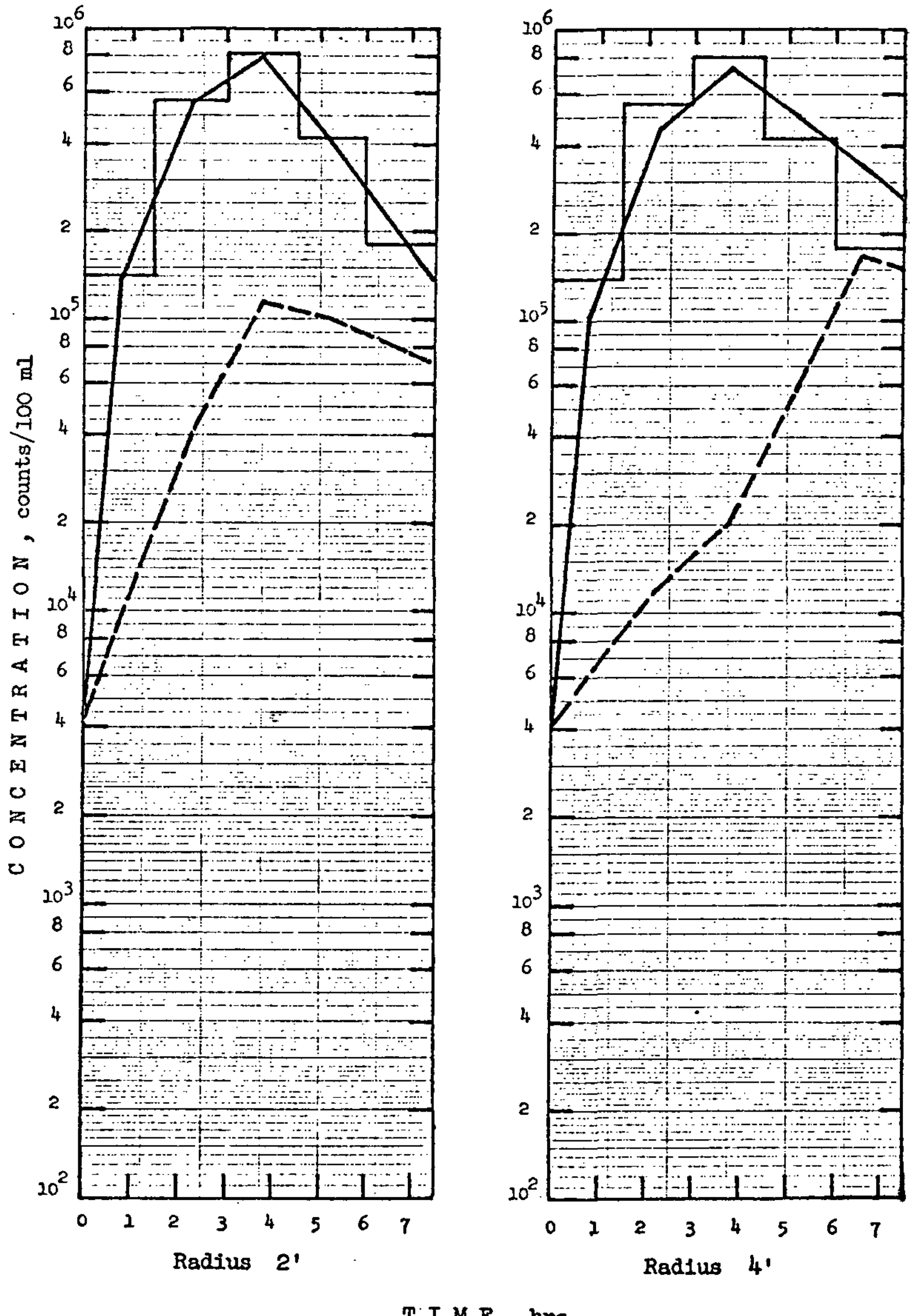


Figure 4B-18 Total Coliform Concentration vs. Time
(Run #6 - Type III Injection Pollutograph)

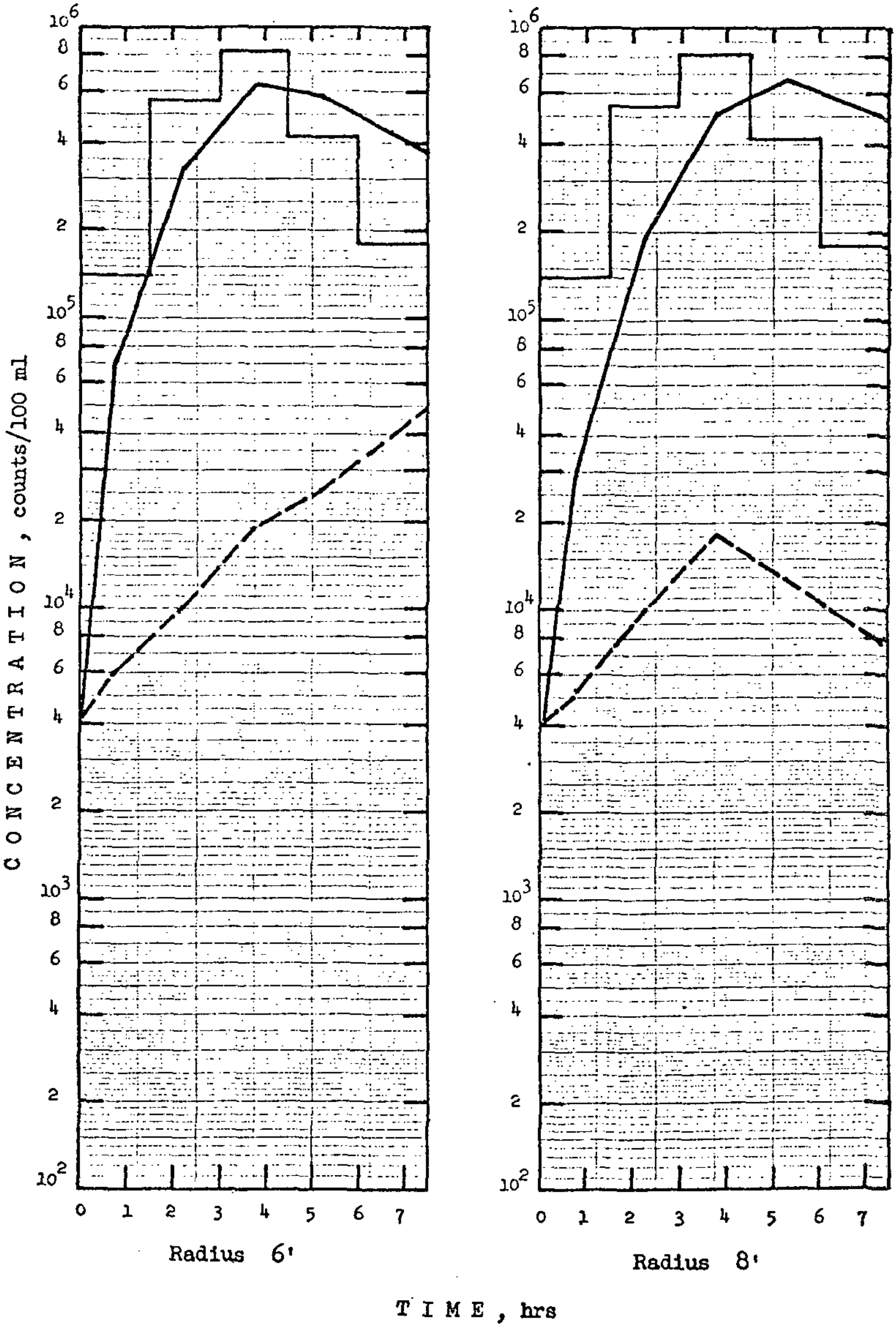


Figure 4B-18 (Cont.)

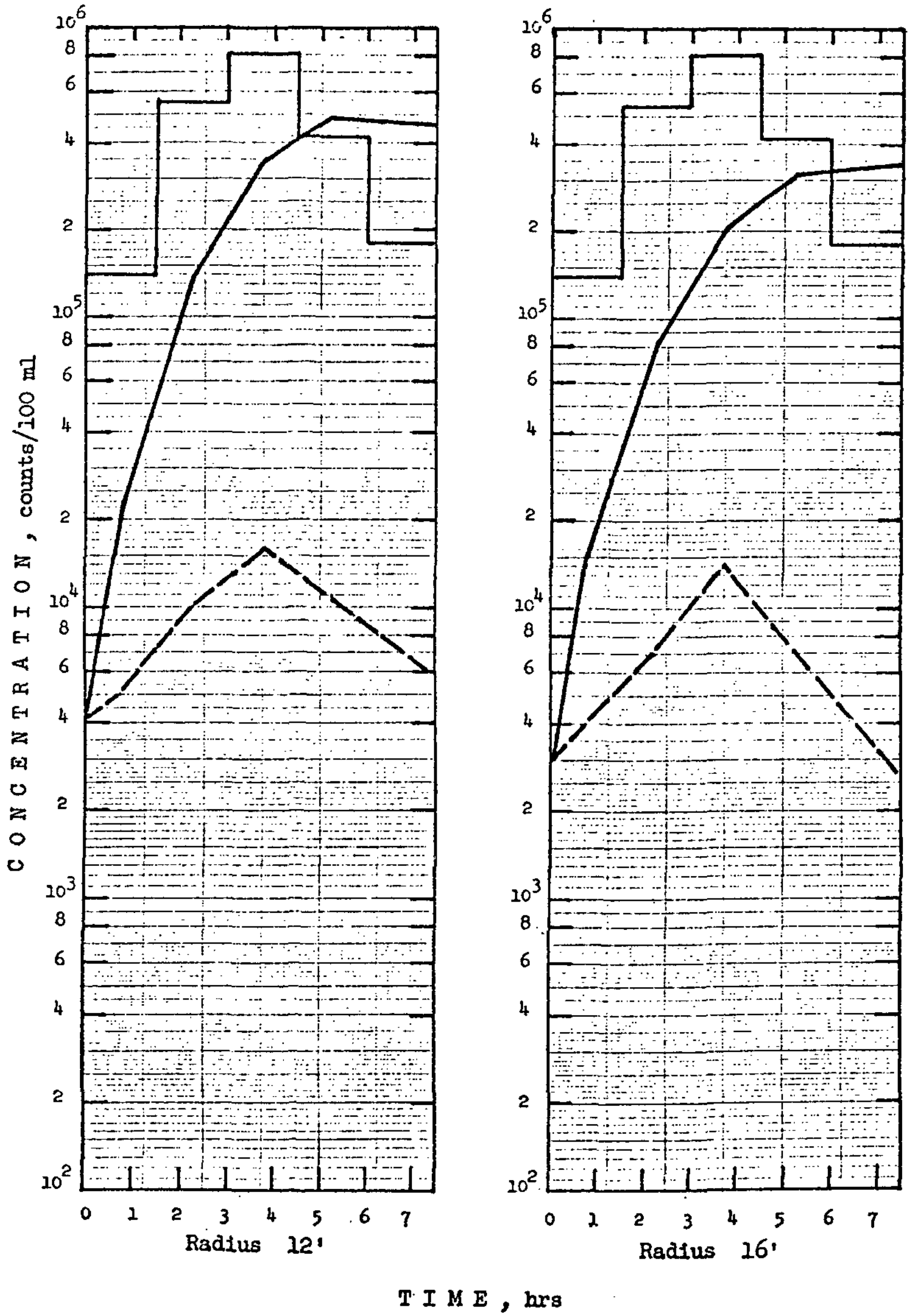


Figure 4B-18 (Cont.)

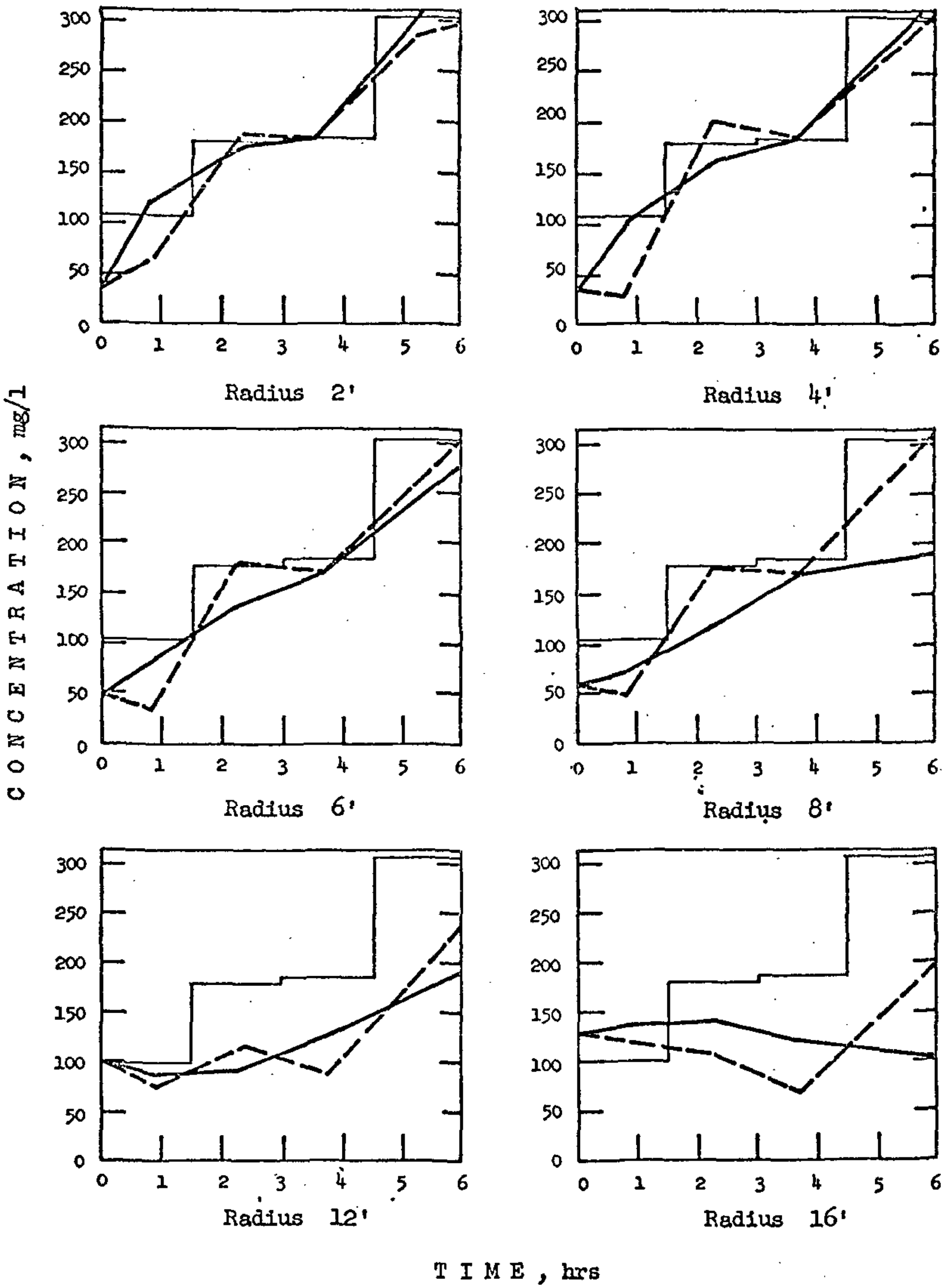


Figure 4B-19 - Total Solid Concentration vs. Time
(Run #1 - Type II Injection Pollutograph)

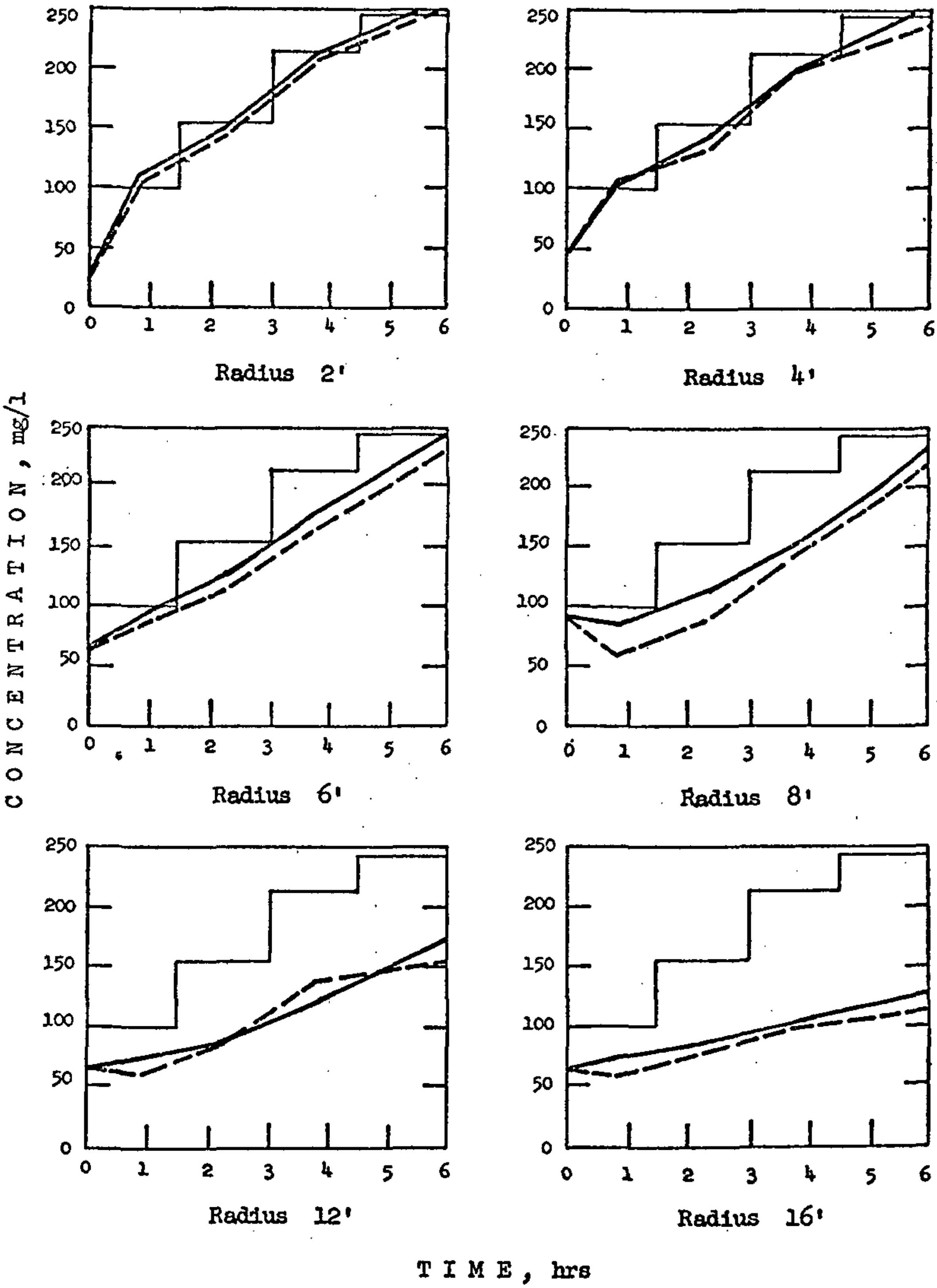


Figure 4B-20 Total Solids Concentration vs. Time
(Run #2 - Type II Injection Pollutograph)

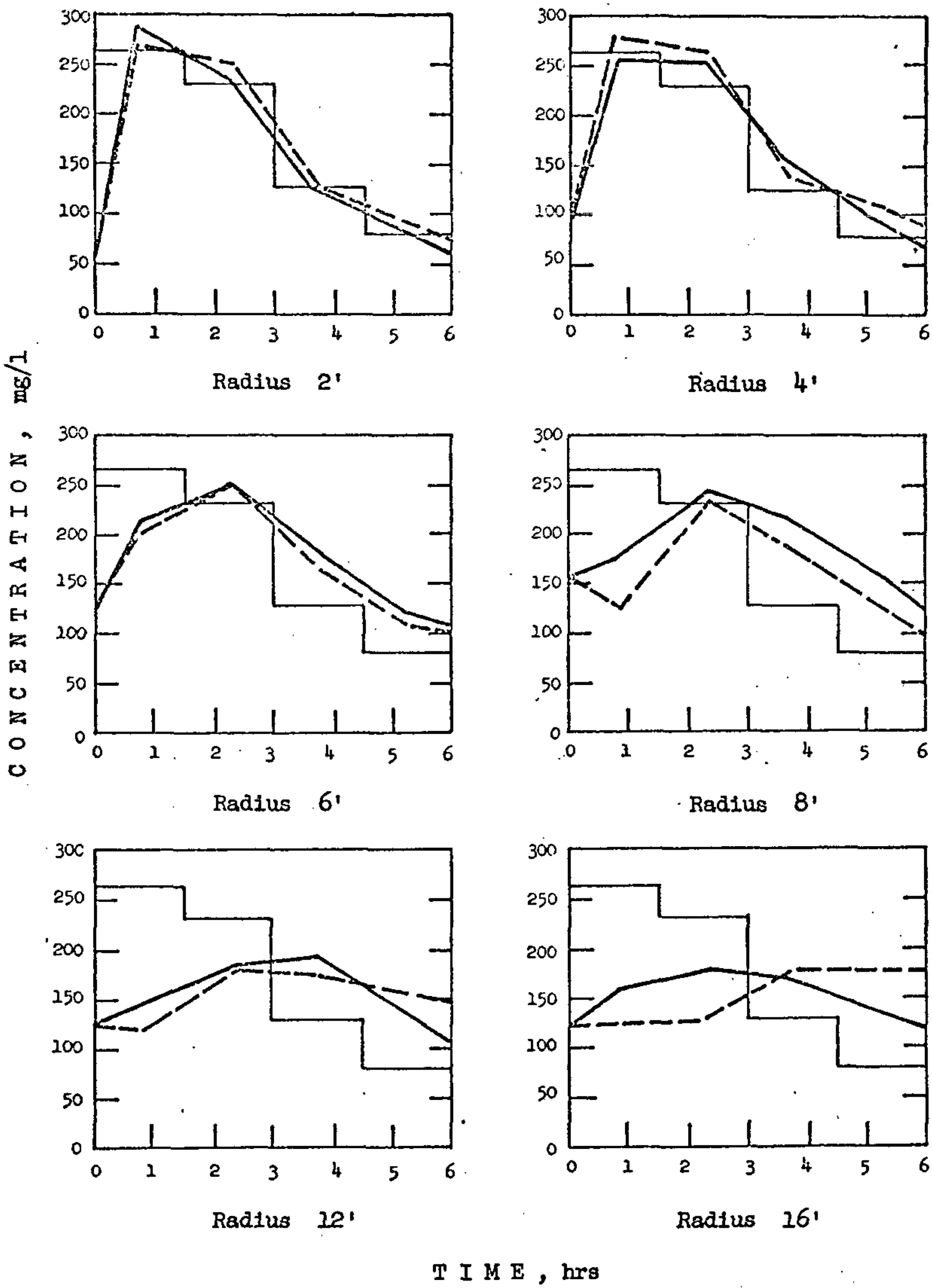


Figure 4B-21 Total Solids Concentration vs. Time
(Run #3 - Type I Injection Pollutograph)

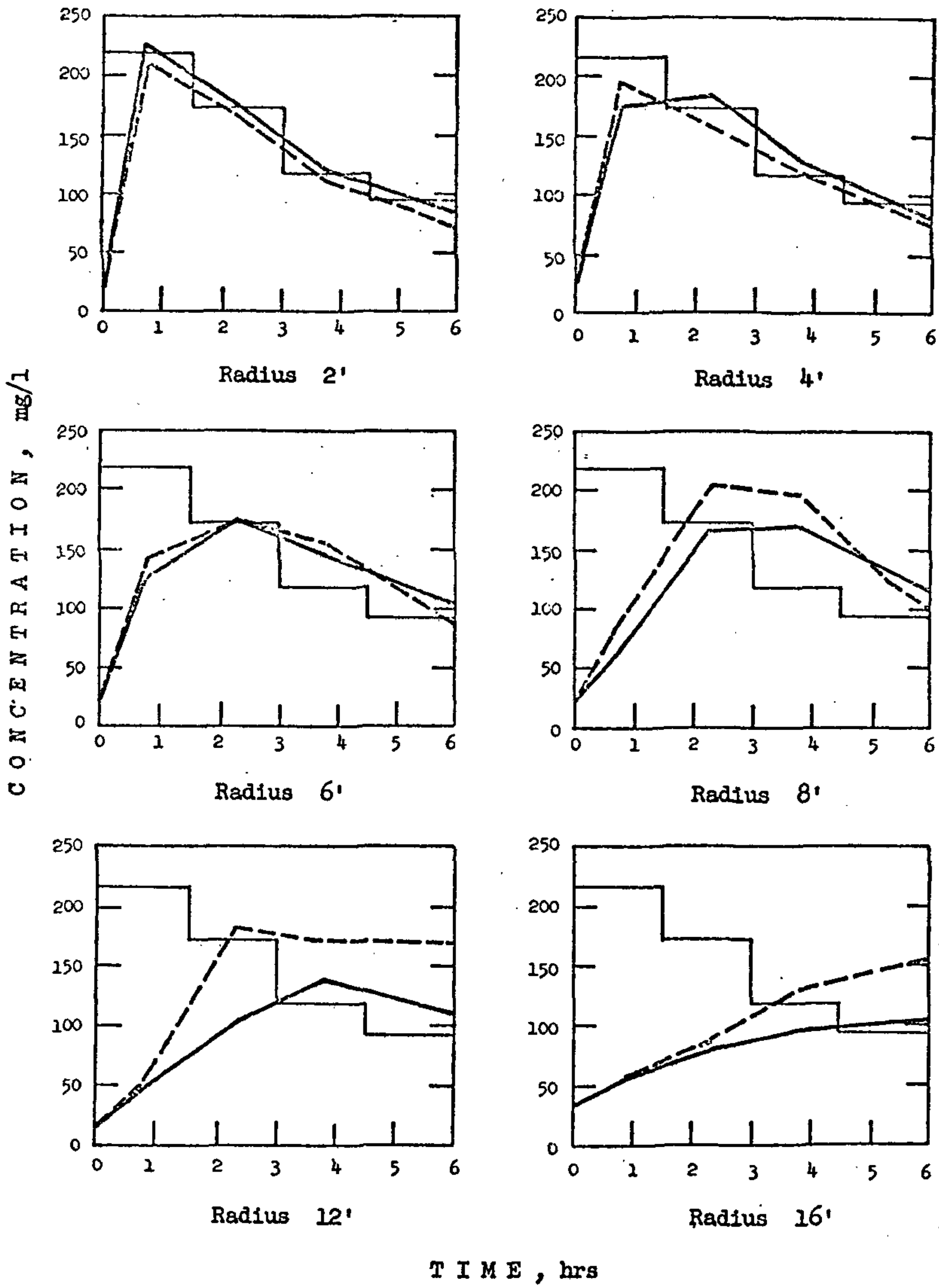


Figure 4B-22 Total Solids Concentration vs. Time
(Run #4 - Type I Injection Pollutograph)

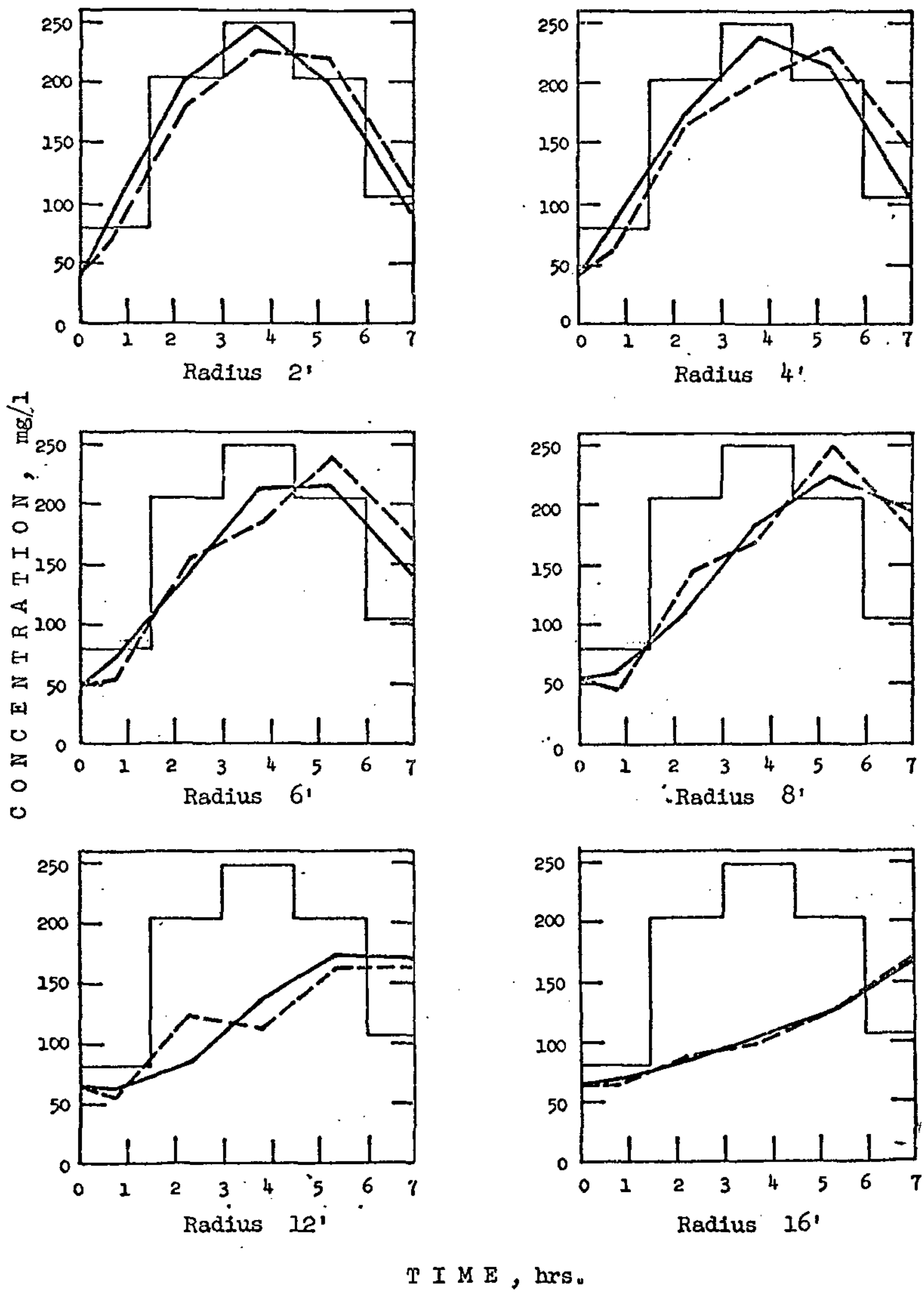


Figure 4B-23 Total Solids Concentration vs. Time
(Run #5 - Type III Injection Pollutograph.)