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Stream Pollution And Abatement From Combined Sewer Overflows

BUCYRUS, OHIO

U.S. DEPARTMENT OF THE INTERIOR • FEDERAL WATER QUALITY ADMINISTRATION

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STREAM POLLUTION AND ABATEMENT FROM COMBINED SEWER OVERFLOWS
BUCYRUS, OHIO

A Study of Stream Pollution
From Combined Sewer Overflows and
Feasibility of Alternate Plans for
Pollution Abatement in Bucyrus, Ohio

FEDERAL WATER QUALITY ADMINISTRATION
DEPARTMENT OF THE INTERIOR

by

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This report has been reviewed by the Federal Water Pollution Control Administration and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Water Pollution Control Administration.

ABSTRACT

This report contains the results of a detailed engineering investigation and comprehensive technical study to evaluate the pollutional effects from combined sewer overflows on the Sandusky River at Bucyrus, Ohio and to evaluate the benefits, economics and feasibility of alternate plans for pollution abatement from the combined sewer overflows. The City of Bucyrus is located near the upper end of the Sandusky River Basin which is tributary to Lake Erie. Bucyrus has an incorporated area of about 2,340 acres, a population of 13,000, and a combined sewer system with an average dry weather wastewater flow of 2.2 million gallons per day. A year long detailed sampling and laboratory analysis program was conducted on the combined sewer overflows in which the overflows were measured and sampled at 3 locations comprising 64% of the City's sewered area and the river flow was measured and sampled above and below Bucyrus.

The results of the study show that any 20 minute rainfall greater than 0.05 of an inch will produce an overflow. The combined sewers will overflow about 73 times each year discharging an estimated annual volume of 350 million gallons containing 350,000 pounds of BOD and 1,400,000 pounds of suspended solids. The combined sewer overflows had an average BOD of 120 mg/l, suspended solids of 470 mg/l, total coliforms of 11,000,000 per 100 ml and fecal coliforms of 1,600,000 per 100 ml. The BOD concentration of the Sandusky River, immediately downstream from Bucyrus, varied from an average of 6 mg/l during dry weather to a high of 51 mg/l during overflow discharges. The suspended solids varied from an average of 49 mg/l during dry weather to a high of 960 mg/l during overflow discharges. The total coliforms varied from an average of 400,000 per 100 ml during dry weather to a high of 8,800,000 per 100 ml during overflow discharges.

Various methods of controlling the pollution from combined sewer overflows are presented along with their degree of protection, advantages, disadvantages and estimates of cost. The methods presented include (1) complete separation, (2) interceptor sewer and lagoon system, (3) stream flow augmentation, (4) primary treatment, (5) chlorination, and (6) offstream treatment. It was concluded that the most economical method of providing a high degree of protection to the Sandusky River is by collecting the combined sewer overflows with a large interceptor and using an aerated lagoon system to treat the waste loads from the overflows.

This report was submitted in fulfillment of Contract 14-12-401 between the Federal Water Pollution Control Administration and Burgess & Niple, Limited.

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SECTION I

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

1. Any 20 minute rainfall greater than 0.05 inches will produce an overflow of wastewater into the Sandusky River at Bucyrus. A rainfall of this intensity and duration or greater will occur on the average of once every 5 days.

2. A typical summer thundershower occurred on June 13, 1969 and produced 1.1 inches of rain, had a duration of 78 minutes and an average intensity of 0.84 inches per hour. The runoff from this storm discharged into the Sandusky River, through the combined sewer overflows, 5,200,000 gallons of combined sewer wastewater, 1580 pounds of BOD and 23,000 pounds of suspended solids.

3. A storm on August 9, 1969 which produced 0.50 inches of rain in about 75 minutes, increased the BOD concentration of the Sandusky River downstream from Bucyrus from 11 mg/l (530 pounds per day) at a river flow of 9 cfs to 51 mg/l (35,500 pounds per day) at a river flow of 130 cfs.

4. The combined sewers will overflow about 73 times each year discharging an estimated total annual volume of 350 million gallons or about 1 million gallons per day.

5. The combined sewer overflows have an average BOD of 120 mg/l, suspended solids of 470 mg/l, total coliforms of 11,000,000 per 100 ml and fecal coliforms of 1,600,000 per 100 ml.

6. The combined sewer overflows at Bucyrus discharge an estimated 350,000 pounds of BOD and 1,400,000 pounds of suspended solids annually into the Sandusky River.

7. The BOD concentration of the Sandusky River, immediately downstream from Bucyrus, varied from an average of 6 mg/l during dry weather to a high of 51 mg/l during overflow discharges. The suspended solids varied from an average of 49 mg/l during dry weather to a high of 960 mg/l during overflow discharges. The total coliforms (by membrane filter technique) varied from an average of 400,000 per 100 ml during dry weather to a high of 8,800,000 per 100 ml during overflow discharges.

8. The estimated yearly discharge of 15,700 pounds of nitrate nitrogen (12,200 pounds from overflows and 3,500 pounds from wastewater plant) from Bucyrus is rather insignificant when compared to the 136,000 pounds and 192,000 pounds found in the river coming from the upper drainage basin on April 19, 1969 and May 19, 1969, respectively.

9. The nitrate nitrogen concentration of the Sandusky River, upstream from Bucyrus, varied from a low of 0.4 mg/l as NO_3 to a high of 32 mg/l. The high concentrations occurred during high river flows in the spring of the year. The estimated nitrate nitrogen discharged from the upstream drainage area is 2,300,000 pounds annually.

10. The combined sewer overflows discharge about 30,000 pounds of phosphates (PO_4) into the river annually. The wastewater treatment plant discharges about 160,000 pounds of PO_4 each year. An estimated 110,000 pounds of PO_4 per year came from the upstream drainage area.

11. Sludge accumulation in the river from combined sewer overflows at Bucyrus is estimated to be approximately 47,000 cubic feet per year.

12. The flushing effect of the sewer system during intense rainfalls causes the majority of the waste load to be discharged during the peak overflow period as evidenced by the peak concentration of the various water quality characteristics which tend to coincide with the peak overflow rate.

13. The effects of the combined sewer overflows on the Sandusky River in and below the City of Bucyrus are visually apparent. There are indications of gross pollution, such as sludge banks, sections of the river are devoid of oxygen, extensive algae growth, and some sections of the river are completely devoid of life.

14. The probability of thundershower type storms occurring is highest during the summer months. There is a 74% probability of the 2 year, 1 hour thundershower, which has a total rainfall of 1.23 inches, occurring in June, July and August.

15. The median flow in the Sandusky River at Bucyrus in June, July and August is 13 cfs, 6.9 cfs and 4.8 cfs, respectively.

16. The assimilative capacity of the Sandusky River (the ability of the river for self-purification) is limited to approximately 25 pounds of BOD per day per cfs at flows less than 10 cfs. Forty percent of the time the flow in the Sandusky River at Bucyrus is less than 10 cfs.

17. The weighted average runoff for all storms measured was 19 percent. For storms with over 1.0 inches of rainfall, the weighted average runoff through the combined sewers equals 20 percent, 28 percent and 25 percent for Sewer Districts 8, 17 and 23, respectively.

18. The volume and character of pollutants discharged to surface watercourses from combined sewer systems in other similar communities would no doubt be very similar to that found to exist at Bucyrus, Ohio.

19. The various methods of reducing or controlling pollution from combined sewers considered in this study and the estimated project cost of each are as follows:

    A. Complete separation of the combined sewer system into a sanitary sewer system and storm sewer system.

        1. Construct new sanitary sewers using the existing system as storm sewer system -- $9,300,000.

        2. Construct new storm sewer system using existing system as sanitary sewer system -- $8,800,000.

    B. Interceptor Sewer and Lagoon System

1. Gravity Interceptor	$3,600,000
2. Pump Station	1,000,000
3. Lagoon System	620,000
	$5,220,000

C. Stream Flow Augmentation	$5,000,000
D. Primary Treatment of Overflows	$8,810,000
E. Chlorination of Overflows	$3,000,000
F. Offstream Treatment	
1. Pump Station and Low Head Dam	$1,080,000
2. Lagoon System	620,000
	$1,700,000

20. Sewer separation, at Bucyrus, as a solution to combined sewer overflows will only reduce the waste loads discharged to the river by about 50%.

21. Construction and operation of the "Interceptor and Lagoon System" or "Off-Stream Treatment" as a demonstration project would be the most economical method of reducing or controlling pollution of the Sandusky River at Bucyrus and would provide answers to certain design, operation and benefit unknowns.

22. Stream flow augmentation as a method of controlling pollution from combined sewer overflows is not feasible at Bucyrus due to lack of suitable reservoir sites.

23. Primary treatment of overflows will only reduce the waste loads discharged into the Sandusky River through the combined sewer overflows by 50 to 70 percent.

24. Chlorination of overflows will reduce the bacteria count discharged into the river by combined sewer overflows but will not reduce significantly, any of the other pollutional characteristics of the overflows. Therefore, adequate treatment cannot be provided by chlorination alone.

Recommendations

1. The "Interceptor Sewer and Lagoon System" for abating pollution from combined sewer overflows should be adopted for Bucyrus.

   The benefits from controlling pollution due to combined sewer overflows by the use of an "Interceptor and Lagoon System" are many.

   (a) Reduces pollution of the river both within the city of Bucyrus and downstream.

   (b) Stream protection surpasses that to be achieved by combined sewer separation in that all runoff up to the design storm will be intercepted and treated.

   (c) Increases the value of the stream to the public in the City and downstream from the City.

   (d) Reduces a health hazard within and below the City.

   (e) A clean stream provides the possibility through use of landscape architecture to beautify the stream, enhance its esthetic value and make it a real asset to the community.

2. The lagoon type of treatment should be demonstrated as capable of providing the degree of treatment required by constructing the "Off-Stream Treatment" concept as the first phase of the overall abatement program at Bucyrus, Ohio.

   The cost of the interceptor sewer represents a major portion of the "Interceptor Sewer and Lagoon System" method of abatement from combined sewer overflows. The "Off-Stream Treatment" method of protecting the downstream water quality without regard for the inner-city reach of the river provides a method whereby the initial project cost can be reduced substantially. The interceptor could be added for complete protection at a later date, as the final phase of the project.

   The City of Bucyrus is ideally situated in the Sandusky River watershed to demonstrate and evaluate pollution abatement from combined sewers on a watershed basis.

   (a) There are no large municipalities upstream to contribute pollutants.

   (b) The upstream watershed of approximately 90 square miles is used for general farming.

   (c) The river downstream from Bucyrus is presently used by several cities as a source of water supply and the river is destined to become a major source of water supply in the future.

   (d) The benefits of pollution abatement from sanitary waste and urban storm runoff (combined sewer overflow) to downstream water uses could be adequately demonstrated.

   (e) The reclamation and protection provided a principal river for all downstream water used would be impressive and could be used as an example for other watersheds.

   (f) Watershed protection rather than pilot or scale concepts is recommended to more fully evaluate the design storm, hydraulic variables of storm runoff, large scale operation cost, etc.

   The "Off-Stream Treatment" concept which is proposed as the first phase of the Bucyrus project has many applications where the stream or river must receive treatment to achieve the desired water quality standards. A full scale project such as proposed at Bucyrus would demonstrate the benefits to be derived and design criteria could be developed which would be valuable for other projects.

3. The interceptor system should be constructed as the second and final phase of abatement when it has been adequately demonstrated that the lagoon type of treatment is adequate and capable of providing the water quality protection required.

4. Until such time as effective methods are constructed to control or eliminate the pollution problem the channels, waterways, or stream beds into which the overflows are discharged should be protected from erosion to prohibit the ponding of such overflows in pools which become septic and cause odors and are very unsightly. Periodic removal of debris from the stream channel especially in the urban area should be accomplished at frequent intervals.

5. Municipalities with combined sewer systems should construct separated systems when extending service to new areas of growth or replacing existing sewers and new storm sewers constructed should be discharged to outlets other than existing combined trunk sewers where and when feasible.

6. Automatic rain gauges, monitoring stations and sampling stations should be established both upstream and downstream from Bucyrus on the Sandusky River to provide additional base data for future evaluation studies on abatement projects undertaken.

7. Install continuous level recorders at the three overflow weirs that were constructed for this investigation. These recorders will provide a continuous record of overflow volume.

8. The officials of the City of Bucyrus should be fully informed of the results of this study and the recommendations contained herein and that they be given the opportunity to participate in a demonstration project for abatement of pollution from their combined sewer system.

SECTION II

INTRODUCTION

To achieve the water quality standards established in Ohio for streams, rivers and lakes all of the communities with combined storm and sanitary collection systems have been placed under orders by the Ohio Water Pollution Control Board to seek methods of abating pollution from their combined sewer overflows. Physical separation of the system is of course one acceptable method.

In a recent study to develop a total water management plan for an area of 9,144 square miles in northwestern Ohio, preliminary cost estimates were prepared for the physical separation of combined sewer systems in 48 communities with a total population (1965) of some 812,000 persons. In some instances the combined system may be converted to a separate sanitary system and in others to a separate storm system.

The magnitude of the total project for making this conversion is demonstrated by the $200,000,000 estimated cost to the 48 communities. The City of Bucyrus, Ohio, which was selected as a site for this study was one of the 48 communities included in the northwestern Ohio study. The estimated cost of combined sewer separation in Bucyrus was $5,400,000 or $415 per capita using 1965-66 prices.

The Sandusky River flows through Bucyrus and discharges into Sandusky Bay and Lake Erie. Future water management plans for the principal cities in the watershed are based on utilizing the natural flow in the river and upground storage reservoirs as the major source of water for the area. The reduction in the pollutants discharged into the river thus becomes very important if the desired water quality is to be achieved and maintained for the intended use of the river water.

The need for pollution abatement due to combined sewer overflows is evident. There is need to determine if there are methods of abating pollution from combined sewers which would accomplish the task better than physical separation and at a lesser cost.

SECTION III

PURPOSE AND SCOPE

The study is based on the possibility of interception of all or part of the combined sewer overflow for treatment prior to release to the stream. The advantages, disadvantages and economics of abating pollution from combined sewer overflows by this method will be compared to physical separation of the combined system.

One of the primary objectives of this study is to determine the relationship of rainfall events to overflow events and the volume of flow in the Sandusky River. Once this relationship has been established, the design storm with its resultant overflow rates, volumes and waste characteristics can be selected for the design of the intercepting devices and treatment facilities.

Research of historical records of rainfall and flow in the Sandusky River provided a pattern of rainfall and river flow which could be used to evaluate data collected during this study period. Available data on water quality in the Sandusky River was compiled. Rainfall measuring stations and Sandusky River gaging stations were established to record rainfall and river flows during the study period.

Weirs for measuring overflows during rainfall were installed at the overflow points of three selected sewer districts. Samples were collected during selected overflow events to determine overflow characteristics and effects on the stream.

The results of this data collection are presented and discussed in the following sections of this report. From this data the facilities for the collection and treatment of combined sewer overflows have been sized and cost estimates prepared for comparison with physical separation costs.

SECTION IV

STUDY AREA

The City of Bucyrus, selected as a typical small midwest community with a combined sewer system, is located on the Sandusky River near the upper end of the 1,421 square mile Sandusky River Basin. The river is tributary to Lake Erie as shown in Figure 1. The 90 square mile drainage area upstream from Bucyrus is level agricultural land. Bucyrus is the county seat of Crawford County, and has an incorporated area of about 2,340 acres. The City is located on an end moraine and the topography is generally flat to slightly rolling. The climate is humid with warm summers and mildly cold winters. The mean annual temperature is 51° F and the mean annual precipitation is 36 inches. The Study Area is shown in Figures 1, 2, 3, 4, and 5.

Bucyrus has a tax valuation of approximately $40,000,000 and a 1968 tax rate of 33.90 mills per $1,000. The median income in Crawford County is $9,252 per year per household.

From 1920 through 1950 the population of Bucyrus remained practically constant between 9,700 and 10,400 persons. The 1960 census showed a population of 12,276 persons and the current estimated population is 13,000 persons.

The City is moderately industrialized. Some of the larger industries are Timken Roller Bearing Company, Swan Rubber Company, General Electric Company, Galion Iron Works and Manufacturing Company, Ryder Brass Foundry, Cobey Corporation, Bucyrus Blades, Inc., Crawford Steel Company, Ohio Locomotive and Crane Company, and Bucyrus Ice Company.

The water supply for the municipal waterworks system is obtained from the Sandusky River upstream from the City. Water is pumped from the River and stored in upground storage reservoirs. The water treatment plant is a lime-soda ash softening plant.

The dry weather wastewater in the combined sewer system is intercepted at 24 points along the river and conveyed downstream in an interceptor sewer to the wastewater treatment plant. The plant uses the conventional activated sludge process for treatment of the wastewater. The plant effluent is discharged into the Sandusky River. Most of the sewers are at minimum grade due to the flat terrain.

The Sandusky River downstream from Bucyrus is a source of water supply for the cities of Upper Sandusky, Tiffin, and Fremont. At the present time there are no significant water development facilities on the River other than for these public water supplies. However, six (6) multipurpose upground reservoirs have been proposed by the Ohio Department of Natural Resources to provide water development for the area. One of the purposes of this reservoir system is to provide for a sustained flow of 3.75 MGD (5.7 c.f.s.) in the Sandusky River during a 20-year frequency drought. This would provide for increased public water supply, for improved boating and fishing, and in a few places, for swimming in the Sandusky River. An area near Bucyrus is designated as a site for one of these reservoirs. The total design capacity of the proposed Bucyrus reservoir is 2,054 million gallons, 476 million gallons for sustained flow, 1,400 million gallons for public water supply and 178 million gallons for a conservation pool. The total cost of the reservoir was estimated to be $2,458,000 in 1966.

The principal pollution problems in the Sandusky River are sediment, oxygen consuming materials, bacteria, phosphates and nitrates. The stream drains rich agricultural lands which contribute significant amounts of sediment and nutrients (phosphates and nitrates). The area around Bucyrus is intensely cultivated, the main crops being corn, wheat, and soy beans. Significant oxygen demand and high bacterial concentration occur below Bucyrus due to discharge of treated and untreated sewage. At the present time, all communities discharging sewage effluents to the Sandusky River provide secondary treatment facilities.

SECTION V

STUDY PROCEDURES

The procedures followed in accomplishing this study are discussed in the order in which they were performed.

The overflows from the 24 sewer districts in Bucyrus could not be studied in detail. Therefore, a preliminary analysis of the districts was made to determine which districts were representative. The following items were considered: area, land use, hydraulics of the trunk sewers and the interceptor, accessibility of the overflow points, and the availability of channels to accommodate weirs for measurement of overflow volumes and rates. All overflow points and the wastewater treatment plant were visited both in wet and dry weather.

Three districts were finally selected -- Numbers 8 (179 acres), 17 (452 acres), and 23 (378 acres). (See Figures 2, 3, 4, and 5) These are the three largest districts in Bucyrus, representing 64% of the total sewered drainage area. They include different types of waste discharges in varying proportions.

A detailed analysis was made to determine the drainage characteristics of Numbers 8, 17, and 23 sewer districts. Except for the times of concentration, the characteristics of the remaining districts were estimated by comparing their land use to that of Numbers 8, 17, and 23 sewer districts. A topographical map and a sewer map were used for the analysis. The maps were substantiated by in-field observations.

A hydraulic analysis of the sewer system was made to determine the flow capacities of the trunk sewers, the connectors, and the interceptor. A field survey was made of Numbers 8, 17, and 23 sewer districts and the interceptor. The sewer map was used for the remaining districts. On two occasions, once during dry weather and once during wet weather, the depth of flow in the interceptor was measured at several points and a hydraulic gradient drawn. The Manning equation, with $n = 0.013$, was used to compute the flow in all pipes.

The existing meteorological and hydrological records for Bucyrus were obtained and summarized to establish a base line to which the data measured during the study period could be compared. These records included rainfall, river flow and quality, and water and wastewater treatment plant flows and treatment plant efficiencies. These records were obtained from the U. S. Weather Bureau, the U. S. Geological Survey, the Ohio Health Department, the Federal Water Pollution Control Administration and the City of Bucyrus.

A literature survey was made and a brief summary was compiled of the technical literature available on combined sewers. Special attention was given to the technical knowledge and operating experience of existing facilities similar to units suggested as part of the alternate solutions.

An industrial waste survey was made to determine the type and volume of waste to expect during dry weather and overflow sampling. Only the industries in the three study districts were surveyed. Each industry was visited and in-plant inspections made when necessary.

Two aquatic biology surveys of the Sandusky River were made to determine the type of aquatic life present as an indicator of water quality. The surveys covered the section of the river from ten miles upstream to thirteen miles downstream from Bucyrus. The surveys were performed by Rendell Rhoades, Chairman of the Biology Department, Ashland College, Ashland, Ohio. The first survey was in the fall of 1968 and the second in the spring of 1969.

A system was established to alert personnel in Columbus, Ohio, of approaching rain in Bucyrus. By this system, personnel at the U. S. Weather Bureau at Port Columbus, Columbus, Ohio, upon request, informed project personnel of the probability, the type, and the time of arrival of rainfall in Bucyrus, six to twelve hours in advance. This enabled project personnel to install the necessary equipment and collect initial samples from the sewers or the river prior to the arrival of the rain. Also personnel of the City of Bucyrus notified project personnel when the rain actually occurred in Bucyrus.

A continuous record of the rainfall on the three sewer districts during the study period was obtained. Three rain gages were used, one in each of the three sewer districts. The gages were the weighing type, Bendix Model 775C Universal Recording Rain and Snow Gage. The charts had a 1:1 ratio for rainfall depth and one chart revolution equaled 24 hours. The charts were changed weekly. See Figure 6 for a picture of the rain gage in Number 23 sewer district.

Samples and flow measurements were taken of the dry weather wastewater flow discharged from the three sewer districts, the influent and effluent of the waste water treatment plant, and the Sandusky River at the upstream and downstream gages. The purpose of these samples and flow measurements was to determine the average strength and volume of the waste at different times of the day and at different times of the year.

A weir was installed in each trunk sewer. The weirs were a 90° V-notch weir, a 24 inch rectangular weir, and an 18 inch rectangular weir for Numbers 8, 17, and 23 sewer districts, respectively. (See Figure 7) A sample was collected and the flow measured every 15 minutes for the trunk sewers and hourly for the wastewater treatment plant and river. The samples were composited proportional to flow on an eight-hour shift basis. Both samplings were for 24 hour periods.

A rectangular weir was built at each of the three overflow points to measure overflow during rainfall. (See Figures 7 & 8). The weirs were constructed of one-inch plywood, which was bolted onto steel angles imbedded in concrete. The weir plates were 8, 16, and 10 feet long for Numbers 8, 17, and 23 overflows, respectively.

Water level recorders were mounted in instrument shelters 42 inches behind the weirs. The recorders were Stevens Type F Recorder, Model 68, with a 9.6 time scale and a 1:2 gage scale. All recorders were equipped with automatic starters which would start the clocks at predetermined water levels. The recorders were reset at least once every week. (See Figure 8)

The overflows from many storms were sampled during the study period to determine the quality of the overflow and pollution loads. Only the overflows from Numbers 8, 17, and 23 sewer districts were sampled. From July, 1968, through January, 1969, samples were collected manually. After February 1, 1969, Serco Automatic Samplers, Model NW-3, were installed in the instrument shelters at the overflows. (See Figure 8) These samplers collected a 300 m.l. sample every five minutes for two hours during overflow. If the overflow continued longer than two hours, samples were collected manually at less frequent intervals.

An automatic starter was devised for the samplers that started the clocks when the water level reached a predetermined height behind the weirs. The samplers could therefore be left unattended prior to and during an overflow. The samplers required a vacuum be maintained in the sample bottles. Because the samplers would lose the vacuum after one or two days, they had to be installed within 24 hours prior to the overflow.

A continuous record of the flow in the Sandusky River above and below Bucyrus was obtained for the study period. An existing recording gage operated by the U. S. Geological Survey located at the first bridge below the wastewater treatment plant was utilized for downstream flow measurements. (See Figure 10) Through the cooperation of the U. S. Geological Survey, project personnel had access to the recorder and received copies of the charts when removed. A new gaging station was installed on the River 300 feet upstream from the first overflow point on the combined sewer system. (See Figure 10) A rating curve for the gage was plotted using standard gaging techniques. Flow metering equipment was borrowed from the U. S. Geological Survey. The recorders used at both gaging stations were Stevens Type A35, with 1:6 gage scales. The time scales for the gages were 4.8 and 2.4 inches per day, for the upstream and downstream gages, respectively.

The Sandusky River was sampled above and below Bucyrus during the study period to determine the water quality. The most commonly used sampling points were the upstream gage, the Au Miller Park ford, and the first six bridges below the city. All of the dry weather samples and some of the wet weather samples were collected manually. A separate sample for dissolved oxygen was collected with a dissolved oxygen sampler and the temperature of the water measured.

After February 1, 1969, the samples were collected at the upstream and downstream gages during wet weather by Serco Automatic Samplers. These samplers were the same type as used for the overflows except they were set to collect one sample per hour for 24 hours. The samplers were located on platforms overhanging the water. (See Figure 6) They were installed and activated shortly before the rain started. Additional samples were collected for dissolved oxygen.

The Wastewater Treatment Plant bypass overflow measurement was determined by installing a recorder, float chamber, and instrument shelter at the bypass manhole. (See Figure 9) This recorder measured the water level in the chamber in relation to the invert of the overflow pipe. A rating curve based on the flow characteristics of the overflow pipe was drawn. The recorder used was a Stevens Type F Recorder, with a 1:1 gage scale and a 1.2 time scale. The chart was changed weekly.

Laboratory analyses were performed on all overflow and river samples collected for 18 different physical and chemical tests. The parameters analyzed were (1) biochemical oxygen demand (BOD), (2) chemical oxygen demand (COD), (3) total solids, (4) suspended solids, (5) total volatile solids, (6) volatile suspended solids, (7) total phosphates, (8) nitrate nitrogen, (9) ammonia nitrogen, (10) organic nitrogen, (11) pH, (12) alkalinity, (13) hardness, (14) chlorides, (15) specific conductance, (16) total coliforms, (17) fecal coliforms, and (18) fecal streptococci. All laboratory tests were done in accordance with the "12th edition of Standard Methods for the Examination of Water and Wastewater".[1]

Laboratory analyses of the samples were started immediately upon receiving the samples. Samples that were not completely analyzed the first day were preserved by acid and/or refrigeration. When necessary individual samples collected were composited according to the overflow pattern. The membrane filter technique was used for the total coliform, fecal coliform and fecal streptococci tests.

SECTION VI

DRY WEATHER CONDITIONS

Collection System

Since the entire Bucyrus sewer system is combined, the system capacity is adequate for the normal dry weather wastewater flow. There were no complaints of dry weather odors or backups during the study. In several locations the system has been extended beyond the natural drainage boundaries and the sewers are too shallow to permit gravity drains from basements.

The major trunk sewers in the older sections of the city are of brick construction; the remaining sewers are concrete or vitrified clay pipe. All are laid close to or at minimum grade. Because the larger pipes do not maintain scouring velocities at low flows, solids accumulate in the sewers at many locations in the system.

Interceptor System

The interceptor sewer system consists of control structures in the trunk sewers which divert the dry weather flow through connector pipes to the interceptor sewer. The interceptor generally parallels the river through the city to the wastewater treatment plant. Two types of controls or diversion structures are used. One is a weir built across the combined sewer pipe, the other is a depression in the bottom of the pipe. The size of the connecting pipe regulates the amount of flow diverted.

All of the connector pipes flow by gravity from the bottom of the trunk sewer to the interceptor. Most of the connectors are double or triple the capacity required for the normal dry weather flow. There were no reports of overflows during dry weather due to insufficient connector capacity during the study.

The main interceptor is designed to handle four to five times the dry weather flow. It collects the flow from all 24 sewer districts and discharges it to the wastewater treatment plant wet well and pump station.

The dry weather flows from Numbers 8 and 23 sewer districts were observed overflowing directly into the river several times during the study period. There were reports of other sewer districts also overflowing directly into the river at various times. In each case the connector pipe was plugged with a large object or an accumulation of solids. City personnel reported some connectors require frequent cleaning.

Wastewater Treatment Plant

The Bucyrus wastewater treatment plant was designed as a conventional activated sludge plant. The raw sewage is pumped from the wet well to a comminutor then flows by gravity through an aerated grit chamber, three circular primary settling tanks, three aeration tanks, and two circular final settling tanks and to the Sandusky River. The sludge is anaerobically digested and dewatered on sand beds. Plans are being prepared for post-chlorination of the plant effluent in compliance with orders from the Ohio Water Pollution Control Board.

The daily operational records of the treatment plant for the October, 1968, to September, 1969, study period have been averaged and are summarized below.

Flow	BOD - mg/l			Suspended Solids - mg/l		
MGD	Raw	Settled	Final	Raw	Settled	Final
2.20	119	106	30	128	182	35

Sandusky River

The dry weather water quality condition of the Sandusky River varies with the season and flow. During the winter and spring months the flow in the river is high and the river's condition is fair to good. During the summer and fall months the flow in the river is low and the river's condition deteriorates.

One half of a mile below the upstream gage the water treatment plant discharges lime sludge and wash water into the river. During high flow the sludge is carried downstream with no noticeable affects. During low flow the sludge settles out on the bottom of the river. (See Figure II) Neither fish nor aquatic insects can survive under these conditions. The sludge affects the river for approximately a distance of one mile, or to the U. S. 30 bridge. Plans are now being prepared for lagooning the lime sludge and wash water which will terminate their discharge to the River.

The wastewater treatment plant effluent has the greatest affect on the water quality of the river during dry weather. During the months of August, September, and October, the flow in the river is too low to assimilate all of the BOD in the treatment plant effluent. Dissolved oxygen levels are consistantly below 4.0 mg/l for a distance of five miles downstream from the treatment plant effluent outfall and frequently reach 0 mg/l during the night. Sludge banks are formed on the river bottom for a distance of three miles downstream.

SECTION VII

WET WEATHER CONDITIONS

Collection System

The Bucyrus sewer system has been extended beyond its original design capacity. In at least one area the sewers are extended over the natural drainage divide. Many of the sewers have capacities which are inadequate for an average one-year storm. Therefore, there is street ponding at several locations during the higher intensity rainfalls.

Except for the temporary inconvenience, the street ponding does not cause any problems. A few corrections and additions to the system would make it adequate for the present population. However, the surcharged sewers do cause backups in many homes through the basement drains. There were several reports of basements flooded with a mixture storm water and sewage during high intensity storms.

Interceptor System

The interceptor sewer has the same capacity as the sum of the connector pipes flowing into it at every point in the system. The total pipe capacity at the lower end of the system is 19 c.f.s., or four to five times the average dry weather wastewater flow.

During a storm, however, the interceptor has a capacity only two times the average dry weather wastewater flow. The capacity is restricted by the limited rates pumped to the wastewater treatment plant and the resultant overflow level at the bypass manhole, all of which reduced the hydraulic gradient of the sewer.

Two factors control the diversion capacity of the interceptor during wet weather. One is the capacity of the connector pipes, and the other is the water level in the interceptor. From field measurements made during an overflow, the hydraulic gradient for the flow in the interceptor was found to be high enough to affect the connector pipe capacities of many of the sewer districts. The flow in Number 5 connector pipe was reversed, with the interceptor overflowing through the .sewer district overflow. Very little overflow occurs at the wastewater treatment plant in comparison to the volumes overflowing at the 24 individual sewer districts.

The water level in the interceptor during an overflow has contributed to one problem. Following the overflow the interceptor remains full for several days because of the limited rate pumped to the wastewater treatment plant. The low velocities in the trunk sewers, connector pipes and the interceptor sewer allow the solids to settle out.

The controls and overflows function as designed. The depression type of control appears to function better than the weir type of control during wet weather. The weir type control blocks part of the trunk sewer and in some cases reduces its capacity by one-half. All of the overflows are part of the original trunk sewer, and most of them are equipped with flap gates to prevent back flow from the river during high water.

Wastewater Treatment Plant

During wet weather the sluice gate to the wet well at the wastewater treatment plant must be closed to limit the amount of flow into the plant. Experience has shown that when the gate is closed three-fourths of the way, the flow to the plant will be between 3.0 and 3.5 MGD. This is the maximum capacity of the plant.

During wet weather the BOD of the waste coming into the plant decreases. The increase in flow rate decreases the settling time and efficiency and reduces the suspended solids content in the aeration tanks. The quality of the effluent remains approximately the same, but the efficiency of treatment decreases.

Sandusky River

All of the overflows discharge directly into the river. The sum of the overflows give a very distinct hydrograph at the downstream gage. If the rainfall is a localized thunderstorm, the river returns to its previous flow. If the rainfall is more generalized, the overflow hydrograph will be followed 16 to 40 hours later by the hydrograph of the runoff from the upstream drainage basin.

The water quality of the river during wet weather varies with the season and flow. The more flow in the river, the more dilution water is available for the overflows. Therefore, the condition of the river will be the poorest following an overflow during the summer and early fall months.

The overflow wastes have several effects on the river. The most obvious one is the debris and organic solids which settle in the river in and below the city. These create odors and unsightly conditions long after the overflow is past. A second effect is the decrease in quality of the water moving downstream. BOD's, solids, coliform, etc., are increased. In turn, dissolved oxygen is decreased. The aquatic life is affected. Therefore, the usefulness of the water for recreation and water supply is impaired.

SECTION VIII

METEOROLOGICAL AND HYDROLOGICAL HISTORY

The past meteorological and hydrological records for the Bucyrus area and the Sandusky River have been reviewed and summarized to provide base line data.

Meteorological History

Daily, monthly and annual rainfall data for Bucyrus were obtained from the periodical "Climatological Data" published by the Weather Bureau of the U. S. Department of Commerce. These records date back to 1931. The average monthly rainfall records since 1931 have been summarized and are shown in Table I. The average annual rainfall for the Bucyrus area is 35.7 inches.

In addition to summarizing the meteorological records since 1931, the records for the ten-year period from January 1959 to September 1968 were studied in more detail and used for comparison with the weather conditions occurring during the study period. The average monthly rainfall for the study period is shown in Table I and also graphed in Figure 13. The average annual rainfall for the ten-year period is 32.8 inches.

For the ten-year period studied in detail, July had the highest average monthly rainfall with 3.88 inches and October the lowest with 1.96 inches. The wettest month during this ten-year period was July, 1966, when 9.29 inches of rain fell while the driest month was October, 1963, when only a trace fell.

Detailed study of the ten-year period indicates that on the average there are 75 days per year when the rainfall is greater than 0.10 inches, 22 days per year when rainfall is greater than 0.50 inches and six days per year when rainfall is greater than 1.00 inches. Also, July is the wettest month in terms of amount and intensity of rainfall.

The weather bureau station at Bucyrus reports only daily rainfall totals. Due to the lack of hourly rainfall data at this station, intensity-duration information could not be developed. Therefore, the "Rainfall Frequency Atlas of the United States", Technical Paper No. 40, published by the U. S. Department of Agriculture, was used to develop rainfall-duration-frequency relationships for the Bucyrus area. Figure 14, entitled "Rainfall Depth - Duration - Frequency Curves" and Figure 15, entitled "Intensity - Duration Curves" were derived from the above mentioned source.

Hydrological History

Data on flow rates in the Sandusky River were obtained from the U. S. Department of Interior Geological Survey publication titled "Surface

Water Records of Ohio" and the Ohio Department of Natural Resources Printed Bulletin No. 37, 40, and 42. The Geological Survey records cover the periods August, 1925, to November, 1935; July, 1938 to December, 1951; and December, 1963, to September, 1966. The Natural Resources summary is based on gaging records through 1965.

The average flow in the Sandusky River at Bucyrus is 80.2 cfs for the 26 years of records. On a yearly bais, the minimum average flow of 33.8 cfs occurred in 1937 while a maximum average flow of 145 cfs occurred in 1959.

The average daily flows of the Sandusky River at Bucyrus are graphed in Figure 12 for each month. The contrast between the low daily flows in July to November and the high daily flows in December to June is evident from this graph. The months in order of decreasing average daily flow are: (1) March - 202 cfs, (2) February - 151 cfs, (3) April - 150 cfs, (4) January - 127 cfs, (5) May - 86 cfs, (6) June - 82.5 cfs, (7) December - 78 cfs, (8) November - 36.5 cfs, (9) July 26.5 cfs, (10) August - 18.8 cfs, (11) October - 12.2 cfs, and (12) September - 9.6 cfs.

The maximum daily flow ever observed was 4,600 cfs on December 14, 1927, and the minimum of 0.6 cfs on September 29, 1941, and September 25, 1946. The ten-year, seven-day duration low flow average discharge is 0.70 cfs.

Figure 13 compares average daily flows for each month with average rainfall for each month. Monthly flow variation is shown to be much greater than monthly rainfall variation. Maximum rainfall is less than twice the minimum rainfall, while the ratio of maximum to minimum river flow is over 20 to 1.

Only a little over 1 percent of the total annual discharge of the Sandusky River at Bucyrus occurs during the month of September. Five percent of the total annual discharge occurs during the three month period - August to October. About 12 percent of the total annual discharge occurs during the five month period - June to November. Over 50 percent of the total annual discharge occurs during the three month period - February to April. The minimum average monthly discharge, the minimum average daily discharge and the minimum observed discharge all occurred in the month of September.

The flow duration probability values for the Sandusky River at Bucyrus are presented in Table 2, entitled "Percent of Time Indicated Sandusky River Flow at Bucyrus is Equaled or Exceeded". This table shows that the median river flow or the flow that is equaled or exceeded 50 percent of the time is 16.8 cfs. The median flow of 16.8 cfs is extremely lower than the average flow of 80.2 cfs.

TABLE I

AVERAGE MONTHLY RAINFALL AT BUCYRUS, OHIO

	Inches of Rainfall	
	1931 - 1968	1959 - 1968
January	2.88	2.52
February	2.34	2.12
March	3.15	2.34
April	3.18	3.55
May	3.35	3.37
June	4.50	3.09
July	3.22	3.88
August	3.23	2.28
September	2.73	2.52
October	2.40	1.96
November	2.45	3.10
December	2.24	2.11
Average Annual Rainfall	35.67	32.84

TABLE 2

PERCENT OF TIME INDICATED SANDUSKY RIVER FLOW AT BUCYRUS IS EQUALED OR EXCEEDED

Percent	Flow (cfs)
5	350
10	170
15	108
20	79.0
25	59.5
30	45.0
40	27.0
50	16.8
60	10.1
70	6.10
75	4.75
80	3.80
85	3.05
90	2.43
95	1.80

SECTION IX

WEATHER CONDITIONS DURING STUDY PERIOD

Project personnel made 16 wet weather trips to Bucyrus during the study period July, 1968, to September, 1969, to collect samples of predicted overflows. There were 10 days out of the 16 that overflows actually occurred and were sampled.

Grab samples were collected manually during 5 overflow events that occurred prior to February 8, 1969. Samples of the remaining 5 overflow events were collected by automatic samplers and project personnel.

Rainfall Data for Study Period

The total rainfall per month that occurred during the study period is shown in Table 3. These monthly rainfall totals have been compared to the past monthly averages and are presented in Table 3 as percentages of average rainfall.

The period November, 1968, through January, 1969, was wet, while February and March, 1969, were dry. April, 1969, marked the start of an unusually wet four-month period. The rainfall during these four months averaged 162 percent above normal.

Sandusky River Flow During Study Period

A summary of the Sandusky River flow during the study period is presented in Table 4. This table shows the average, minimum, and maximum river flow for each month of the study period. Also included in the table is a comparison of the average flow during the study period with the historical average. December, 1968 and January, April, May, and August, 1969, were extremely wet months ranging from 150 percent to 288 percent above normal flow.

TABLE 3

RAINFALL DURING STUDY PERIOD

Date	Inches of Rainfall	Percentage Average Rainfall
July 1968	3.54	110
August	1.97	61
September	3.11	114
October	0.90	38
November	1.45	159
December	3.14	140
January, 1969	3.20	111
February	0.94	40
March	1.33	42
April	5.49	173
May	4.91	147
June	5.77	128
July	6.36	198

TABLE 4

SANDUSKY RIVER FLOW DURING STUDY PERIOD

Date	Average cfs	Minimum cfs	Maximum cfs	Percent of Past Average
July, 1968	27	3	330	100
August	9	3	95	47
September	9	2	310	90
October	6	2	36	50
November	33	2	350	91
December	146	6	1,800	187
January, 1969	197	16	1,650	150
February	106	24	700	70
March	62	10	400	66
April	272	44	1,850	182
May	248	19	2,800	288
June	50	12	275	62
July	60	6	435	222

SECTION X

DRAINAGE CHARACTERISTICS OF THE SEWER DISTRICTS

History

The Bucyrus sewer system started as a combined system. Because the city was built adjacent to the Sandusky River, drainage was not a problem. The natural drainage system was converted to a combined sewer system which discharged both storm and sanitary wastes directly into the river. This led to the development of numerous small sewer districts along each side of the river, each district with its own separate overflow.

In 1935, the wastewater interceptor sewer was built along the Sandusky River. Through control structures and diversion pipes, the sanitary wastewater flow was collected during dry weather and discharged into the river at a point west of the city. Later this flow was diverted from the river to a wastewater treatment plant, thus completing the system. The system is the same now as in 1935, with the exception of improvements to the wastewater treatment plant, which were constructed in 1961.

General Description

Bucyrus is located near the drainage divide between the Lake Erie and the Ohio River Drainage Basins, as shown in Figure 1. The change in elevation from one end of the city to the other is less than 20 feet, except in the immediate vicinity of the river. The topography resembles a plateau with the river and its flood plain winding through it. Most of the building and development is on the plateau area. Most of the sewers are laid at minimum grade, until they reach the edge of the flood plain.

The existing sewer system is composed of 24 separate sewer districts as shown in Figure 2. The size of the districts varies from 2.5 to 452 acres. The average size is 65 acres, the median size is 20 acres. All of the drainage districts border on the river at some point, with the exception of Number 24. The trunk sewer to district Number 24 was installed after the interceptor system was constructed and serves the extreme northwest part of the city.

The sewers in all of the existing sewer districts are extended to the natural drainage divide or adjacent areas with the exception of Numbers 1, 4, 17, and 24. Additional wastewater or storm water could not be placed into the systems without pumping. Numbers 1, 4, and 17 districts cover the extreme east side of Bucyrus. Portions of these districts contain unsewered areas which are occupied by the fairgrounds and farmland. The development of this land will require adequate means of drainage.

Detailed Description

A detailed analysis was made of the drainage characteristics of Numbers 8, 17, and 23 sewer districts. This was done to relate the three drainage districts to each other and to the drainage districts in any other city. During this analysis the sanitary drainage area, the storm drainage area, the number and type of waste contributors, the land use, the land cover, the area slope and the area shape were determined for each of the three districts. The storm drainage areas and land use classifications were also determined for the other 21 sewer districts. This analysis is summarized in Tables 5, 6, and 7.

The three sources of information for this analysis were topographical maps of Bucyrus and the surrounding area, a sewer map of Bucyrus, and field observations. The topographical maps were prepared from aerial photographs and were completed in the fall of 1968. They have a horizontal scale of 1" = 200 feet and two-foot contour intervals. All streets and buildings were shown, as well as other topographical features. The City of Bucyrus sewer map has a horizontal scale of 1" = 300 feet and shows the locations of sewers and manholes, inverts of manholes, and the size and grade of the sewers. This map was field checked where necessary.

The following procedure was used for the detailed analysis. First, the sewer systems were redrawn onto the topographical maps. The topographical maps were then taken to the field and a complete survey made of the three sewer districts. Every street in the sewer districts was inspected. Each property was labeled as residential, commercial, industrial, institutional, undeveloped, or railroad. All non-residential property and residential property with unusual land cover or area were further classified as to the type of land cover. This was done by sketching in the boundaries of these various areas and labeling them. Many manhole covers were lifted to determine the accuracy of the sewer map. Field checks were made around the boundaries of the drainage basins to determine which areas actually drained into their sewer systems.

Following are the definitions of terms used for land use in this study:

residential - any family dwelling unit - one for each one or two family unit and one for each family in multi-family units.

commercial - all places of business excepting those whose major business is the manufacture of a product to be sold elsewhere - one for each business occupying a ground-level storefront.

industrial	- any place whose major business is the manufacture of a product to be sold elsewhere - one for each name regardless of number of separate properties occupied.
institutional	- all schools and churches
undeveloped	- without permanent improvements
railroad	- track area not owned by private enterprise

The remainder of the detailed analysis of the sewer districts was completed in the office. First the boundaries of the sanitary and storm drainage districts were determined. The boundaries were then planimetered to determine their areas. The number of each type of property in each district was counted. The area of each type of land cover and the total area were then measured.

More than half of the storm drainage area of each sewer district is normal residential property. This property was not measured directly. The sums of the areas of the other types of property were subtracted from the total areas to determine the area occupied by normal residential property. These residential areas were then divided by the number of normal residences in each sewer district to determine an average lot size. Spot checks were then made to determine the average lot land cover. (See Table 6)

The three sewer districts were classified according to their land use. All three districts contain residential areas. Number 23 sewer district is classified as suburban residential because of the low density of houses per unit of area. Based on these classifications, the remaining 21 sewer districts were also classified. The remaining districts were compared to Numbers 8, 17, and 23 districts by comparing their land use on the large scale topographical maps. These classifications and the drainage areas of all the districts are given in Table 7.

The average slopes of the three drainage districts were determined by dividing each district into smaller drainage areas. The average slope of each of these smaller areas was determined by measuring the fall from the remotest drainage point to the sewer outlet. These values were then weighted on the basis of their areas and an average slope computed. (See Table 5)

The final part of the analysis of the sewer districts was the determination of the shapes of the three study areas. This was done by approximating the drainage areas with regular polygons. Maximum widths and lengths were measured. (See Table 5)

TABLE 5

GENERAL DRAINAGE CHARACTERISTICS OF SELECTED SEWER DISTRICTS

	Sewer Districts		
Number of Customers	No. 8	No. 17	No. 23
Residential	577	1,228	561
Commercial	14	173	23
Industrial	4	1	5
Institutional	3	15	1
Total	598	1,417	590
Population*			
Population	2,020	4,300	1,960
Persons / Acre	11.7	9.1	5.0
Drainage Basin Slope			
Weighted Average - %	0.85	0.65	0.25
Drainage Basin Shape			
Maximum Length - feet	3,600	8,600	8,000
Maximum Width - feet	5,000	4,500	3,600
Ratio Length to Width	0.7	1.9	2.2

* Based on 3.5 people per residence

TABLE 6

LAND USE AND LAND COVER OF SELECTED SEWER DISTRICTS

	Sewer Districts		
Land Use - % of Total Area	No. 8	No. 17	No. 23
Residential	59.6	55.1	53.4
Commercial	6.3	11.5	4.8
Industrial	7.8	7.2	17.6
Institutional	4.6	2.3	0.7
Undeveloped	12.9	11.1	15.4
Railroad	0.2	3.8	0.2
Streets	8.6	9.0	7.9
Total	100.0	100.0	100.0
Land Cover - % of Total Area			
Impervious			
Buildings	14.7	14.1	11.2
Asphalt & Concrete	10.5	10.6	6.4
Streets	8.5	9.0	7.9
Water	0	0	0.6
Total	33.7	33.7	26.1
Pervious			
Weeds	20.4	18.5	17.8
Lawn	39.8	35.3	49.9
Packed Earth	0.8	0.9	0.3
Gravel	0.8	10.1	5.9
Cornfield	4.5	1.5	0
Total	66.3	66.3	73.9
Normal Residential Lot			
Lot Area (sq. ft.)	8,400	9,000	16,000
Lot Dimensions (ft.)	60 x 140	60 x 150	90 x 178
House and Garage Area (sq. ft.)	1,400	1,500	1,900
Asphalt & Concrete Area (sq. ft.)	350	975	850
Lawn Area (sq. ft.)	5,000	5,450	12,300
Gravel Area (sq. ft.)	0	225	50
Weeds & Garden Area (sq. ft.)	1,650	850	900

TABLE 7

DRAINAGE AREAS AND CLASSIFICATIONS

District No.	Sanitary Drainage Area - Acres	Storm Drainage Area - Acres			Classification of Sewered Area*
		Sewered	Non-Sewered	Total	
1		73.3		73.3	SD
2		2.5		2.5	R
3		19.4		19.4	R
4		113	82	195	R
5		32.1		32.1	50% R, 50% C
6		21.0		21.0	R
7		3.2		3.2	50% R, 50% C
8	188	179		179	R
9		3.0		3.0	R
10		7.1		7.1	50% R, 50% C
11		6.2		6.2	C
12		41.9		41.9	R
13		8.8		8.8	75% R, 25% C
14		70.2		70.2	R
15		10.8		10.8	75% R, 25% C
16		5.0		5.0	50% R, 50%
17	475	452	162	614	R
18		5.7		5.7	SR
19		24.5		24.5	R
20		12.1		12.1	SR
21		7.8		7.8	SR
22		72.4		72.4	SR
23	395	378		378	SR
24		20.7		20.7	SD
Totals		1,570	244	1,814	

*Symbols:
SD - Semi-developed
SR - Suburban Residential
R - Residential
C - Commercial

SECTION XI

HYDRAULIC ANALYSIS OF THE SEWER AND INTERCEPTOR SYSTEMS

Sewer Systems

The hydraulic analysis of the sewer systems consisted of two parts: (1) determining the maximum sewer capacities, and (2) determining the times of concentration. The Manning equation with n = 0.013 was used to compute the flow and velocity in the pipes.

Because of the steady growth of the city many of the sewers have been extended beyond the area for which they were originally designed. In some cases, the systems have been extended beyond the drainage divide. The result has been street ponding and basement flooding during wet weather. Number 23 sewer district is an example. In the southern part of this district, the sewer system has actually been extended beyond the drainage divide between the Sandusky River and the Little Scioto River. In this area the combined sewer is only three feet deep. Because of the flatness of the sewer grades and the small size of the pipes, the City has limited the number of street inlets which results in ponding rather than surcharging the sewer system. However, there are frequent reports of basement flooding in the area.

The maximum capacity of each of the 24 sewer systems was computed. (See Table 8) The main trunk sewer was the controlling capacity for most of the sewer systems. However, in a few systems the flow is limited by the capacity of the sewers discharging into the main trunk sewer. Number 23 sewer district is an example.

The times of concentration for all 24 sewer districts were determined. The time of concentration consists of two parts: (1) the time of overland flow, and (2) the time of concentration in the sewer. The time of overland flow depends on the length of travel from the most remote area in the drainage district to the nearest storm sewer inlet, the type of ground cover, and the slope of the land. These values were measured for Numbers 8, 17, and 23 sewer districts. The times of overland flow for these three districts were computed using the formulas given under the hydrograph method in the ASCE Sewer Design Manual.[2] A maximum of 30 minutes was assumed. Assuming that these values were typical for the city, they were then applied to the other 21 sewer districts. The time of overland flow was also determined for the impervious areas.

The time of concentration in the sewer is equal to the travel time from the most remote inlet in the system to the point where the last inlet lateral joins the main trunk sewer. These times were computed from the velocities in the pipes, assuming that the pipes were flowing full.

The summary of the values obtained for time of overland flow, time of concentration in sewer, and total time of concentration is given in Table 8. Also given is the total travel time for each district. This value was determined by adding the time of concentration to the travel time in the sewer from the last inlet lateral to the overflow.

Interceptor System

The existing interceptor system functions as designed. There were no reports of overflows during dry weather except when a connector pipe became plugged. An analysis of the hydraulics of the interceptor was necessary to determine its capacity and function during wet weather.

During the dry weather sampling on October 9 and 10, 1968, a check was made on the connector capacity of the three districts studied. The districts have a connector capacity equal to 0.9 cfs for Number 8 district, 4.2 cfs for Number 17 district, and 3.4 cfs for Number 23 district. These compare with a maximum dry weather flow, measured on March 5, 1969, of 0.5 cfs for Number 8 district, 1.0 cfs for Number 17 district, and 1.4 cfs for Number 23 district. Therefore, the connector capacities for these three districts are more than twice the maximum dry weather flows.

The existing interceptor pipe is designed to handle a flow of 19 cfs at the lower end of the system. At each point in the system, the interceptor pipe capacity is equal to the sum of the connector pipe capacities. The wastewater treatment plant records show that the interceptor has a maximum daily flow in the spring of 4.5 cfs and an average daily flow of 3.5 cfs. Therefore, the interceptor has a capacity equal to four times the maximum hourly flow and five and one-half times the average daily flow.

An overflow structure is located on the interceptor near the wastewater treatment plant. During wet weather, flow to the plant is controlled at about 3 MGD. Flows in the interceptor which exceed 3 MGD are diverted to the river. The overflow is a 30" corrugated metal pipe with the invert approximately 5.5 feet above the invert of the interceptor. This overflow is located near the junction manhole for the northwest trunk sewer. There are two 24" overflows on the northwest trunk sewer at an elevation only 0.8 feet above the wastewater treatment plant overflow. These also act as overflows for the interceptor during wet weather.

The control structures for diverting the dry weather flow consist of two different types. One type is a simple rectangular weir built across the trunk sewer pipe, and is usually made of bricks. The connector pipe to the interceptor is cut into the wall of the trunk sewer pipe. When the connector pipe is surcharged to the height of the weir, overflow to the river will occur.

The other type of control structure consists of a depression in the bottom of the sewer. The connector pipe is cut into the bottom of the depression, makes a right angle bend, and continues to the interceptor. When the connector pipe is surcharged to the level of the bottom of the sewer, overflow will occur. Most of the overflow structures are equipped with flap gates at the mouth of the overflow to prevent the river from flowing into the pipe during high river stage. Numbers 8 and 17 overflows have the weir type control; Number 23 overflow has two of the depression type controls.

Both types of control structures have proven successful for diverting the dry weather flow. However, the weir type control structure is a restriction in the trunk sewer pipe during high flows during an overflow event. Most of the trunk sewers were designed to carry only the flow from the upstream drainage system. The weir, in many cases, covers one-half the area of the trunk sewer pipe. The effects of this were seen during this study. For example, every time the trunk sewer pipe at Number 17 overflow is flowing full, the cover on the manhole behind the control structure is blown off. See Table 9 for a comparison of the capacities of the trunk sewers for Numbers 8, 17, and 23 sewer districts with the control structure, without the control structure, and the maximum flows observed from the overflows during the past year.

TABLE 8

MAXIMUM SEWER SYSTEM CAPACITIES
AND TIMES OF CONCENTRATION

District No.	Maximum Sewer Capacity cfs	Time of Overland Flow Min.	Time of Concentration in Sewer Min.	Time of Concentration Min.	Total Travel Time Min.
1	23	20	14	34	36
2	4	20	1	21	21
3	5	20	7	27	33
4	42	20	15	35	37
5	25	20	9	29	29
6	4	20	7	27	27
7	1	20	1	21	21
8	105	30	24	54	54
9	5	20	2	22	22
10	5	20	4	24	24
11	61	10	4	14	14
12	7	20	6	26	26
13	3	20	3	23	23
14	29	20	16	36	37
15	39	20	5	25	25
16	3	20	4	24	24
17	140	30	34	64	67
18	5	20	4	24	24
19	6	20	18	38	41
20	5	20	6	26	27
21	4	20	7	27	28
22	12	20	12	32	36
23	65	21	39	60	63
24	8			30	
For Impervious Areas Only:					
8		6	24	30	30
17		4	34	38	41
23		4	39	43	46

TABLE 9

MAXIMUM OVERFLOW RATES

Sewer District No.	Maximum Overflow Rate - cfs		
	Existing System	Control Structure Removed	Measured by Weirs
8	50	105	50.8[1]
17	75	140	71.0[2]
23	65	65	71.1[3]

[1] Measured on July 11, 1969, for a 25-year storm.

[2] Measured on April 5, 1969, for less than a 1-year storm.

[3] Measured on July 11, 1969, for a 25-year storm. Also measured 69.3 cfs on May 17, 1969, for less than 1-year storm.

SECTION XII

ANALYSIS OF RAINFALL AND OVERFLOW DATA

Tabulation of Hydraulic Data

The rainfall and overflow data recorded on the charts was transposed into tabular form for study and analysis. The rain gages recorded the rainfall as a mass curve with the ordinate as total inches of rainfall and the abscissa as time. The smallest chart division of time was 20 minutes. The charts were further sub-divided into five or ten-minute increments for low or moderate intensity rainfalls. For high intensity rainfalls the charts were read from inflection point to inflection point. Estimated accuracy of time readings are $\pm$ 2 minutes, and accuracy of setting the clocks to the correct time is $\pm$ 5 minutes. The vertical total rainfall scale was graduated to the nearest 0.05 of an inch, and could be read to the nearest .01 of an inch.

Every rainfall that had correspondingly good overflow records was tabulated. This included the clock time, the number of minutes in the time interval, the accumulated rainfall as recorded on the charts, the rainfall during that interval, and the average intensity of the rainfall during that interval. When no overflow record was obtained, only the total amount of rainfall was recorded.

The charts from the flow level recorders recorded depth of overflow above the weir plates versus time. The depth on the vertical scale was graduated to the nearest 0.02 of a foot, and could be read to the nearest 0.01 of a foot. The time scale was graduated to the nearest 15 minutes, and could be read to the nearest 2 mintues. When there were rapid fluctuations in flow, the charts were further sub-divided into five-minute intervals. The accuracy of setting 0.00 on the recorder chart to the top of the weir plates was $\pm$ .01 of a foot. The accuracy of setting the time on the charts to correspond with the correct time was $\pm$ 2 minutes.

The flow records were also recorded in a tabular form. This included clock time, minutes in time interval, depth of overflow above weir, overflow rate per foot of weir, total overflow rate, average rate of overflow for time interval, overflow volume for time interval, and accumulated overflow volume for overflow event. Since the flow level recorders recorded height above weir rather than flow, these had to be converted to flow by use of tables for rectangular weirs.

Rainfall versus Overflow Graphs

The next step of the data analysis was plotting the rainfall and overflow data for each overflow event recorded. Both rainfall and runoff were plotted on the same graph, with rainfall plotted above overflow on the same time scale. The rainfalls were plotted in the form of hyetographs, the overflows in the form of hydrographs. The time scale remained constant for all graphs and the vertical scale varied according to the peak rainfall intensity and the peak overflow rate. The rainfall and overflow graphs for March 24 and June 13, 1969, are given as examples in Figures 16 through 21.

Each overflow hydrograph was divided into ten-minute segments. The rate of flow at the mid-point of each ten-minute interval was read and assumed to be the average for that interval. These values were summed to determine the total volume of overflow for the entire overflow event. The volume was then compared to the volume computed previously in the tabulation of the raw data to check its accuracy. A summary of the rainfall and overflow data was prepared for each of the three overflows for each overflow event. (See Tables 10, 11, and 12)

Analysis of Rainfall Data

The rainfall data corresponding to the measured overflows were also plotted as intensity-duration curves (See Figure 22) These rainfalls represent a variety of different types of storms. May 7 and June 13 were short, intense thunderstorms. April 5 and May 17 were long duration rainfalls, with short periods of intense rain. The majority of the storms, such as February 8 and March 24, were of light to medium intensity and lasted between 2 and 12 hours.

None of the storms measured between February 8 and June 13, 1969, surpassed a one-year storm over their entire length. Two of the storms did exceed a one-year storm at some point. May 17 exceeded a one-year storm during the maximum 12 minutes, and June 13 exceeded a one-year storm during the maximum 38 to 80 minutes.

TABLE 10

RAINFALL AND OVERFLOW DATA
NO. 8 SEWER DISTRICT

		Rainfall			Overflow				
Date	No.	Total Depth In.	Avg. Intensity In./hr.	Duration Min.	Total Vol. cu.ft.	Inches on Basin	Peak cfs	Duration Min.	% Overflow
2/8					No Record				
3/24	1	0.25	0.14	110	22,400	0.034	3.5	150	14
	2	.14	.09	90	20,300	.031	4.6	180	22
	3	.17	.15	70	29,300	.045	7.5	160	26
4/5	1	.10	.60	10	10,000	.015	5.7	70	15
	2	.47	1.68	17	61,300	.094	22.8	105	20
	3	.30	.60	30	31,400	.048	11.0	95	16
	2,3	.77	.80	57	92,700	.141	22.8	125	18
28	1	.03	.09	20	No Record				
	2	.14	.28	31	11,300	.017	4.8	115	12
5/7	1	.16	.32	30	6,800	.011	3.0	107	7
	2	.04	.06	40	2,300	.004	1.1	65	10
8	1	.09	.23	23	No Record				
	2	.12	.05	140	No Record				
17	1	.27	.23	70	No Record			220	
	2	.40	.57	42	32,000	.049	19.4	185	12
	3	1.18	.51	138	172,000	.264	32.4	325	22
6/13	1	.39	1.56	15	35,200	.054	19.0	100	14
	2	.81	.97	50	96,900	.149	27.2	175	18
	1,2	1.20	.90	80	132,100	.203	29.5	200	17
8/9	1	.13	.16	55	6,400	.010	2.7	100	8
	2	.06	.10	35	3,100	.005	1.4	60	8
	1,2	.19	-	-	9,500	.015			8
	3	.50	.40	75	54,000	.083	22.7	140	17

TABLE II

RAINFALL AND OVERFLOW DATA
NO. 17 SEWER DISTRICT

		Rainfall			Overflow				
Date	No.	Total Depth In.	Avg. Intensity In./hr.	Duration Min.	Total Vol. cu.ft.	Inches on Basin	Peak cfs	Duration Min.	% Overflow
2/8		0.17	0.09	120	37,000	0.023	12.1	105	14
3/24	1	.25	.14	110	43,200	.026	11.0	135	10
	2	.14	.09	90	48,400	.030	13.5	135	21
	3	.17	.15	70	53,800	.033	12.0	145	19
4/5	1	.09	.54	10	14,000	.009	14.6	80	10
	2	.42	.93	27	142,000	.087	71.0	95	20
	3	.26	.52	30	102,000	.062	40.5	100	24
	2,3	.68	.72	57	244,000	.149	71.0	135	22
28	1	.03	.09	20	0	0			0
	2	.14	.28	31	34,200	.021	18.9	85	15
5/7	1	.16	.32	30	18,100	.011	24.5	60	7
	2	.04	.06	40	2,900	.002	2.2	40	5
8	1	.09	.23	23	41,500	.025	29.2	80	28
	2	.12	.05	140	12,100	.007	2.7	102	6
17	1	.27	.23	70	84,000	.050	28.0	135	19
	2	.44	.29	92	236,000	.144	66.5	415	33
	3	1.16	.65	108	655,000	.400	64.0	265	34
6/13	1	.37	2.77	8		Trace			0
	2	.83	.91	55	158,000	.096	63.2	190	12
	1,2	1.20	.90	80	158,000	.096	63.2	190	8
8/9	1	-	-	-	0	0			0
	2	-	-	-	0	0			0
	1,2	.30	-	-	0	0			0
	3	.56	No Record		148,000	.090	50.1	110	16

TABLE 12

RAINFALL AND OVERFLOW DATA
NO. 23 SEWER DISTRICT

Date	No.	Rainfall Total Depth In.	Rainfall Avg. Intensity In./hr.	Rainfall Duration Min.	Overflow Total Vol. cu.ft.	Overflow Inches on Basin	Overflow Peak cfs	Overflow Duration Min.	Overflow % Overflow
2/8		0.24	.10	150	35,300	0.026	8.5	165	11
3/24	1	.26	.14	115	23,100	.017	5.2	135	7
	2	.16	.09	110	24,600	.018	6.0	140	11
	3	.15	.13	70	30,600	.022	7.8	150	15
4/5	1	.10	.60	10	16,800	.012	8.1	65	12
	2	.27	.54	30	91,800	.067	32.3	140	25
	3	.17	.41	25	62,700	.046	20.0	135	27
	2,3	.44	.44	60	154,500	.113	32.3	175	26
28	1	.04	.12	20	1,700	.001	1.2	45	3
	2	.12	.29	25	24,400	.018	10.1	110	15
5/7	1	.20	.55	22	30,000	.025	13.2	110	13
	2	.05	.15	20	8,900	.007	4.4	75	14
8	1	.11	1.32	5	34,800	.025	15.1	105	23
	2	.11	.05	130	13,300	.010	2.3	170	9
17	1	.22	.22	60	No Record				
	2	.39	.59	40	No Record				
	3	1.06	.62	102	534,000	.390	69.3	265	37
6/13	1	.18	2.70	4	19,000	.014	11.0	67	8
	2	.72	.85	51	171,000	.125	59.7	160	17
	1,2	.90	.72	75	190,000	.139	61.2	177	15
8/9	1	No Record			7,700	.006	3.7	95	
	2	.04	.03	80	7,100	.005	1.7	145	13
	1,2	-	-	-	14,800	.011			
	3	.36	.31	70	96,200	.070	35.3	170	19

SECTION XIII

WASTEWATER CHARACTERISTICS OF COMBINED SEWER OVERFLOWS AND RECEIVING STREAM

The sampling program for the collection of overflow and river samples began in July, 1968, and continued through August, 1969. From July, 1968, through January, 1969, flows were estimated and samples were taken manually to determine the concentration range of water quality characteristics. The weirs at the three selected sewer districts were completed in January, 1969, and after that date the overflows were automatically measured and sampled. Table 13 presents the frequency and duration of the sampling program and shows when and where samples were collected and measured. There were five days - February 8, March 24, May 7, June 13 and August 9 - when overflow events were both measured and sampled.

The results of the laboratory analyses of all samples collected during the study period have been summarized into a number of tables and graphs, each of which will be discussed in this section of the report.

Dry Weather Sampling

Dry weather flow measuring and sampling to provide base data on waste loads and flow was done on October 9, 1968, and March 4, 1969. The dry weather sewage flows in sewer districts Numbers 8, 17, and 23 were measured and sampled at 15-minute intervals for 24 hours. The Sandusky River, upstream and downstream, was measured and sampled at one-hour intervals for 24 hours. The wastewater treatment plant influent and effluent were sampled at one-hour intervals for 24 hours and the flow measurements taken from the plant records. The samples were composited into eight-hour shifts which provided three composited samples for each sampling point.

The laboratory results of the dry weather sampling have been summarized and presented in Table 14. The waste loads from the three sewer districts were fairly consistent for both sampling days. The BOD waste load averaged 110 pounds per day per 100 acres for the three sewer districts sampled. This compares very close with the March 4, 1969, total dry weather BOD at the wastewater treatment plant of 112 pounds per day per 100 acres of sewered area. The suspended solids waste load averaged approximately 150 pounds per day per 100 acres.

The October 9, 1968, samples showed an unusually high influent BOD at the treatment plant. A study of the laboratory results of the total sampling indicates the unusually high BOD may have been caused by a slug of industrial waste from a sewer district other than those three sampled. Also, during the latter part of 1968, which includes September and October, the wastewater treatment plant was not operating effectively and an unusually high waste load was being discharged into the river, as indicated by the effluent BOD on the October 9th sampling. However, this problem was corrected and by January, 1969, the treatment plant was operating normally.

The total coliforms averaged 44 million per 100 ml for sewer districts Numbers 8 and 17, and 7 million per 100 ml for sewer district Number 23. The lower count in sewer district Number 23 is due to a large quantity of industrial water.

Infiltration of groundwater into the sewerage system can be estimated by comparing the total flow received at the wastewater treatment plant on the two dry weather sampling days. The October 9, 1968, sampling occurred during an extremely dry month and represents conditions of practically no infiltration. The total flow of 2.05 million gallons received at the wastewater treatment plant on October 9, 1968, agrees closely with the water plant output of 2.0 million gallons on the same day. The March 4, 1969, sampling occurred following spring thawing and represents saturated ground water conditions. The flow received at the wastewater treatment plant on March 4 totaled 2.47 million gallons. The water plant output for this same period was 2.0 million gallons. Therefore, the infiltration is the difference between the October and March sampling or approximately 420,000 gallons. This amounts to an infiltration rate of 270 gallons per day per acre during the times when the ground water table is high. This infiltration rate is not unreasonable for a collection system.

Overflow Samples

The laboratory results of the overflow samples from the three selected sewer districts have been summarized and presented in Table 15. This table presents the average, minimum, maximum, and median values of the chemical and bacteriological characteristics of all the individual overflow samples collected during this study. Sewer district Number 8 has an average BOD concentration of 170 mg/l which is considerably higher than the average BOD concentration of sewer districts Numbers 17 and 23, each of which have an average BOD of 107 and 108 mg/l respectively. This difference of BOD concentration is due to the fact that periodic discharges of slaughter house wastes occur in sewer district Number 8. It is interesting to note that there is very little difference between the average BOD concentration of the overflow samples and the average BOD concentration of the dry weather samples. However, the average suspended solids concentration of 480 mg/l for the overflow samples is much higher than the average of 160 mg/l for the dry weather samples.

The average total coliform count for Numbers 8 and 17 overflows was 8.8 million per 100 ml and 16 million per 100 ml, respectively. This is only 20 percent to 30 percent of the dry weather sample total coliforms.

The more significant water quality characterisitcs of the overflow samples, which include BOD, suspended solids, total solids, the nitrogen series, total phosphates and chlorides, have been graphed in comparison to time after start of overflow and are shown in Figures 23 through 43. These graphs very clearly show the first flushing effects of the storm water on the water quality of the overflows. The peak concentrations of the various water quality characteristics tend to coincide with the peak overflow rate.

To determine the effects of settling on the overflow samples, two overflow samples from Number 17 sewer district were settled and the supernatant withdrawn at 30-minute intervals for two hours. The supernatant was analyzed for BOD and suspended solids and the results are shown in Figure 44. The results indicate that approximately 60 percent to 70 percent of the BOD and suspended solids could be removed with 30 minutes of settling time.

A summary of the waste loads discharged into the Sandusky River from each of the five complete overflow events sampled and measured have been calculated and summarized in Table 16. This table shows that the August 9, 1969, overflow event discharged into the Sandusky River, from just three sewer districts, 2,300 pounds of BOD in approximately two hours. This is more BOD than that received at the wastewater treatment plant from 24 hours of dry weather flow. Extrapolating the 2,300 pounds of BOD to include all 24 sewer districts gives a total of 3,500 pounds of BOD discharged to the river.

Sandusky River Samples

The laboratory analyses of the Sandusky River samples taken upstream and at various locations downstream, during wet and dry weather, have been summarized and are presented in Table 17. Dry weather samples represent conditions of the river without overflow effects and wet weather samples present the river conditions during times of overflow. Because of the difficulty in treatment operation at the wastewater treatment plant during the latter part of 1968, mentioned previously in this report, only those samples collected after January, 1969 have been used in this table.

The major differences in the upstream water quality characteristics during dry and wet weather are in the suspended solids, nitrates and bacteria counts. The average dry weather suspended solids of 32 mg/l increase to an average 465 mg/l during wet weather. The average dry weather concentration of nitrates is 7.2 mg/l as NO_3 and is increased to an average 21.7 mg/l during wet weather. This increase in nitrates seems to be due to agriculture runoff. The total coliform count is reduced from a dry weather average of 59,000 per 100 ml to 3,400 per 100 ml during wet weather. This reduction in bacteria is due to the added dilution water from the upper drainage area.

The comparison between the dry and wet weather river samples indicates that the waste loads from the overflow affect the river quality as far downstream as the fifth bridge, which is approximately seven miles downstream from the wastewater treatment plant. During periods of overflows the average BOD concentration at the first bridge downstream from the wastewater treatment plant is increased from a dry weather average of 6 mg/l to 14 mg/l, the suspended solids increase from 49 mg/l to 192 mg/l, and the total coliforms increase from a dry weather average of 400,000 per 100 milliliters to 4.5 million per 100 milliliters. The average coliform count at the fifth bridge downstream from the wastewater treatment plant is increased from an average 4,500 per 100 milliliters to 86,000 per 100 milliliters.

The diurnal fluctualtion of dissolved oxygen in the Sandusky River is presented in Figures 45 and 46. Figure 45 shows the dissolved oxygen at the upstream gage on October 9 and 10, 1968, at the time of the first dry weather sampling. The dissolved oxygen ranged from 8.2 mg/l to 10.3 mg/l and saturation ranged from 80 percent to 109 percent. The dissolved oxygen at the first bridge downstream from the wastewater treatment plant remained at zero (0.0) mg/l for the entire 24 hour period.

Figure 46 shows the diurnal affect on dissolved oxygen in the Sandusky River from the upstream gage to the seventh bridge downstream from the wastewater treatment plant, a total distance of about 12 miles. The river samples were taken on August 5 and 6, 1969, during dry weather and river flow of approximately 10 cfs which is considered low flow. This dissolved oxygen profile shows the normal dissolved oxygen concentration of the river during low flows.

In addition to the two figures presented above, dissolved oxygen profiles of the river downstream from the wastewater treatment plant during both wet and dry weather conditions are shown in Figure 47.

TABLE 13

DATA SUMMARY

Date	Overflows Measured	Overflows Sampled	River Sampled	Sewer Sampled
July 10, 16, 26 - 1968			X	
" 18		X	X	
September 5			X	X
" 12			X	
" 16				X
" 24		X	X	X
October 2, 9			X	X
" 10		X	X	X
November 15		X		
January 16, 17 - 1969		X		
February 8	X	X	X	
March 4, 5			X	X
" 20, 21			X	
" 24	X	X	X	
" 25, 29	X			
April 2, 28	X		X	
" 5	X			
" 8, 15, 18, 19, 21			X	
May 5, 9, 12, 19, 21			X	
" 7	X	X	X	
" 17, 18	X			
June 3, 12, 16, 23, 26			X	
" 13	X	X	X	
" 14, 15	X			
July 1, 8, 15, 22, 25, 28, 31			X	
" 3, 5, 11, 17, 27	X			
August 1, 6, 10, 20, 27			X	
" 9	X	X	X	
" 16	X			
September 2, 6, 16	X			
" 3, 9, 18			X	
" 17	X		X	

TABLE 14

SUMMARY OF DRY WEATHER WASTE LOADS

DATE & LOCATION	AVERAGE FLOW MGD	BOD mg/l	BOD lbs/day	BOD lbs/day/ 100 ac
1. Upstream				
October 9, 1968	1.27	2.2	23	
March 4, 1969	24.56	3	614	
2. Sewer District #8				
October 9, 1968	0.13	192	208	116
March 4, 1969	0.22	121	221	124
3. Sewer District #17				
October 9, 1968	0.27	198	445	98
March 4, 1969	0.50	118	491	108
4. Sewer District #23				
October 9, 1968	0.66	60	329	87
March 4, 1969	0.69	84	482	128
5. WWTP-RAW				
October 9, 1968	2.05	232	3960	252
March 4, 1969	2.47	109	1969	112
6. WWTP-FINAL				
October 9, 1968	2.15	105	1882	
March 4, 1969	2.59	24	518	
7. Downstream - 1st Bridge				
October 9, 1968	10.87	42	3807	
March 4, 1969	29.63	6.0	1482	

TABLE 14 (CONTINUED)

SUMMARY OF DRY WEATHER WASTE LOADS

	SUSPENDED SOLIDS mg/l	SUSPENDED SOLIDS lbs/day	SUSPENDED SOLIDS lbs/day/ 100 ac.	TOTAL SOLIDS mg/l	TOTAL VOLATILE SOLIDS mg/l	SPECIFIC CONDUCTIVITY mohos/cm
1.	12	127		593	213	757
	10	2048		442	138	490
2.	155	168	94	768	252	1,003
	109	199	111	753	223	897
3.	84	189	42	738	242	1,010
	193	804	178	862	273	910
4.	62	344	91	287	145	573
	246	1417	375	758	343	665
5.	96	1638	104	847	333	1,117
	143	2941	187	755	203	897
6.	70	1255		735	237	1,063
	55	1188		822	257	872
7.	13	1178		673	213	960
	5	1235		457	132	538

TABLE 14 (CONTINUED)

SUMMARY OF DRY WEATHER WASTE LOADS

	COD mg/l	TOTAL COLIFORMS per 100 ml	FECAL COLIFORMS per 100 ml	FECAL STREP per 100 ml
1.	36	60,000	10,000	1,500
	63	23,000	450	1,000
2.	516	42×10^6	5×10^6	550,000
	318	33×10^6	3.0×10^6	300,000
3.	781	61×10^6	1.7×10^6	1.1×10^6
	304	42×10^6	4.2×10^6	350,000
4.	660	7×10^6	5.7×10^6	110,000
	537	7×10^6	500,000	18,000
5.	628	10×10^6	10×10^6	1.5×10^6
	201	200×10^6	8×10^6	360,000
6.	327	4.2×10^6	-	270,000
	253	15×10^6	2.0×10^6	26,000
7.	192	6.5×10^6	6.0×10^6	320,000
	46	1.5×10^6	110,000	17,000

TABLE 14 (CONTINUED)

SUMMARY OF DRY WEATHER WASTE LOADS

	NITRATE NITROGEN	AMMONIA NITROGEN	ORGANIC NITROGEN	TOTAL NITROGEN AS N		
	mg/l as NO_3	mg/l as N	mg/l as N	mg/l	lbs/day	lbs/day/ 100 ac
1.	0.6	1.2	Trace	1.2	13	
	5.1	0.4	2.0	3.6	737	
2.	0.9	55	36	91.2	98	55
	1.7	30	33	62.9	115	64
3.	1.0	56	48	104	234	52
	1.6	35	27	62.4	260	57
4.	0.8	38	14	52	286	76
	2.6	21	16	37.4	215	57
5.	0.7	48	34	82.5	1410	90
	0.5	30	21	47	968	62
6.	0.6	58	31	89.1	1598	
	0.6	24	22	51	1102	
7.	0.3	43	17	60.4	5475	
	5.1	1.8	3.0	5.4	1334	

TABLE 14 (CONTINUED)

SUMMARY OF DRY WEATHER WASTE LOADS

	TOTAL PHOSPHATES AS PO4			CHLORIDES	TOTAL ALKALINITY	pH
	mg/l	lbs/day	lbs/day/ 100 ac	mg/l as Cl	mg/l as $CaCO_3$	
1.	0.5	5		36	251	7.6
	0.6	123		29	159	7.9
2.	82	89	50	122	330	7.3
	13.7	25	14	120	249	7.7
3.	47	106	10	107	300	7.2
	19.2	80	4	113	243	7.9
4.	8.7	49	13	74	160	7.3
	8.7	50	13	70	155	8.0
5.	31	530	34	169	249	6.8
	19	391	25	136	216	7.0
6.	31	556		137	227	7.2
	19	410		133	218	7.3
7.	17	1541		112	233	7.1
	1.6	395		39	170	7.7

TABLE 15

SUMMARY OF LABORATORY ANALYSES
ON OVERFLOW SAMPLES

LOCATION	BOD mg/l	COD mg/l	SUSPENDED SOLIDS mg/l	VOLATILE SUSPENDED SOLIDS mg/l	TOTAL SOLIDS mg/l
1. Overflow No. 8					
No. of analyses	47	13	42	13	40
Average	170	372	533	182	1647
Minimum	11	64	20	70	150
Maximum	560	735	2440	440	3755
Median	140	394	360	180	1260
2. Overflow No. 17					
No. of analyses	54	20	44	24	33
Average	107	476	430	238	863
Minimum	11	120	90	80	310
Maximum	265	920	990	570	1960
Median	100	440	400	160	780
3. Overflow No. 23					
No. of analyses	52	21	32	20	25
Average	108	391	477	228	916
Minimum	23	105	120	70	370
Maximum	365	795	1050	640	1965
Median	78	355	385	200	830

TABLE 15 (CONTINUED)

SUMMARY OF LABORATORY ANALYSES
ON OVERFLOW SAMPLES

	NITRATE NITROGEN mg/l as NO3	AMMONIA NITROGEN mg/l as N	ORGANIC NITROGEN mg/l as N	TOTAL COLIFORM /100 ml	FECAL COLIFORM /100 ml	FECAL STREP /100 ml
1.	41	14	14	14	13	14
	4.54	3.13	10.3	8.8×10^6	2.4×10^6	0.50×10^6
	0.05	0.10	2.1	0.75×10^6	0.5×10^6	$.01 \times 10^6$
	13.50	21.3	68	34.0×10^6	7.1×10^6	1.2×10^6
	3.30	1.10	5.6	3.6×10^6	2.4×10^6	0.50×10^6
2.	52	21	21	12	12	12
	3.79	1.08	8.9	16×10^6	2.1×10^6	0.35×10^6
	0.05	0.10	2.8	0.6×10^6	0.1×10^6	0.01×10^6
	21.0	2.2	19.3	49×10^6	9.9×10^6	1.3×10^6
	3.10	1.1	6.7	7.5×10^6	0.4×10^6	$.02 \times 10^6$
3.	49	23	23	11	13	11
	3.89	2.7	7.3	6.0×10^6	0.50×10^6	0.21×10^6
	0.05	0.10	0.1	0.2×10^6	0.1×10^6	$.003 \times 10^6$
	21.50	9.0	18.5	25×10^6	2.5×10^6	1.6×10^6
	2.40	1.8	5.9	3.6×10^6	0.30×10^6	$.04 \times 10^6$

TABLE 15 (CONTINUED)

SUMMARY OF LABORATORY ANALYSES
ON OVERFLOW SAMPLES

	TOTAL PHOSPHATES PO4 mg/l	pH	TOTAL ALKALINITY mg/l as CaCO3	TOTAL HARDNESS mg/l as CaCO3	CHLORIDES mg/l as Cl	SPECIFIC CONDUCTANCE mohoms/cm
1.	31	42	31	31	34	42
	11.5	6.9	127	183	203	1721
	1.0	6.5	70	115	24	200
	35	7.3	184	290	1400	4600
	8.8	7.0	114	168	99	810
2.	42	47	35	35	39	39
	9.0	7.1	123	160	120	502
	2.0	6.6	40	58	9	132
	27.2	7.4	280	290	460	1500
	7.7	7.1	110	150	110	300
3.	41	47	29	29	31	46
	10.5	7.0	123	176	147	825
	2.4	6.6	64	105	12	174
	30	7.4	250	260	660	2100
	7.5	7.0	100	170	71	800

TABLE 16

SUMMARY OF WASTE LOADS FOR EACH OVERFLOW EVENT

OVERFLOW EVENT DATE & OVERFLOW NO.	FLOW Overflow Period In Minutes	Maximum cfs	Total Volume 1000 cf
1. February 8, 1969			
# 8	120	3.0	13
#17	105	12.0	37
#23	165	8.4	35
TOTAL			85
2. March 24, 1969			
# 8	150	4.4	22
#17	135	11.0	43
#23	135	5.2	23
TOTAL			88
3. May 7, 1969			
# 8	107	3.0	7
#17	60	24.5	18
#23	110	13.2	30
TOTAL			55
4. June 13, 1969			
# 8	200	27.8	132
#17	190	63.2	158
#23	177	61.2	190
TOTAL			480
5. August 9, 1969			
# 8	140	22.7	54
#17	110	50.1	148
#23	170	35.3	96
TOTAL			398

TABLE 16 (CONTINUED)

SUMMARY OF WASTE LOADS FOR EACH OVERFLOW EVENT

	BOD			SUSPENDED SOLIDS		
	Average mg/l	Total lbs.	lbs/100/ ac.	Average mg/l	Total lbs.	lbs/100/ ac.
1.	120	98	55	570	464	260
	51	118	26	615	1416	313
	86	190	50	670	1480	390
		406			3360	
2.	146	201	112	675	931	520
	161	415	92	670	1539	340
	104	149	40	505	725	192
		765			3195	
3.	118	50	28	430	184	103
	172	194	43	454	514	114
	116	216	57	660	1234	325
		460			1932	
4.	41	331	185	375	3100	1700
	31	312	69	413	4200	900
	36	420	111	652	7700	2050
		1063			14778	
5.	177	600	336	-	-	-
	112	1040	230	306	2850	630
	112	670	178	-	-	-
		2310				

TABLE 16 (CONTINUED)

SUMMARY OF WASTE LOADS FOR EACH OVERFLOW EVENT

	VOLATILE SUSPENDED SOLIDS		TOTAL PHOSPHATES AS PO4		NITRATE NITROGEN AS NO3	
	Average mg/l	Total lbs.	Average mg/l	Total lbs.	Average mg/l	Total lbs.
1.	-	-	8.3	7	3.0	2
	-	-	6.7	15	3.1	7
	-	-	6.5	14	2.5	5
				36		14
2.	390	540	12.0	16	2.0	3
	289	779	11.3	30	2.7	7
	280	404	11.8	17	2.5	4
		1733		63		14
3.	200	85	7.3	3	1.4	1
	291	329	12.2	14	0.8	1
	368	689	15.1	28	0.5	1
		1103		45		3
4.	126	1023	2.3	19	9.1	74
	96	967	2.0	21	9.3	94
	160	1873	9.7	113	16.9	198
		3863		153		366

TABLE 17

SUMMARY OF WET AND DRY WEATHER RIVER ANALYSES

	BOD mg/l		SUSPENDED SOLIDS mg/l	
	Dry Weather	Wet Weather	Dry Weather	Wet Weather
1. Sandusky River - Upstream				
No. of analyses	33	22	20	13
Average	4	5	32	465
Minimum	1	2	5	20
Maximum	14	13	160	1,960
Median	3	4	20	240
2. Sandusky River - Downstream 1st Bridge downstream from wastewater treatment plant				
No. of analyses	27	43	14	38
Average	6	14	49	192
Minimum	2	4	8	5
Maximum	12	51	190	960
Median	5	10	22	90
3. Sandusky River - Downstream 2nd Bridge from WWTP				
No. of analyses	9	8	8	8
Average	7	5	44	62
Minimum	3	3	10	20
Maximum	22	8	195	135
Median	-	-	22	40
4. Sandusky River - Downstream 3rd Bridge from WWTP				
No. of analyses	12	17	4	17
Average	4	6	36	36
Minimum	1	3	27	20
Maximum	8	10	45	50
Median	4	6	-	40
5. Sandusky River - Downstream 5th Bridge from WWTP				
No. of analyses	13	19	5	11
Average	5	6	18	90
Minimum	2	2	15	25
Maximum	13	12	25	300
Median	4	6	17	50

TABLE 17 (CONTINUED)

SUMMARY OF WET AND DRY WEATHER RIVER ANALYSES

	TOTAL VOLATILE SOLIDS mg/l		TOTAL SOLIDS mg/l		TOTAL PHOSPHATES mg/l as PO4	
	Dry Weather	Wet Weather	Dry Weather	Wet Weather	Dry Weather	Wet Weather
1.	15	8	12	9	17	14
	183	94	510	576	0.8	0.9
	2	35	400	405	0.2	0.1
	225	125	610	1,080	3.2	2.7
	-	-	-	-	0.6	0.8
2.	9	17	9	19	13	40
	128	158	506	746	1.6	3.3
	30	30	410	415	0.2	0.8
	195	270	710	1,335	5.9	10.0
	130	165	490	630	1.3	2.6
3.	-	2	-	2	3	6
	-	168	-	480	2.0	2.2
	-	-	-	-	-	-
	-	-	-	-	-	-
	-	-	-	-	-	-
4.	-	-	-	-	2	15
	-	-	-	-	2.0	4.1
	-	-	-	-	-	1.8
	-	-	-	-	-	8.0
	-	-	-	-	-	4.8
5.	-	-	-	-	-	14
	-	-	-	-	-	3.7
	-	-	-	-	-	1.3
	-	-	-	-	-	7.3
	-	-	-	-	-	2.6

TABLE 17 (CONTINUED)

SUMMARY OF WET AND DRY WEATHER RIVER ANALYSES

	NITRATE NITROGEN mg/l as NO3		AMMONIA NITROGEN mg/l as N		ORGANIC NITROGEN mg/l as N	
	Dry Weather	Wet Weather	Dry Weather	Wet Weather	Dry Weather	Wet Weather
1.	26	19	5	9	4	9
	7.2	21.7	0.58	0.32	1.51	1.76
	0.4	0.5	0.13	0.0	0.0	0.0
	32.0	28.8	1.20	2.60	6.07	4.80
	3.3	14.5	-	-	-	-
2.	20	41	5	11	5	11
	6.7	7.5	1.51	2.40	2.40	3.79
	0.2	0.3	1.10	0.60	0.80	0.20
	32.0	24.8	2.12	6.60	3.03	14.7
	3.3	7.7	1.51	1.40	3.03	2.80
3.	3	6	-	2	-	2
	1.0	0.9	-	1.0	-	0.8
	-	-	-	-	-	-
	-	-	-	-	-	-
	-	-	-	-	-	-
4.	4	15	-	-	-	-
	3.7	1.9	-	-	-	-
	1.3	0.5	-	-	-	-
	5.8	3.4	-	-	-	-
	-	1.8	-	-	-	-
5.	3	14	-	1	-	2
	6.9	6.2	-	0.6	-	1.7
	1.4	0.5	-	-	-	-
	10.2	22.2	-	-	-	-
	-	4.4	-	-	-	-

TABLE 17 (CONTINUED)

SUMMARY OF WET AND DRY WEATHER RIVER ANALYSES

	TOTAL COLIFORMS /100 ml		FECAL COLIFORM /100 ml		FECAL STREP /100 ml	
	Dry Weather	Wet Weather	Dry Weather	Wet Weather	Dry Weather	Wet Weather
1.	3	4	3	3	3	3
	59,000	3,400	8,000	900	1,600	170
	23,000	1,200	450	800	1,000	130
	95,000	6,300	14,000	1,000	2,400	200
	-	-	-	-	-	-
2.	8	11	6	7	6	6
	0.4×10^6	4.5×10^6	36,000	161,000	11,000	55,000
	2,000	0.05×10^6	2,000	10,000	1,000	1,000
	1.5×10^6	8.8×10^6	110,000	320,000	24,000	157,000
	-	-	-	-	-	-
3						
4.	5	1	4	-	4	1
	15,000	130,000	390	-	310	1,400
	5,600	-	180	-	70	-
	40,000	-	500	-	500	-
	-	-	-	-	-	-
5.	4	1	4	-	3	-
	4,500	86,000	380	-	230	-
	3,000	-	175	-	180	-
	5,300	-	760	-	300	-
	-	-				

TABLE 17 (CONTINUED)

SUMMARY OF WET AND DRY WEATHER RIVER ANALYSES

	COD		pH		TOTAL ALKALINITY mg/l as CaCO3	
	Dry Weather	Wet Weather	Dry Weather	Wet Weather	Dry Weather	Wet Weather
1.	12	4	26	14	13	10
	127	65	7.9	7.8	192	155
	17	18	7.1	7.1	152	98
	422	164	8.9	8.5	254	192
	-	-	7.8	7.8	182	170
2.	11	14	24	37	9	19
	244	114	7.8	7.3	165	135
	24	28	7.4	7.1	150	99
	770	220	8.5	7.8	180	166
	130	120	7.7	7.3	164	140
3.	2	4	6	6	3	6
	156	156	8.0	7.9	183	166
	32	24	7.4	7.5	172	140
	280	430	8.4	8.2	200	186
	-	80	7.9	7.9	-	-
4.	1	4	5	14	2	6
	48	112	7.7	7.7	173	189
	-	18	7.5	7.3	160	174
	-	210	8.2	8.2	186	240
	-	-	7.7	7.8	-	-
5.	-	5	5	10	1	7
	-	86	7.6	7.6	164	142
	-	24	7.5	7.4	-	86
	-	220	7.7	7.8	-	168
	-	-	7.6	7.7	-	-

TABLE 17 (CONTINUED)

SUMMARY OF WET AND DRY WEATHER RIVER ANALYSES

	TOTAL HARDNESS mg/l as CaCO3		CHLORIDES mg/l as Cl		SPECIFIC CONDUCTIVITY mohoms/cm	
	Dry Weather	Wet Weather	Dry Weather	Wet Weather	Dry Weather	Wet Weather
1.	10	8	14	13	18	11
	316	283	30	31	621	524
	250	252	13	21	420	309
	372	320	37	35	770	610
	314	290	-	-	630	550
2.	8	17	12	34	14	32
	286	236	38	53	602	504
	260	145	10	23	380	245
	335	305	57	158	750	825
	282	248	38	40	620	520
3.	3	6	3	6	5	6
	297	285	39	43	674	607
	288	256	38	39	610	560
	304	302	40	46	770	620
	-	-	-	-	-	-
4.	2	6	3	14	5	14
	270	293	44	56	651	653
	240	272	37	37	580	600
	300	304	50	77	725	742
	-	-	-	-	-	-
5.	1	7	5	9	6	9
	268	271	60	47	639	581
	-	185	37	22	563	370
	-	296	69	74	720	740
	-	-	65	43	688	600

SECTION XIV

AQUATIC BIOLOGY SURVEY OF THE SANDUSKY RIVER

A summary of the aquatic biology survey of the Sandusky River is shown in Table 18.

Sampling stations were established to determine biological productivity and other pertinent information in the Sandusky River upstream and downstream from Bucyrus. Samples were collected in the fall of 1968 and in the spring and summer of 1969. The study section consisted of 26 miles of the Sandusky River extending ten miles upstream and thirteen miles downstream and including three miles of river within the city. The stream population and stream conditions affecting biological productivity were observed at ten points.

The results of the aquatic biology survey corroborates the results of the water quality studies of this report. The river upstream from Bucyrus has a relatively undisturbed fauna of the types normally found in unpolluted waters. The river inside the City of Bucyrus shows indication of gross pollution and has sections completely devoid of life. The river downstream from Bucyrus, during periods of low flow, is biotically dead for six to eight miles below the wastewater treatment plant.

TABLE 18

SUMMARY OF AQUATIC BIOLOGY SURVEY
OF THE SANDUSKY RIVER

Date and Location	Life Forms Found	Remarks
10 Miles Upstream from Bucyrus March 18, 1969	Crayfish, snails and clams	No evidence of pollution.
Upstream Gage October 26, 1968	Pea clams, snails, leeches minnows and crayfish	Gravel and rock bottom covered with algae
March 18, 1969	Crayfish, snails, pea clams and muskrat	Relatively undisturbed stream fauna.
July 25, 1969	Crayfish, snails and minnows	River bottom has been washed clean by recent floods.
No. 8 Overflow October 26, 1968	Only a few immature insect larvae	Water plant waste lime sludge had filled most of the niches between the gravel and stone river bottom. Blue-green algae scum very apparent.
March 18, 1969	No apparent life	River very turbid from lime sludge. Oil slick on water.
No. 17 Overflow October 26, 1968	Pea clams, Phepa, leeches, crayfish, minnows and an array of immotile aquatic insects.	Filamentacious algae extremely abundant.

TABLE 18 (Continued)

Date and Location	Life Forms Found	Remarks
AuMiller Park - 2000' Below No. 17 Overflow		
October 26, 1968	Pea clams, Phepa, leeches and crayfish	This location contains a biotic abundance including many forms of algae and other plankton. The water is very clear.
November 19, 1968	Pea clams, leeches, snails, minnows and darters	A tremendous abundance of life forms. River very clear at this location.
March 18, 1969	Pea clams, crayfish and snails	The biotic abundance found in October now greatly reduced. The algae has been swept away and rocks are clean.
July 25, 1969	Crayfish, pea clams, leeches minnows, and various forms of plankton	No evidence of pollution. High flood waters have brought river back to normal.
Downstream Gage - First Bridge downstream from WWTP		
October 26, 1968	Sludge worms	From a biotic standpoint, the river is dead. The river is black and the stench is evident before one sees the stream. Bottom of river is covered with black sludge deposits.
November 19, 1968	Sludge worms	A dead river.

TABLE 18 (Continued)

Date and Location	Life Forms Found	Remarks
Downstream Gage - First Bridge (Cont'd.)		
March 18, 1969	No life forms found	The river was rather clear and most of the sludge deposits have been washed out from the recent high water.
July 9, 1969	No sample taken	High river flow and no evidence of pollution.
July 25, 1969	Crayfish and minnows	Recent high waters have flushed out sludge deposits and continued high water has brought life forms.
Fourth Bridge Downstream - 5.5 miles below WWTP		
October 26, 1968	Sludge worms	Biologically, this location seems more lifeless than the downstream gage location.
March 18, 1969	No life found	River black and barren.
July 25, 1969	Crayfish, leeches, frogs, minnows and various plankton	Flood water has cleaned river bottom and brought in new life forms.
Fifth Bridge Downstream - 7.2 miles below WWTP		
November 19, 1968	Only immature insect larvae	River was grayish and contained sludge deposits.

TABLE 18 (Continued)

Date and Location	Life Forms Found	Remarks
Eighth Bridge Downstream - 9.6 miles below WWTP		
November 19, 1968	Caddis Larvae	Caddis larvae indicate the stream is returning to normal. River was clear and there was no evidence of sludge deposits.
March 18, 1969	Crayfish and all types of aquatic insects	No evidence of pollution.
July 25, 1969	Crayfish, clams, minnows and aquatic insects	River in good condition.
Tenth Bridge Downstream - 12.9 miles below WWTP		
November 19, 1968	Minnows, sunfish, crayfish, caddis and aquatic insects	River is fully recovered at this location.
March 18, 1969	Minnows, various types of fish, crayfish and aquatic insects	River is fully recovered at this location.

SECTION XV

RELATIONSHIP OF RAINFALL AND RUNOFF

The design of interception and treatment or holding facilities for combined sanitary and storm runoff water requires a complete sewer hydrograph.[3] This requires knowing the rainfall-runoff relationship. Sewer hydrographs were developed first for the three sewer districts studied in detail, then for all of Bucyrus. Throughout the discussion the word "overflow" will be used for any water flowing into the river from the sewer system and "runoff" will be used for any water flowing into the sewer system. Overflow is assumed to equal runoff for rainfalls greater than 0.25 inch.

There are three relationships between rainfall and runoff which must be determined before a complete runoff hydrograph can be defined: one, the relationship of the start of the runoff hydrograph to the start of the rainfall; two, the relationship of the shape of the runoff hydrograph to the duration of the rainfall; and three, the relationship of the peak a and volume of the runoff hydrograph to intensity and duration of the rainfall. Whenever possible these relationships will be derived from the measured data.

Start of Overflow

A table was prepared for each of the three selected overflow points listing all of the overflow events measured. Included in these tables were the times between the start of the significant rainfall and the start of the overflow. The period of time varies with the rainfall intensity and pattern. The following values present in Table 19 are average for rainfalls of intensities greater than 0.5 inch per hour.

TABLE 19

TIME TO START OF OVERFLOW

Sewer District No.	Time Between Start of Rainfall and Start of Overflow Minutes
8	10
17	20
23	25

The time between the start of the significant rainfall and the start of the overflow could be called the reaction time of the sewer system. It is equal to the period of time for a significant amount of runoff to reach the overflow point.

Before runoff starts, the depression storage must be filled. The runoff needed to cause overflow is equal to the storage capacity in the system and either the volume of the interceptor or the capacity of the connector pipe. An analysis was made of the rainfalls producing little or no overflow and having no antecedent rainfall. The following values presented in Table 20 are the average amounts of rainfall required to cause overflow:

TABLE 20

RAINFALL TO CAUSE OVERFLOW

Sewer District No.	20-Minute Rainfall Required to Produce Overflow Inches
8	.04
17	.06
23	.05

Hydrograph Shape

The unit hydrograph was used to describe the shape of the overflow hydrographs from each of the three areas. Each unit hydrograph was derived from the measured overflow data. Only the overflows from short intense rainfalls were used. In many cases the rainfall events produced compound hydrographs. These hydrographs were separated and drawn as individual hydrographs.

Each overflow hydrograph was divided into ten-minute intervals. The average rate of flow in each ten-minute interval was determined and the total volume of overflow computed. The hydrograph ordinates were then adjusted to give a total overflow volume equivalent to 1.00 inch of runoff from the sewer district.

Every overflow hydrograph for each sewer district then had the same volume, but many different shapes. The shape of the hydrographs is determined by the length of the rainfall. According to the unit hydrograph theory, any rainfall less than the length of a unit storm will produce the same shape hydrograph. If the rainfall continues past this critical period of time, each additional period of unit storm will produce a unit hydrograph; the sum of these unit hydrographs will produce the hydrograph for that rainfall. Each additional unit hydrograph will delay the peak of the runoff hydrograph by the duration of the unit storm. Therefore, the period of time from the start of the overflow to the peak of the overflow will equal the period of significant rainfall for any rainfall equal to or greater than a unit storm.

A graph was prepared for each overflow, plotting the length of the significant rainfall in minutes versus the time between the start and the peak of the overflow in minutes. (See Figure 48 for the summary of these graphs) For Number 8 overflow, the length of significant rainfall was equal to the peak time. However, for Number 17 overflow the peak time is five minutes less than the length of the significant rainfall, and for Number 23 overflow the peak time was five minutes greater than the length of the significant rainfall. These variations in peak time for Numbers 17 and 23 overflows were due to the longer lengths of time to start overflow (See Table 19) and variations in the drainage characteristics.

The smallest periods of time between the starts and the peaks of the overflows measured were five minutes for Number 8 overflow, two minutes for Number 17 overflow, and ten minutes for Number 23 overflow. (See Figure 48) The maximum length of significant rainfall producing these minimum peak times were five minutes for Numbers 8 and 23 overflows, and seven minutes for Number 17 overflow. Since five minutes was also the shortest time of significant rainfall measured, the conclusions are the length of a unit storm is equal to or less than five minutes for Numbers 8 and 23 overflows, and is equal to seven minutes for Number 17 overflow.

A unit hydrograph for each overflow was derived from the overflow hydrographs. These unit hydrographs were based on the hydrographs produced by rainfalls less than or equal to the unit storm for the sewer district. The values given in Figure 48 for the times between the start of an overflow and its peak were used. Twenty minute lengths of rainfall were later found to be more convenient to work with than the unit storm lengths of rainfall. Therefore, the unit hydrographs for 20 minutes of rainfall were summed to equal one hydrograph for each overflow. (See Figures 49, 50, and 51) These hydrographs have a volume of 1.00 inch of runoff and will be referred to as "20 minute unit hydrographs" in future discussion.

Hydrograph Peak and Volume

The two most important elements of a runoff hydrograph are its peak and volume. The two cannot be separated. Since the shape of the hydrograph has already been determined, knowing either the peak or the volume of the hydrograph will completely define it for any rainfall.

The two most commonly used methods for determining the peak or the volume are: (1) the rational formula and (2) the hydrograph method. The application of both of these methods to the three sewer districts in Bucyrus was studied and the results compared to the measured data. Finally a modification of the hydrograph method was studied and adopted for use in designing storm water facilities for Bucyrus.

1. Rational Formula

The rational formula is the most commonly used method for designing storm water facilities. The formula is easy to understand, simple to use, and coefficients and variables are available from standard references for either preliminary or detailed design. The most frequent objection to the rational formula is that only a peak rate of flow can be computed.[3] This objection would be overcome if used with the derived unit hydrographs.

In its most commonly used form, the rational formula appears as $Q = CIA$, "Where Q is the rate of runoff at a specific point in time, A is the drainage area tributary to the specific point at the specific time, I is the average intensity of rainfall over the tributary drainage area for the specified time, and C is the coefficient of runoff or ratio of rate of runoff to rate of rainfall applicable to the particular situation."[4] The specified time referred to in the definition of I equals the time of concentration. In other words, the rational formula states that the rate of runoff is equal to the rate of supply if the length of rainfall is greater than the time of concentration.

Since this study involves only the wastewater discharged to the river, only the runoff at the overflow point for each sewer district was determined. The values for sewered areas given in Table 7 were used for A. The values of I used were obtained from the intensity duration curves for Bucyrus, Figure 15, using the times of concentration obtained from Table 8. Values for C were still required.

The preferred method of obtaining a C value for the three sewer districts is to use measured rainfall and runoff data. However, this method was found to be impossible without studying the sewer systems in great detail. The times of concentration given in Table 8 are 54 minutes, 64 minutes, and 60 minutes for Numbers 8, 17, and 23 sewer districts, respectively. These times of concentration exceed the length of any continuous rainfall measured during the past year for the three overflows with an intensity greater than 0.20 inch per hour. Therefore, the only way to obtain a C value by this method would be to determine by a complete hydraulic analysis how much of the sewer district area was contributing at the time the rainfall stopped. An analysis of this type exceeds the realm of this study. Therefore, values of C for various types of land cover have been assumed, based on published data, and matched to the types of land cover for the three sewer districts.[2] The C values used are presented in Tables 21 and 22.

TABLE 21

RUNOFF COEFFICIENTS

	C
Impervious	
Buildings (roofs	0.85
Asphalt and Concrete (drives and walks)	0.80
Streets (asphalt and brick)	0.80
Water	1.00
Pervious	
Lawns (heavy soil - 2%)	0.15
Weeds (unimproved areas)	0.20
Packed earth (playgrounds)	0.30
Gravel (railroad yard areas)	0.30
Corn Fields	0.20

Using the values for sewered area, given in Table 7, a weighted value of C for each of the three sewer districts was obtained. (See Table 22)

TABLE 22

WEIGHTED RUNOFF COEFFICIENTS

Sewer District	Weighted C
8	0.39
17	0.41
23	0.35

These values compare well with other values of C given for residential type areas. For example, Linsey gives a C value for flat residential area, 30 percent impervious, of 0.40.[5] This compares to 0.39 and 0.41 for Numbers 8 and 17 sewer districts, each with 33.7 percent impervious area. Since Number 23 sewer district is only 26.1 percent impervious, it has a lower C value.

The runoff to be expected from a one year frequency storm was computed using the weighted values of C. (See Table 23) The runoffs from the total area and just the impervious areas are given. These values are compared to the measured runoff from the May 17 and June 13 storms.

From Table 23 two things become evident. First, it is possible to get a higher peak from the impervious areas alone than from the total areas. The intensity duration curves for high intensity storms drop so rapidly during the first 20 to 30 minutes that the peak overflow rate from a smaller area with a shorter time of concentration can be greater than the peak rate for the entire area.

Second, the computed peak flows for both the impervious areas and the total areas are more than double the flows measured on May 17 and June 13, and far exceed the maximum capacity of the sewer system. Since the peak overflow rates for Numbers 17 and 23 sewer districts were close to the maximum sewer capacities on May 17 and June 13, a comparison of these values to the computed values cannot be made. However, the peak overflow rates for Number 8 overflow on these two dates are 20 cfs less than the maximum sewer capacity. Therefore, the conclusions are the C value for Number 8 sewer district is less than one-half the standard value for this type of area.

A check was made on the maximum sewer capacities of all the sewer districts using a two-year storm. (See Table 24) The classifications given in Table 7 were used to estimate a C value for each of the other sewer districts based on the weighted C values obtained for Numbers 8, 17, and 23 sewer districts. Nineteen of the twenty-four trunk sewers have capacities less than the peak runoff from a two-year storm.

The conclusions from the above analysis are that the rational formula is not an acceptable method of computing the runoff from the sewer districts in Bucyrus. First, the peak flows for a two-year storm exceed the maximum sewer capacities of many of the districts. Second, the peak flows computed with the standard runoff coefficients that do not exceed the maximum sewer capacities are much greater than the measured values. Finally, with the rational formula there is no simplified way of determining how much of the area is contributing for rainfall durations less than the times of concentration.

TABLE 23

COMPARISON OF THE RATIONAL FORMULA
TO MEASURED DATA

No. 8 Sewer District	A Acres	Time of Concent. min.	I I Yr. Storm in/hr.	C	Q cfs
Rational Formula					
Entire Area	178.9	54	1.08	0.39	75
Impervious Only	60.4	30	1.57	0.82	78
May 17, 1969					32
June 13, 1969					30
Max. Sewer Capacity*					50
No. 17 Sewer District					
Rational Formula					
Entire Area	452.5	64	0.95	0.41	176
Impervious Only	151.5	38	1.36	0.82	169
May 17, 1969					67
June 13, 1969					63
Max. Sewer Capacity*					75
No. 23 Sewer District					
Rational Formula					
Entire Area	377.8	60	1.00	0.35	132
Impervious Only	98.5	43	1.26	0.82	102
May 17, 1969					69
June 13, 1969					61
Max. Sewer Capacity					65

* With Control Structure

TABLE 24

OVERFLOW PEAKS USING RATIONAL FORMULA
2 Year Storm for Bucyrus

Sewer District No.	A Acres	Time of Concent. min.	I in/hr.	C	Q cfs
1	73.3	34	1.5	.25	27.5
2	2.5	21	2.4	.40	2.4
3	19.4	27	2.1	.40	16.3
4	113	35	1.8	.40	81.3
5	32.1	29	2.0	.60	38.5
6	21.0	27	2.1	.40	17.6
7	3.2	21	2.4	.60	4.6
8	178.9	54	1.3	.39	90.6
9	3.0	22	2.3	.40	2.8
10	7.1	24	2.2	.60	9.4
11	6.2	14	2.9	.80	14.4
12	41.9	26	2.1	.40	35.2
13	8.8	23	2.2	.50	9.7
14	70.2	36	1.7	.40	47.8
15	10.8	25	2.1	.50	11.3
16	5.0	24	2.2	.60	6.6
17	452.5	64	1.2	.41	222.5
18	5.7	24	2.2	.35	4.4
19	24.5	38	1.7	.40	16.7
20	12.1	26	2.1	.35	8.9
21	7.8	27	2.0	.35	5.5
22	72.4	32	1.9	.35	4.8
23	377.8	60	1.2	.35	158.7
24	20.7	30	2.0	.25	10.4

2. Hydrograph Method

The most widely used hydrograph methods are the Los Angeles Method[6] and the Chicago Method.[7] Both of these methods are for the design of an entire sewer system, catch basin by catch basin. The hydrographs for each sewer lateral are summed and the composit hydrograph peak used for design.

Since unit hydrographs have been derived for the overflows of the three districts, a composite hydrograph was not needed. However, the surface infiltration curves used to determine the volume of runoff from each small drainage area can be applied to the entire drainage area provided the area of each type of land cover is known.

The results by using the standard infiltration-capacity curve from the ASCE design manual[2] for a pervious surface in a standard residential area was compared with observed data. This curve was plotted in terms of accumulative mass infiltration capacity and checked against the rainfall and runoff data for Number 17 overflow on May 17 and June 13. Mass diagrams of the rainfall on these two dates were drawn to the same scale as the filtration curve. (See Table 25 for the comparison of the derived overflow volume versus measured overflow volume) The three rainfalls of May 17 were considered as new rainfalls and separate mass curves drawn for each one.

An analysis of the results in Table 25 indicates that there is a great deal of variation in the overflow volumes for the two storms. On May 17, 30 percent to 60 percent of the rainfall on the impervious areas ran off. However, the same infiltration curve applied to the rainfall of June 13 yielded a volume of runoff from the pervious area greater than the total runoff measured. Both storms were approximately one-year storms. Comparisons were made for the other two districts and similar inconsistencies were noted. Therefore, on the basis of this analysis, the standard residential infiltration curve was not applicable.

3. Modified Hydrograph Method

A relationship between peak overflow rate and rainfall was derived from the measured data. Three graphs were plotted for each overflow. These were the maximum 10, 20, and 30 minute rainfall intensities versus the peak overflow rates which they produced. The intensities of rainfalls with durations less than the stated times were averaged over the time period. Compound rainfalls and overflow hydrographs were separated.

A straight line relationship was found between maximum rainfall intensity for a given duration and peak overflow rate. The least amount of deviation was produced by a rainfall of 20 minute duration. (See Figures 52, 53, and 54) All three graphs distinguish between antecedent rainfall and no antecedent rainfall conditions. This distinction disappears, however, at higher intensity rainfalls for Numbers 8 and 17 sewer districts. All

TABLE 25

OVERFLOW VOLUME - NO. 17 OVERFLOW
Using Standard Infiltration Curve *

	May 17, 1969 1	2	3	June 13 1969
1. Rainfall, total - Inches	0.27	0.44	1.16	1.20
2. Runoff, Pervious Area* - Inches depth on pervious area	0	0.14	0.28	0.38
3. Runoff, Pervious Area - %	0	32	24	32
4. Runoff, Pervious Area - Inches depth on drainage district	0	0.09	0.19	0.25
5. Overflow, Measured by weir - Inches depth on drainage district	0.05	0.14	0.40	0.10
6. Runoff, Impervious Area - Inches depth on drainage district	0.05	0.05	0.21	0
7. Runoff, Impervious Area - Inches depth on impervious area	0.15	0.15	0.63	0
8. Runoff, Impervious Area - %	56	34	54	0

* Standard Infiltration - Capacity Curves for Pervious Surface, Residential areas (standard curve), Design and Construction of Sanitary and Storm Sewers, ASCE MOEP No. 37.

three graphs start at or near the origin for antecedent rainfall conditions. For no antecedent rainfall conditions, overflow starts when the 20 minute rainfall exceeds the values given in Table 19. Following are the mathematical formulas for the relationships shown on the three graphs:

Maximum Twenty Minute Rainfall versus Peak Overflow Rate

Q = Peak flow, cfs

I = Maximum average 20 minute rainfall intensity, In./Hr.

No. 8 Overflow

1) No Antecedent Rainfall

$I \leq 0.39, \quad Q = 18\,(I - 0.12)$

$I > 0.39$, Same as with Antecedent Rainfall

2) Rainfall within 24 hours

$I \leq 0.75, \quad Q = 12(I)$

$I > 0.75, \quad Q = 20\,(I - 0.30)$

No. 17 Overflow

1) No Antecedent Rainfall

$I \leq 0.39, \quad Q = 110\,(I - 0.18)$

$I > 0.39$, Same as with Antecedent Rainfall

2) Rainfall within 24 hours

$Q = 60\,(I - 0.03)$

No. 23 Overflow

1) No Antecedent Rainfall

$Q = 33\,(I - 0.15)$

2) Rainfall within 24 hours

$Q = 40(I)$

The maximum ten minute rainfall intensity cannot be used to describe a 20 minute duration rainfall hydrograph. If the unit hydrographs for an overflow are graphically summed, the resulting hydrograph is the S curve.[5] Assume each unit hydrograph is for a five minute duration unit storm. Since the rainfall intensity remains constant, the four hydrographs for the 20 minute duration rainfall will have the same maximum ten minute rainfall intensity as the ten minute duration rainfall with only two hydrographs. The peaks for the two rainfalls are obviously different.

The maximum 20 minute intensity can be used to describe a ten minute rainfall, however. Assuming the two hydrographs from the ten minute rainfall have the same volume as for the four hydrographs from the 20 minute rainfall, the composite peak for the four hydrographs would be only a little less than the composite peak for the two hydrographs.

A 30 minute rainfall duration produces more deviation than a 20 minute rainfall duration because of the nature of the rainfalls measured. Only one high intensity rainfall lasted longer than 20 minutes and still produced only one hydrograph peak. The remaining high intensity rainfalls lasting longer than 20 minutes produced multiple peaks because of the irregularity of their intensities.

The peak overflow rates shown in Figures Numbers 52, 53, and 54 are limited by the maximum sewer capacities. With the control structures, these are equal to 50, 70, and 65 cfs for Numbers 8, 17, and 23 overflows, respectively. The data plotted for Number 17 overflow in Figure 53 clearly shows the limiting effect of the sewer capacities.

The volumes of overflow were related to rainfall by means of the unit hydrograph. Since the peaks of the unit hydrographs are directly proportional to their volume, there was also a straightline relationship between the rainfall and overflow volumes. These relationships were expressed as the following mathematical formulas:

Twenty Minute Rainfall versus Overflow Volume

O = Overflow Volume, Depth on sewer district in inches

P = Rainfall, inches

No. 8 Overflow

1) No Antecedent Rainfall

$P \leq 0.13, \quad O = 0.18\,(P - 0.04)$

$P > 0.13$, same as with Antecedent Rainfall

2) Rainfall within 24 hours

$P \leq 0.25, \quad O = 0.12\ P$

$P > 0.25, \quad O = 0.20\ (P - 0.10)$

<u>No. 17 Overflow</u>

1) No Antecedent Rainfall

$P \leq 0.13, \quad O = 0.51\ (P - 0.06)$

$P > 0.13$, same as with Antecedent Rainfall

2) Rainfall within 24 hours

$O = 0.28\ (P - 0.01)$

<u>No. 23 Overflow</u>

1) No Antecedent Rainfall

$O = 0.20\ (P - 0.05)$

2) Rainfall within 24 hours

$O = 0.25\ P$

These formulas were used to determine the overflow hydrographs for the one-year, two-year, and five-year frequency twenty-minute storms. (See Figures 55, 56, and 57)

SECTION XVI

RIVER RESPONSE TO RAINFALL

An analysis was made of the discharge records for the Sandusky River above and below Bucyrus to determine the relationship between rainfall and runoff for Bucyrus and the upstream drainage basin. The records used for this analysis covered the time period from February 4, 1969, to September 24, 1969. The rainfalls were segregated into 24 hour time intervals. The average rainfall for the entire drainage basin was obtained by averaging the rainfall from the three rain gages. All hydrographs were separated from the base river flow.

All of the storms passing over Bucyrus move in an Easterly direction. Since the upstream drainage basin is east of Bucyrus, the storms will pass over Bucyrus before any rain falls in the upstream drainage basin. This phenomena produces two distinct runoff hydrographs at the downstream gage, first the urban runoff hydrograph, and then the upstream drainage basin runoff hydrograph.

Urban Runoff Hydrograph

The urban runoff hydrograph includes the runoff from: the sewered area in the combined sewer overflows; the non-sewered drainage areas between the two stream gages which flows directly to the river; and the area adjacent to the river for one or two miles above the upstream gage. The percent of the urban hydrograph from combined sewer overflow varied considerably but was generally about one-half the volume of the urban runoff hydrograph. The volume of runoff from the area adjacent to the river above the upstream gage also varies greatly. The peak flow rates measured at the upstream gage varied from four cfs to 267 cfs for rainfalls greater than 1.00 inch. This runoff peaks at the upstream gage 1.5 hours after the rainfall has stopped in Bucyrus and becomes part of the total urban hydrograph.

Figure 63 is the distribution graph for the urban runoff from a unit storm in Bucyrus. The significant runoff reaches the downstream gage approximately one hour after the start of the rain. The river peaks two hours later. In seven hours the river returns to its pre-storm flow. The maximum hydrograph peak observed during the study period was 332 cfs on July 11. On this date, approximately one-half of this peak was from the upstream drainage area.

Upstream Drainage Basin Runoff Hydrograph

Following the urban runoff hydrograph the river returns to its pre-storm flow until the hydrograph from the upstream drainage basin arrives. The time of arrival depends entirely on the velocity in the river. The lag time of the peak flow following the end of the rain varies from 40 hours to 17 hours for river flows of four to 300 cfs at the upstream gage. For flows greater than 500 cfs the lag time will be approximately 16 hours. The travel time between the upstream and downstream gages also varies with the velocity. The peak will take an additional 6.5 hours to travel to the downstream gage at 10 cfs, 1.5 hours at 500 cfs, and one hour at 2000 cfs.

The hydrograph for the upstream drainage basin is a much flatter hydrograph than the urban runoff hydrograph. The minimum time between start of hydrograph and peak is five hours. The peak of the hydrograph varies with the season of the year and the length of time since the preceding rainfall.

The relationship between rainfall and runoff for the study period of February through May was entirely different from the relationship for June through September. For February through May, there was a straight line relation between rainfall and peak flow for rainfalls greater than 0.75 inch and peak flows greater than 500 cfs. During this period normally any rainfall greater than 1.5 inches will produce a peak river flow greater than 1500 cfs.

Most of the storms measured from June through September were short, intense thunderstorms. The rainfall intensities for these types of storms varied greatly between rain gages. The peak flows for the upstream drainage basin hydrographs also varied greatly for amounts of rainfall. For example, a 1.2 inch rainfall produced no peak flow on June 13 and 500 cfs peak flow on July 20. The minimum peak and maximum peak flows possible from a 1.5 inch rainfall are 25 cfs and 700 cfs, respectively.

SECTION XVII

EVALUATION AND CORRELATION OF WASTELOAD DATA

Waste Loads versus Overflows

The strength and amount of waste load discharged from combined sewer overflows depends on a number of factors, including duration and intensity of rainfall, volume of runoff, number of days between overflow events, efficiency of street cleaning operation, and design characteristics of the sewer system. This portion of the report will evaluate the effect of rainfall, runoff and number of days between overflows on the strength and amount of waste load discharged to the river.

The relationships between BOD, total solids, suspended solids, chlorides, phosphates, the nitrogen series, and length of overflow for the three selected sewer districts have been graphed and are shown in Figures 23 through 43. The BOD and suspended solids concentration generally reach a peak about 20 minutes after start of overflow and then tend to drop at a fairly rapid rate for two to three hours and then approach a lower limit. Samples taken approximately 12 hours after start of overflow indicate that the BOD and suspended solids concentration will decrease to about 15 mg/l and 50 mg/l, respectively.

Generally, the longer the period of time between overflows the larger the waste load for a particular overflow volume. The influence of this parameter is shown by comparing the BOD waste load of the June 13 and August 9 overflow events. Table 16 shows that the June 13 overflow volume was 480,000 cubic feet and the BOD load was 1063 pounds, while the August 9 event overflow volume was 398,000 cubic feet with a BOD load of 2310 pounds. The rainfall on June 13 was 1.20 inches with a peak intensity of 2.77 inches per hour while the rainfall on August 9 was only 0.56 inch with a peak intensity of 2.28 inches per hour. There was a period of five days of dry weather preceding the June 13 overflow event and twelve days of dry weather preceding August 9.

Waste Loads versus Rainfall

Figure 58 and Figure 59 show the apparent relationship between the BOD and suspended solids discharged from the three combined sewer districts sampled and total rainfall per storm. These figures have been plotted using the data from the five complete overflow events sampled during the study period. These relationships have been developed from five overflow occurrences and do not consider many other factors that may influence the waste loads from combined sewer overflows. However, the graphs definitely indicate a trend.

In addition to the relationship between rainfall and BOD, the number of days of dry weather preceding overflow has been incorporated into the rainfall-BOD and rainfall-suspended solids relationships and is shown on Figures 58 and 59. The number of days between rainfall appears to have a greater influence on the BOD than on the suspended solids.

Effect of Overflows on River Water Quality

One of the major objectives of this study is to determine what effect the combined sewer overflows have upon the water quality of the Sandusky River. Samples were taken at selected locations upstream, intown, and downstream during dry and wet weather along with visual observations of the condition of the river before, during and after overflows. Also, an aquatic biology survey of the river, which is presented elsewhere in the report, was made to determine the overflow effects on life forms in the river.

Table 17 very clearly shows the pollutional effects of the combined sewer overflows on the quality of the Sandusky River. The BOD concentration at the first bridge downstream (3/4 mile below treatment plant) is more than doubled by the overflows and the suspended solids have quadrupled while the total coliform count has increased ten-fold. The August 9 overflow event increased the BOD concentration at the first bridge downstream, from 11 mg/l with a river flow of nine cfs to 51 mg/l with a flow of 130 cfs. This is a BOD rate increase from 530 pounds per day before overflow, which is equal to the effluent load from the treatment plant, to 35,500 pounds per day during overflow.

Figure 46 presents a typical dissolved oxygen profile of the river during times of low flow (10 cfs or less). Normally, due to the wastewater treatment plant's effluent, the dissolved oxygen of the river below the wastewater treatment plant is extremely low for about five to seven miles before the river starts to recover.

Figure 47 compares dissolved oxygen profiles of the river during wet and dry weather. The graph shows that the combined sewer overflows tend to lengthen time of recovery for the river.

The assimilation capacity of a river is defined in this report as the BOD waste load that the river is capable of treating by self-purification processes and not depress the dissolved oxygen concentration of the river below 4 mg/l. Laboratory analyses and waste load calculations of selected river samples have shown that the assimilation capacity of the Sandusky River at Bucyrus is approximately 25 pounds of BOD per day per cfs at low flow (less than 10 cfs). This is a population equivalent of 150 per cfs. The calculations and analyses also indicate that the assimilation capacity of the river increases with flow and temperature.

The quantity of nitrate nitrogen discharged into the river from combined sewer overflows and the wastewater treatment plant effluent is negligible compared to the amount of nitrate nitrogen contributed to the river from rural runoff. The annual nitrate nitrogen load discharged to the river by overflows is about 12,200 pounds as NO_3 based on an annual overflow volume of approximately 365 million gallons and an average NO_3 concentration of 4.0 mg/l. The wastewater treatment plant contributes about 3,500 pounds per year based on a flow of 2.2 MGD and NO_3 of 0.5 mg/l. This gives a total of 15,700 pounds of NO_3 per year. In contrast, on April 19, 1969, approximately 136,000 pounds of nitrate nitrogen (NO_3) passed the upstream gage and on May 19, 1969, approximately 192,000 pounds of NO_3 passed the upstream gage. These large amounts occurred during times of high river flow in the spring and early summer.

The amount of total phosphates discharged into the river by combined sewer overflows and the wastewater treatment plant effluent is significant when compared to the total phosphates contributed by rural runoff. The wastewater treatment plant effluent discharges into the river about 160,000 pounds of PO_4 per year while the combined sewer overflows discharge about 30,000 pounds of PO_4 per year. The phosphates from the overflows are based on an annual overflow volume of 365 million gallons and an average PO_4 concentration of 10.0 mg/l.

On April 19, 1969, approximately 5,600 pounds of PO_4 passed the upstream gage and on May 19, 1969, about 34,600 pounds of PO_4 came off the upstream drainage area. Assuming an average PO_4 concentration of 0.7 mg/l and using the average river flow of 80 cfs there is about 110,000 pounds of PO_4 passing the upstream gage per year.

Therefore, the wastewater treatment plant effluent discharges about 50 percent of the total phosphates in the river while the combined sewer overflows contribute about 10 percent of the total phosphates.

SECTION XVIII

GENERAL DESIGN CONDITIONS

Certain basic design conditions must be established before any alternate solutions can be evaluated for the combined sewer system. These conditions include the stream water quality which must be protected; the design storms that result in waste discharges from which the stream must be protected; the design storms corresponding overflow volume, peak rates of overflow and the average maximum waste loads.

The stream water quality for the Sandusky River, downstream from Bucyrus was established by the Ohio Water Pollution Control Board as (1) Minimum Conditions Applicable to All Waters At All Places and At All Times and (2) Aquatic Life "A", which have the following criteria:

(1) MINIMUM CONDITIONS APPLICABLE TO ALL WATERS AT ALL PLACES AND AT ALL TIMES

1. Free from substances attributable to municipal, industrial or other discharge that will settle to form putrescent or otherwise objectionable sludge deposits;

2. Free from floating debris, oil, scum and other floating materials attributable to municipal, industrial or other discharges in amounts sufficient to be unsightly or deleterious;

3. Free from materials attributable to municipal, industrial, or other discharges producing color, odor or other conditions in such degree as to create a nuisance;

4. Free from substances attributable to municipal, industrial or other discharges in concentrations or combinations which are toxic or harmful to human, animal or aquatic life.

(2) AQUATIC LIFE "A"

The following criteria are for evaluation of conditions for the maintenance of a well balanced warm-water fish population at any point in the stream except for areas immediately adjacent to outfalls. In such areas cognizance will be given to opportunities for the admixture of effluents with stream water:

1. Dissolved oxygen: Not less than 5.0 mg/l during at least 16 hours of any 24-hour period, nor less than 3.0 mg/l at any time;

2. pH: No values below 5.0 nor above 9.0 and daily average (or median) values preferably between 6.5 and 8.5;

3. Temperature: Not to exceed 93° F. at any time during the months of May through November, and not to exceed 73° F. at any time during the months of December through April;

4. Toxic substances: Not to exceed one-tenth of the 48-hour median tolerance limit, except that other limiting concentrations may be used in specific cases when justified on the basis of available evidence and approved by the appropriate regulatory agency.

Since the Sandusky River through Bucyrus is available for body contact use and fishing, some consideration should be given to recreational uses which require, according to the Ohio Water Pollution Control Board the following:

WATERS FOR RECREATIONAL USES

The following criterion is for evaluation of conditions at any point in waters designed to be used for recreational purposes, including such water-contact activities as swimming and water skiing:

> Bacteria: Coliform group not to exceed 1,000 per 100 ml as a monthly average value (either MPN or MF count); nor exceed this number in more than 20 percent of the samples examined during any month; nor exceed 2,400 per 100 ml (MPN or MF count) on any day.

Design Storms

After consideration of the general conditions which prevail during various rainfall patterns, two design storms were selected -- a two-year, one-hour storm for the peak overflow rate, a one-year, 24-hour storm for the total volume of overflow. The one-hour storm is the spring, summer and fall type of thunderstorm, with very high intensities and short duration. The amount and intensity of rainfall during thunderstorms varies greatly over the drainage basin. The one-year, 24-hour storm resembles a more generalized rainfall. The intensities will not vary greatly during the 24 hours and over the drainage basin. Both types of storms are most likely to occur during the summer months of June, July, August, and September. (See Table 26)

Protecting the river from the peak overflows of a two-year, one-hour frequency rainfall is a logical choice since this rate of flow is the maximum capacity of most of the trunk sewers of the combined sewer system. Also very few thunderstorms will last longer than one hour and any additional rainfall after one hour will not add significant peak flow.

The probability of the two-year, one-hour storm occurring is the greatest during the summer months when the flow in the river is the lowest. Dilution water is usually not available from the upstream drainage basin during an overflow from this type of storm.

The volume of overflow from the one-year, 24-hour frequency rainfall was selected, in addition to the peak rate of overflow from the two-year, one-hour storm, so that the river would be completely protected from pollution due to combined sewer overflows for a return period of one year.

Peak Rate of Overflow

The rainfall for a two-year, one-hour storm was read from the intensity duration curves at twenty minute intervals. (See Figure 15) The estimated total rainfall equals 1.23 inches. The three 20-minute rainfalls were arranged to correspond to the storm pattern given in the ASCE Sewer Design Manual,[2] which the peak intensity occurs at three-eighths of the total storm time. This was approximated by placing the second highest intensity rainfall in the first twenty minutes of the storm, with the highest intensity and lowest intensity intervals following.

The peak flows for Numbers 8, 17, and 23 sewer districts were computed, using the relationships developed in the modified hydrograph method section. A hydrograph for each overflow was derived for each of the 20-minute rainfalls. No antecedent rainfall conditions were assumed. Each hydrograph was then lagged behind the start of its corresponding rain by the amount given in Table 19. The sum of all three hydrographs for each overflow equaled the hydrograph for the total storm. (See Figures 60, 61, and 62)

A review of the composite hydrographs in Figure 61 and 62 showed that the peak overflow rates for Numbers 17 and 23 overflows were limited by the maximum capacities of their sewer systems. Therefore revised hydrographs were drawn for these two overflows. These hydrographs have the same runoff volume as the original hydrographs. The peak of the hydrograph was assumed delayed until the sewer had sufficient capacity. The shape of the hydrograph was defined by assuming the hydrograph had the same recession curve as the original hydrograph, only delayed by the volume of the peak.

Since Numbers 8, 17, and 23 sewer districts were the only districts for which unit hydrographs had been determined, the following assumptions were made for the remaining 21 districts:

1. All hydrographs have the basic shape of a triangle.

2. The hydrograph begins ten minutes after the start of the rainfall.

3. The peak occurs five minutes following the end of the significant rainfall or 45 minutes after start of rainfall.

4. The hydrograph will end at a time equal to the time of concentration following the end of the rainfall.

5. The total runoff volumes are equal to the following percentages of rainfall:

Semi-developed	20%
Suburban-Residential	25%
Residential	30%
Commercial	50%

These assumptions are based on the characteristics of the hydrographs for Numbers 8, 17, and 23 sewer districts. The runoff percentages are based on the classification system given in Table 7.

The peak overflow rates and overflow volumes for a two-year, one-hour storm were computed for the remaining 21 sewer districts using the above assumptions. (See Table 27) The peak overflow rates of 13 out of the 24 sewer districts were limited by the maximum sewer capacities, and the peak rates of two additional districts were equal to the maximum sewer capacities. The hydrographs for these districts were adjusted by assuming the hydrographs had the same recession curves as the original hydrographs, only delayed by the volumes of the peaks.

<u>Volume of Overflow</u>

For the one-year, 24-hour storm, it was not necessary to plot a hydrograph for each twenty minutes of rainfall for Numbers 8, 17, and 23 sewer districts. Instead the following runoff percentages were assigned the three districts:

Number 8 Sewer District	20%
Number 17 Sewer District	28%
Number 23 Sewer District	25%

These runoff percentages are based on the runoff formulas developed by the "Modified Hydrograph Method".

The runoff percentages given in assumption five under Peak Rate of Overflows were used to compute the runoff from the one-year, 24-hour storm for the remaining districts. The volumes of runoff obtained for these districts and Numbers 8, 17, and 23 sewer districts are given in Table 28. A one-year, 24-hour storm has a total rainfall depth of 2.3 inches. Antecedent rainfall conditions were assumed.

<u>Design Waste Loads</u>

The waste loads discharged from the two design storms, previously presented, have been calculated for two different conditions, the average and the maximum.

TABLE 26

PROBABILITY OF THE DESIGN STORMS

	2 Year, 1 Hour Storm		1 Year, 24 Hour Storm	
Month	Probability of Occurring %*	Order of Magnitude	Probability of Occurring %*	Order of Magnitude
January	0	11	5	11
February	0.5	9	4	12
March	1	8	11	4
April	1	6	8	7
May	2	5	8	6
June	9	3	11	3
July	15	1	15	1
August	13	2	12	2
September	7	4	8	5
October	1	7	7	8
November	0.5	10	6	9
December	0	12	5	10
	50		100	

*Reference: Rainfall Frequency Atlas of the United States, Technical Paper No. 40, U. S. Department of Agriculture, pages 59-61.

TABLE 27

OVERFLOW PEAKS AND VOLUMES
FOR THE
2 YEAR, 1 HOUR STORM

Sewer District No.	Overflow Volume % Rainfalls	Overflow Volume Depth Inches	Overflow Volume 1000 c.f.	Overflow Peak cfs
1	20	0.25	65	23+
2	30	0.37	3	1
3	30	0.37	26	5+
4	30	0.37	151	42+
5	40	0.48	57	21
6	30	0.37	28	4+
7	40	0.48	6	1+
8	*	0.20	131	49
9	30	0.37	4	2
10	40	0.48	13	5
11	50	0.62	14	5
12	30	0.37	56	7+
13	35	0.43	14	3+
14	30	0.37	94	29+
15	35	0.43	17	6
16	40	0.48	9	3+
17	*	0.33	544	140+
18	25	0.31	6	2
19	30	0.37	33	6+
20	25	0.31	14	5
21	25	0.31	9	3
22	25	0.31	81	12+
23	*	0.29	397	70+
24	20	0.25	18	7
TOTAL			1,790	

= 13.4 Million Gallons

* Overflow volume computed using formulas found on page 86.
+ Peak adjusted to Maximum Sewer Capacity

TABLE 28

OVERFLOW VOLUMES
FOR THE
1 YEAR, 24 HOUR STORM

Sewer District No.	Area Acres	Overflow Volume % Rainfalls	Overflow Volume Depth Inches	Overflow Volume 1000 c.f.
1	73.3	20	0.46	122
2	2.5	30	0.69	6
3	19.4	30	0.69	49
4	113	30	0.69	282
5	32.1	40	0.92	107
6	21.0	30	0.69	53
7	3.2	40	0.92	11
8	178.9	20	0.46	299
9	3.0	30	0.69	8
10	7.1	40	0.92	24
11	6.2	50	1.15	26
12	41.9	30	0.69	105
13	8.8	35	0.80	26
14	70.2	30	0.69	176
15	10.8	35	0.80	31
16	5.0	40	0.92	17
17	452.5	28	0.64	1,050
18	5.7	25	0.58	12
19	24.5	30	0.69	61
20	12.1	25	0.58	25
21	7.8	25	0.58	16
22	72.4	25	0.58	152
23	377.8	25	0.58	794
24	20.7	20	0.46	35
TOTAL				3,487

= 26 Million Gallons

The average waste loads for the design storms have been calculated using the average BOD and suspended solids concentrations of all of the overflow samples analyzed and as summarized in Table 15. The average BOD is 125 mg/l and the average suspended solids is 480 mg/l. These concentrations apply to the first flush out which occurs in approximately two to three hours after start of overflow. BOD and suspended solids concentrations of 20 mg/l and 150 mg/l, respectively, are used as averages for overflow volumes that occur after three hours duration.

The average BOD and suspended solids waste loads for the two-year, one-hour design storm, which has an overflow volume of 13.4 million gallons from all 24 sewer districts, are 14,000 pounds and 53,500 pounds, respectively. For the one-year, 24-hour design storm, which has an overflow volume of 26 million gallons from all 24 sewer districts, the BOD load is 14,000 pounds and the suspended solids load is 62,500 pounds.

The maximum waste loads that could be expected from the two design storms have been determined from envelope curves that were developed from the results of the BOD and suspended solids data. These envelope curves, shown on Figures 23 through 28, indicate the maximum BOD and suspended solids concentrations versus time after start of overflow that could be expected. The amount of BOD and suspended solids discharged from each of the three selected sewer districts is determined by superimposing the developed envelope curves over the design storm hydrographs and matching flows with concentrations.

The computed maximum waste loads for the two-year, one-hour design storm are as follows:

Sewer District	Pounds of BOD	Pounds of Suspended Solids
No. 8	1,500	10,600
No. 17	6,200	30,000
No. 23	3,700	17,500

The above waste loads give an average BOD of 1,100 pounds per 100 acres and suspended solids of 5,800 pounds per 100 acres. Expanding this to include the entire sewered area of Bucyrus gives a design BOD waste load of 18,000 pounds and a suspended solids load of 90,000 pounds.

The maximum waste loads for the one-year, 24-hour design storm were computed by essentially the same procedure. The maximum BOD expected is 17,100 pounds and the maximum suspended solids expected is 76,000 pounds.

Table 29 presents a summary of the average and maximum waste loads for the design storms.

TABLE 29

DESIGN STORMS AND WASTE LOADS

Design Storms	Total Rainfall Inches	Overflow Volume Million Gallons	BOD lbs.		Suspended Solids lbs.	
			Average	Maximum	Average	Maximum
2-yr., 1 hr.	1.23	13.4	14,000	18,000	53,500	90,000
1-yr., 24 hr.	2.3	26	14,000	17,100	68,000	76,000

SECTION XIX

ALTERNATE SOLUTIONS

This section of the report will present various methods of abatement and/or control of the pollution from combined sewer overflows. The degree of protection, advantages and disadvantages along with the estimated costs are presented for each method.

A. Complete Separation of Sanitary Wastewater and Storm Water

Complete separation of sanitary wastewater and storm water has historically been prime solution for pollution due to combined sewer systems. However, there are disadvantages as well as advantages of complete separation as presented below.

(1) Advantages of Separate Sewer Systems

Separation of sanitary wastewater from storm water permits complete treatment of all sanitary wastewater before being discharged into the receiving stream. The wastewater treatment plant facilities are required to process only the dry weather flow. Elimination of storm water from the treatment plant will enable the plant to operate at a higher efficiency.

Many of the bacteria and other organisms responsible for intestinal and other diseases are found in sanitary wastewater. The separate sewer system delivers the sanitary wastewater to the treatment plant where these organisms can be controlled.

The separation of the sanitary wastewater from the storm water will eliminate basement flooding during times when the combined sewers are inadequate to carry the storm runoff.

(2) Disadvantages of Separate Sewer Systems

Many cities such as Washington, D.C., New York, Philadelphia, Detroit, Milwaukee, Minneapolis and Chicago have found, through engineering studies, that complete separation is usually not economically feasible. The cost estimates for separation of the Bucyrus sewer system, presented in this section of the report, show that the cost is indeed very high.

In addition to the high cost, there are other disadvantages of separation. Sewer separation only partially reduces the pollutional effects from combined sewer overflows. Recent studies, including this report, have shown that storm water runoff from urban areas contains a significant amount of contaminates harmful to stream water quality. The degree of pollution of storm water varies from that of very dilute sewage to strong sewage.

There is also an extreme inconvenience factor to the populace involved when converting to a separate system. Streets are torn up for years, utility services are disrupted periodically, traffic rerouted, etc. The complete separation of the individual house services, roof drainage lines and basement drain tile is almost impossible.

(3) Cost of Sewer Separation

Separation of combined sewer systems can be accomplished in two ways. One, the existing combined sewer system can be used as a storm sewer and a new sanitary sewer system constructed. Two, the existing sewer system can be used as a sanitary sewer and a new storm sewer system constructed. Both of these systems have been investigated and cost estimates prepared.

Constructing a new sanitary system involves paralleling the existing sewer system with sewer pipe sized for sanitary wastewater only. The new sewer would be a few feet deeper than the existing sewer so that the house laterals could be connected to the new pipe. A cross section of the system is shown in Figure 64. This system would include about 208,000 feet of eight inch and ten inch pipe and about 14,000 feet of trunk sewer. Also, about 3,700 house laterals would have to be disconnected from the existing sewer system and relaid to discharge into the new sanitary sewer. The existing interceptor sewer, paralleling the Sandusky River, would be used to convey the sanitary wastewater to the wastewater treatment plant. The existing connector pipes between the interceptor and the existing sewer system would be plugged so that the storm water runoff would go directly to the river at the various overflows.

The estimated cost for a new sanitary system is $9,300,000.00.

Since the existing combined sewer system is designed to handle a one-in-two year storm, the construction of a new storm sewer system would consist of paralleling the existing system with approximately the same size pipe as the existing pipe. The new pipe would be at a more shallow depth since the existing sewer was constructed deep enough to catch sanitary wastewater. All of the existing storm inlets would be disconnected from the existing sewer and connected to the new storm sewer. Also, on some of the larger lines there will be house services that would be cut and these would have to be relaid to the existing sewer. A cross section of this method is shown in Figure 64.

The trunk lines of the new storm water system would discharge directly into the river. The sanitary wastewater would be carried by the existing sewer system and interceptor to the treatment plant.

The estimated cost for the new storm water system is $8,800,000.00.

B. Interceptor Sewer and Lagoon System

This method of abating pollution from combined sewer overflows proposes construction of an interceptor sewer to collect the large overflow volumes, and an aerated lagoon system to treat the waste loads from the overflows. A pump station is required to pump the overflows to the lagoon system. (See Figures 65 and 66)

(1) Gravity Interceptor Sewer

The proposed gravity interceptor sewer would parallel the existing interceptor along the Sandusky River and would terminate near the present junction manhole with the existing northwest trunk sewer.

The primary concern in the design of the interceptor was determining the peak capacity the pipe must handle at every point in the system. This required routing each of the design storms overflow hydrographs down the interceptor and determining their time base relationship to each other. A graphical addition of the hydrographs was used to determine which combination gave the highest peak value at each section of the interceptor. Each section of pipe was designed for the maximum peak flow. The following pipe sizes and peak flows were obtained:

Location	Pipe Sizes inches	Peak Flow cfs
Below #1 Overflow	36	23
Below #8 Overflow	72	150
Below #17 Overflow	108	348
Below #23 Overflow	120	435

From this analysis it was found that there were 39 minutes of travel time between Numbers 1 and 24 overflows. Due to the longer travel times in Numbers 17 and 23 trunk sewers, which delays the peaks, all three overflow peaks coincide. Number 8 overflow takes eleven minutes to travel to Number 17 overflow, and an additional nine minutes to travel to Number 23 overflow. The peak rate of flow in the interceptor below Number 23 overflow was equal to 435 cfs, of which 60 percent was from the three major overflows.

(2) Interceptor Sewer Using Holding Tanks

This is a modification of the proposed gravity interceptor presented above in item (1) in which the size of the interceptor is reduced by using holding tanks located at Numbers 8, 17, and 23 overflows. These tanks were designed to intercept the top one-half of the peak flows and release them at an even rate. The volumes of the three tanks were computed by determining the volumes in the top one-half of the hydrographs for the overflows below the preceding tank, and are equal to 1.1, 1.9, and 1.4 million gallons for Numbers 8, 17, and 23 overflows, respectively.

Completely enclosed, concrete tanks were used for the cost estimate. The tanks are fifteen feet deep, of which the top ten feet is a surge tank, operating by gravity, and the bottom five feet is pumped.

The interceptor for this method was designed by the same methods used for the gravity interceptor system. The following pipe sizes and flows were obtained:

Location	Pipe Sizes inches	Peak Flow cfs
Below #1 Overflow	36	23
Below #8 Overflow	60	76
Below #17 Overflow	84	186
Below #23 Overflow	96	235

(3) Pump Station

A 215 MGD (333 cfs) pump station with a 1.0 million gallon wet well is proposed. The storage volume of the proposed gravity interceptor sewer is also required during the design storm peak flow of 435 cfs.

The capacity of the pump station for the interceptor sewer using holding tanks would be 150 MGD with a 0.5 million gallon wet well.

(4) Aerated Lagoon

A system of aerated and non-aerated lagoons is proposed as a method of treatment for the combined sewer overflow wastewaters. The proposed facilities could also be utilized as tertiary treatment for the effluent from the existing city activated sludge treatment plant. The lagoon system would consist of structures of earthen embankments to form a retention basin with several sections, as shown in Figures 65 and 66. This facility would be similar in construction to existing reservoirs which are used throughout northwest Ohio to store water for municipal water supply. However, using this type of facility for wastewater retention and treatment would require consideration of need for changing water levels, installation of mechanical aerators and removal of accumulated solids. The design requirements to prevent pollution of the stream are discussed in preceding sections of the report.

The design of the system will provide for biological oxidation of the combined sewer overflows and would provide for continued biological oxidation of the effluent from the sewage treatment plant. Three main design parameters are of importance. These are: detention time, the biological oxidation rate (designated as the "k" rate of the wastewaters being treated) and, the air supply. However, the "k" rate essentially establishes the requirements of both the detention time and the air supply.

The rate of reaction constant of the degradation of organic matter, commonly called the "k" rate, varies with the type of organic matter and the state of oxidation which exists in the wastewater, at the time treatment is begun. The "k" rate for the raw wastewater is greatest and will decrease with advance stages of treatment. The "k" rate for normal domestic wastes, as received in a wastewater treatment plant, is usually about 0.1. However, it may vary from half this value to several times this value. The "k" rate found by Havens & Emerson of combined sewer overflows was approximately 0.1. The "k" rate for activated sludge effluents, as found by Havens and Emerson, was 0.031.[9] In a study of the effluents from extended aeration plants by the Ohio Department of Health, the "k" rates were found to be 0.013.[10] In the Ohio Health Department study, the flows were found to be about 25 percent of design, so the detention times were on the order of four days.

The "k" rate also affects the quantity of wastewater that may be discharged into a given stream. A wastewater with a high "k" rate will have a greater effect on a stream than a wastewater with a lower "k" rate. Just as a rapid stream will assimilate more wastewater than a slow stream (with ponding), a given stream will assimilate a higher ultimate BOD load as the "k" rate becomes less. Assimilation of wastewater of BOD load is used here to mean that the dissolved oxygen will not be depressed below a desired level.

The determination of the possible variations of the "k" rate of combined sewer overflows or treated overflows is beyond the scope of this study. Also the variation of "k" rate in partially treated overflow wastewaters or the possible increase in assimilation capacity of a given stream as the "k" rate decreases with increased degrees of treatment is beyond the scope of this study.

From data obtained during this study the range of permissible BOD loading has been determined. The dissolved oxygen in the Sandusky River below Bucyrus will be maintained at the desired level of four mg/l if the BOD wastewater load does not exceed 25 pounds per day per cfs of stream flow. This is equivalent to the waste load of a population of 150 persons.

The rainfall-runoff pattern results in an average of about 1.0 MGD of wastewater, with an average BOD of 125 mg/l and a maximum expected BOD of 170 mg/l. The BOD load will be treated or controlled by two methods. One is by settling and biological oxidation. Part of the BOD will be removed by settling and, in this design, has been assumed to be 50 percent. Biological oxidation of the BOD will follow under aerobic conditions using mechanical aerators.

A "k" rate of 0.10 has been assumed for the first stage treatment or first five days of detention and a "k" rate of 0.05 for the remaining 15 days of detention. The percent of BOD remaining after a given number of days will be according to the equation: $\% = 100-(1-10^{-kt})\ 100$ (when $k = 0.1$ or 0.05 and t = number of days). The degree of treatment of the wastewater from combined sewer overflows in the lagoon system is estimated to exceed 95 percent after 20 days.

A second method of controlling BOD will be to discharge partially treated wastewater to the stream. During some periods of rainfall when enough rainfall has occurred to fill the lagoon, the stream flow will also have increased. When prolonged rainfall occurs such as a one-year, 24-hour storm of 2.3 inches, the quantity of urban runoff will be sufficient to fill the proposed lagoon. However, as a result of such a rainfall the stream flow will increase due to upstream drainage runoff. Without tertiary treatment for the wastewater treatment effluent only when the upstream runoff exceeds 20 cfs can an additional BOD load be discharged at the rate of 25 pounds per day per cfs. The wastewater collected may be discharged from the lagoon after partial treatment to increased stream flow.

If tertiary treatment is provided for the wastewater treatment plant effluent then either this effluent or the treated combined sewer overflow may be discharged to the stream at any flow.

The average runoff from the urban area of Bucyrus is about 1.0 MGD. The lagoon size must allow for a variation in the peak distribution. A lagoon designed for an average detention time of 20 days or 20 million gallons and in addition a volume equal to the two-year, one-hour storm will provide the required variation. This requires a volume of 33 million gallons. This will protect the stream from any overflows of any storm less than the two-year, one-hour storm. This protection does not require any dilution from the upstream drainage area. This volume is about 27 percent greater than that required for the runoff from a one-year, 24-hour storm.

A detailed analysis of the past ten-year precipitation records for the four critical months -- July, August, September, and October -- show that the average monthly rainfall plus the rainfall from the two-year, one-hour storm is greater than the one year maximum monthly rainfall. The records also show that a total monthly rainfall equal to the average monthly rainfall plus the rainfall from a one-year, 24-hour storm would occur only once in eight years. Therefore, a lagoon with a storage volume of 33 million gallons will protect the stream from the two design storms and from the one-year maximum monthly rainfall.

The proposed lagoon will provide for greater protection of the stream if the available dilution water from the upstream drainage area is used. Storms of longer than 24-hour duration, or when rainfall is such that the runoff from the urban area is greater than the average of one MGD plus the two-year, one-hour storm, will produce upstream runoff. One example of this was a storm on September 16-17, 1969, during which 2.3 inches fell in 19 hours. This resulted in a peak upstream flow of 100 cfs and a total stream volume of 65 million gallons over a three-day period. In this case about 3,000 pounds of BOD could have been released to the stream with no detrimental effects. With a BOD concentration of approximately 75 mg/l or less (having received minimum one-day treatment) at least five million gallons of partially treated overflow could have been discharged with the higher stream flows. In actual operation, the BOD may be as low as 50 mg/l which would allow discharging about 7.5 million gallons of partially treated wastes.

A detailed mass diagarm of the past ten years record of stream flow and rainfall was plotted to determine the required lagoon size using the assimilation capacity of the upstream dilution water in conjunction with the treatment capability of the lagoon. The analysis indicated that a 22 million gallon lagoon will protect the stream from pollution by combined sewer overflows if the treatment capacity of the upstream dilution water is used.

At the present time, there is very little factual information available verifying assumed treatment efficiencies. Therefore, until it is demonstrated that the smaller lagoon plus the upstream dilution water will provide the necessary protection, the larger 33 million gallon storage volume is recommended.

The lagoon may be located across the river from the existing wastewater treatment plant. Four cells are proposed, each one 257 feet by 458 feet from centerline of berm to centerline of berm, and cover an area of 15 acres. The total depth is 22 feet, of which the top four feet is freeboard and the bottom five feet is permanent pool. The earth embankment has 2 1/2:1 side slope and a ten-feet wide berm. Ten and one-half feet of the lagoon are below the original ground level.

The lagoon has a total storage capacity of 37.75 million gallons. The permanent pool would allow approximately three days detention time to provide tertiary treatment for the existing wastewater treatment plant effluent. The lagoon dimensions were based on the minimum amount of earth embankment per volume.

Accumulation of solids in the lagoon are estimated to be about two percent per year. The removal of the accumulated solids will be required every five to eight years to maintain an overall efficiency of system of 90 percent.

The cost estimates of the interceptor sewer and lagoon system are as follows:

Gravity Interceptor, Pump Station and Aerated Lagoon

1. Gravity Interceptor		$3,600,000
2. Pump Station		1,000,000
3. Aerated Lagoon		620,000
	TOTAL COST	$5,220,000

Holding Tanks on System, Gravity Interceptor, Pump Station, and Aerated Lagoon

1. Holding Tanks		$1,500,000
2. Gravity Interceptor		3,000,000
3. Pump Station		800,000
4. Aerated Lagoon		560,000
	TOTAL COST	$5,860,000

C. Stream Flow Augmentation

Flow augmentation, as used in this report, is a method of controlling pollution from combined sewer overflows by providing sufficient dilution water from storage impoundments to maintain a desired concentration of dissolved oxygen downstream during overflows of wastewater from the combined sewers. This method of control includes an upground storage reservoir, a low head dam to capture the flow of water in the river, a pump station to deliver river water to the reservoir, a discharge channel to release stored water to river, and a system of rain gages, river flow indicators and controls to release the required amount of dilution water into the river. When wastewater overflows occur, sufficient dilution water must be released from the reservoir so that the combined assimilation capacity of the dilution water and the river flow can maintain a desired level of dissolved oxygen.

The volume of stored dilution water required at Bucyrus is 4,500 million gallons. The reservoir would have an average depth of 30 feet, which includes four feet of freeboard and a two-foot conservation pool. The area required for the reservoir is about 480 acres. See Figure 67 for schematic and typical section of flow augmentation facilities. The volume of stored dilution water was based on an average combined sewer overflow volume of 380 million gallons per year or approximately 1.0 million gallons per day. The combined sewer overflow has an average BOD concentration of 125 mg/l which amounts to approximately 1,000 pounds of BOD per day. The water quality data on the Sandusky River, downstream from Bucyrus, indicates that the river's assimilation capacity at low flows (less than 10 cfs) is approximate 25 pounds of BOD per day per cfs. Using this value for the river's waste load assimilation capacity, the amount of dilution water required, in addition to the probable river flow, was calculated. The probable river flows were taken from the flow duration figures presented in Table 2.

The pump station capacity, based on the flow duration curves, is 45 MGD or 70 cfs.

The preliminary cost estimate for this method of treatment is $5,000,000.00.

The effectiveness of this method of treating combined sewer overflows depends upon the ability of the system to deliver the dilution water at the time of the overflows. The urban area's response to rainfall is almost immediate with an average time of 15 minutes from start of rain to start of overflow. This means that the dilution water must reach the overflow points immediately after start of rainfall.

The nearest site for a 4,500 million gallon reservoir (about one mile square) at Bucyrus is located about five miles upstream, adjacent to the existing water supply reservoirs. The dilution water from this location would have an average travel time to the overflow points of approximately five hours. To deliver the required amount of dilution water to the overflow, in time to be effective, the dilution water would have to be released from the reservoir about five hours before start of rainfall. Obviously trying to determine the start of rainfall five hours before it occurs is impractical and, in most cases, impossible. Therefore, flow augmentation as a method of treating combined sewer overflows is not feasible at Bucyrus.

D. Primary Treatment of Overflows

This method of controlling stream pollution from combined sewer overflows proposes providing primary treatment for the overflows. Primary treatment would include a gravity interceptor sewer to collect the overflows, grit chamber, settling tanks, chlorination facilities, anaerobic digester and sludge drying beds.

The primary treatment facilities have a design capacity capable of providing 1.5 hours of detention time for the two-year, one-hour design storm which discharges 13.4 million gallons of overflow in two hours. Five parallel settling tanks are proposed to provide for the variation in overflow volumes received.

Each tank is 70 feet x 250 feet by 20 feet deep. A chlorination contact tank 70 feet x 100 feet by 10 feet deep would follow each settling tank. An anaerobic digester is proposed to treat about 1,500 pounds per day of volatile suspended solids. The proposed sludge drying beds have an area of approximately 7,500 square feet.

Primary treatment of the overflows could be expected to remove 50 percent to 70 percent of the BOD and suspended solids waste load from the combined sewer overflows. Chlorination of the overflows would significantly reduce the bacteria discharge to the stream.

The cost for primary treatment of the overflows is estimated to be $8,810,000.00.

E. Chlorination of Overflows

Chlorination of overflows is proposed as a method for controlling the large number of bacteria discharged to the stream by combined sewer overflows. The literature survey has indicated that proper chlorination of the overflows will reduce the peak after-growth of coliforms in the stream to 10 percent to 30 percent of the coliforms that would develop if unchlorinated overflows were discharged to the stream.

The proposed chlorination facilities include three chlorine contact tanks located at Numbers 8, 17, and 23 overflow outlets adjacent to the Sandusky River, new interceptor sewers that would collect the overflows from the various overflow points and deliver the overflows to the contact tanks and chlorination facilities capable of providing a chlorine dosage of up to 40 mg/l. The size of the proposed contact tanks at Numbers 8, 17, and 23 overflows are 1.6 million gallons, 1.9 million gallons and 0.7 million gallons, respectively.

The estimated cost for chlorination of the overflows is $3,000,000.00.

F. Off-Stream Treatment

The basic alternate plan of pollution abatement from combined sewer overflows as presented and described herein proposes to intercept and treat the flow from 24 overflow points in Bucyrus thereby eliminating all discharges to the Sandusky River up to the design storms. This provides maximum protection to the Sandusky River both "in City" and downstream. Such protection is the equivalent of collecting and treating the waste water and the discharge from a storm sewer system in a community with separate sewer systems.

The cost of such complete abatement plans should not be related to the cost of physical separation without regard for the pollutional load created by separate storm sewer discharges.

Pollution abatement from combined sewers may be separated into two general categories: (1) Inner-City and Downstream Pollution Abatement, and (2) Downstream Pollution Abatement. The basic approach in this study has been Inner-City and Downstream Abatement. Should only downstream protection be considered at this time as the first phase of an overall pollution abatement plan, a considerable reduction in cost would result.

The same or a pumping station similar to that proposed to pump the flow from the interceptor sewer could be used to divert the flow in the Sandusky River to the lagoons for treatment during periods of combined sewer overflow. The same pump capacity as that proposed for the interceptor sewer without holding tanks (333 cfs) would be capable of diverting the entire river flow which occurs about 95 percent of the time. Pumping would only be accomplished during overflow periods and release from the lagoon would be accomplished during these pumping periods to satisfy the riparian owners water rights.

This concept of downstream water quality protection would not enhance the water quality in the river reach within the City, but it would result in a cost reduction of about $3,500,000 on the basic alternate plan of pollution abatement and about $7,000,000 on the cost of physical separation of the sewer systems. At some future date, the interceptor sewer could be constructed to collect the overflows and convey them to the pumping station and inner-city protection would be realized.

The reduction in initial cost would appear to justify that the downstream protection aspect be thoroughly evaluated by a demonstration project. The demonstration project should include monies for reducing or controlling the channel degradation within the city due to the combined sewer overflows. Channel projects could include regrading, reshaping or paving to reduce pools and low velocity areas where solids would otherwise tend to settle and become septic.

The estimated cost of the pumping station, lagoons and low head dam in the Sandusky River is $1,700,000.

TABLE 30

SUMMARY OF COST ESTIMATES
FOR
ALTERNATE SOLUTIONS

A. Complete Separation of Sanitary Wastewater and Storm Water			
(1) New Sanitary System			$9,300,000
(2) New Storm Water System			$8,800,000
B. Interceptor Sewer and Lagoon System			
(1) Gravity Interceptor, Pump Station and Aerated Lagoon			
	Gravity Interceptor	$3,600,000	
	Pump Station	1,000,000	
	Aerated Lagoon	620,000	
	TOTAL COST		$5,220,000
(2) Holding Tanks, Gravity Interceptor, Pump Station and Aerated Lagoon			
	Holding Tanks	$1,500,000	
	Gravity Interceptor	3,000,000	
	Pump Station	800,000	
	Aerated Lagoon	560,000	
	TOTAL COST		$5,860,000
C. Stream Flow Augmentation			$5,000,000
D. Primary Treatment of Overflows			$8,810,000
E. Chlorination of Overflows			$3,000,000
F. Off-Stream Treatment			$1,700,000

SECTION XX

PROCEDURES FOR EVALUATING SIMILAR SYSTEMS IN OTHER COMMUNITIES

The studies and data collection accomplished at Bucyrus and described herein are typical of the engineering effort necessary to develop evaluation studies and feasible solutions to pollution abatement from combined sewers. Each community's needs and methods of abatement are dependent on growth patterns, topography, capability of existing sewer system, receiving stream, availability of land, degree of protection required, etc.

Since complete separation of storm and sanitary sewers is presently an accepted method of correcting overflows from combined sewers, the total conversion of combined sewers to either storm or sanitary systems if present storm sewer capacity is inadequate may be the most feasible solution. Only a detailed analysis of the present combined system with current design storm conditions and future growth considered will provide the answer as to whether all or portions of the system should remain combined.

After the present combined sewer system has been analyzed and decisions made as to whether all or portions of the combined sewers are to remain as combined systems the points of combined sewer overflow will have been determined.

The local conditions must be studied and one or more of the following steps considered and evaluated:

1. Intercept the overflow at each overflow point by a gravity sewer or pumping station and force main for conveyance to point of treatment, or

2. Construct holding tanks to level out the peak flow from the combined sewer and convey same by gravity sewer or pump station and force main to point of treatment, or

3. Discharge settled effluent from holding tank into receiving stream if studies have shown that stream can assimilate such waste loads and comply with minimum water quality standards, or

4. Build treatment facilities designed to protect the minimum water quality in the stream for all overflow requiring treatment.

The following discussion outlines the procedures, using the results of the Bucyrus data, that can be used to determine and evaluate the above mentioned items. The degree to which the Bucyrus data and solutions can be applied to another community is directly related to the similarity of that community to Bucyrus.

Five basic questions must be answered: one, the peak rate of overflow; two, the total volume of overflow; three, the total waste load discharged from the combined sewers; four, what can be discharged into the receiving water; and five, what is the best method of collecting and treating the waste that cannot be discharged directly into the receiving water. The design storms and return frequencies must be selected with consideration of the five questions.

Analyze Existing Sewer System

The total sewered area must be divided into sewer districts and a detailed analysis made of each district including the trunk sewers. A detailed analysis must also be made on the overflow structures and interceptor sewer.

Often the original design notes or an existing sewer map can be used to determine the size, grade, and location of the existing combined sewers. A field check should be made to insure that no additional overflows have been installed or drainage areas added to the sewer system. Spot checks should be made in each drainage district to determine the type of land use. Field work necessary would depend on available information. The time of concentration must also be determined from the sewer travel time and time of over-land flow.

Select Design Storm and Return Frequencies

The capacity of the existing system must be given consideration in the selection of the design storm, since it controls or limits the maximum overflow rate possible from the sewers. The stream water quality desired and the tolerance of pollution must also be considered along with the minimum stream flows which may exist during periods of overflow. In many communities like Bucyrus, Ohio, there is essentially no stream flow available a large percentage of the time.

All states have established minimum stream standards for stream water. The State of Ohio has five classifications for streams which are based on the water usage. The volume of waste that can be discharged into the receiving water will depend on how much the stream or lake can assimilate without deleterious effects or violating the stream standards. This amount can be determined by using one of the stream purification forumulas. However, in most cases in areas similar to northwestern Ohio a waste load of about 25 pounds of BOD per day per cfs of stream flow at low flow conditions is the maximum permissible loading.

Protecting a stream or lake against every frequency of rainfall is uneconomical. There will be a design frequency for which an increase in the degree of protection cannot be justified. A two-year frequency rainfall is suggested for the design of any overflow collection or interception system and a one-year frequency rainfall for design of the volume of any treatment facilities.

The peak, volume, and quality of the combined sewer overflows are directly related to intensity and duration of the rainfalls. If a recording rain gage has not been located in the area, the total rainfall for various frequencies and durations can be obtained from the Rainfall Frequency Atlas of the United States, Technical Paper No. 40, U. S. Department of Agriculture. For the design of an overflow collection or interception system, a rainfall duration equal to or greater than the maximum time of concentration should be used. A minimum duration of one hour is suggested.

For treatment facilities with long detention times, such as a lagoon, the total volume will be directly related to the volume of rainfall expected during the detention period. One method of obtaining this volume of rainfall is to take a number of years of rainfall records for the area, determine the maximum rainfalls for the given detention time, (20 day period) rank time according to their order of magnitude, assign frequencies of occurrence, and then select the rainfall corresponding to the design frequency. A simpler but less accurate method of obtaining this rainfall is to use the total monthly rainfalls. The monthly rainfall for the design frequency can then be reduced to the detention time of the treatment facility by determining the maximum rainfall that fell in the design month and within the detention time.

Determine the Runoff From Design Storms

For any combined sewer system with multiple overflows, a complete sewer hydrograph is needed for each trunk sewer. Many cities may not be able to measure their major overflows to determine a unit hydrograph. Therefore, some assumptions for hydrograph shape must be made.

If a hydrograph method is already being used successfully for design in the city, it should also be used to determine the hydrographs at the combined sewer overflows. If not, either the Chicago Hydrograph Method, as described in the ASCE Sewer Design Manual, or the Modified Hydrograph Method, as described in this report, is recommended for design.

The Chicago Hydrograph Method may be used since the Chicago terrain and rainfall patterns are representative of a typical midwestern city. A set of curves is given for the peak rate of runoff per acre drained for various depression storages and ground slopes. To determine the total hydrograph shape, mass curves for lateral outflow from uniformly developed areas must be constructed. This requires a detailed analysis of the drainage characteristics of the sewer districts and the hydraulics of the sewer system.

A more simplified method of obtaining a complete runoff hydrograph is the Modified Hydrograph Method developed in this report. The assumptions and procedure are found under "General Design Conditions". The time of the start of overflow and the hydrograph shape were assumed based on the characteristics of the hydrographs for Number 8, 17, and 23 sewer districts. The total volume of runoff for each district was then computed using the runoff percentages and land use classifications developed earlier. These volumes of runoff were used to determine the peak of the hydrograph through the assumed hydrograph shape. For most of the sewer districts in Bucyrus, this peak exceeded the maximum sewer capacities.

A variation of the Modified Hydrograph Method is to use the rational formula for determining the peak of the runoff and through the assumed hydrograph shape determine the volume of runoff. This variation might be more accurate than that used for Bucyrus if coefficients for the rational formula are well defined for the area and the times of concentration do not exceed the length of significant rainfall. It should not be used if the peak rate of runoff exceeds the maximum sewer capacities of many of the sewer districts.

Determine Waste Loads from Design Storms

The characteristics of the waste from the combined sewer overflows of any city similar to Bucyrus will no doubt be similar to that found during this study. Both the BOD and suspended solids concentration reached a peak during the first hour of overflow and then decreased. Both reach a minimum concentration after several hours. For design purposes, an average BOD concentration of 125 mg/l and an average suspended solids concentration of 480 mg/l were assumed for the first three hours of overflow. Overflow after three hours will have an average BOD and suspended solids concentration of about 20 mg/l and 150 mg/l, respectively.

The wasteloads discharged during overflows minus the wasteload which may be discharged to the stream, equal the volume of flow and wasteload which must be collected and treated.

Method of Collection and Treatment

At Bucyrus a gravity interceptor was found to be the most economical method of collecting the combined sewer overflows and carrying the waste to a common point for treatment. A major pump station is required at Bucyrus. A detailed analysis should be made of the pipe sizes, grades, and locations to determine if there are any ways to minimize the cost. For example, if flow through the treatment facilities can be accomplished by gravity then the pump station may be eliminated. Another possibility is splitting the flow into more than one treatment facility and locating the facilities closer to the overflow points. For Bucyrus, the pump station capacity was reduced by one-fourth by using storage capacity of the gravity interceptor and the wet well to store the peak flow.

The capacity of the interceptor must be equal to the peak rate of runoff from a two-year frequency rainfall minus the rate of runoff that the river can assimilate. For example, the median flow in the river at Bucyrus during the summer months is approximately five cfs. The assimilation capacity of the river at this low flow is negligible and the interceptor was designed to handle all of the peak rate of flow from a two-year frequency storm. If some of the overflow wastes can be discharged into the stream, a check must be made of the travel times in the stream between overflow points. If the duration of overflow is greater than the travel times between overflows, the rates of overflow discharged into the stream must be reduced so that their sum is equal to the assimilation capacity of the stream.

Based on the analysis made for Bucyrus, the aerated lagoon is believed to be the most economical method of treating combined sewer overflows. A lagoon can provide the high degree of treatment necessary for the highly variable flows with a minimum of operation.

The volume of the lagoon must be equal to the volume of runoff from the one-year frequency 20-day rainfall minus the volume the river can assimilate. One method of operation is to discharge lagoon effluent continuously into the stream at a constant rate the stream can assimilate during low flow. The lagoon volume will then equal the volume of runoff from the one-year frequency, 20-day rainfall minus the volume discharged into the stream during this 20-day period. The other method of operation is to discharge into the stream from the lagoon at a rate determined by the flow in the stream. The total volume discharged during the 20-day period will then be directly proportional to the flow in the stream.

SECTION XXI

ACKNOWLEDGMENTS

We wish to acknowledge the assistance and cooperation received from the City of Bucyrus during the investigation and preparation of this report. We would especially like to acknowledge the assistance of Bernard C. Piper, Utilities Director and Gerald E. Staiger, Assistant Utilities Director.

The aquatic biology survey portion of this report was performed by Mr. Rendell Rhoades, Chairman of Biology, Ashland College, Ashland, Ohio.

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We wish to acknowledge the assistance and cooperation received from Mr. Howard Kenny, United States Weather Bureau, Port Columbus, Columbus, Ohio.

SECTION XXII

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SECTION XXIII

FIGURES

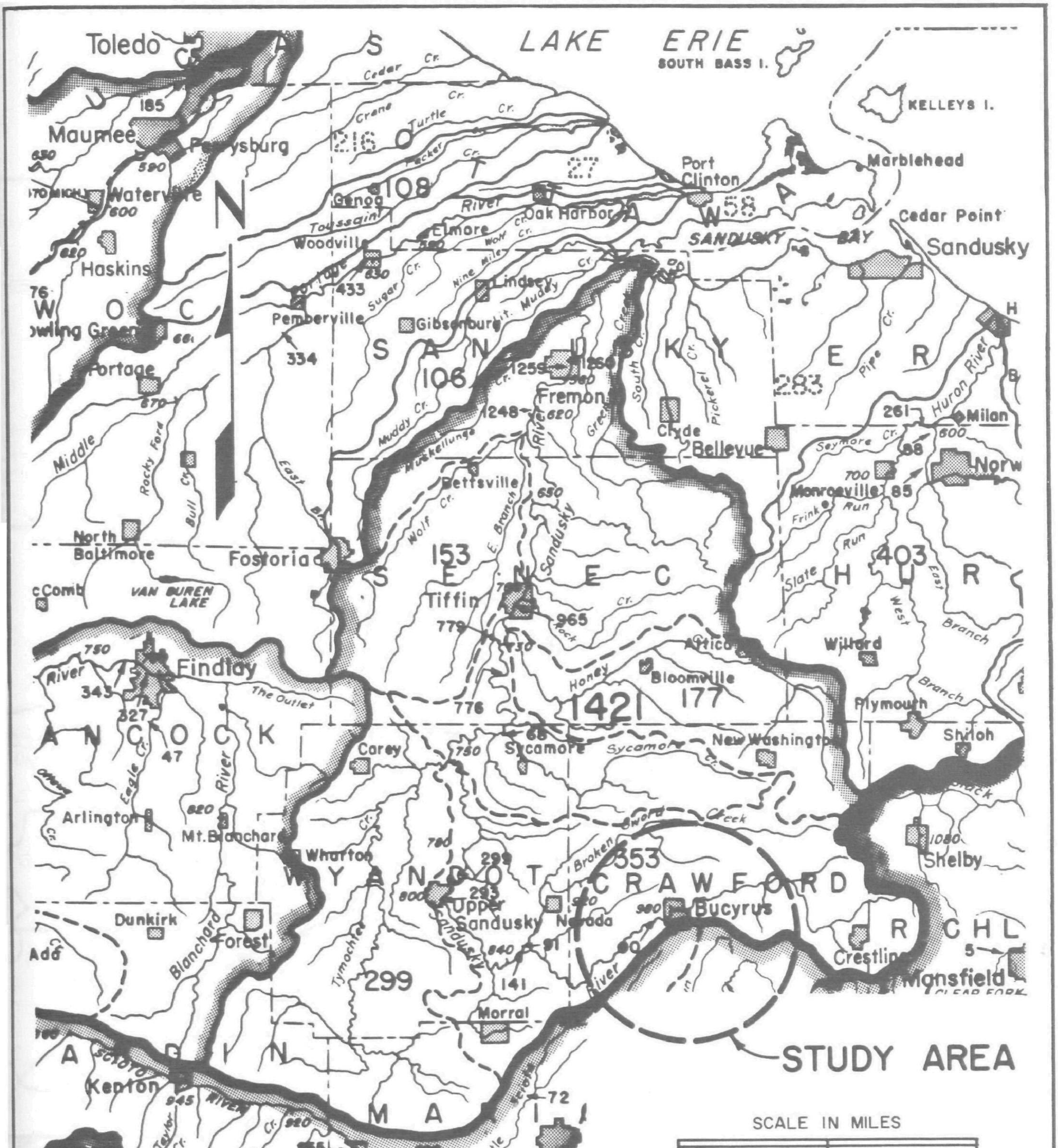


Figure No. 1

SANDUSKY RIVER

Drainage Area

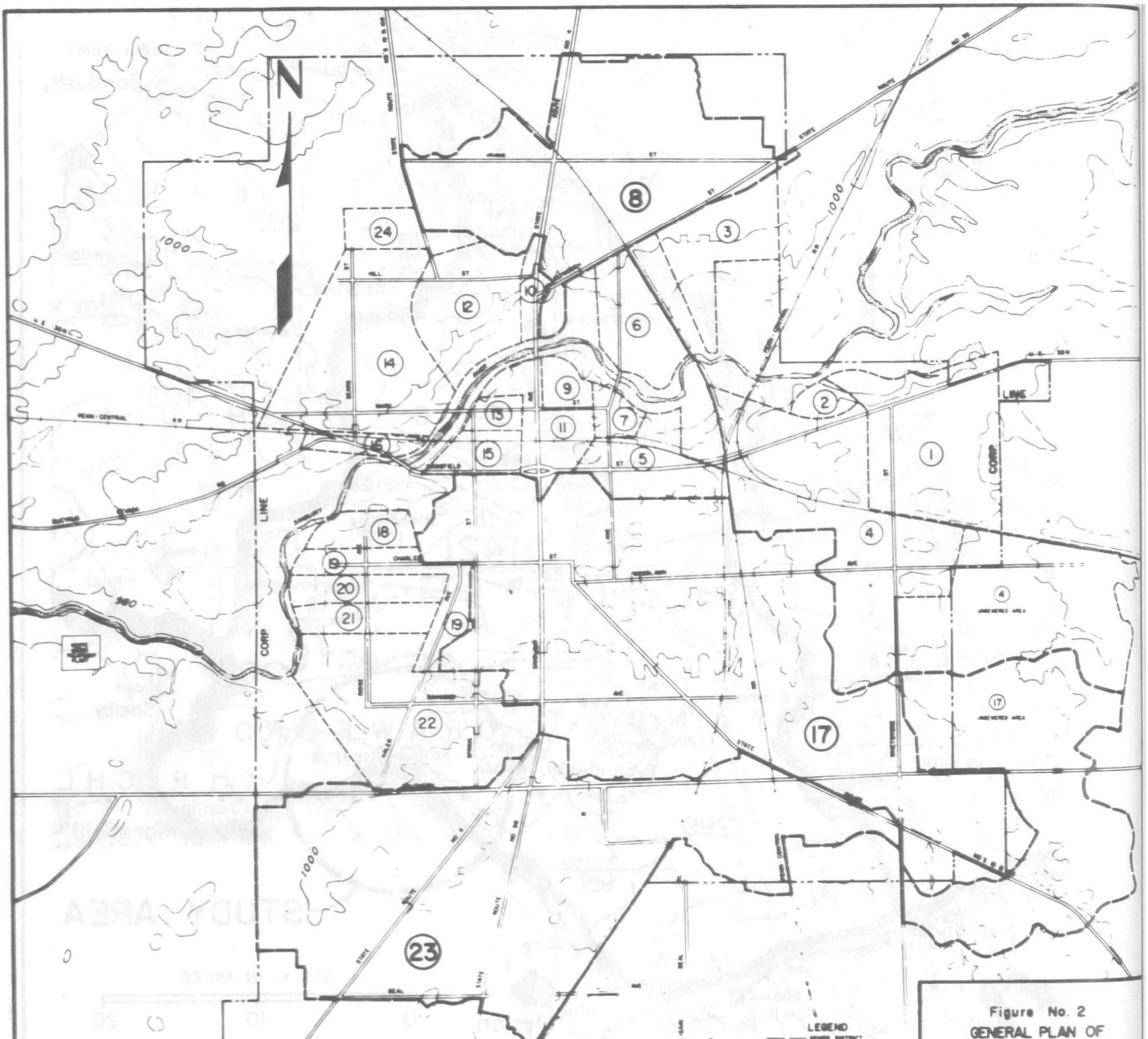
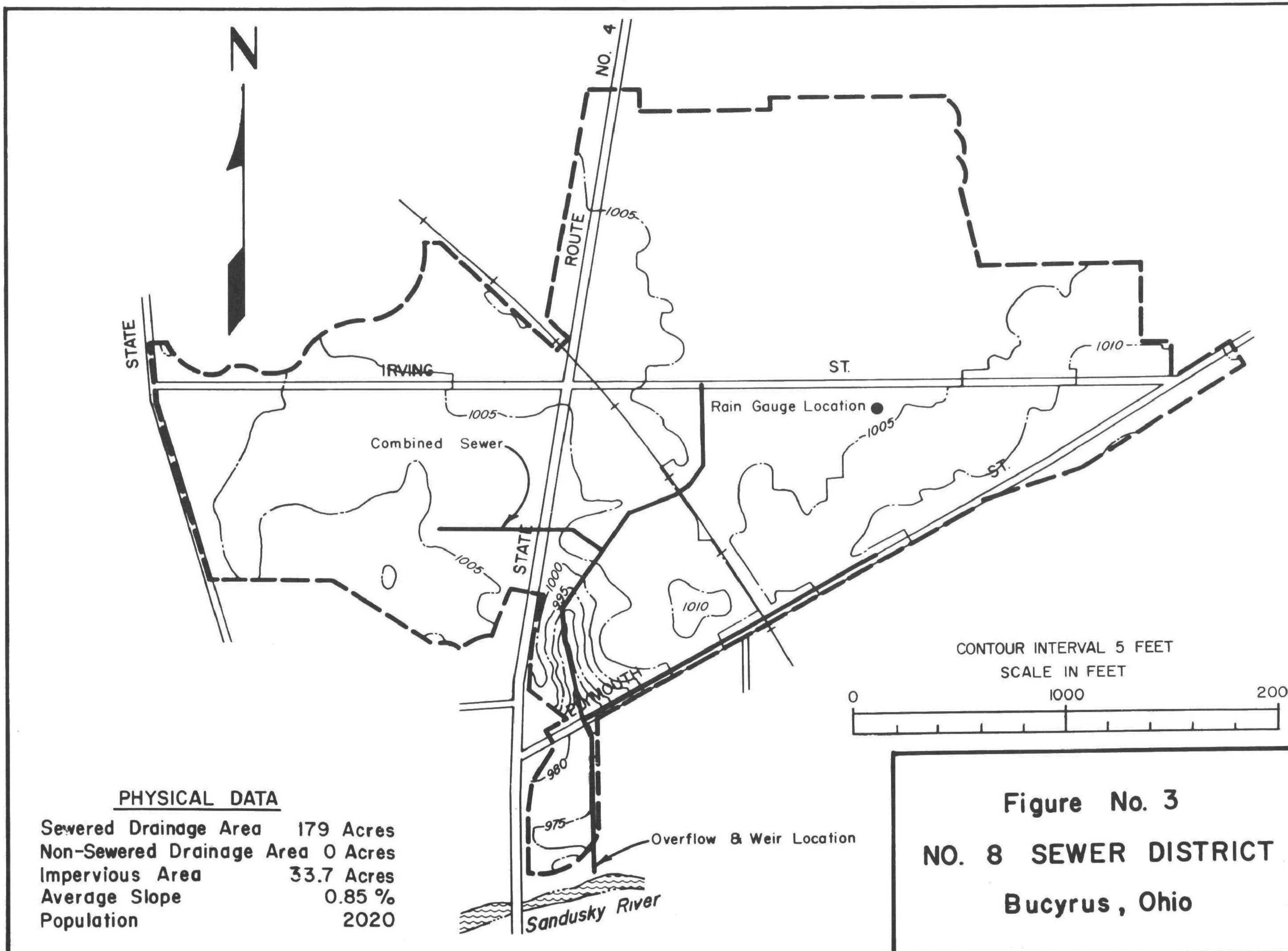


Figure No. 2
GENERAL PLAN OF
COMBINED SEWER DISTRICTS
BUCYRUS, OHIO



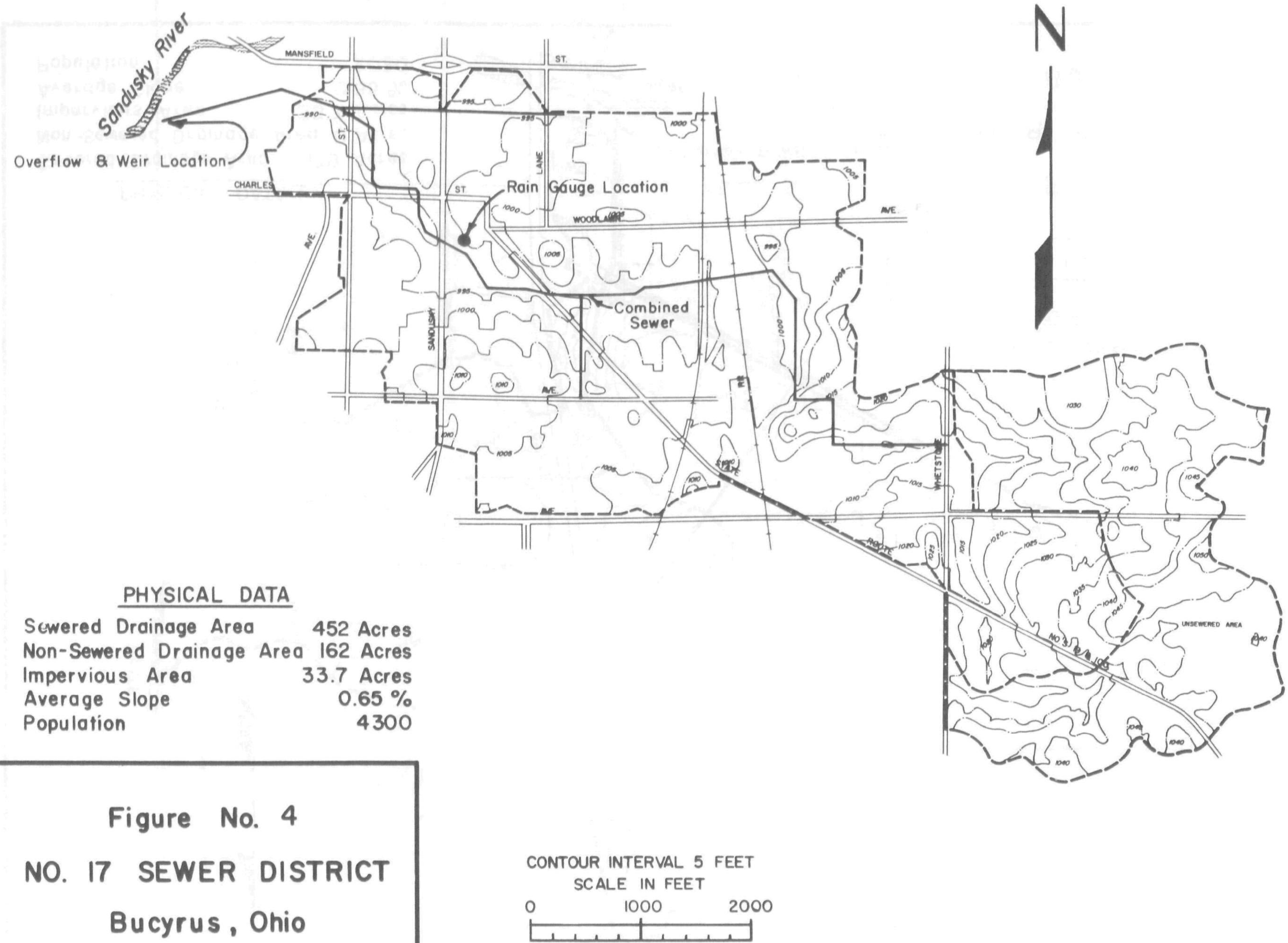
PHYSICAL DATA

Sewered Drainage Area	179 Acres
Non-Sewered Drainage Area	0 Acres
Impervious Area	33.7 Acres
Average Slope	0.85 %
Population	2020

Figure No. 3

NO. 8 SEWER DISTRICT

Bucyrus, Ohio



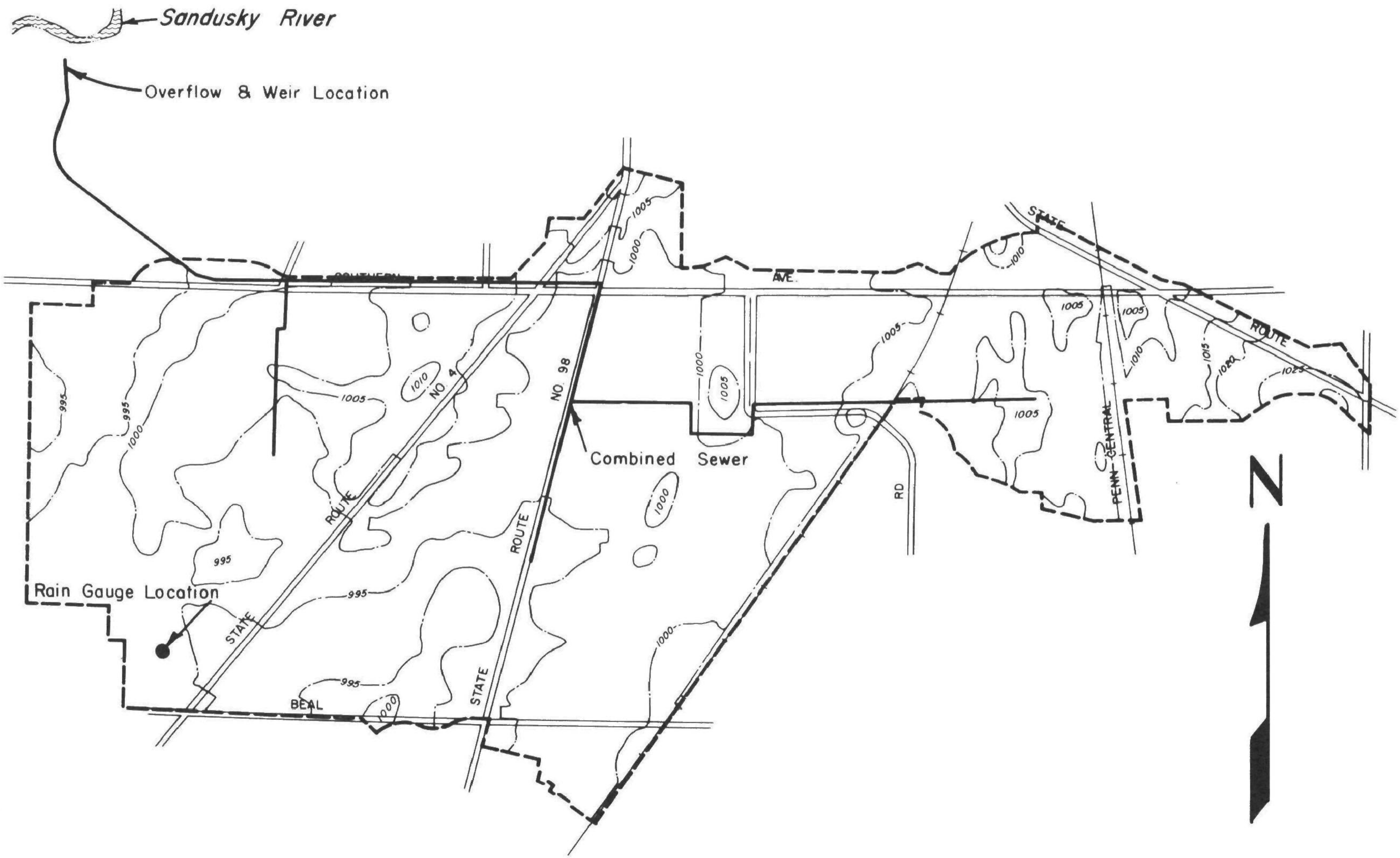
PHYSICAL DATA

Sewered Drainage Area	452 Acres
Non-Sewered Drainage Area	162 Acres
Impervious Area	33.7 Acres
Average Slope	0.65 %
Population	4300

Figure No. 4

NO. 17 SEWER DISTRICT

Bucyrus, Ohio



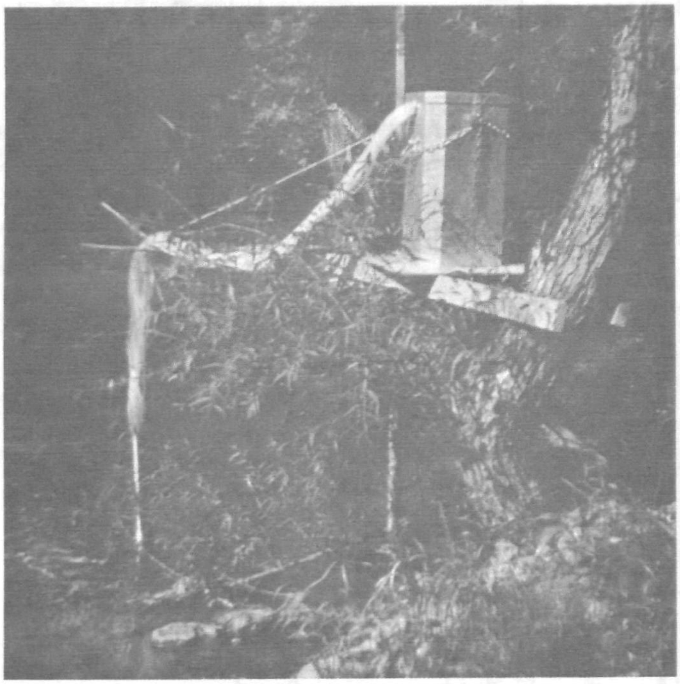
PHYSICAL DATA

Sewered Drainage Area	378 Acres
Non-Sewered Drainage Area	0 Acres
Impervious Area	26.1 Acres
Average Slope	0.25 %
Population	1960

CONTOUR INTERVAL 5 FEET
SCALE IN FEET
0 1000 2000

Figure No. 5
NO. 23 SEWER DISTRICT
Bucyrus, Ohio

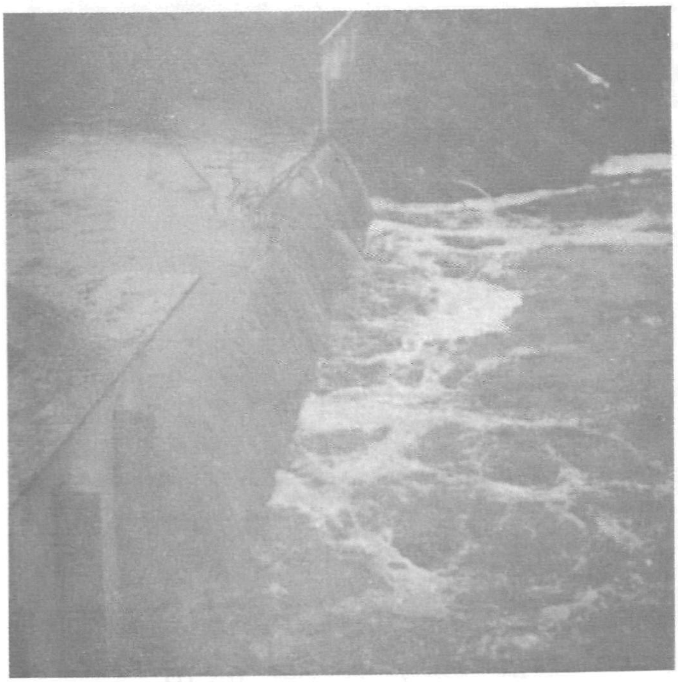
NO. 23 RAIN GAGE



UPSTREAM SAMPLER

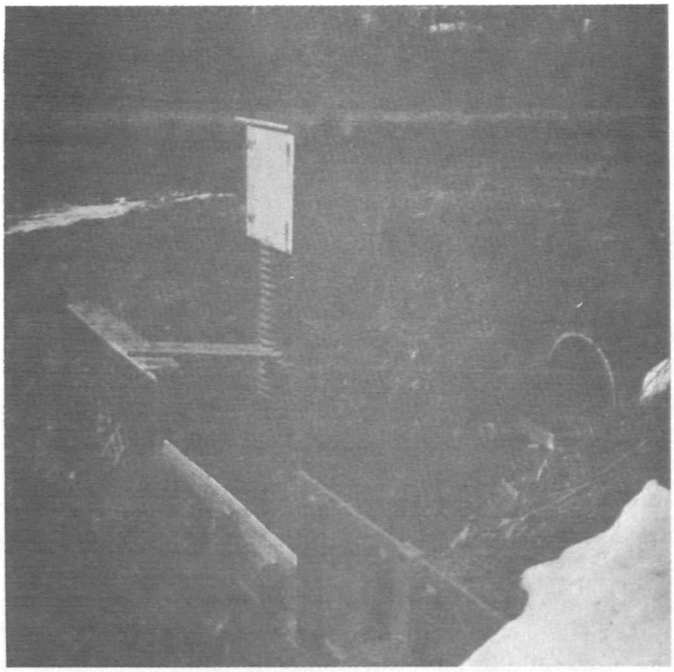
Figure No. 6

NO. 8 DRY WEATHER WEIR

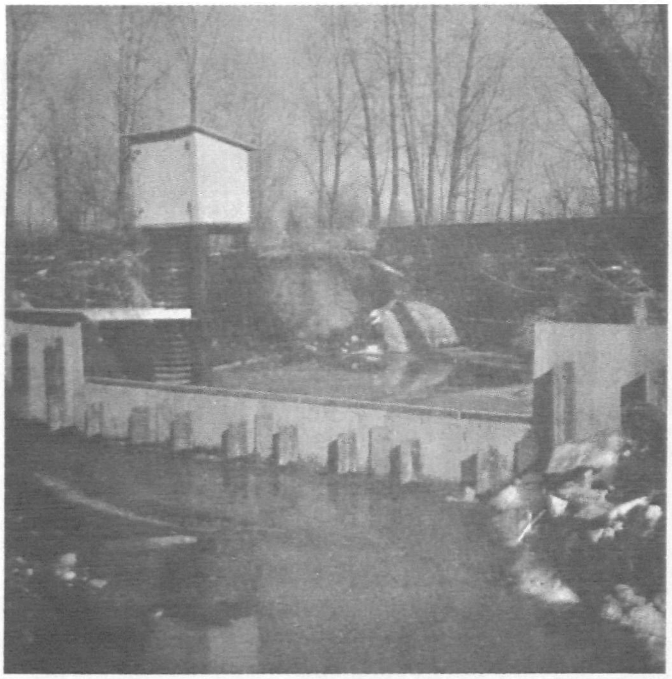


NO. 17 WEIR DURING OVERFLOW

Figure No. 7

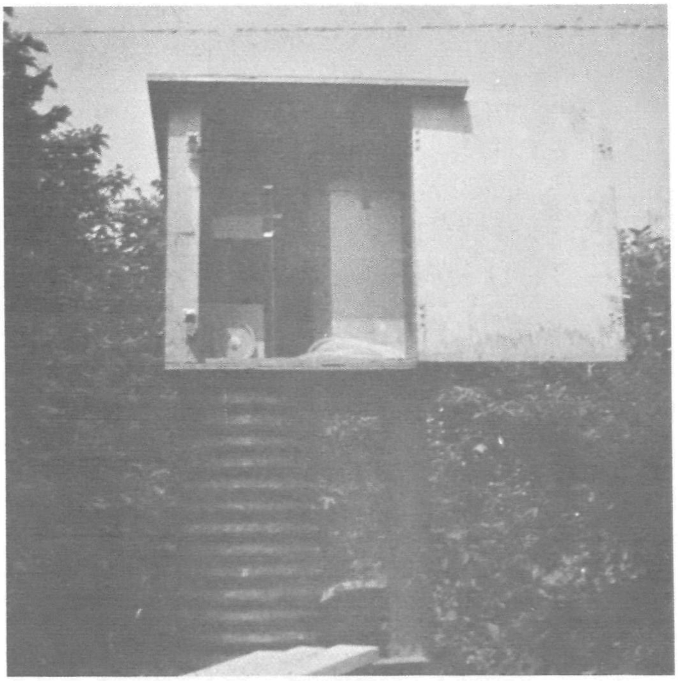


NO. 8 OVERFLOW WEIR



NO. 17 OVERFLOW WEIR

Figure No. 8



NO. 8 INSTRUMENT SHELTER

WASTE WATER TREATMENT PLANT
OVERFLOW RECORDER

Figure No. 9

UPSTREAM GAGE

DOWNSTREAM GAGE

Figure No. 10

RIVER AT NO. 8, LOW FLOW CONDITIONS

RIVER UPSTREAM FROM NO. 8
LOW FLOW CONDITIONS

Figure No. 11

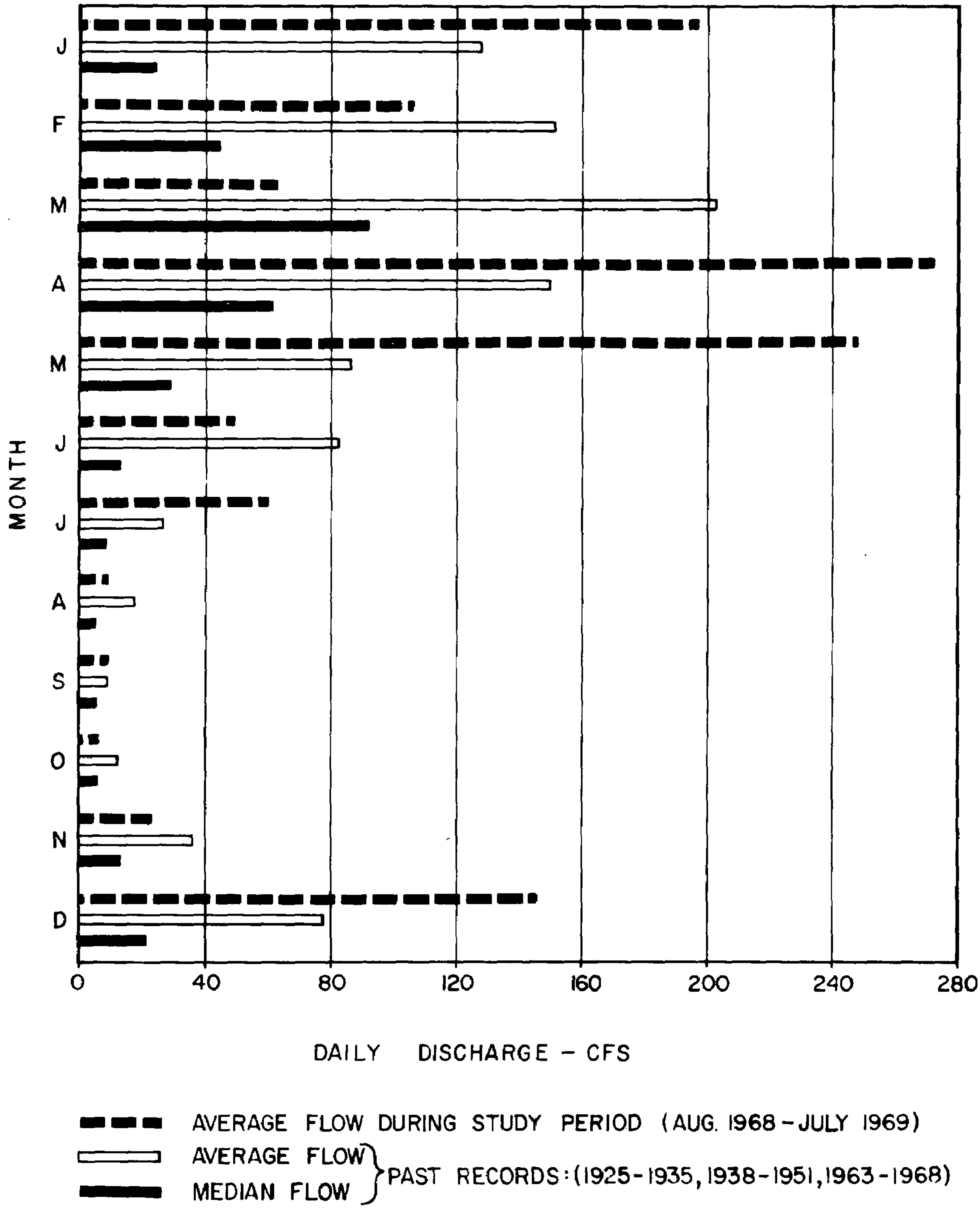


Figure No. 12

Sandusky River Flow at Bucyrus, Ohio

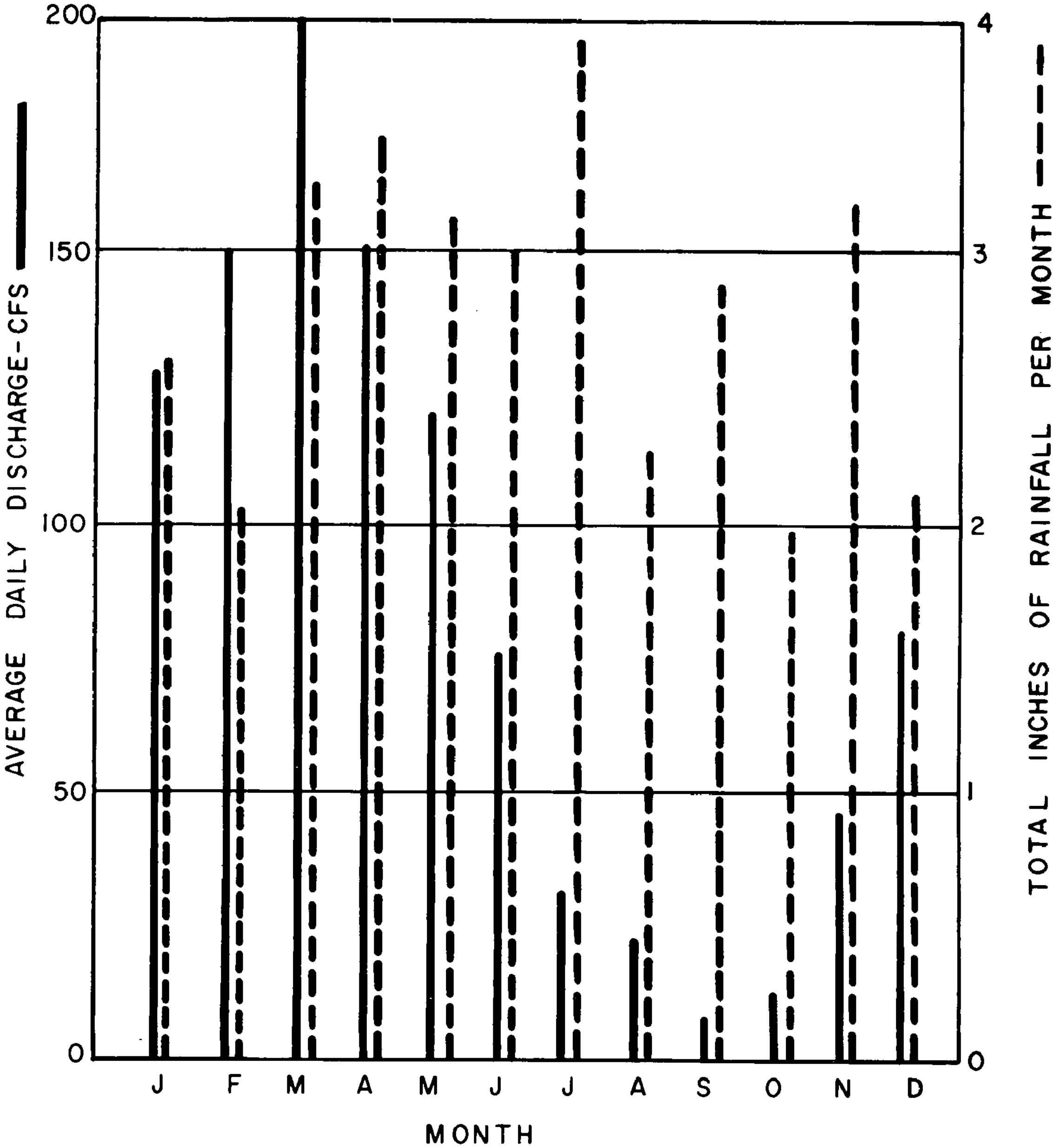
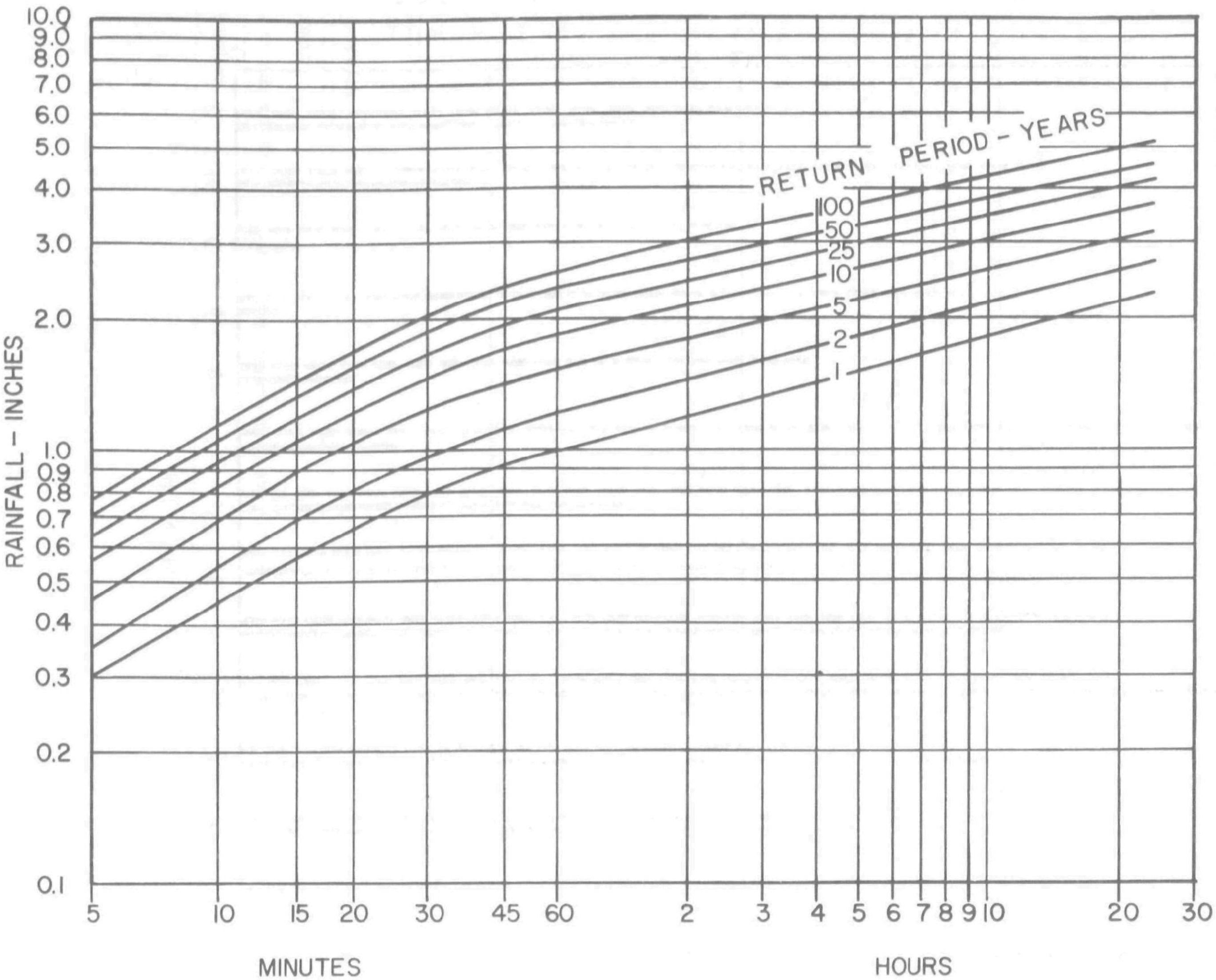


Figure No. 13

Comparison of Monthly Discharge & Monthly Rainfall
Sandusky River at Bucyrus, Ohio

Figure No. 14
Rainfall Depth - Duration - Frequency Curves
for Bucyrus, Ohio



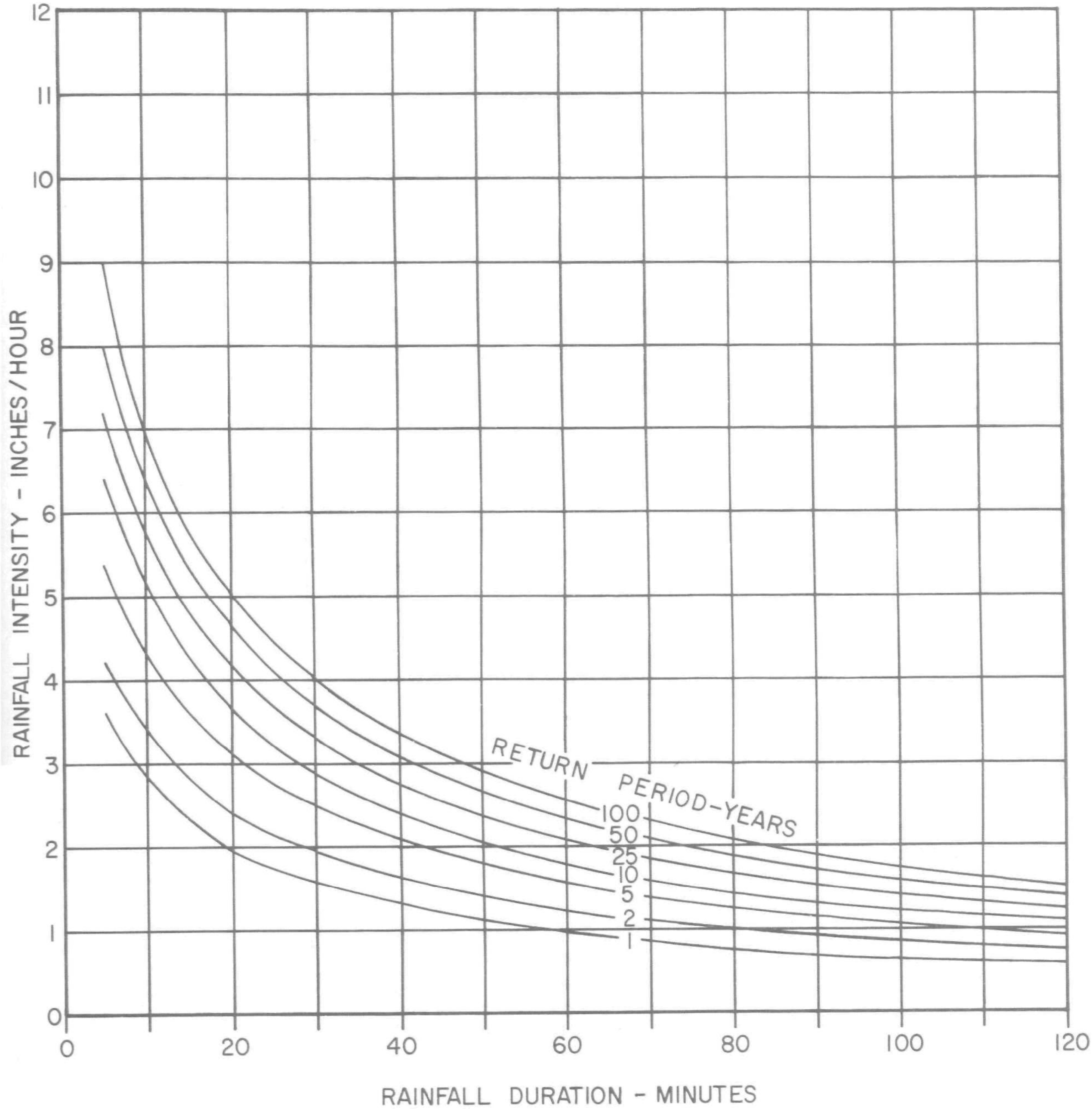


Figure No. 15
Intensity - Duration Curves

for Bucyrus, Ohio

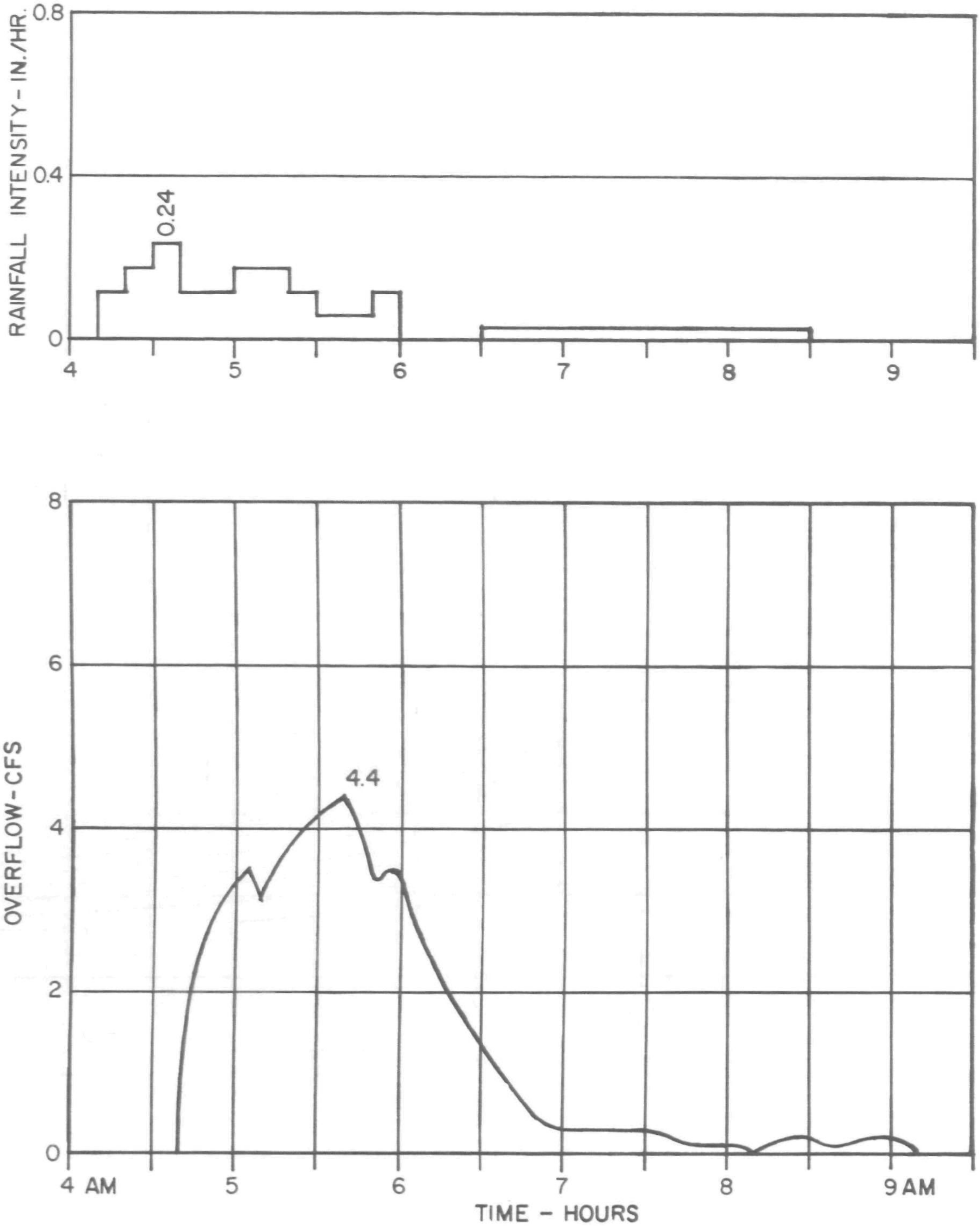
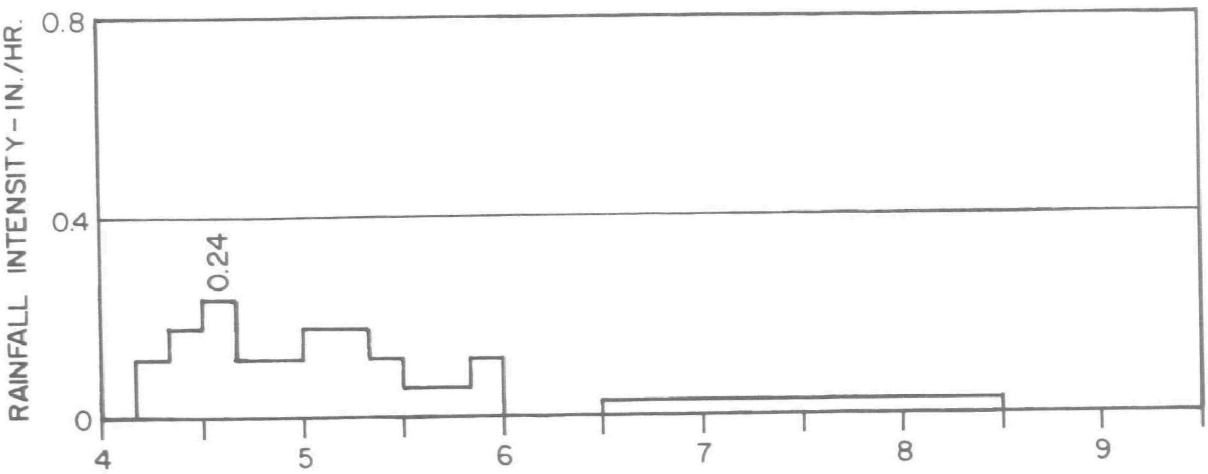


Figure No. 16
Rainfall and Overflow
No. 8 Overflow - March 24, 1969



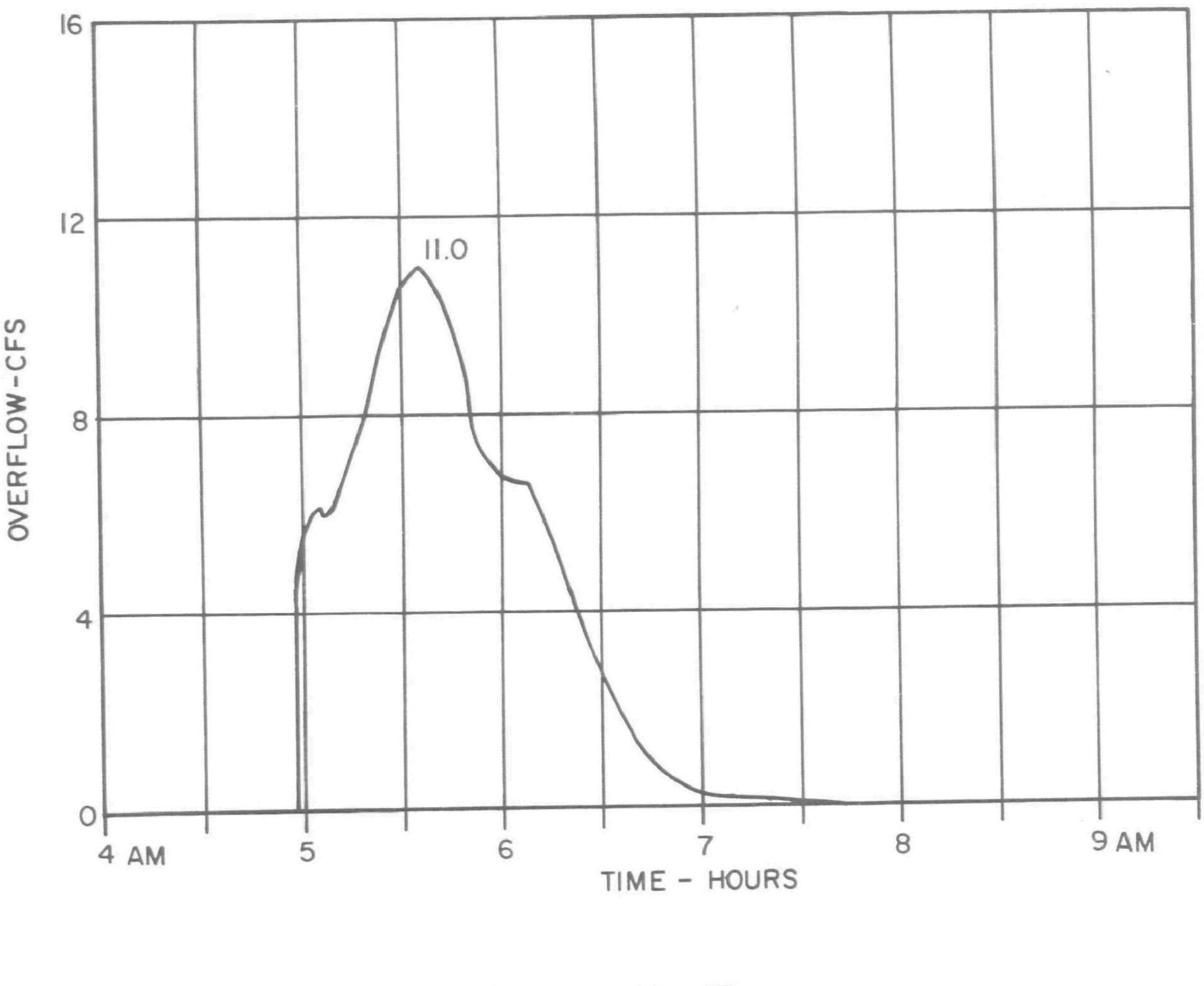


Figure No. 17
Rainfall and Overflow
No. 17 Overflow - March 24, 1969

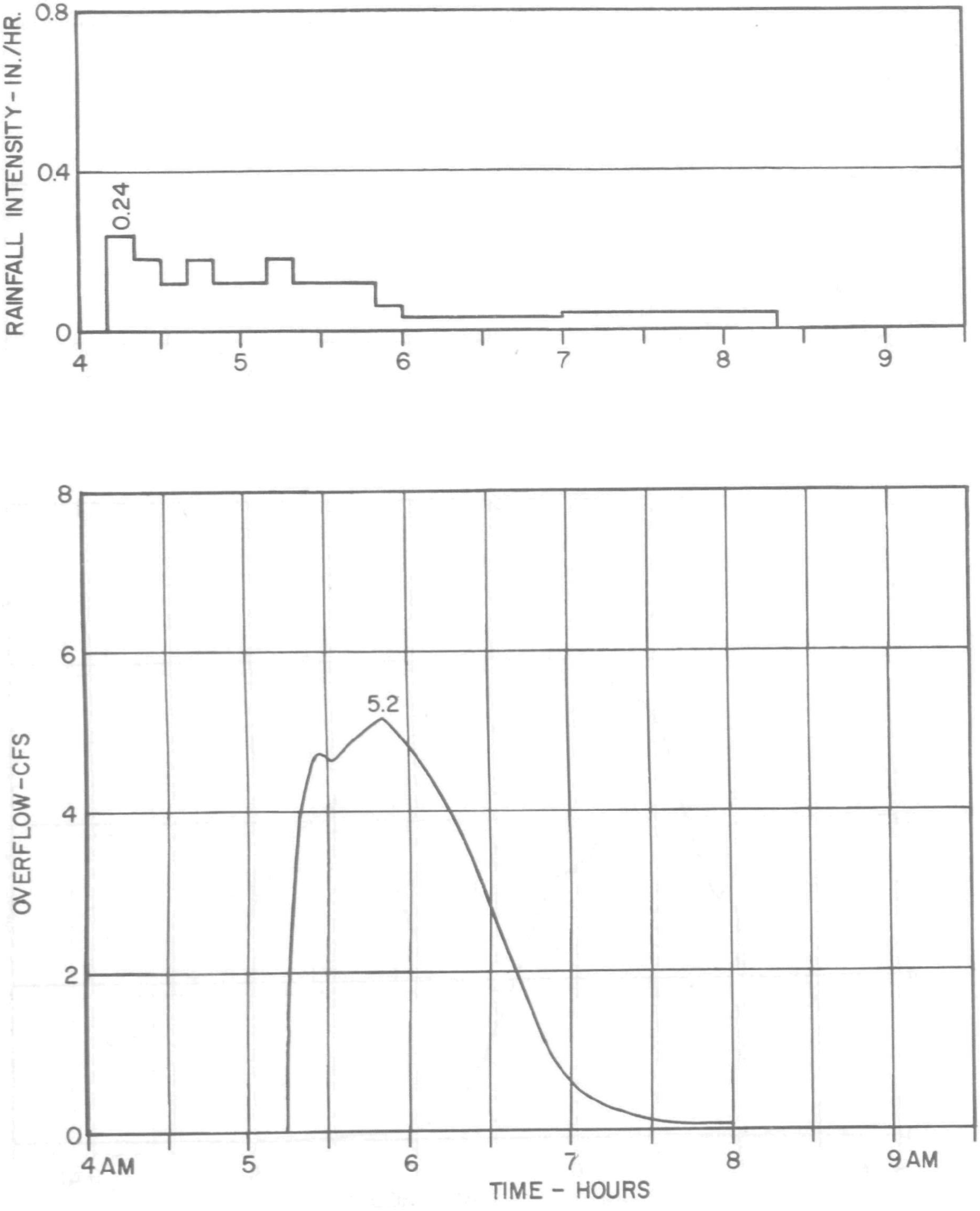


Figure No. 18
Rainfall and Overflow
No. 23 Overflow - March 24, 1969

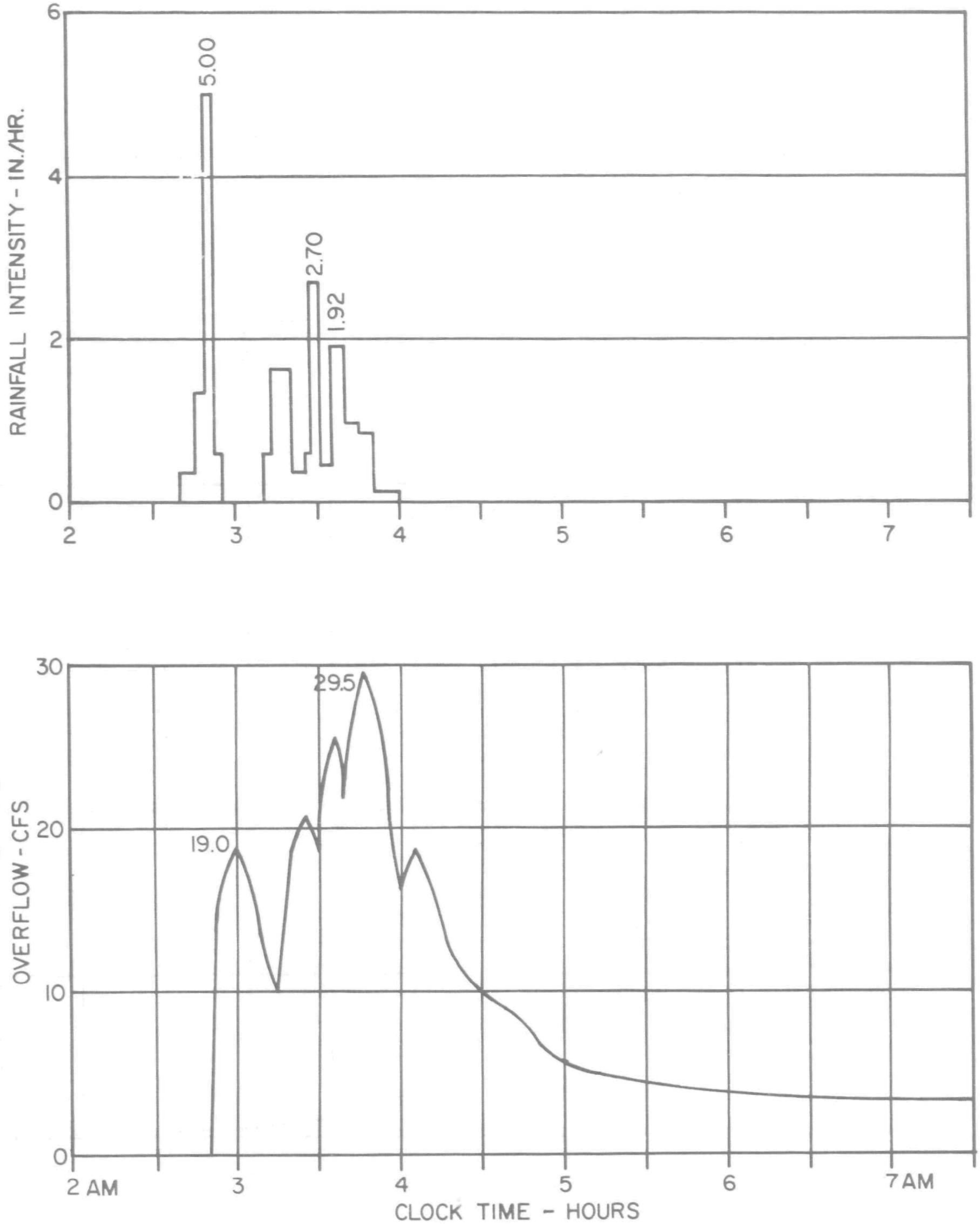


Figure No. 19
Rainfall and Overflow
No. 8 Overflow - June 13, 1969

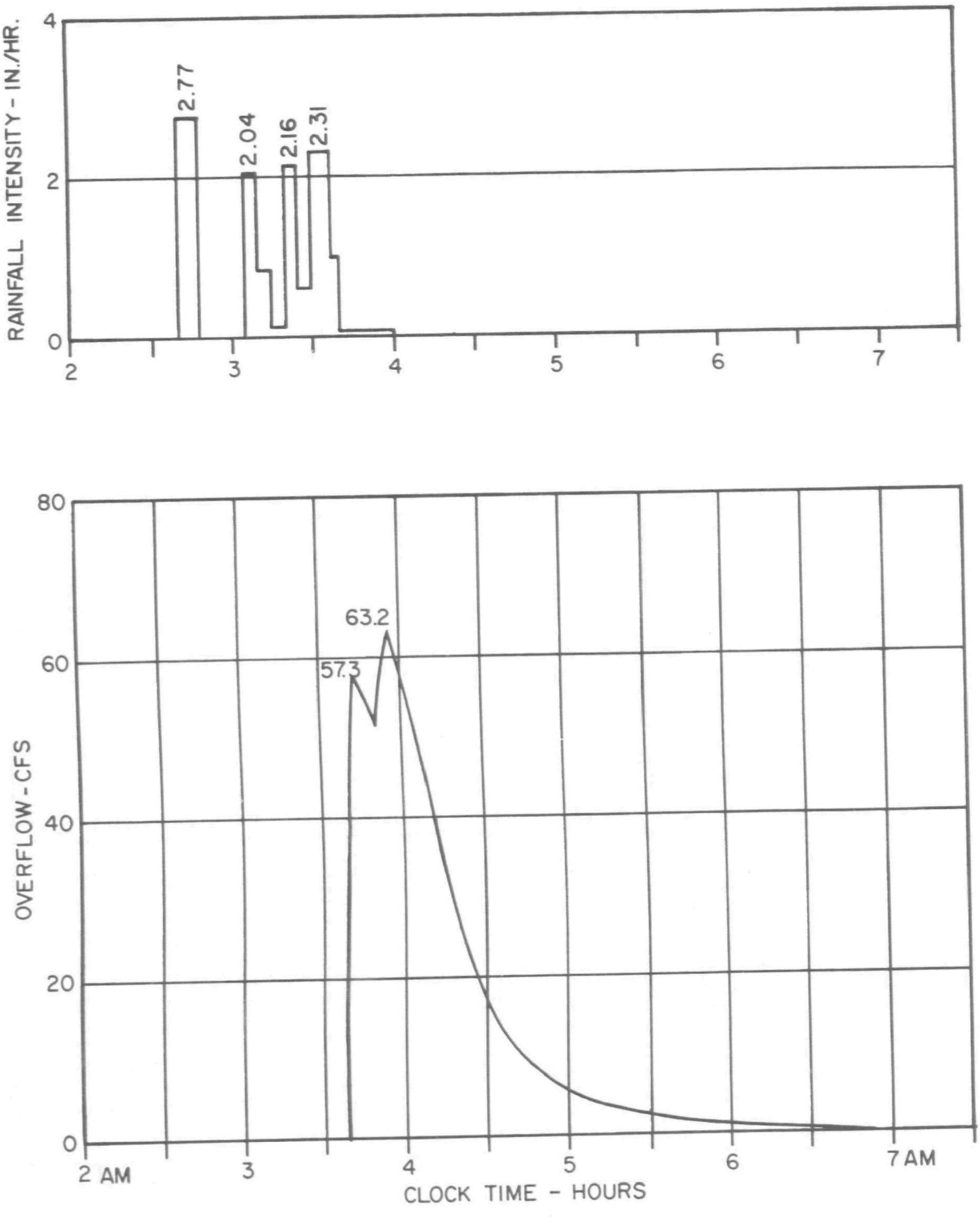


Figure No. 20
Rainfall and Overflow
No. 17 Overflow - June 13, 1969

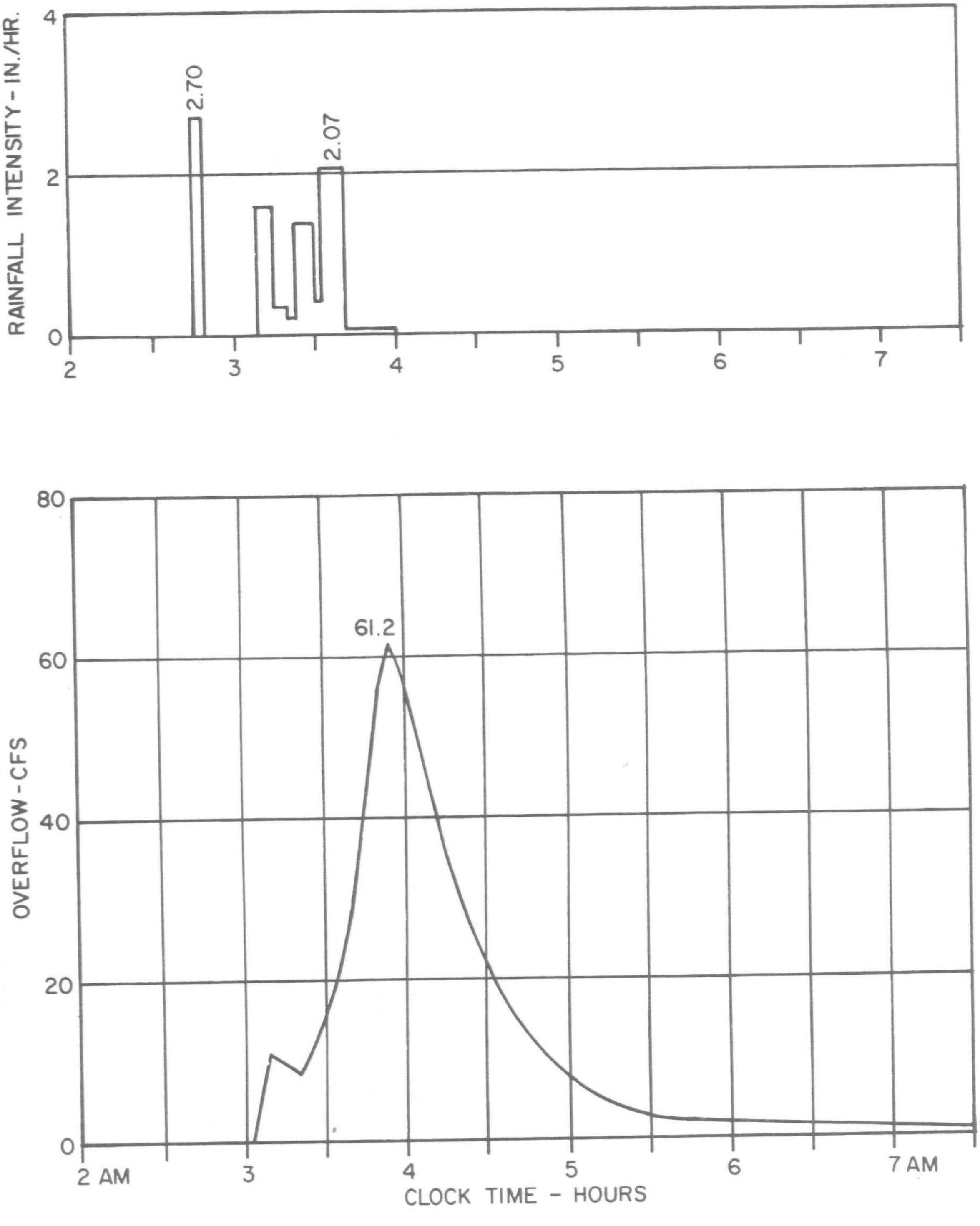


Figure No. 21
Rainfall and Overflow
No.23 Overflow - June 13, 1969

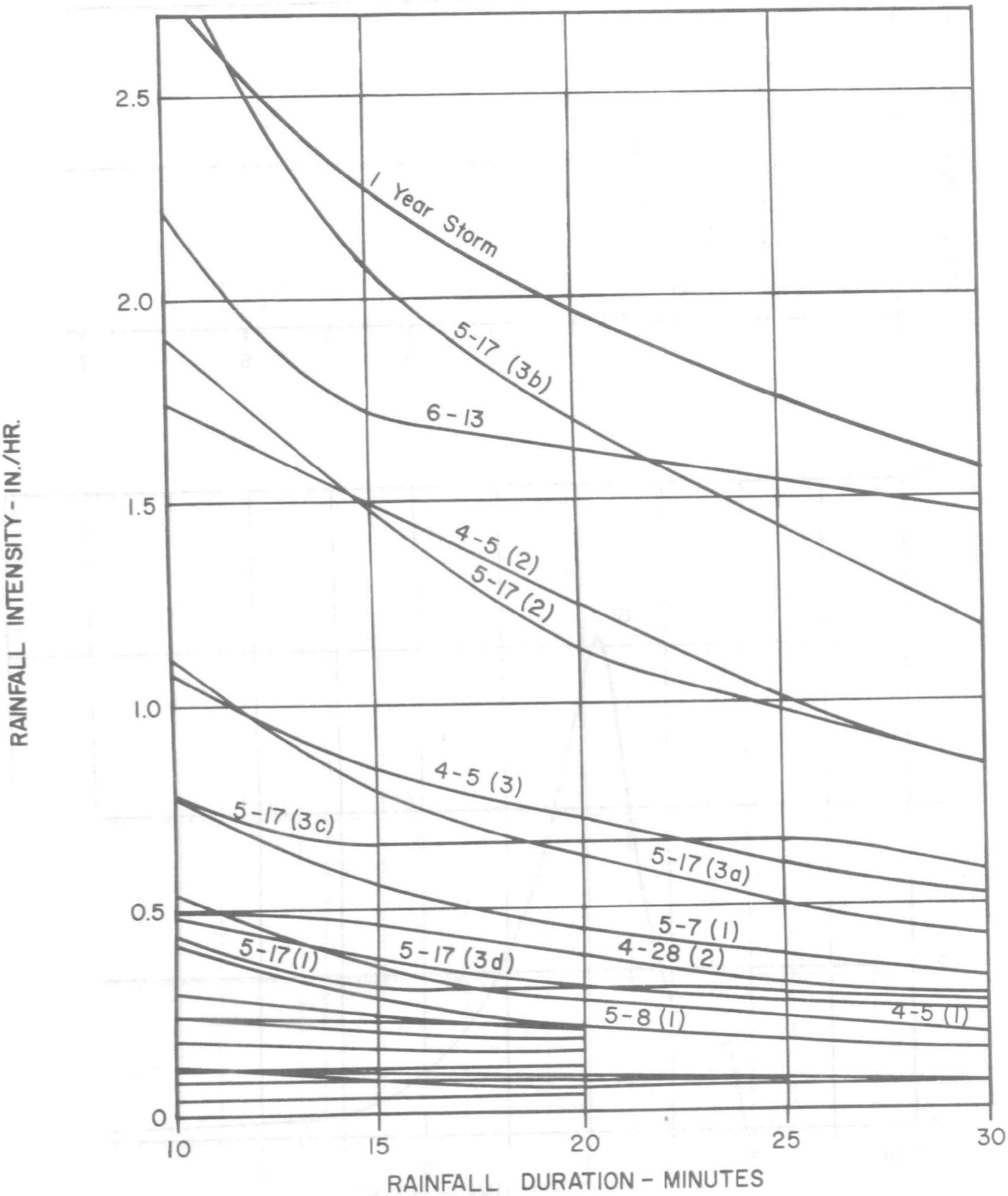


Figure No. 22
Intensity - Duration Curves
Rainfall Corresponding to Measured Overflows
No. 17 Rain Gauge Feb. 8 - June 13, 1969

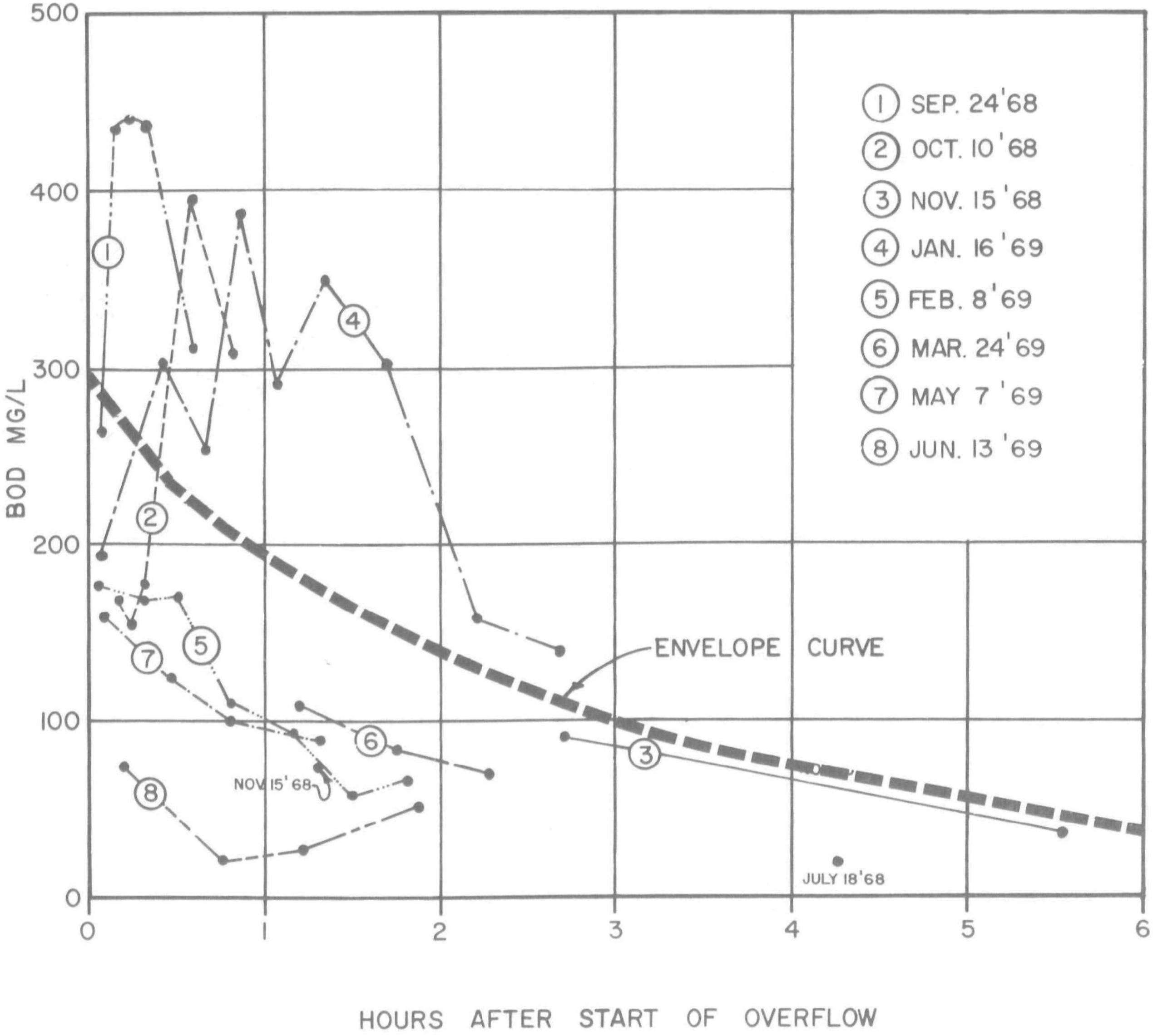


Figure No. 23
BOD Concentration vs. Time
No. 8 Overflow

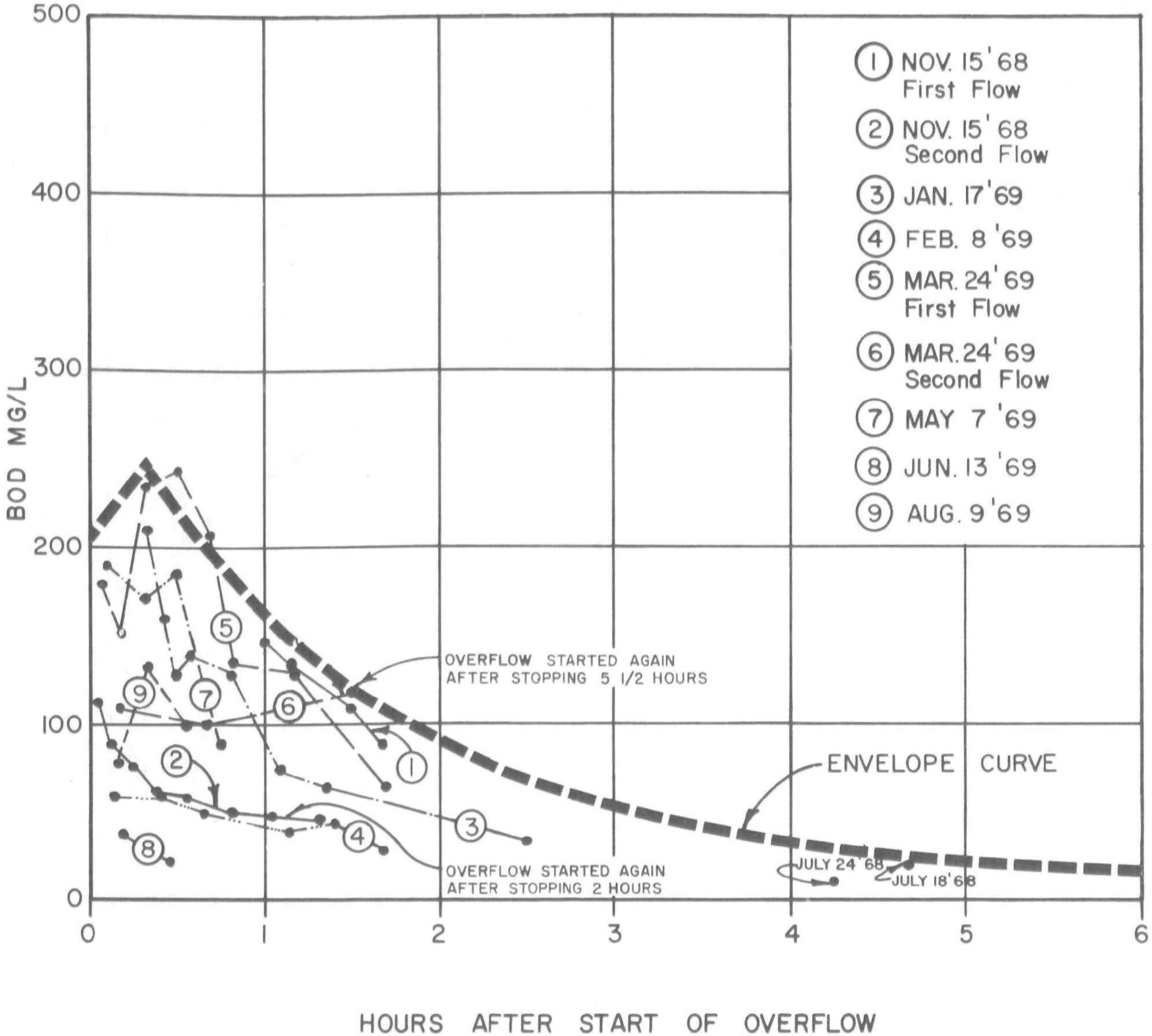


Figure No. 24
BOD Concentration vs. Time
No. 17 Overflow

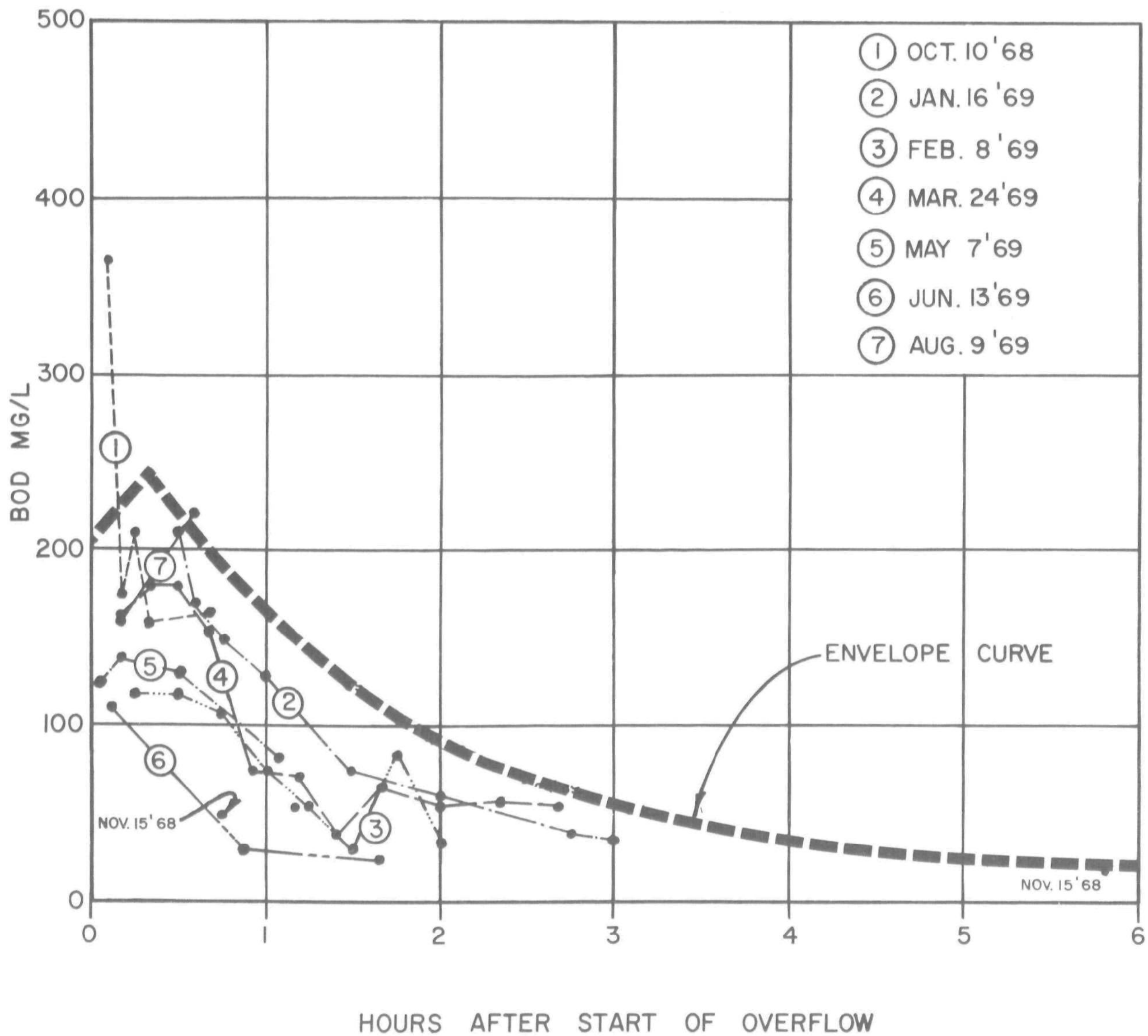


Figure No. 25
BOD Concentration vs. Time
No. 23 Overflow

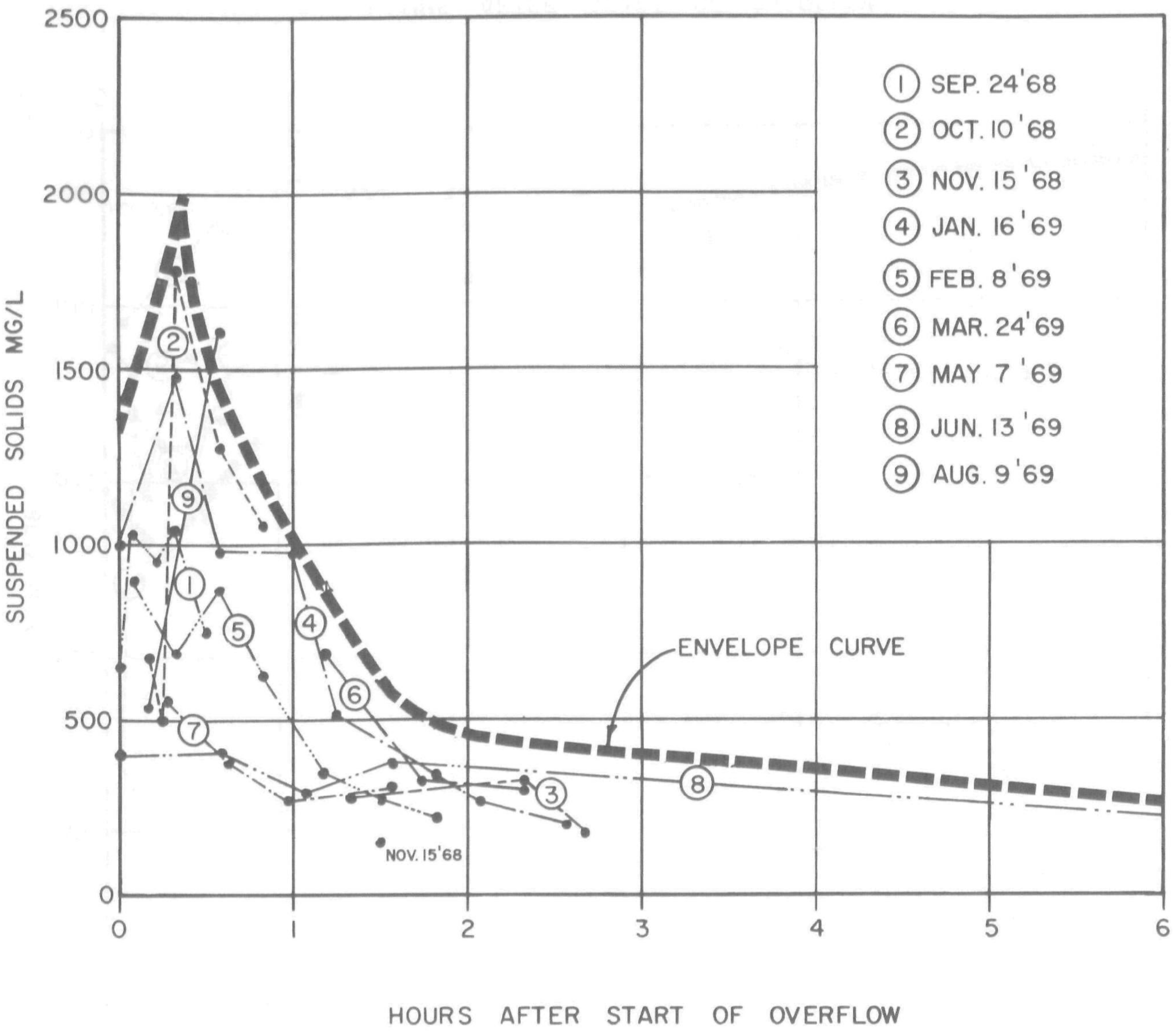


Figure No. 26
Suspended Solids Concentration vs. Time
No. 8 Overflow

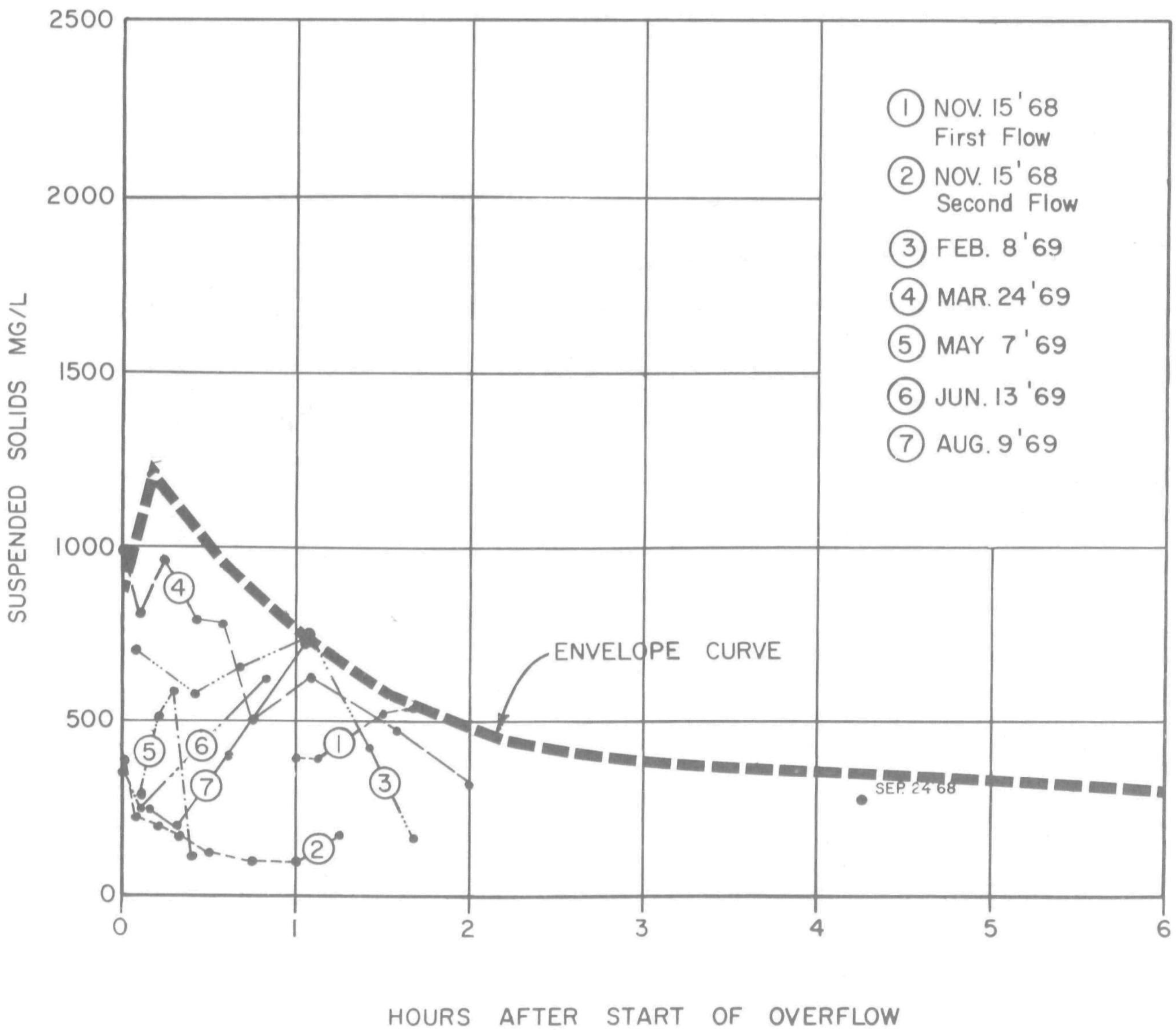


Figure No. 27
Suspended Solids Concentration vs. Time
No. 17 Overflow

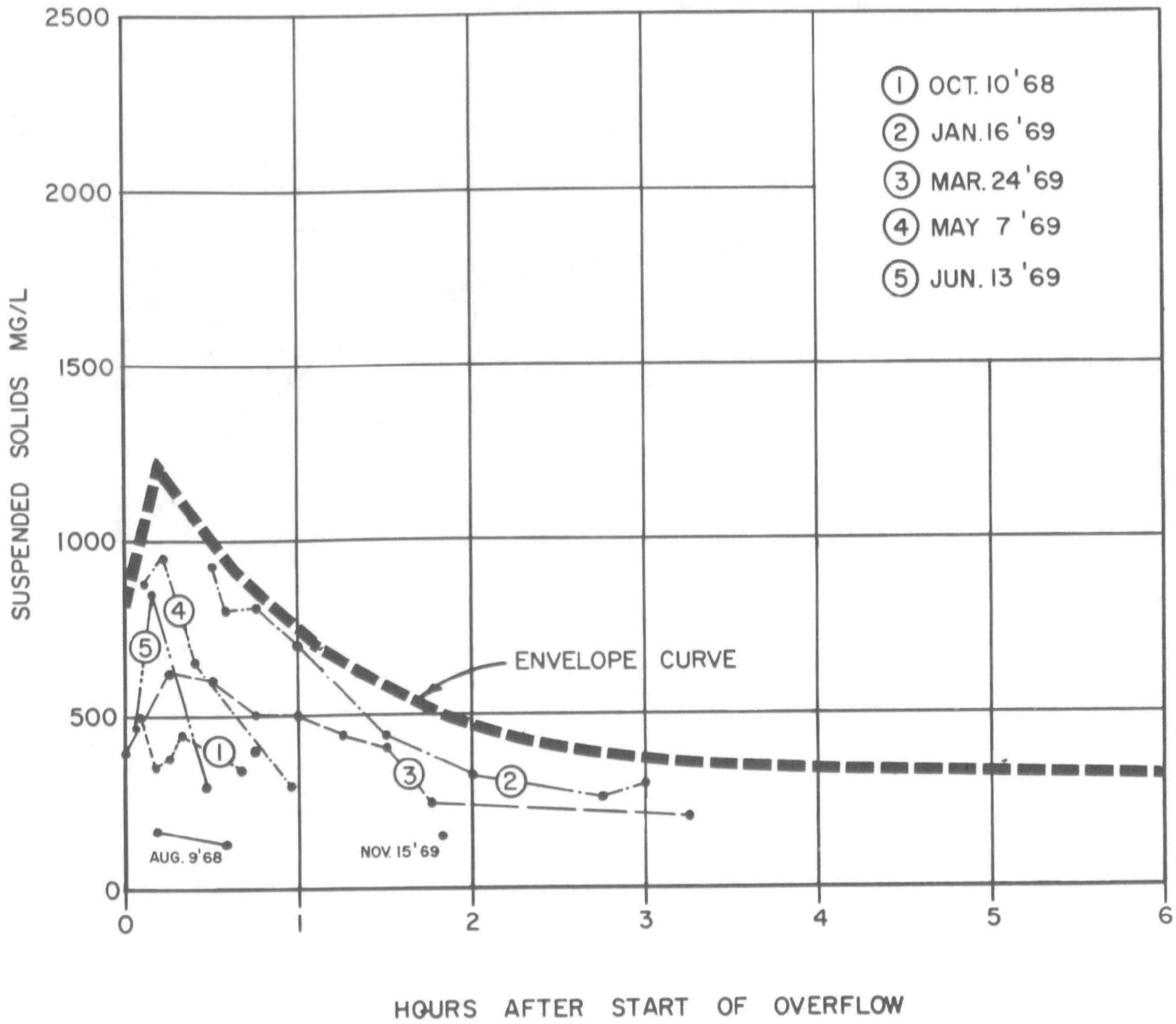
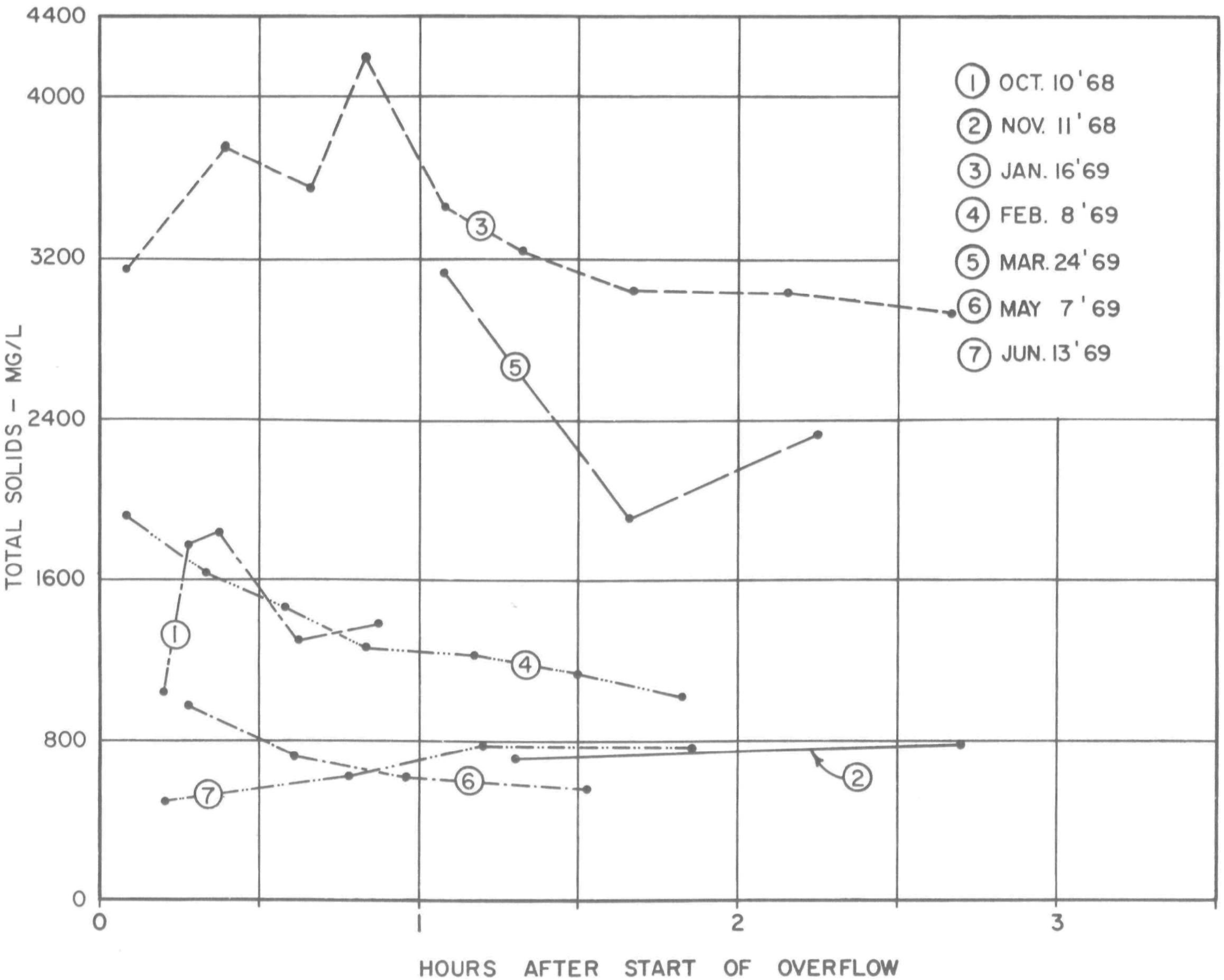


Figure No. 28
Suspended Solids Concentration vs. Time
No. 23 Overflow

Figure No. 29
Total Solids
No. 8 Overflow



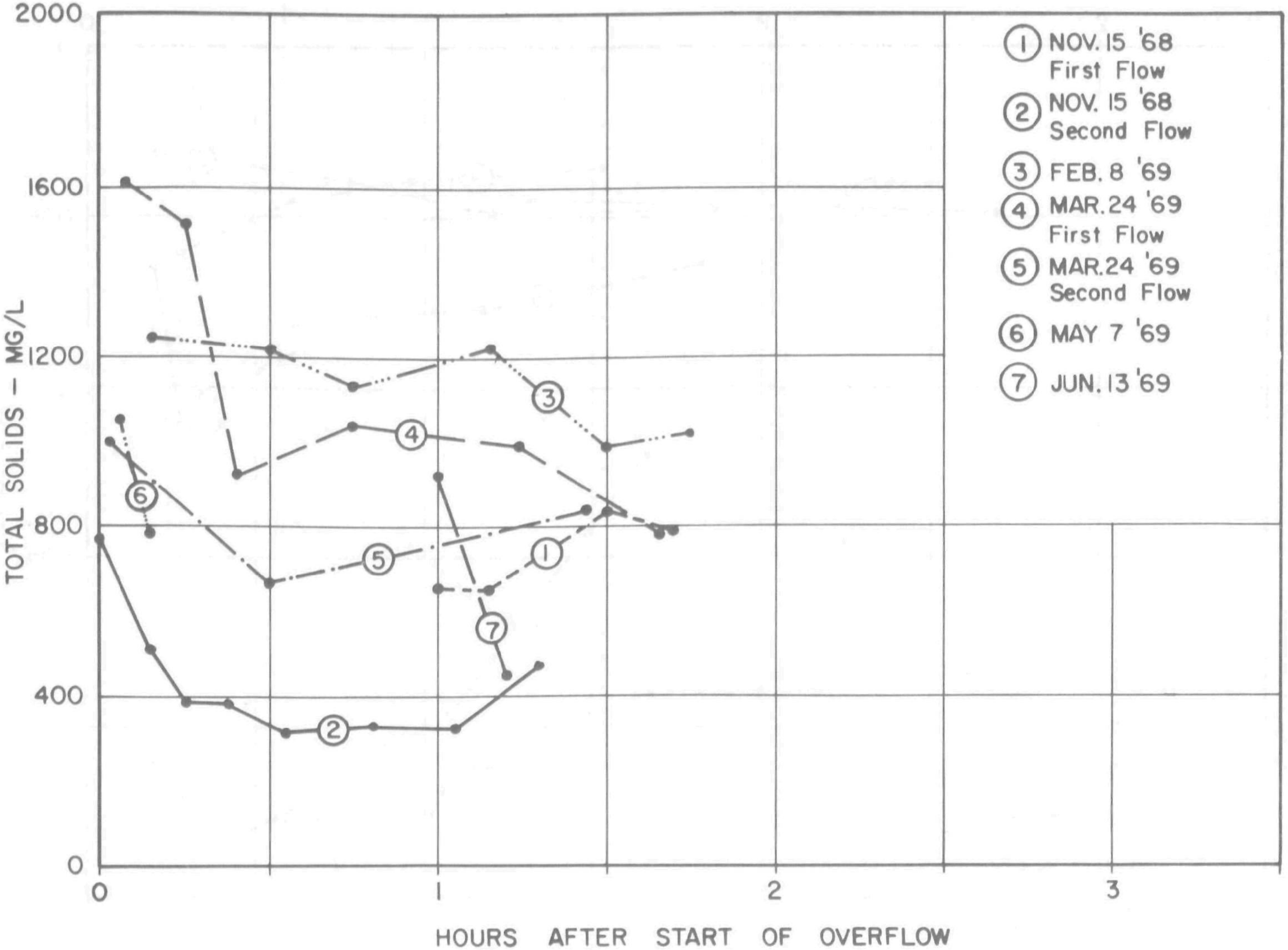


Figure No. 30
Total Solids
No. 17 Overflow

Figure No. 31
Total Solids
No. 23 Overflow

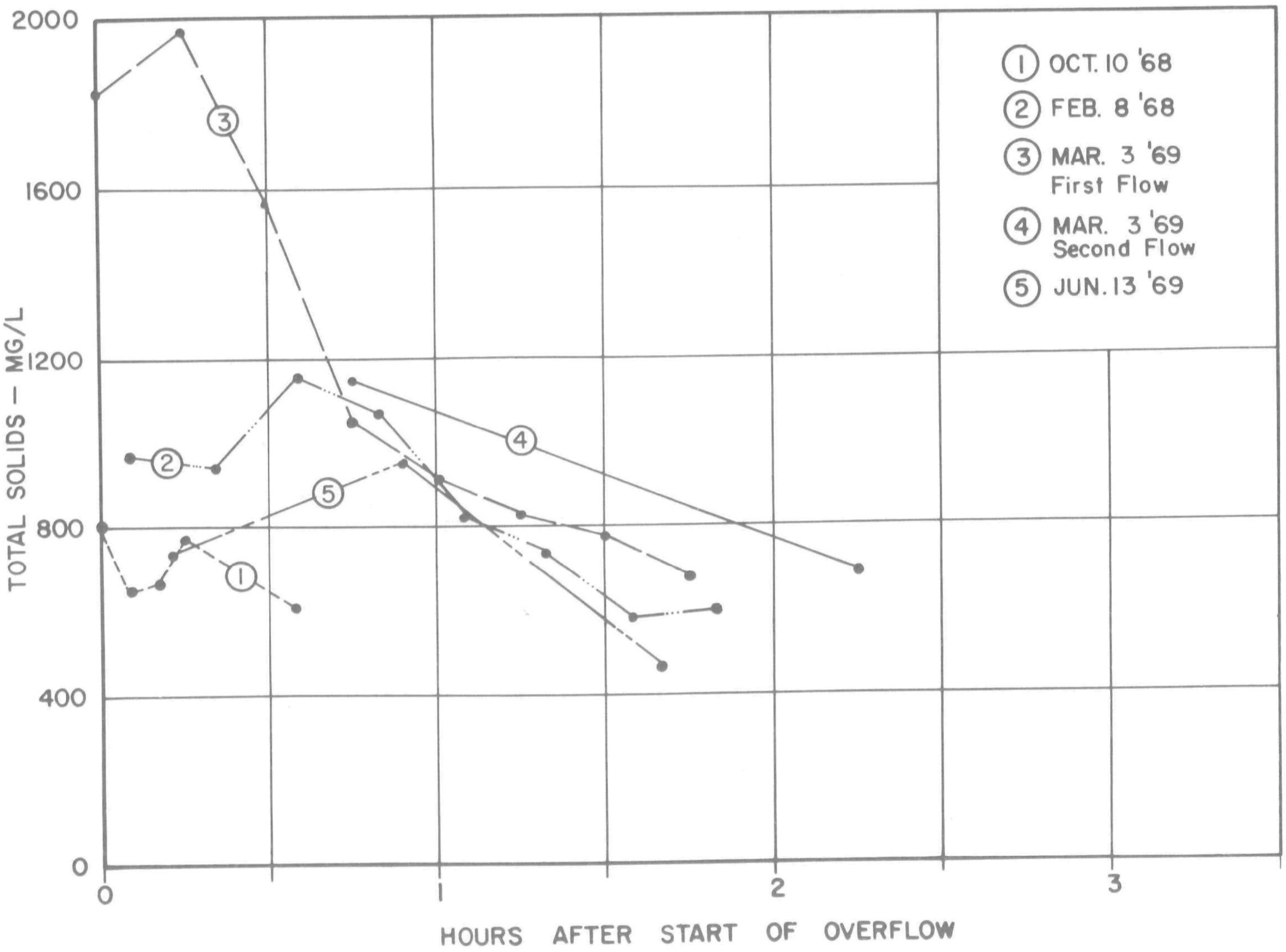
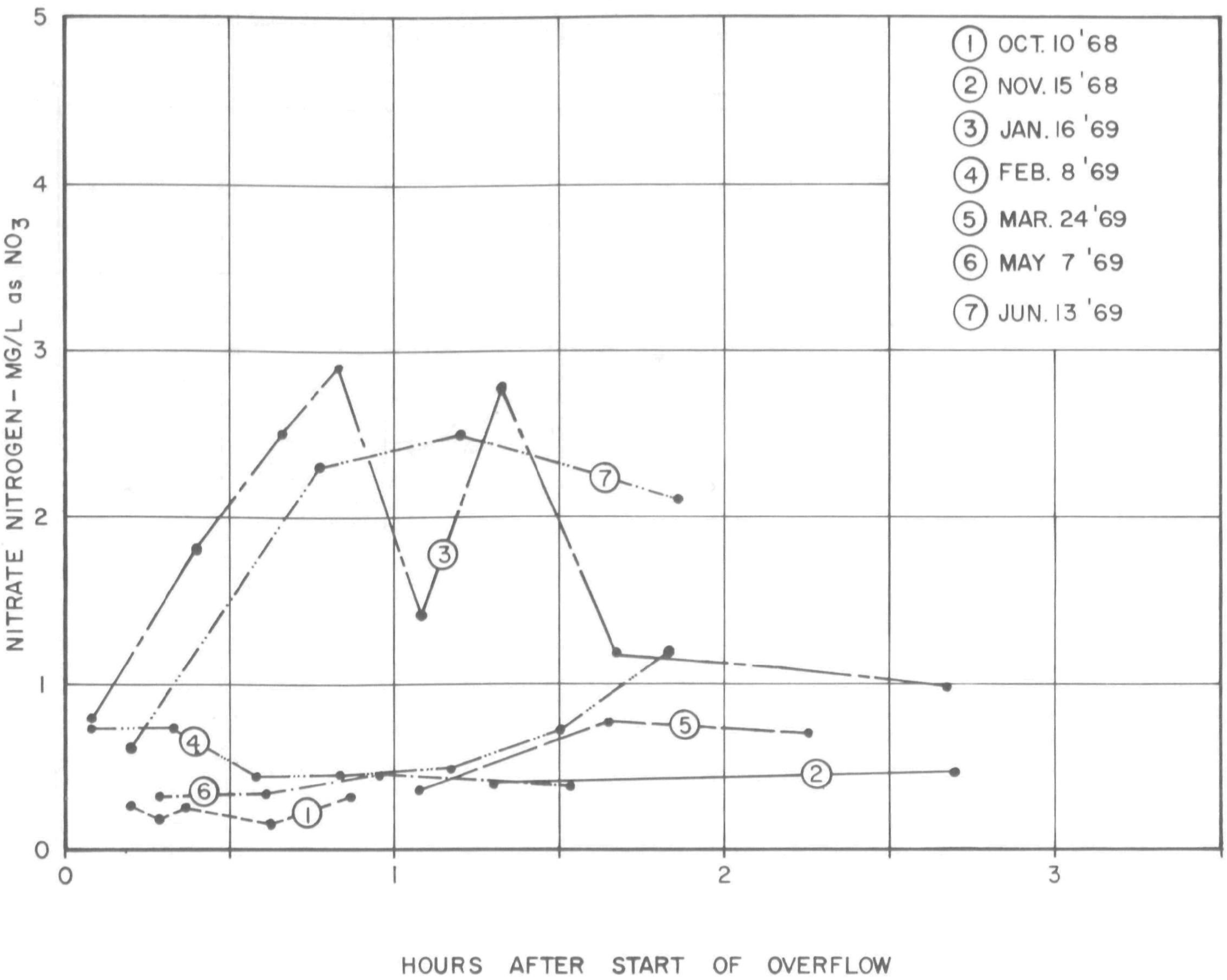


Figure No. 32
Nitrate Nitrogen
No. 8 Overflow



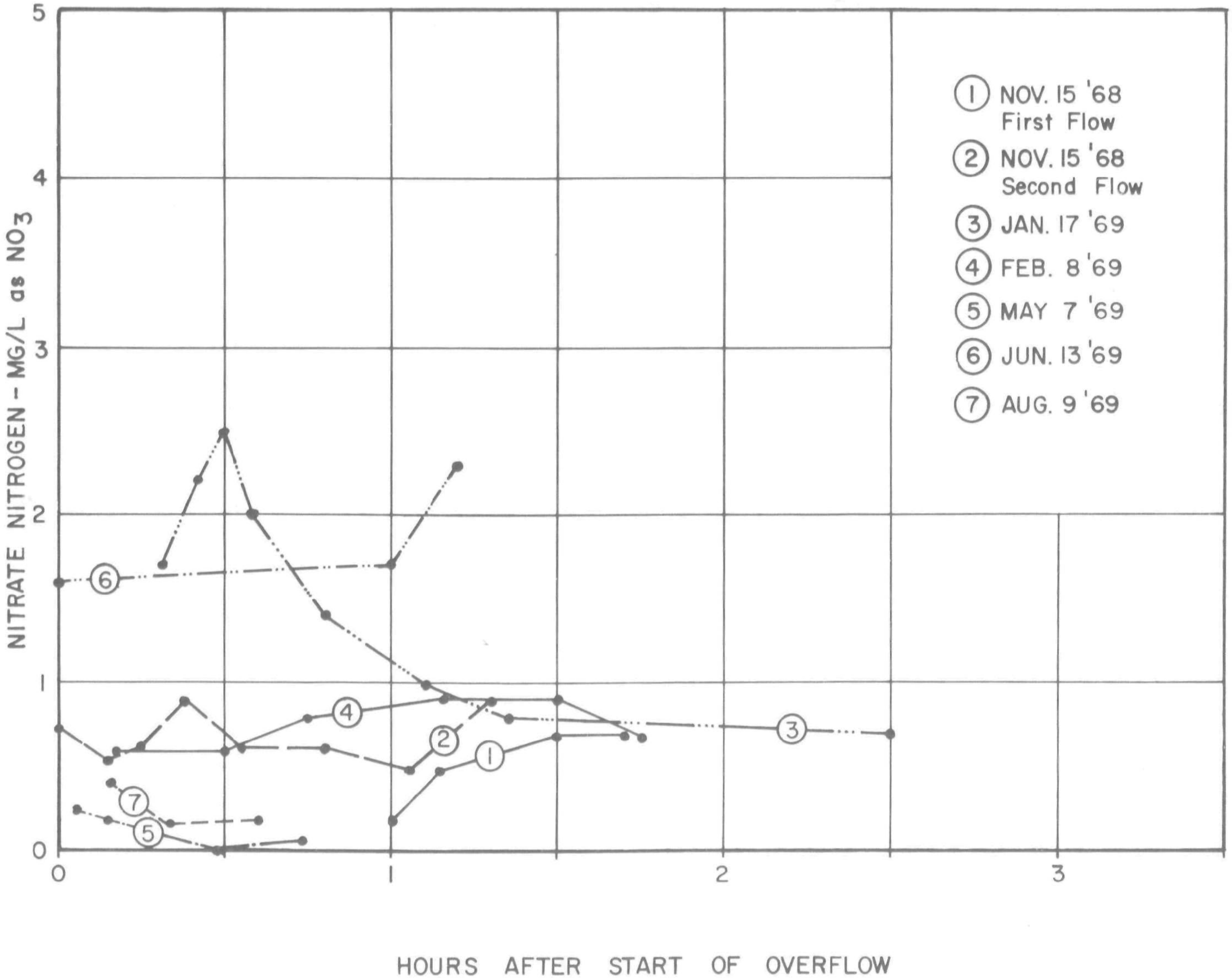


Figure No. 33
Nitrate Nitrogen
No. 17 Overflow

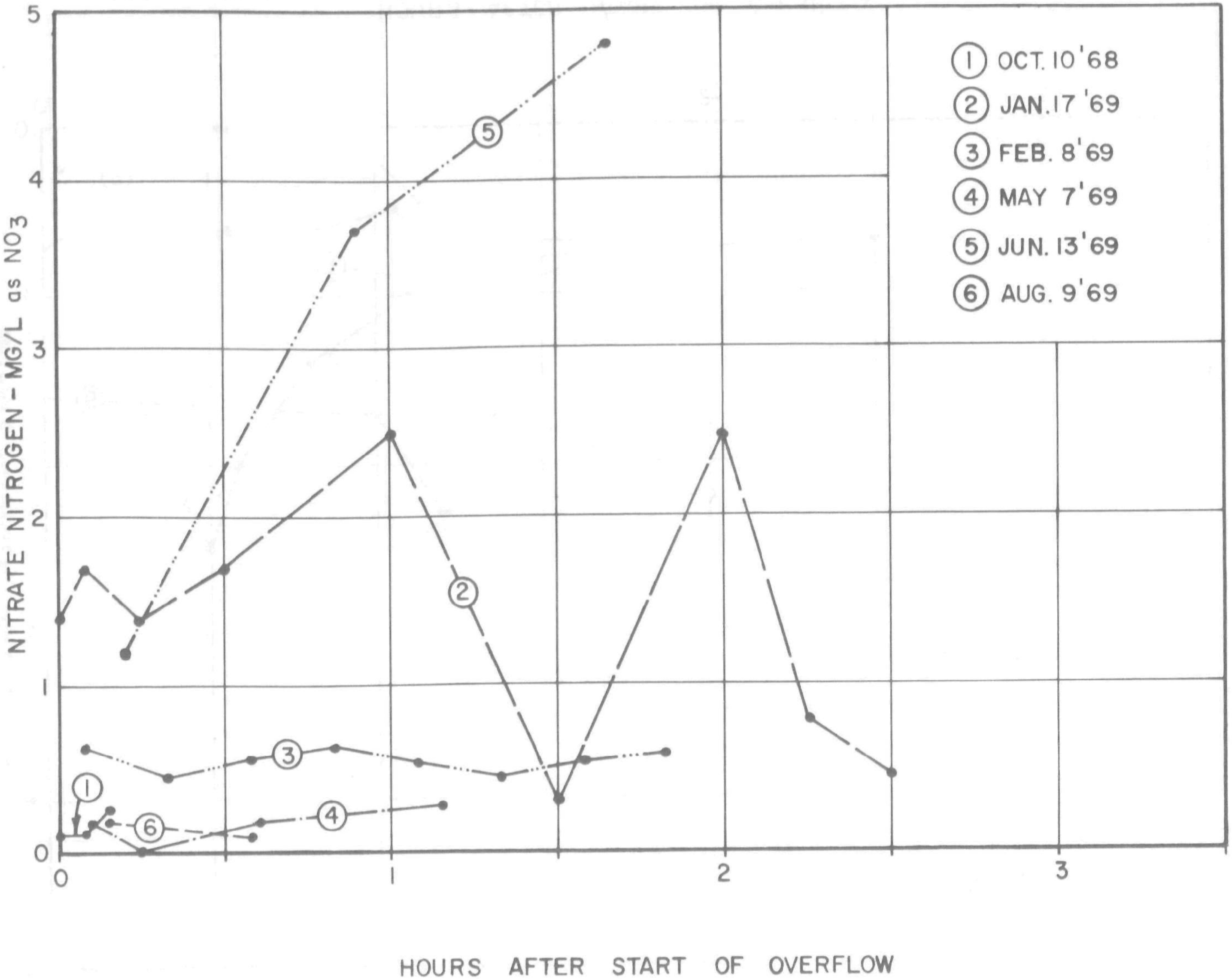


Figure No. 34
Nitrate Nitrogen
No. 23 Overflow

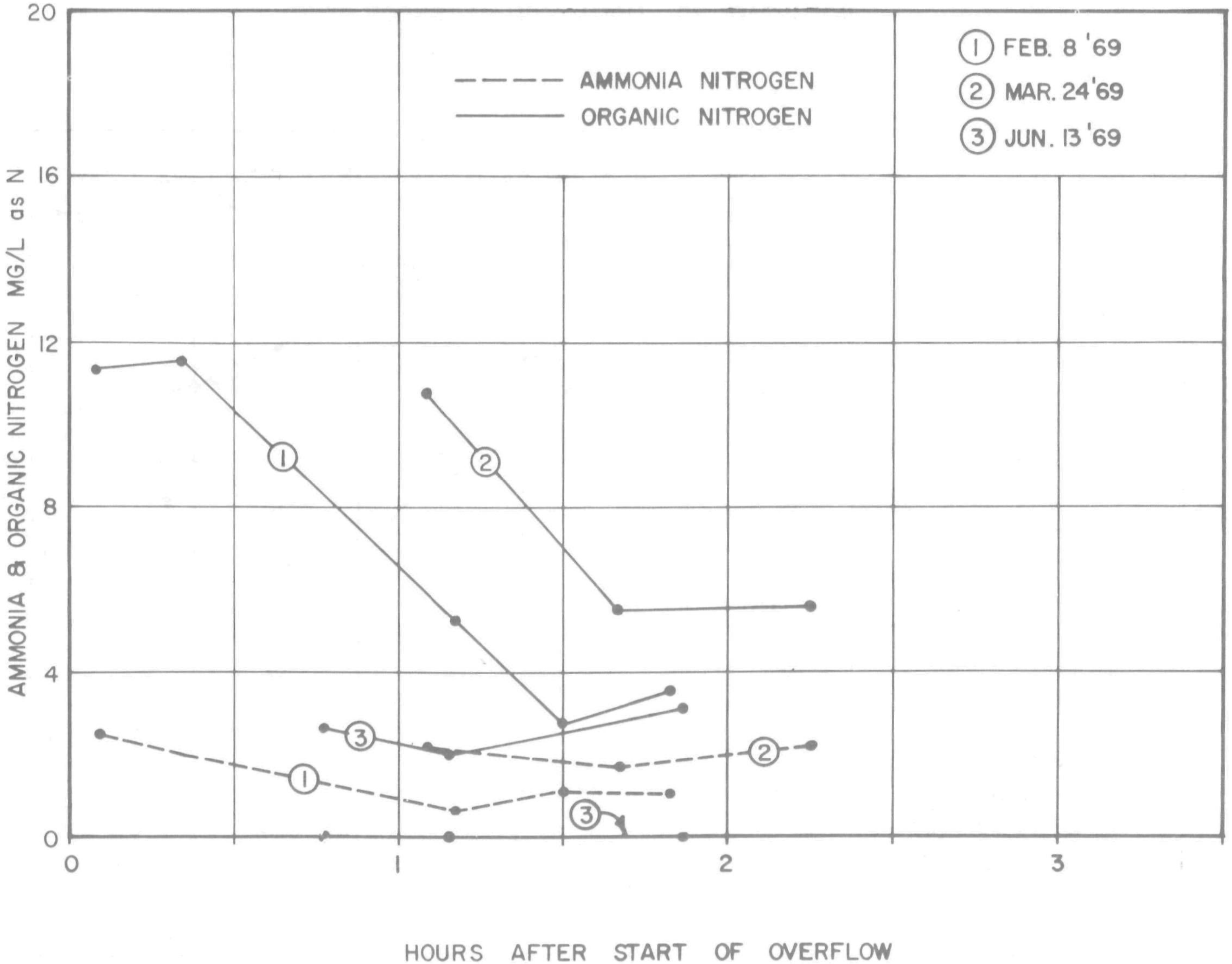


Figure No. 35
Ammonia & Organic Nitrogen
No. 8 Overflow

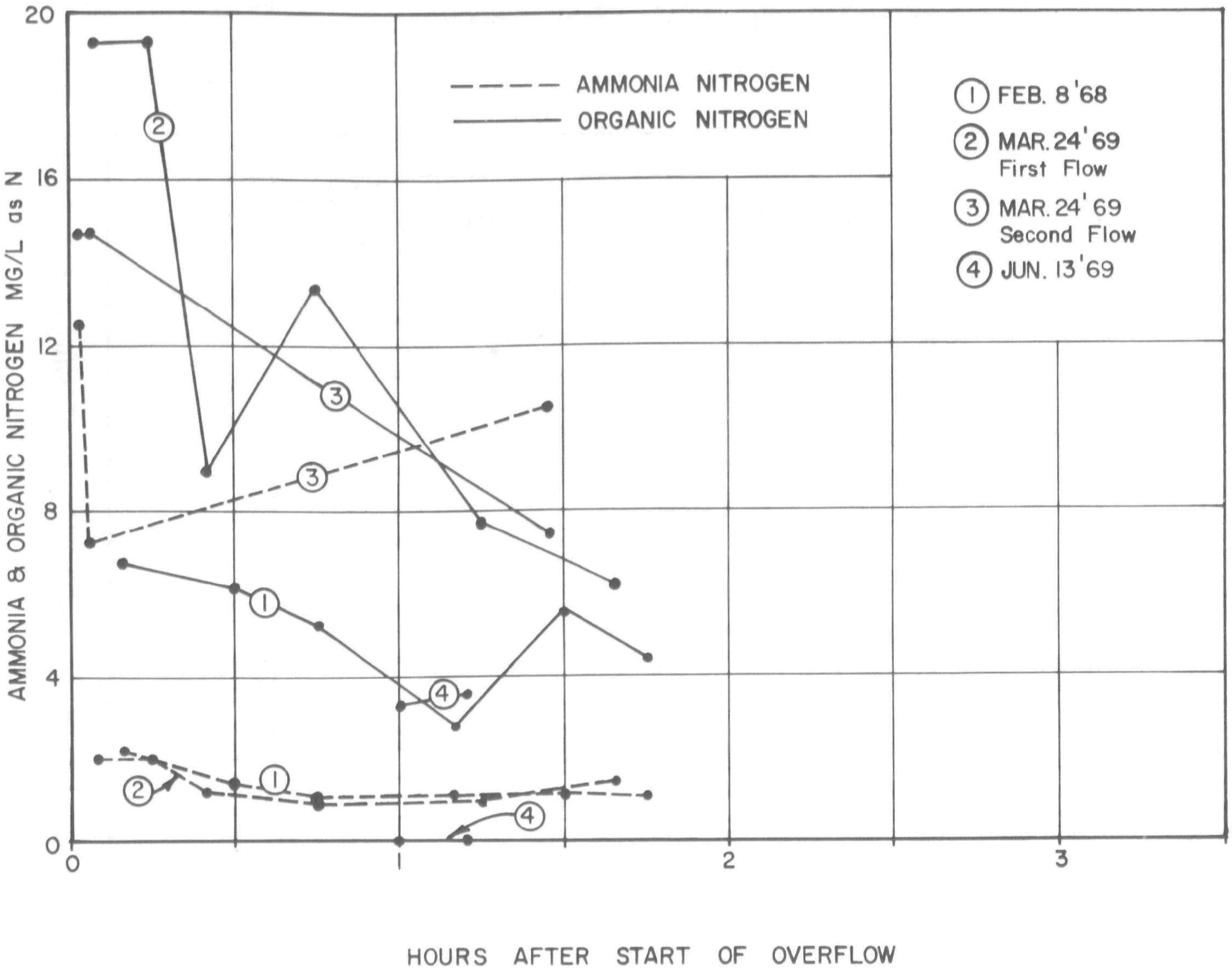


Figure No. 36
Ammonia & Organic Nitrogen
No. 17 Overflow

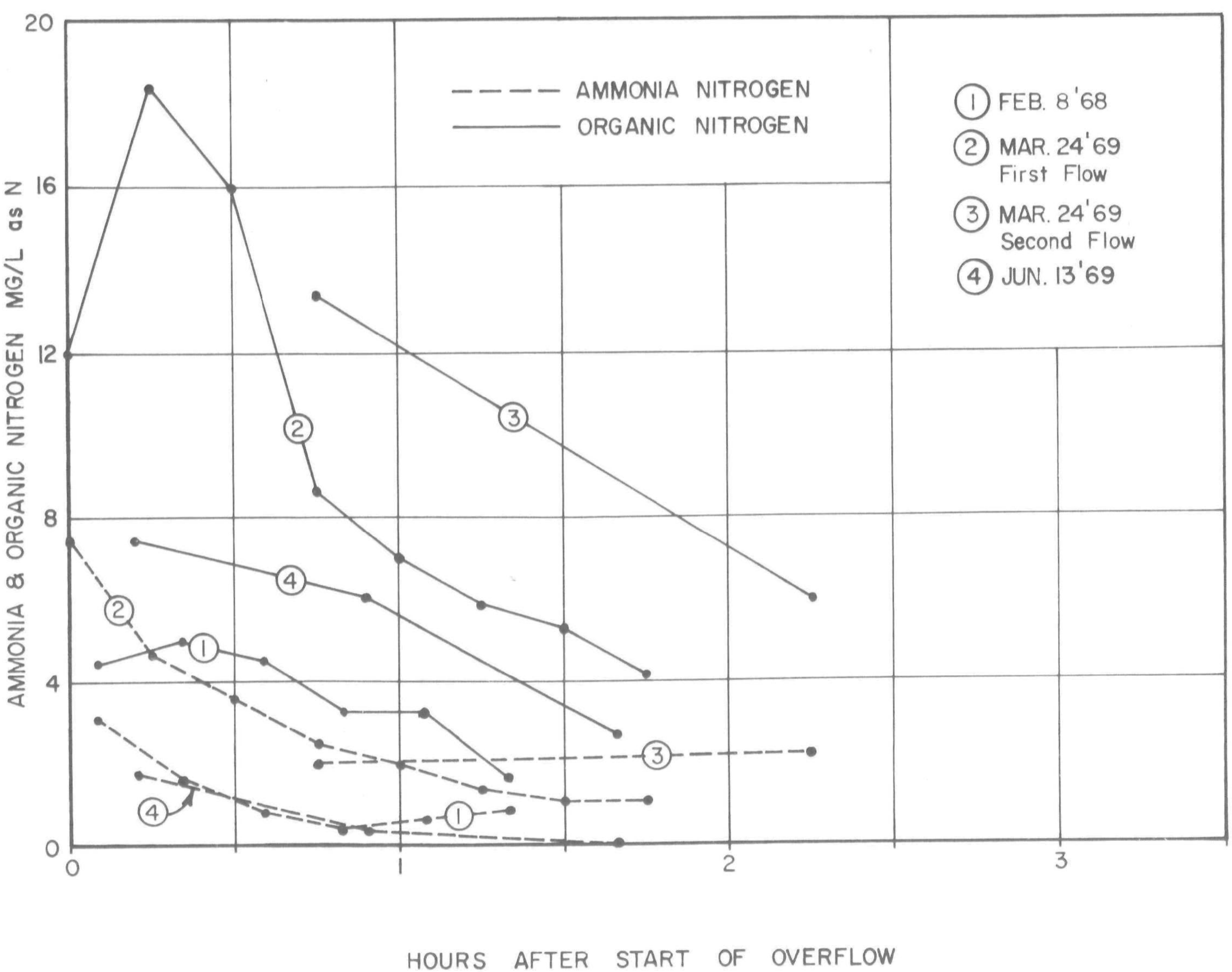


Figure No. 37
Ammonia & Organic Nitrogen
No. 23 Overflow

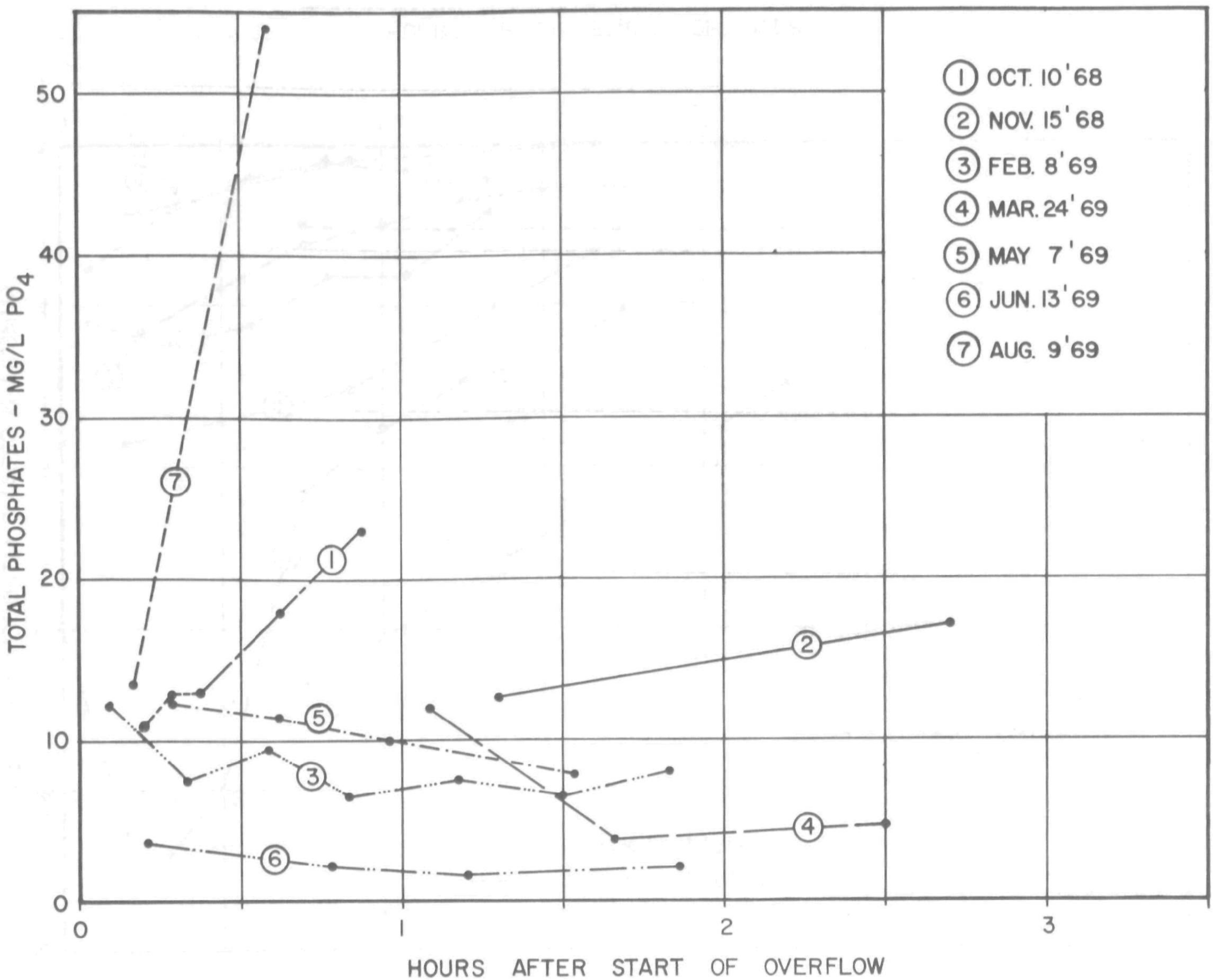


Figure No. 38
Total Phosphates
No. 8 Overflow

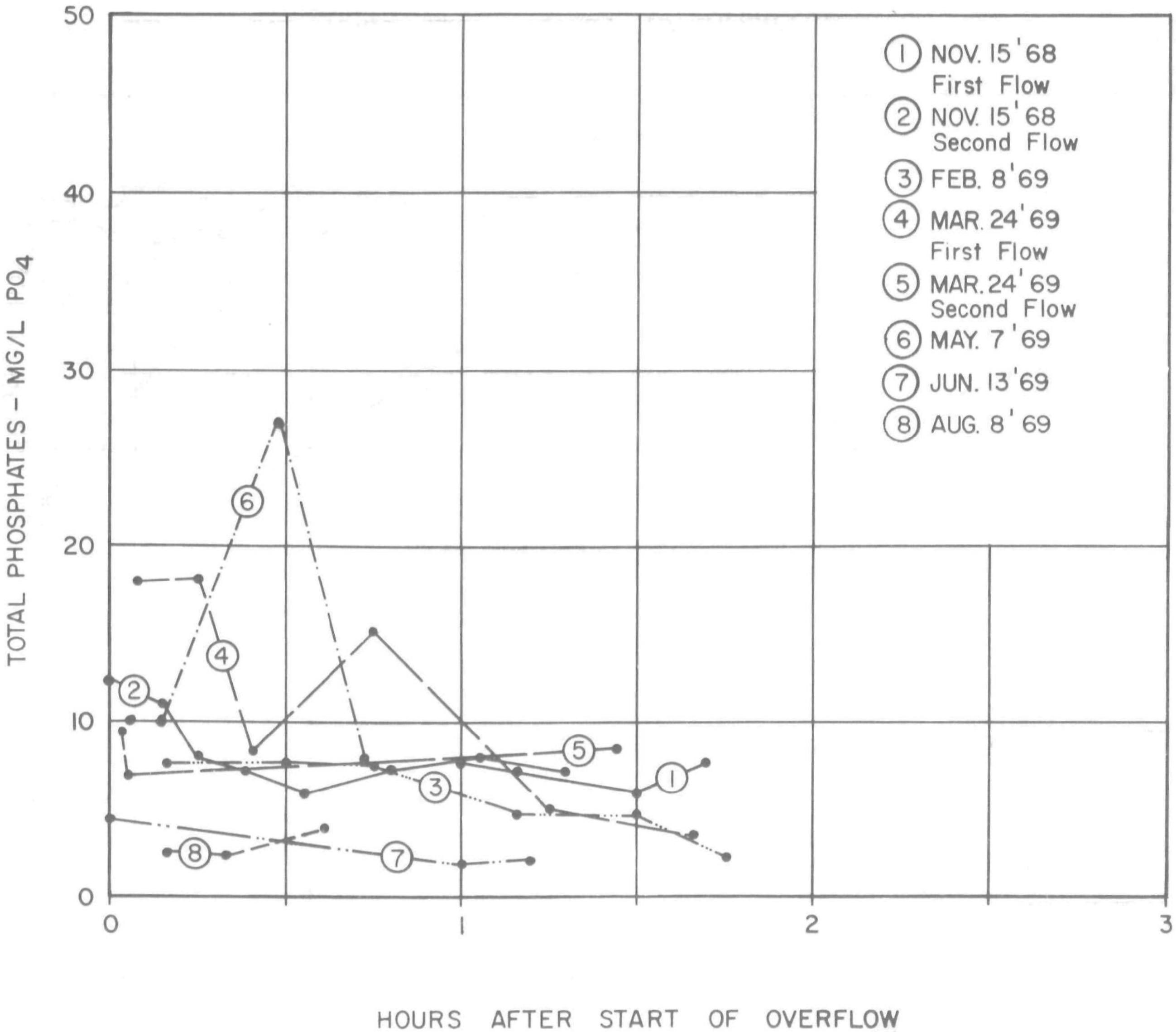


Figure No. 39
Total Phosphates
No. 17 Overflow

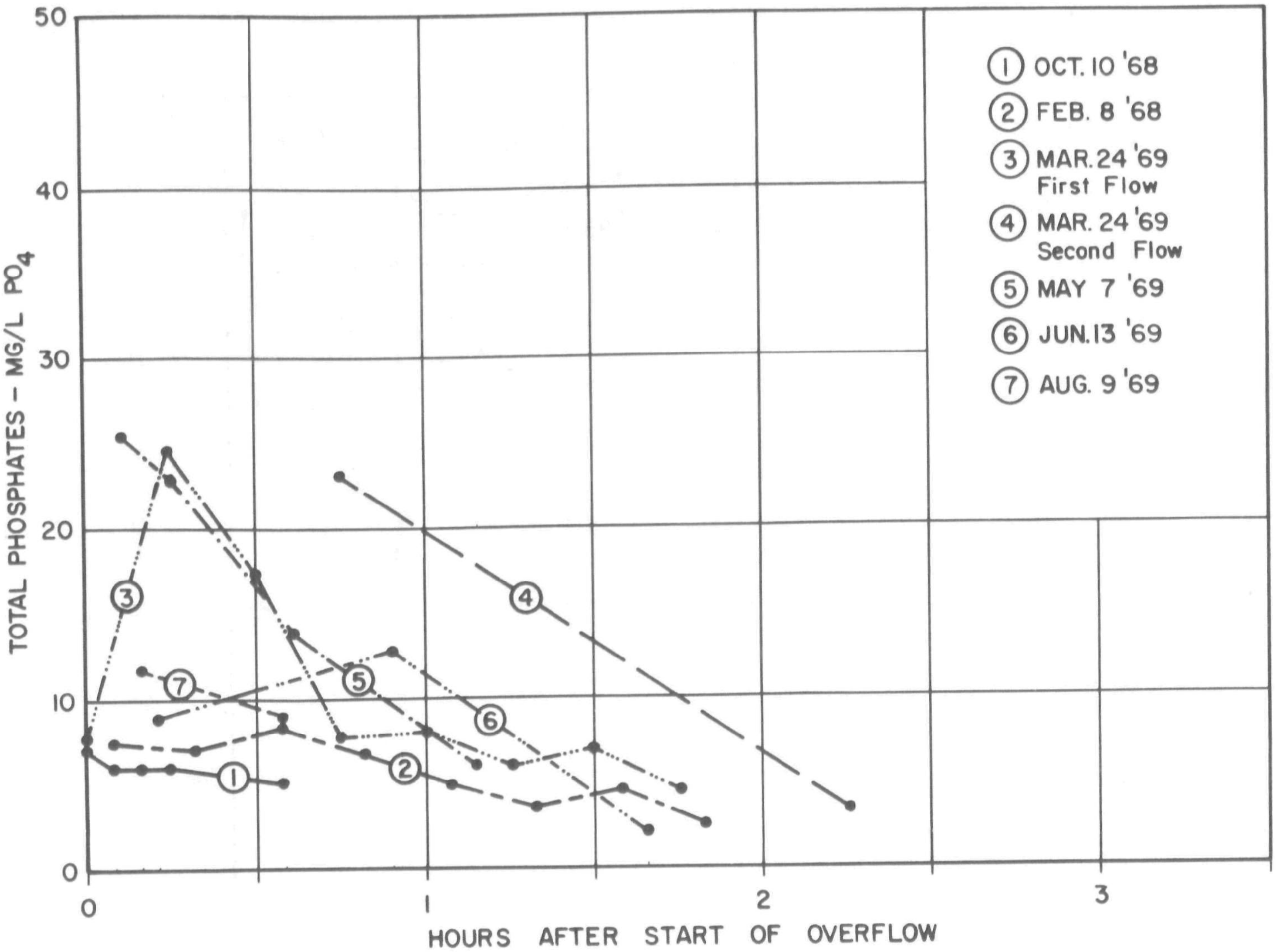


Figure No. 40
Total Phosphates
No. 23 Overflow

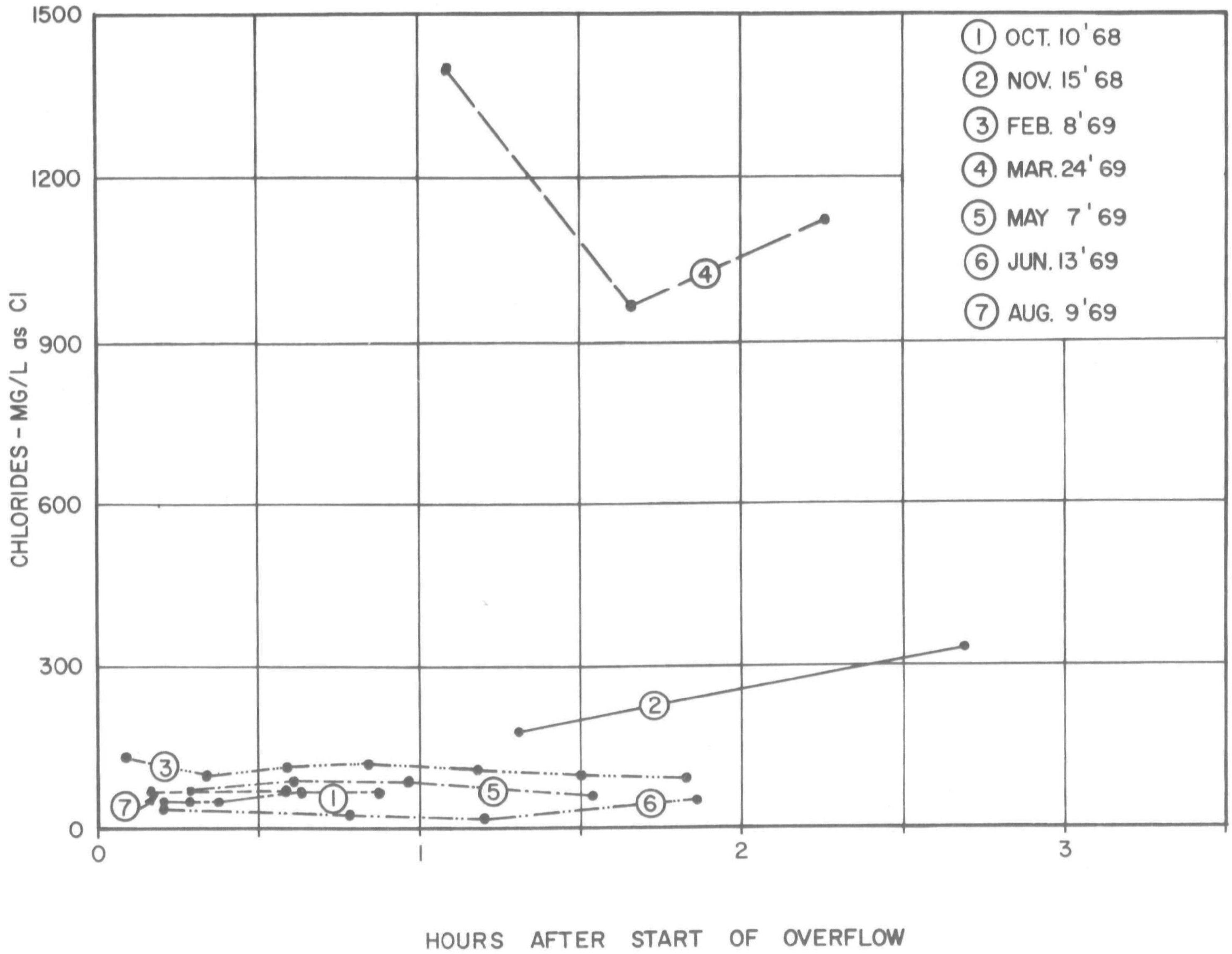


Figure No. 41
Chlorides
No. 8 Overflow

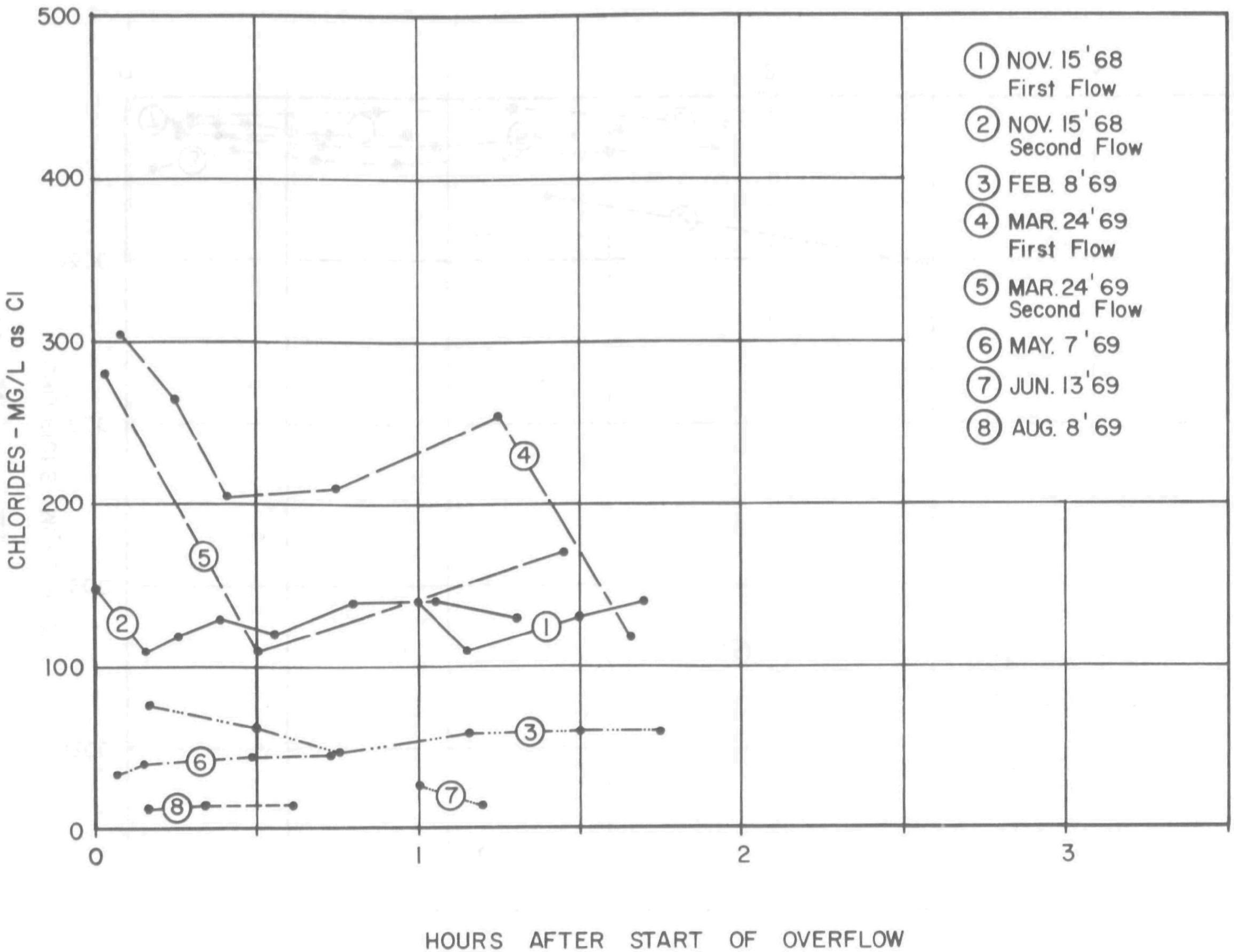


Figure No. 42
Chlorides
No. 17 Overflow

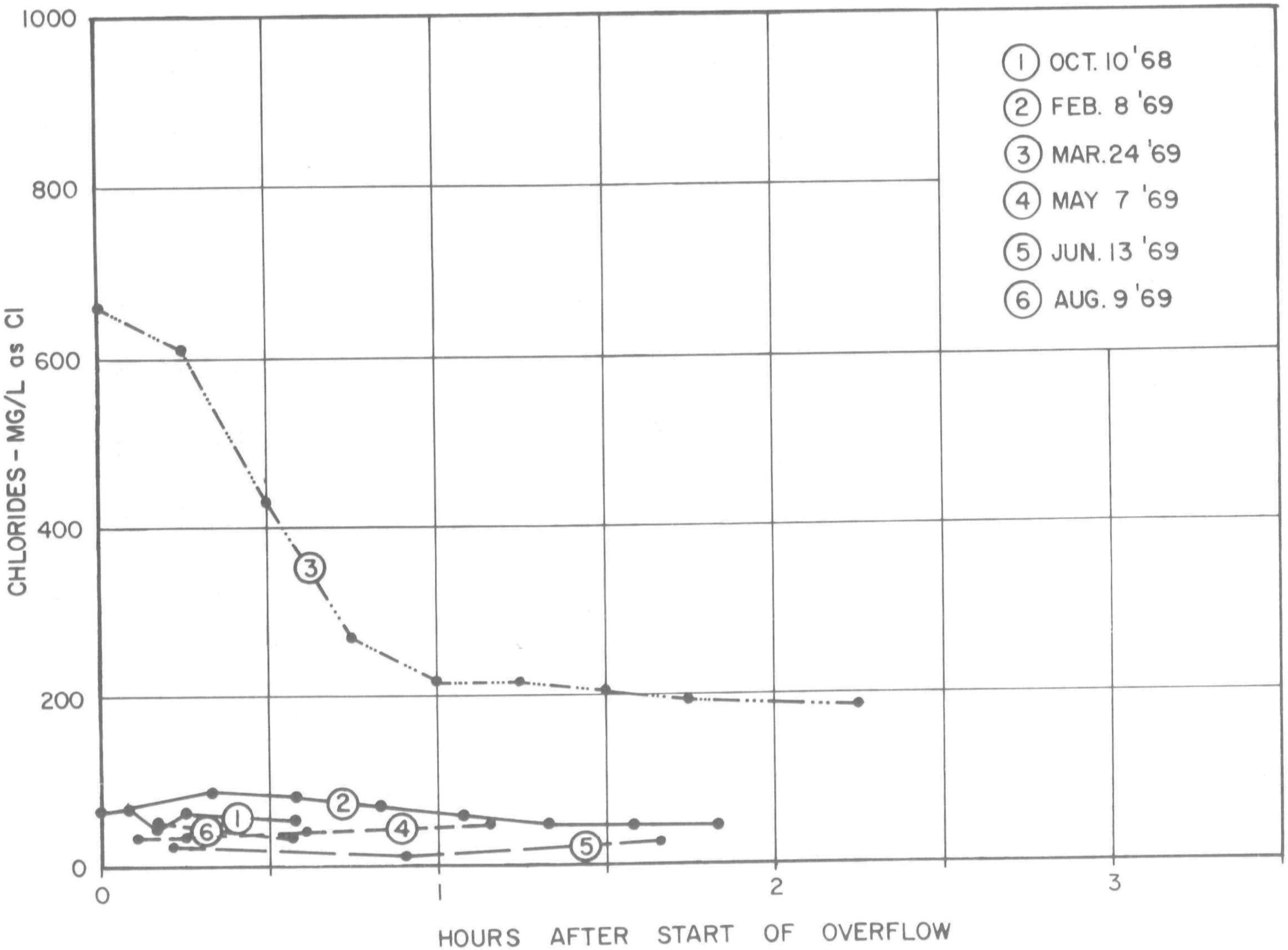


Figure No. 43
Chlorides
No. 23 Overflow

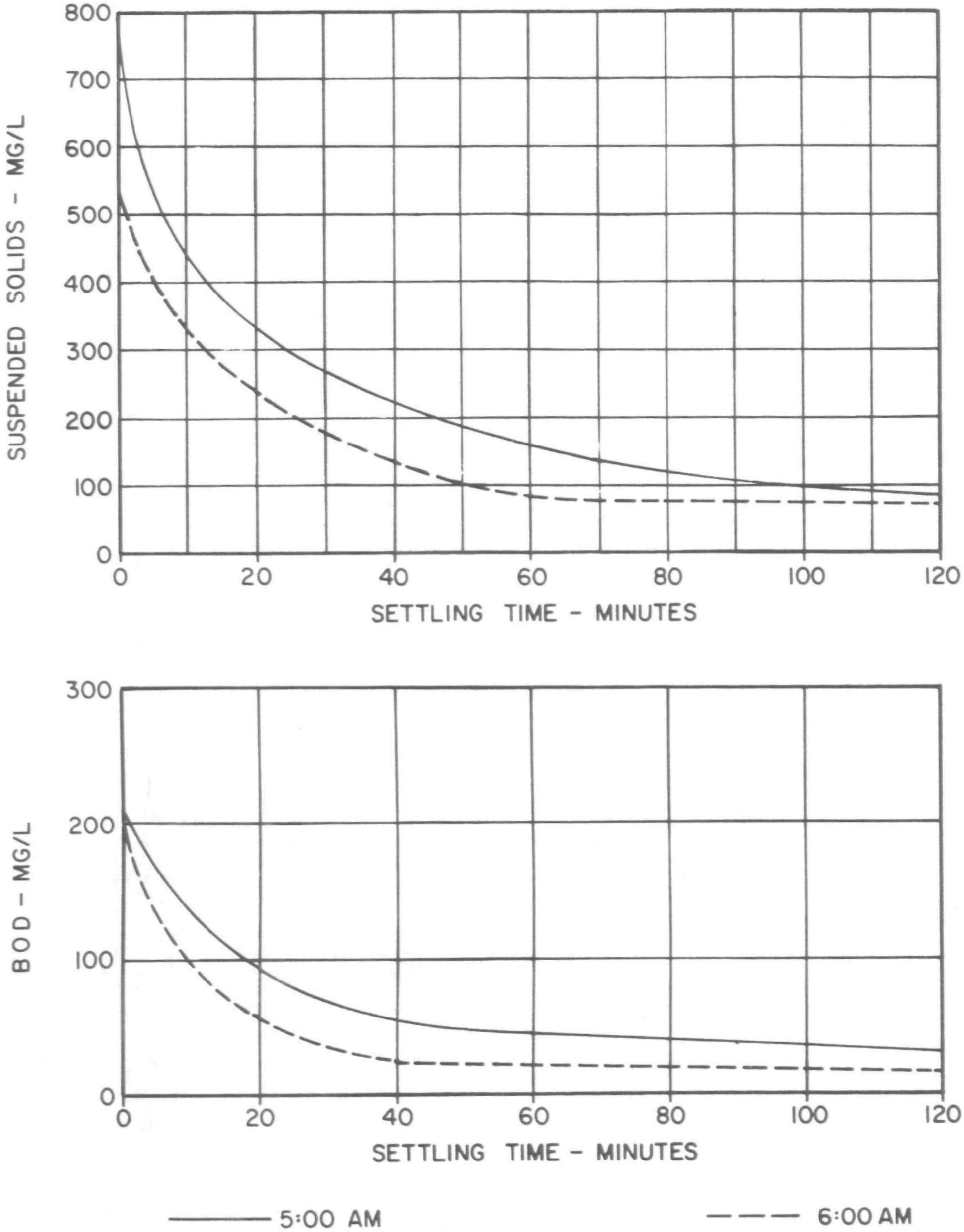


Figure No. 44
Effect of Settling on
BOD & Suspended Solids
No. 17 Overflow - Mar. 24, 1969

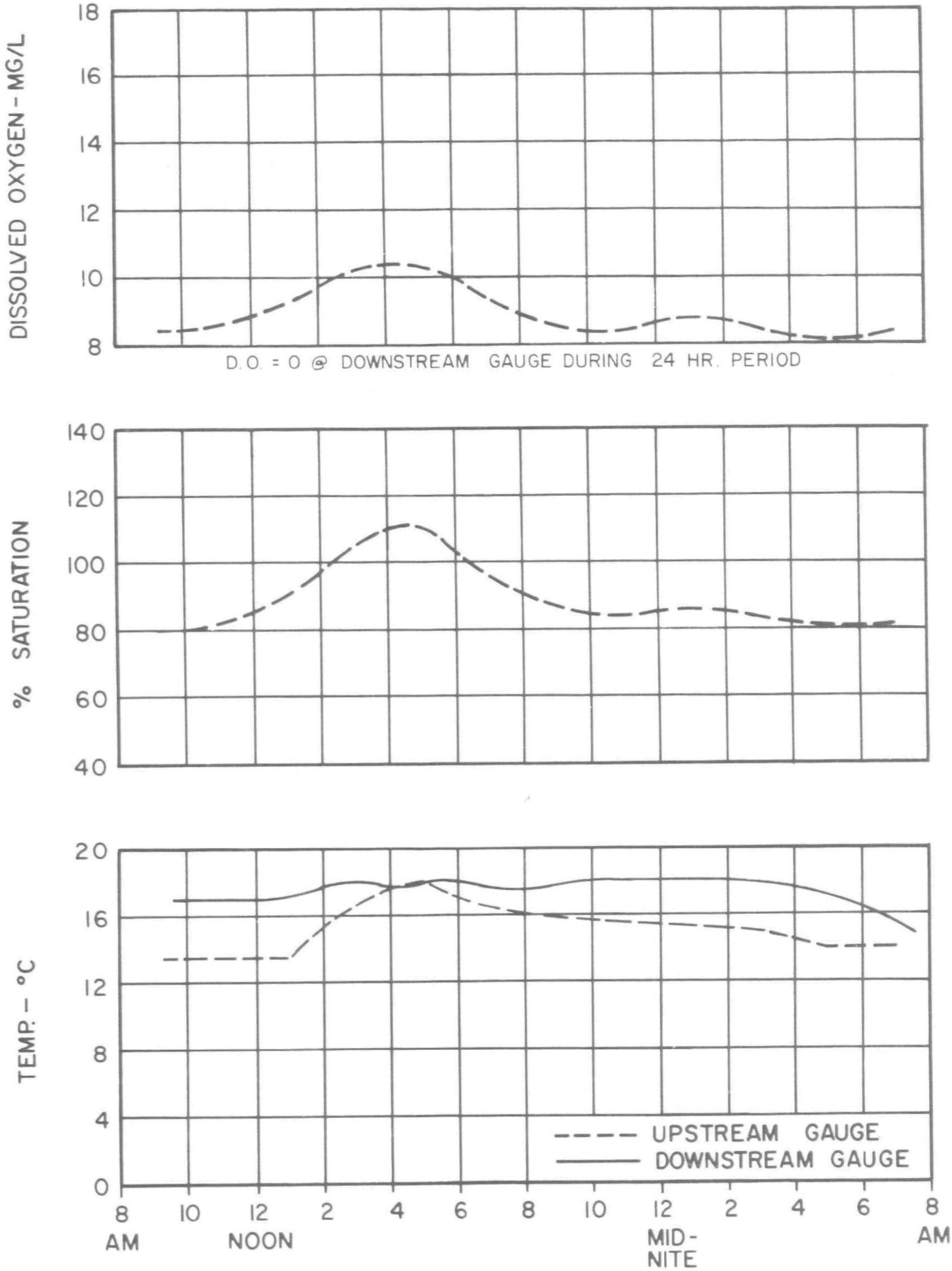


Figure No. 45
Diurnal Fluctuation In Dissolved Oxygen
Sandusky River - Oct. 9 & 10, 1968

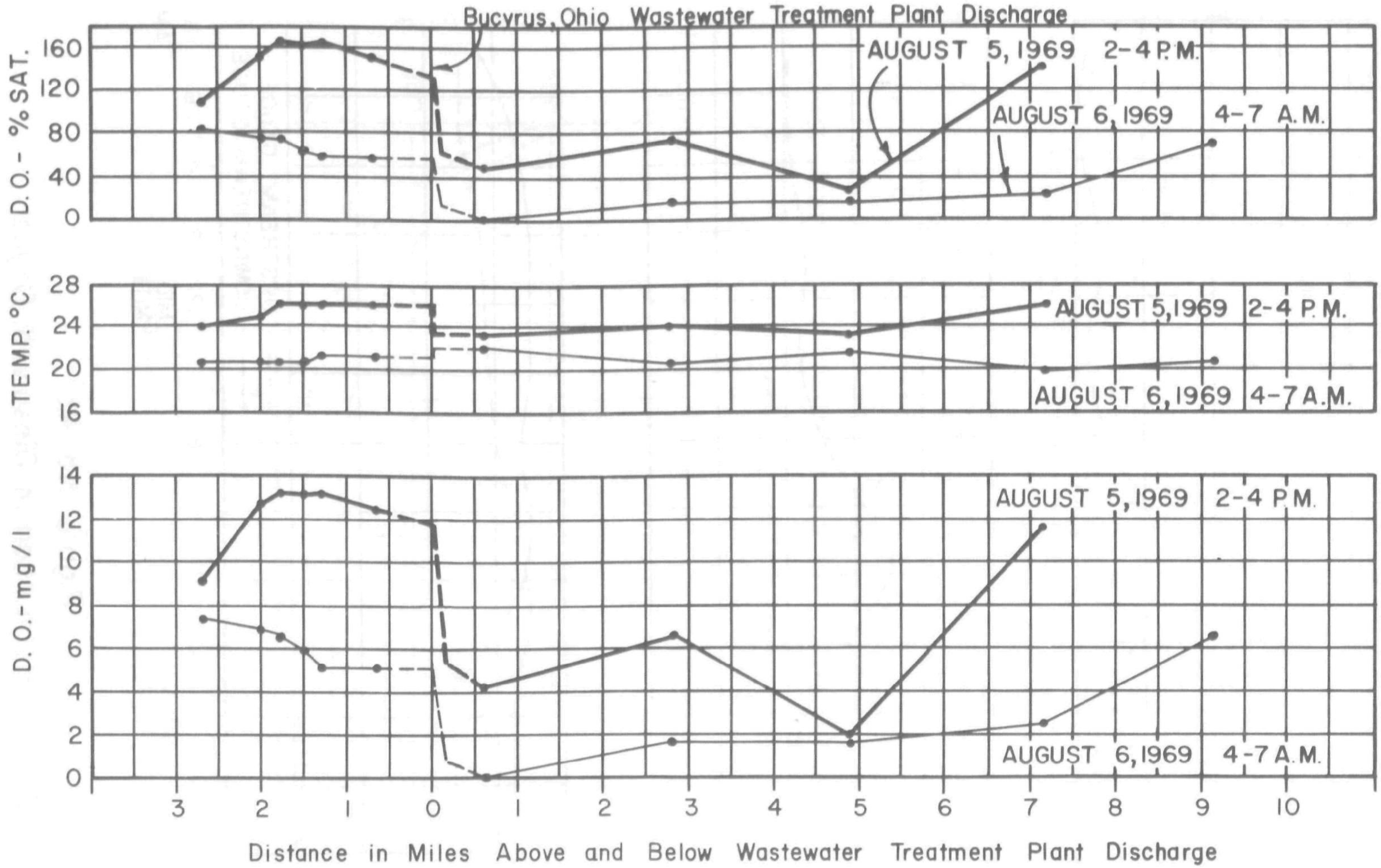


Figure No. 46

Diurnal Affect on Dissolved Oxygen in the Sandusky River

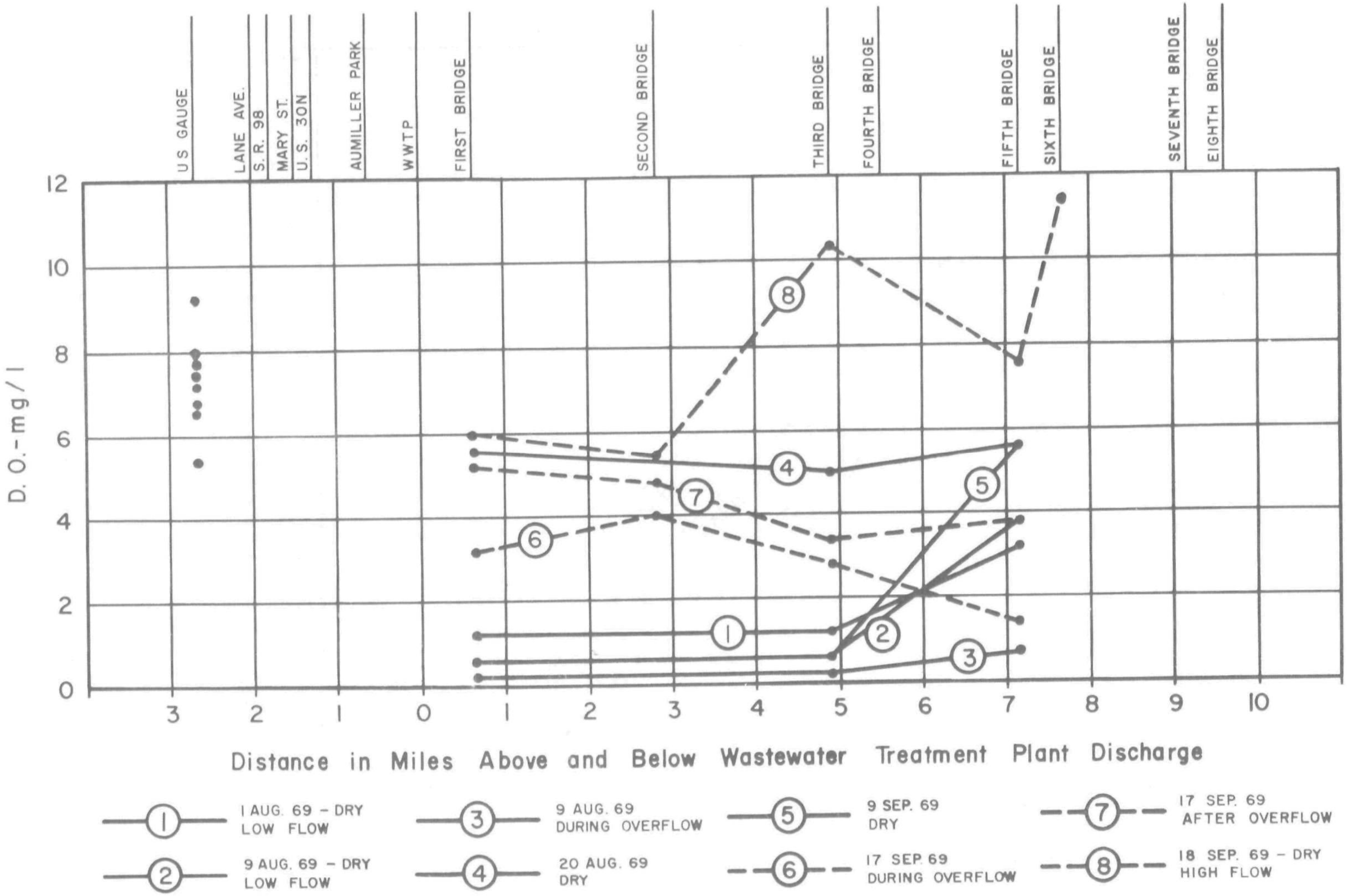


Figure No. 47
Dissolved Oxygen Profile of the Sandusky River
During Dry and Wet Weather

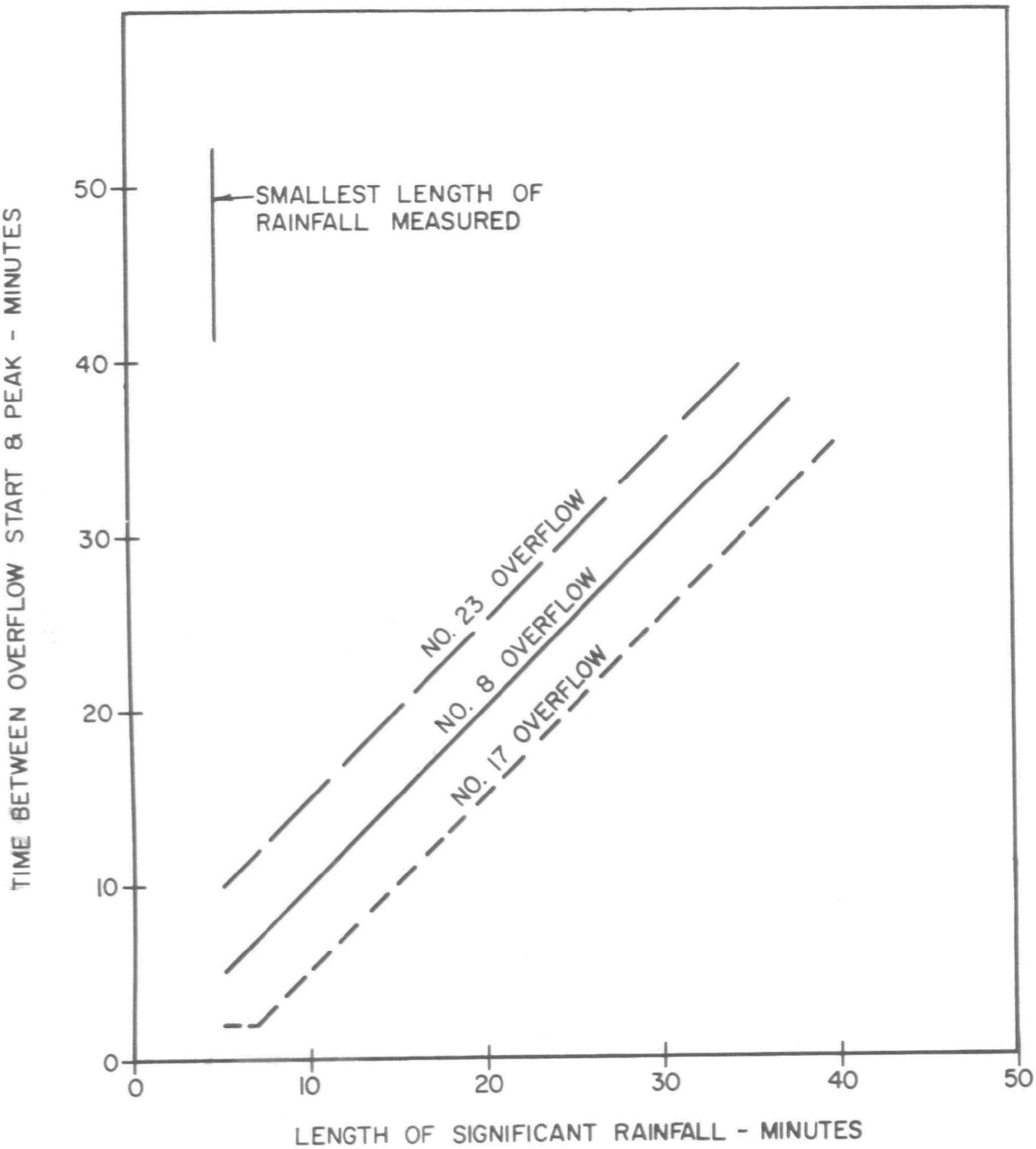


Figure No. 48
Overflow Peak Time
vs. Length of Rainfall
No's. 8, 17, & 23 Overflows

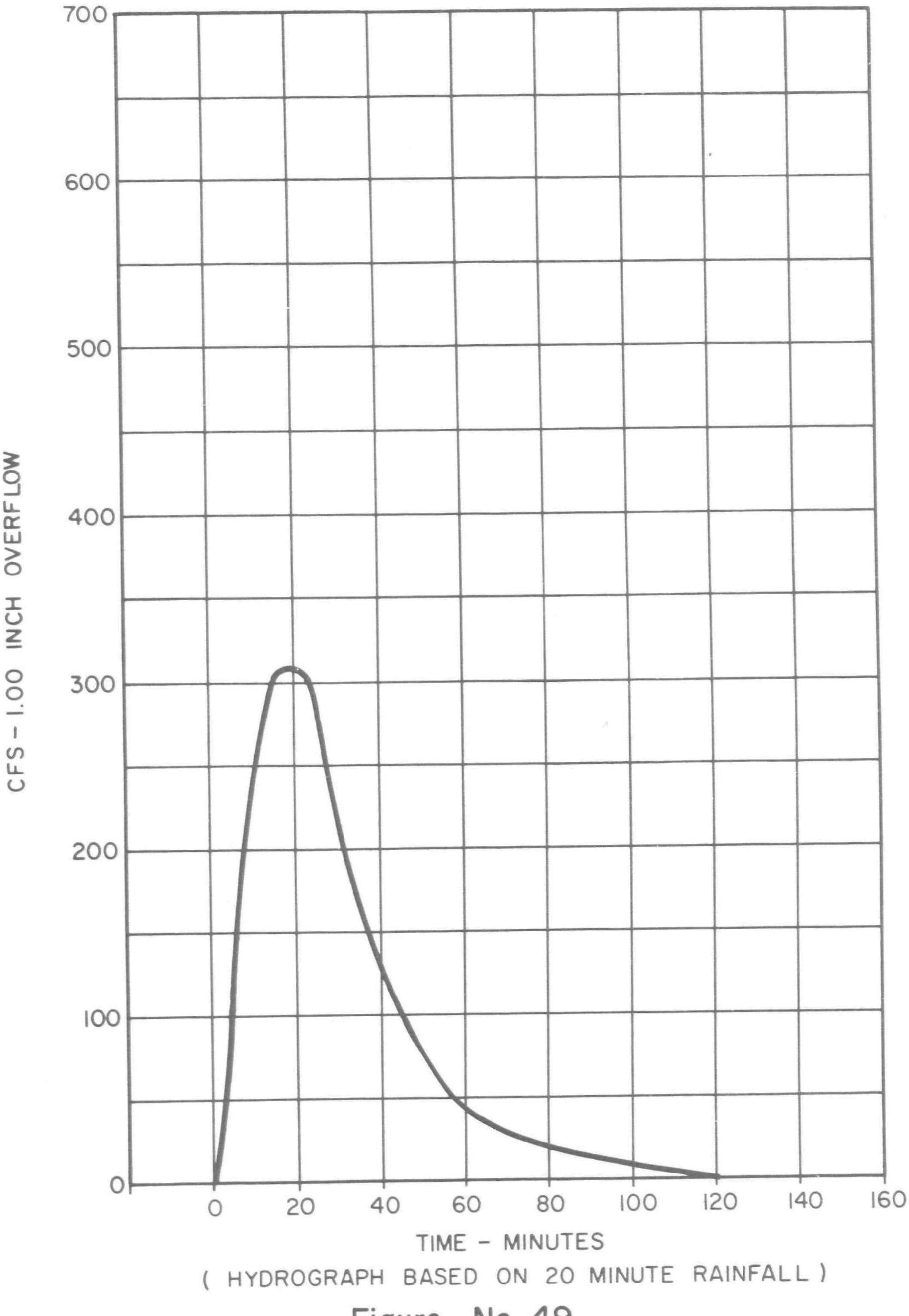


Figure No. 49
Unit Hydrograph
No. 8 Overflow

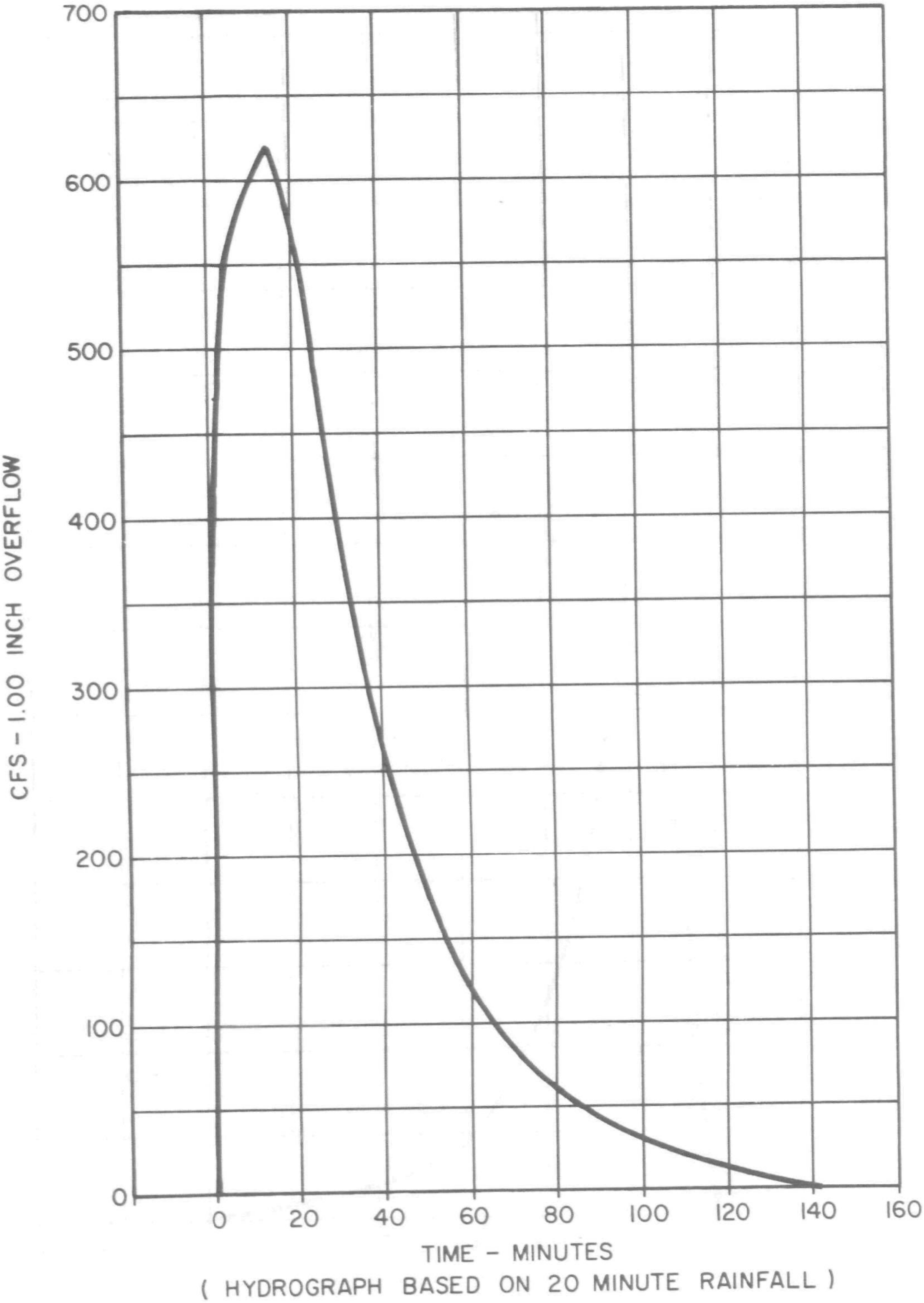


Figure No. 50
Unit Hydrograph
No. 17 Overflow

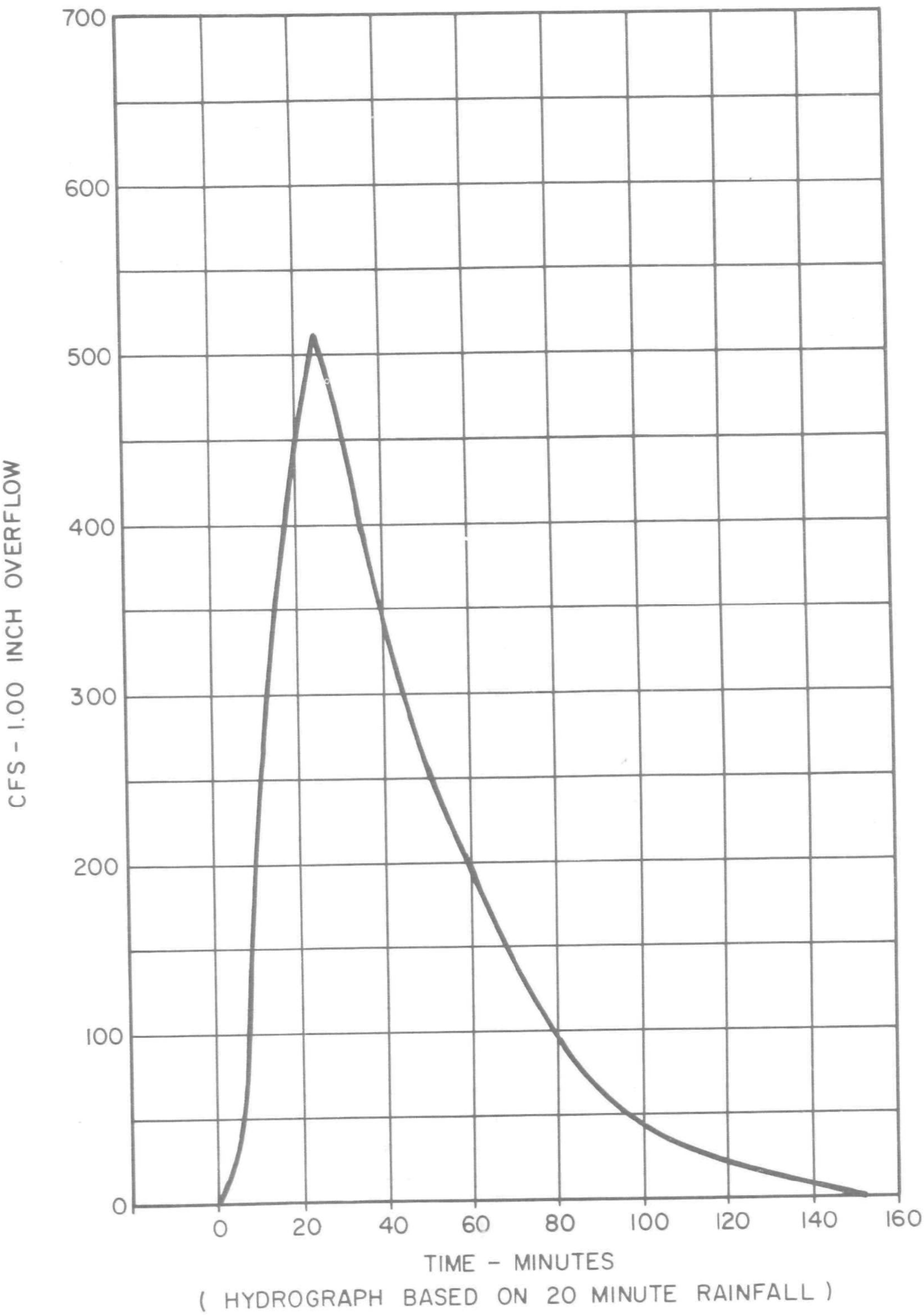


Figure No. 51
Unit Hydrograph
No. 23 Overflow

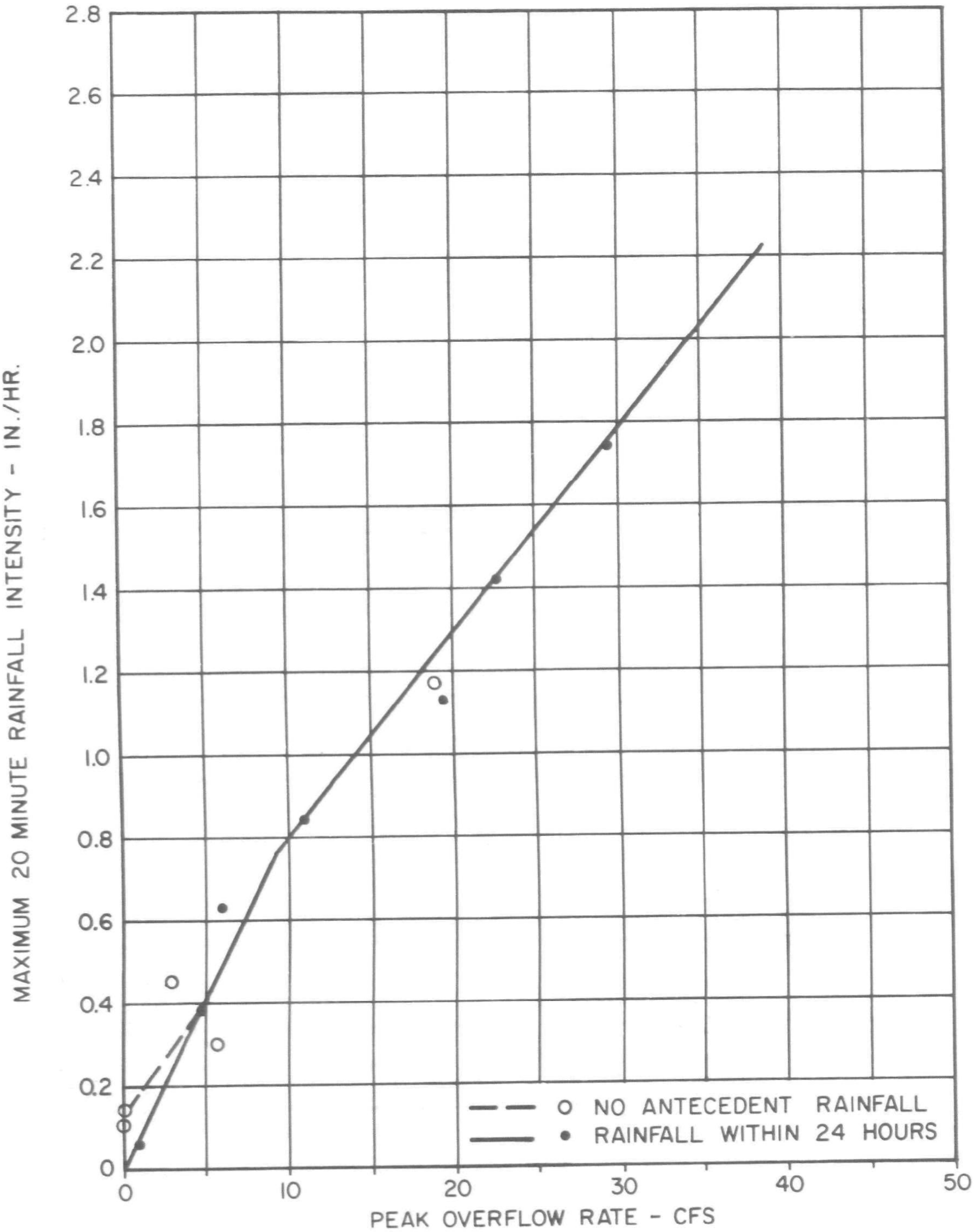


Figure No. 52
Peak Rainfall vs. Peak Overflow Rate
No. 8 Overflow

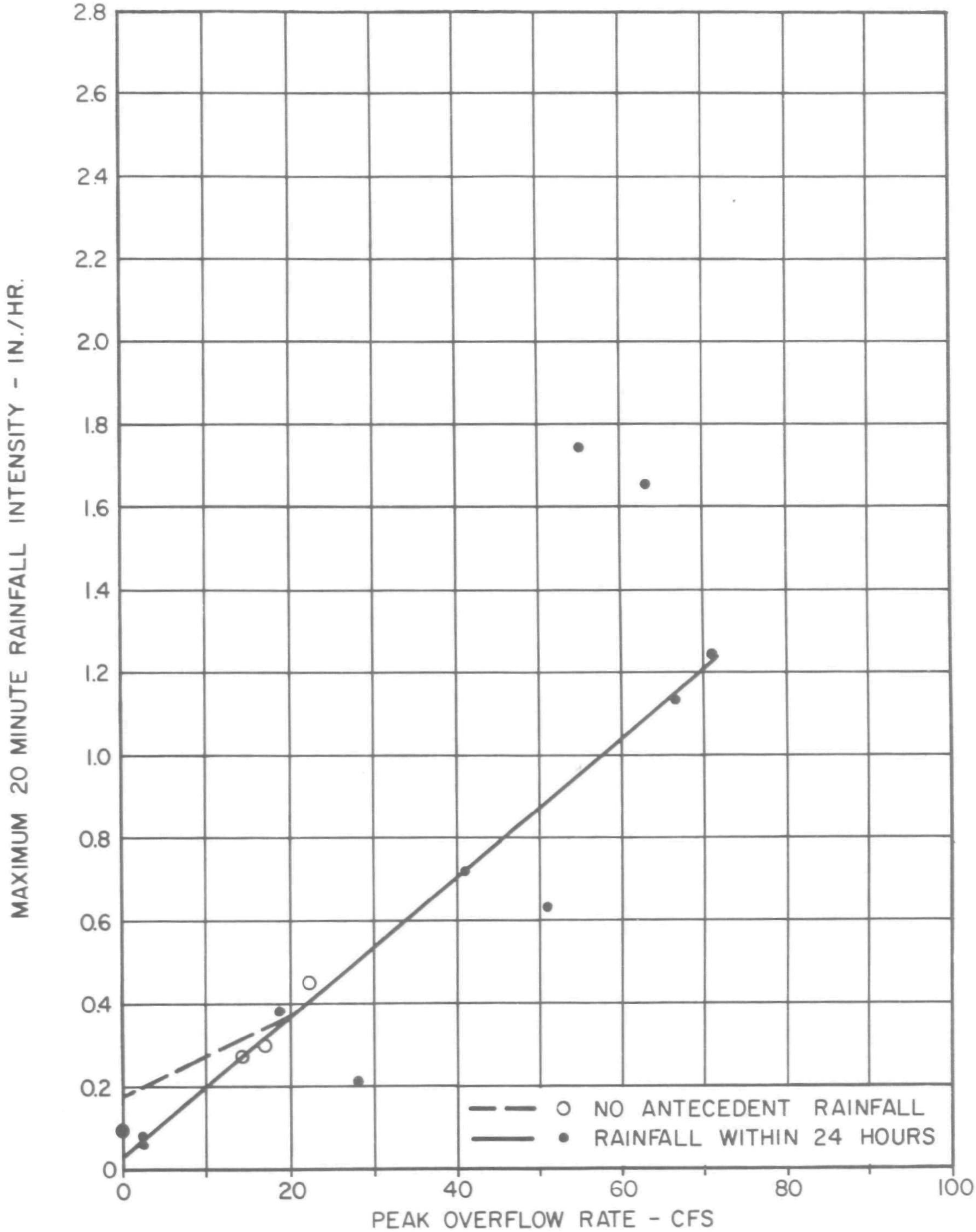


Figure No. 53
Peak Rainfall vs. Peak Overflow Rate
No. 17 Overflow

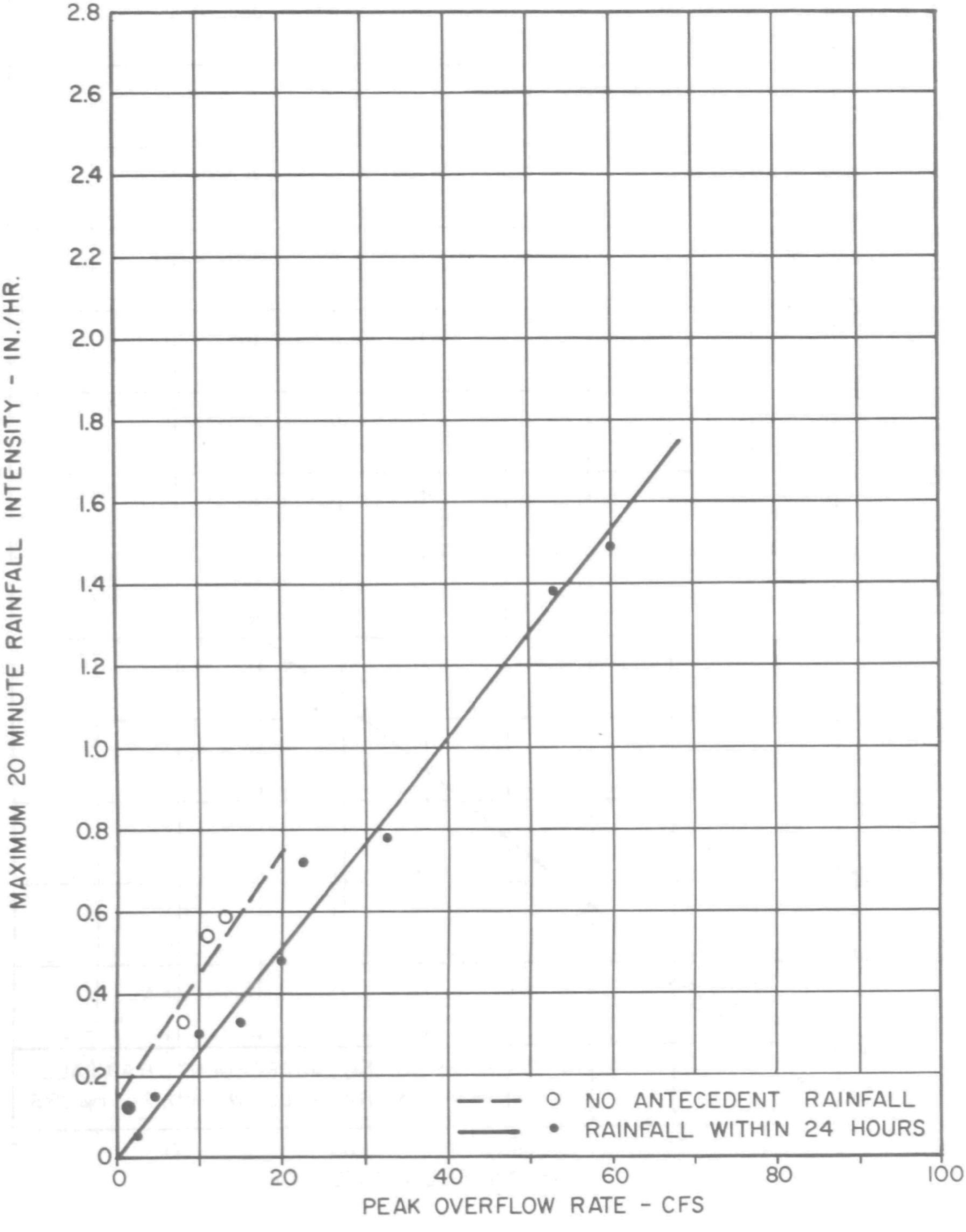


Figure No. 54
Peak Rainfall vs. Peak Overflow Rate
No. 23 Overflow

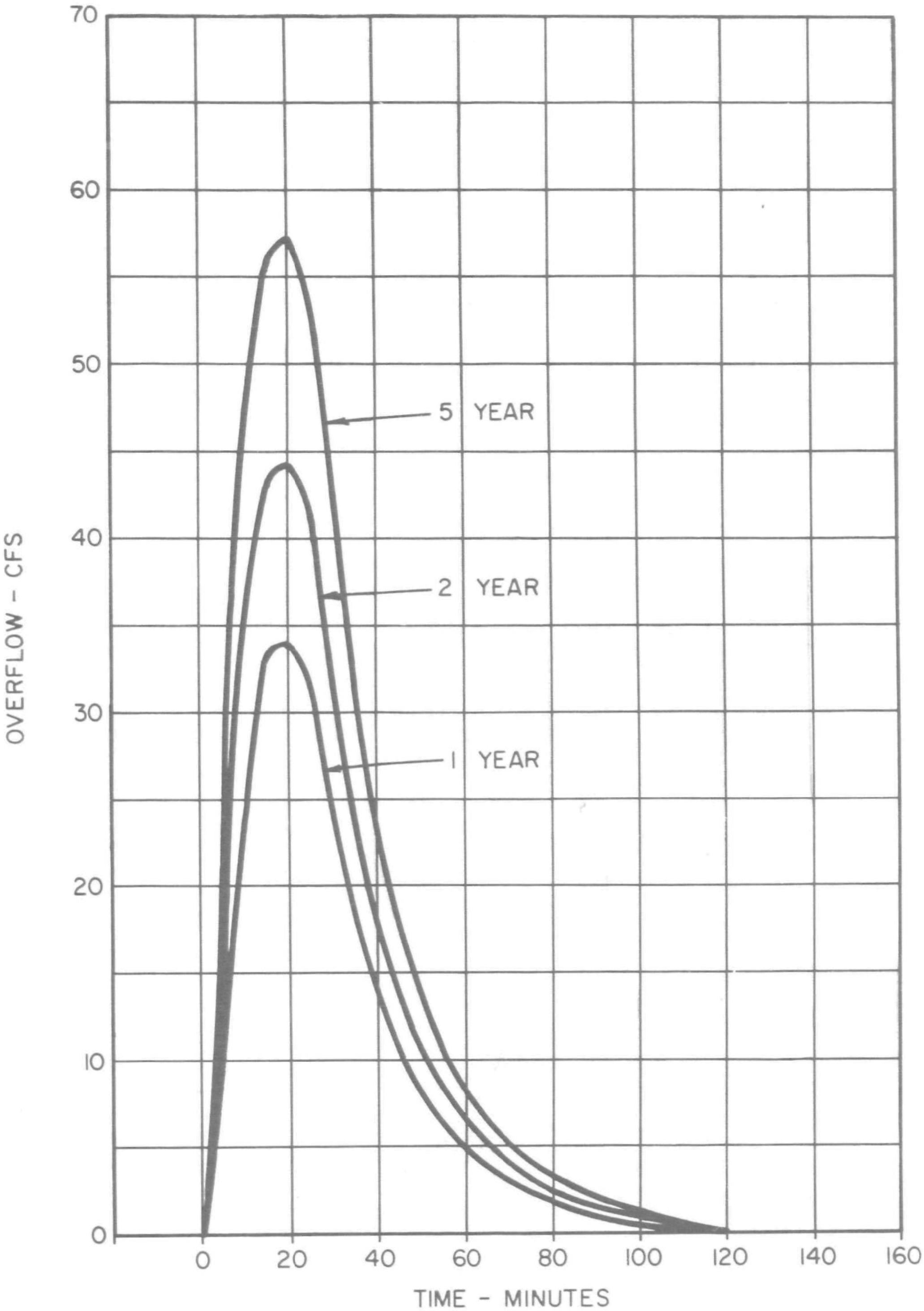


Figure No. 55
Overflow Hydrograph
Twenty Minute Storm
No. 8 Overflow

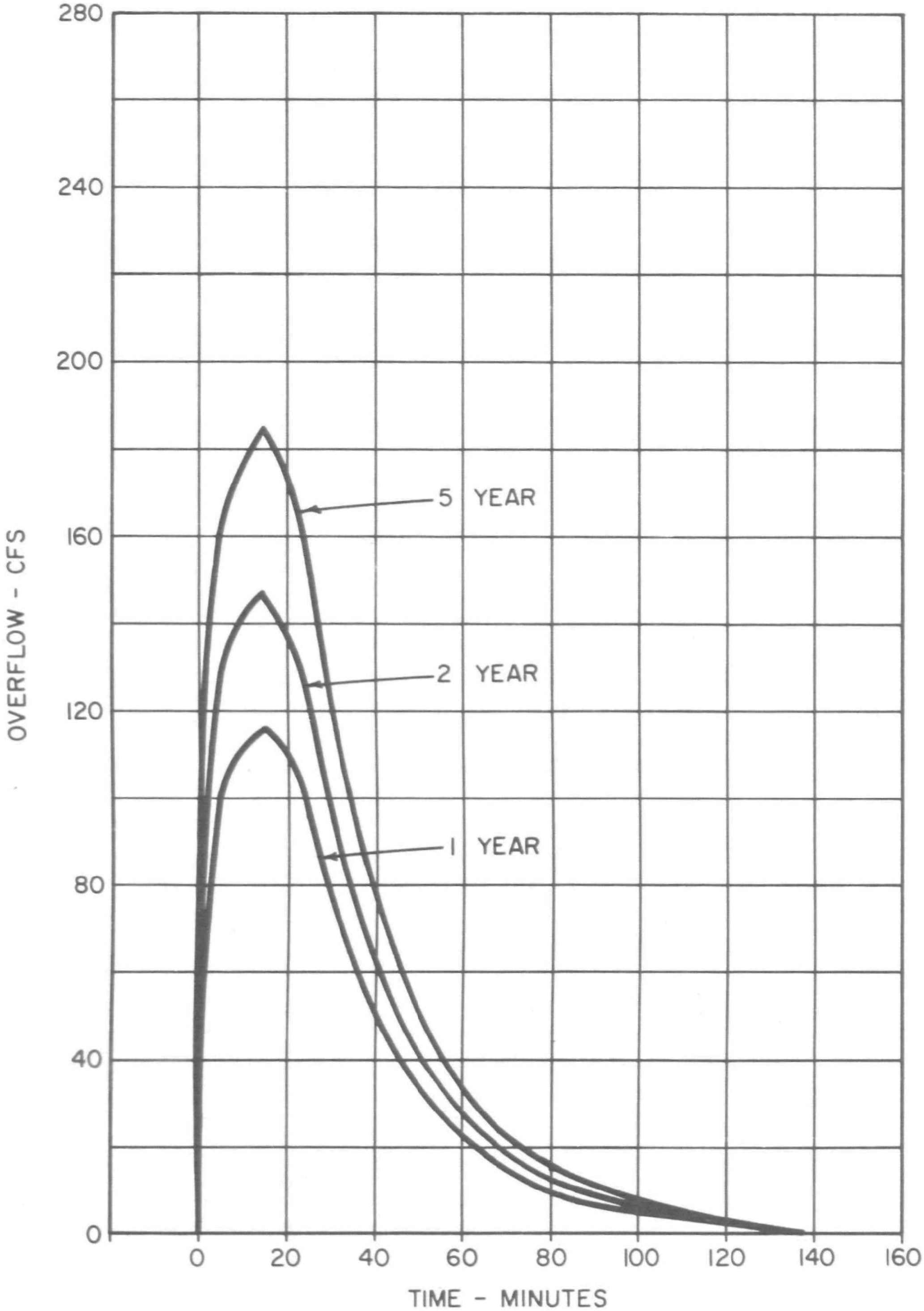


Figure No. 56
Overflow Hydrograph
Twenty Minute Storm
No. 17 Overflow

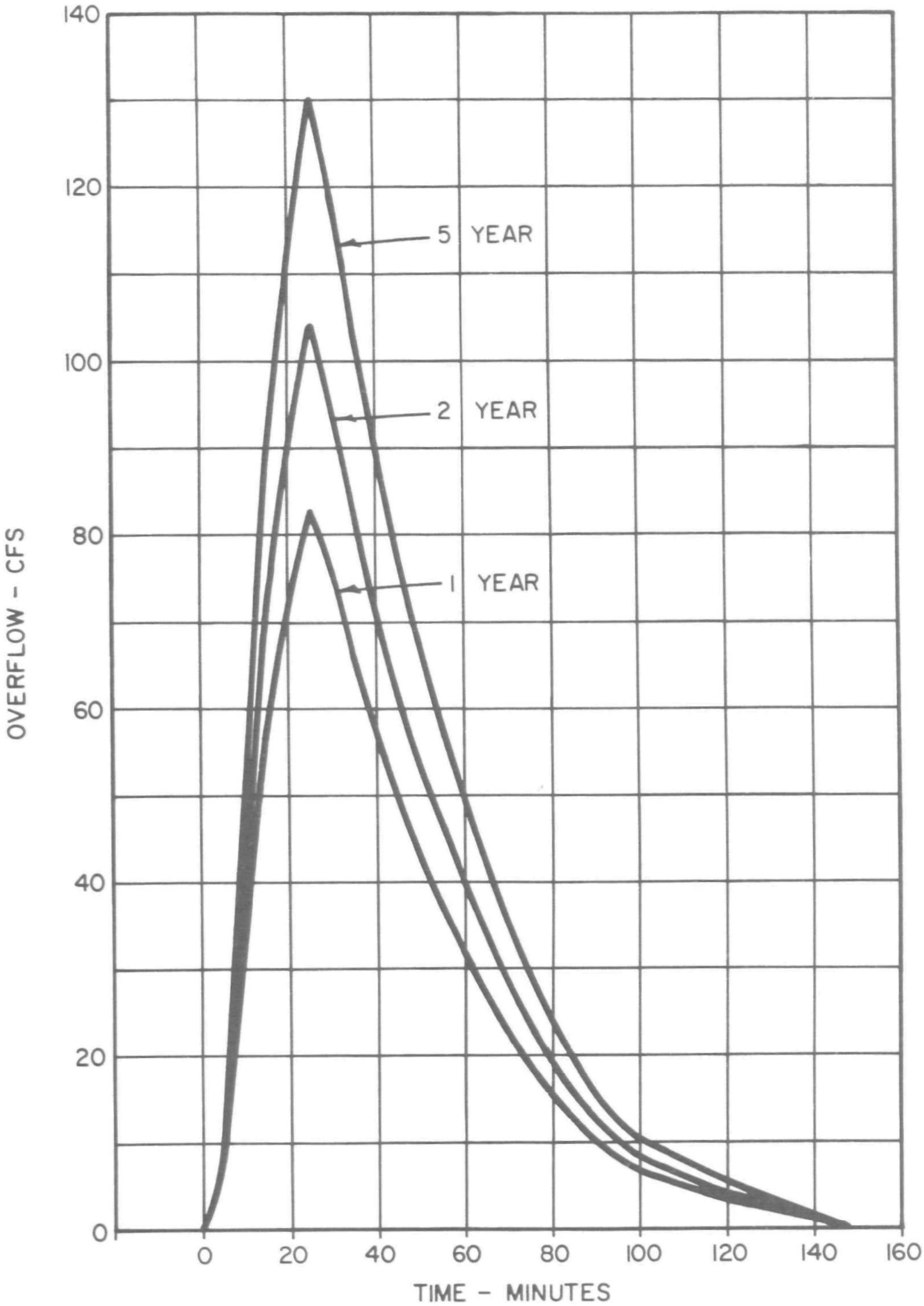


Figure No. 57
Overflow Hydrograph
Twenty Minute Storm
No. 23 Overflow

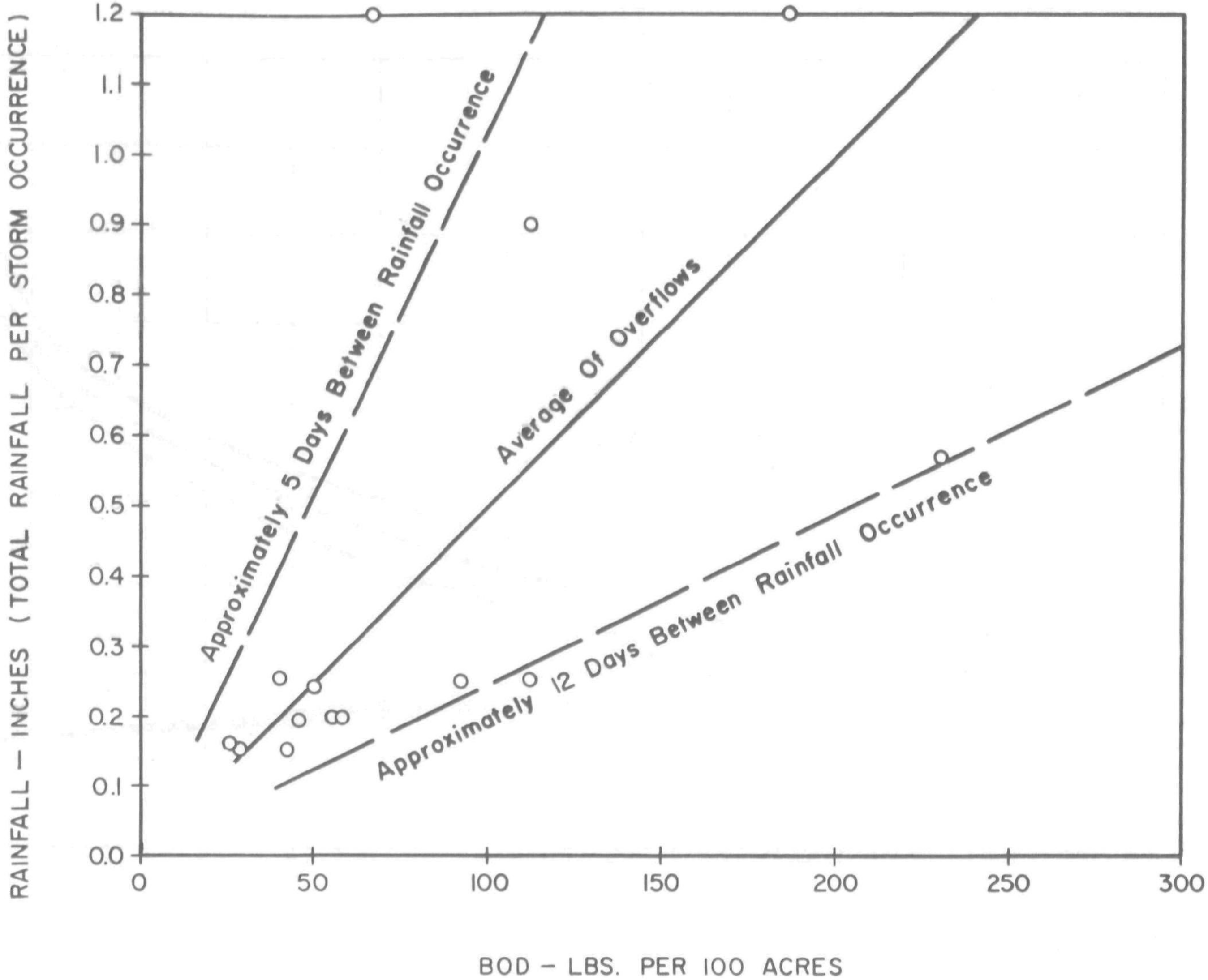


Figure No. 58
Rainfall vs. BOD

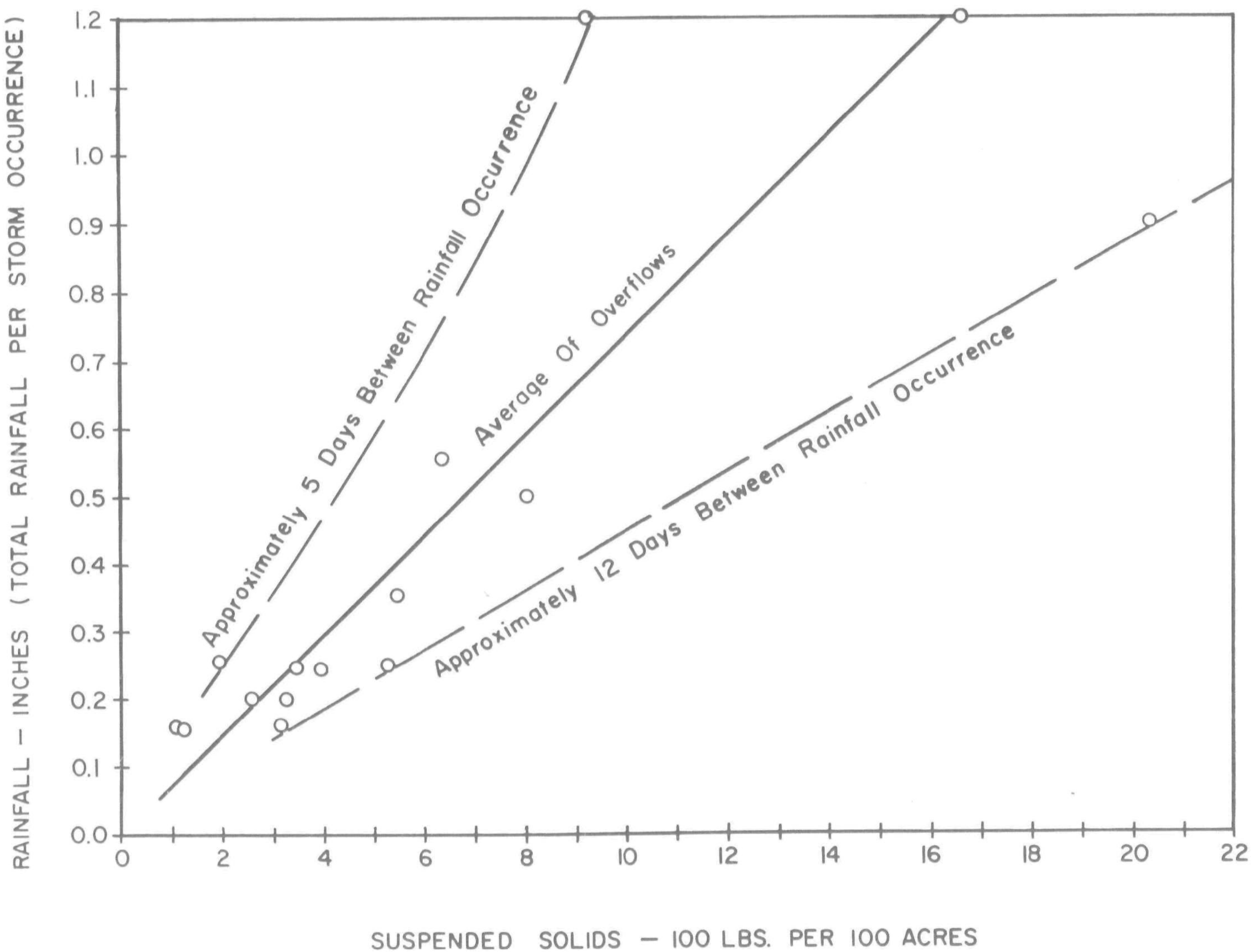


Figure No. 59
Rainfall vs. Suspended Solids

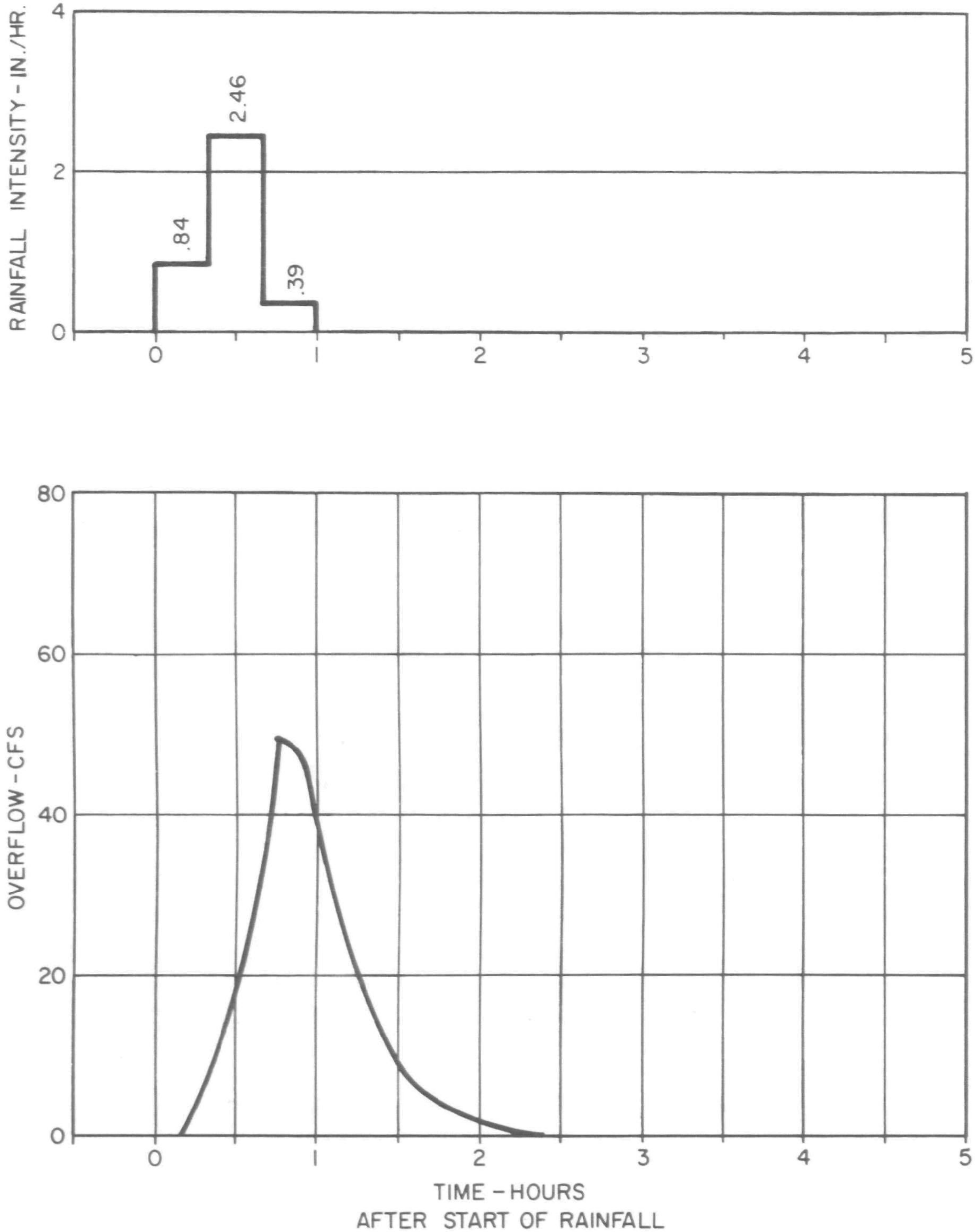


Figure No. 60
Rainfall and Overflow
Two Year, One Hour Storm
No. 8 Overflow

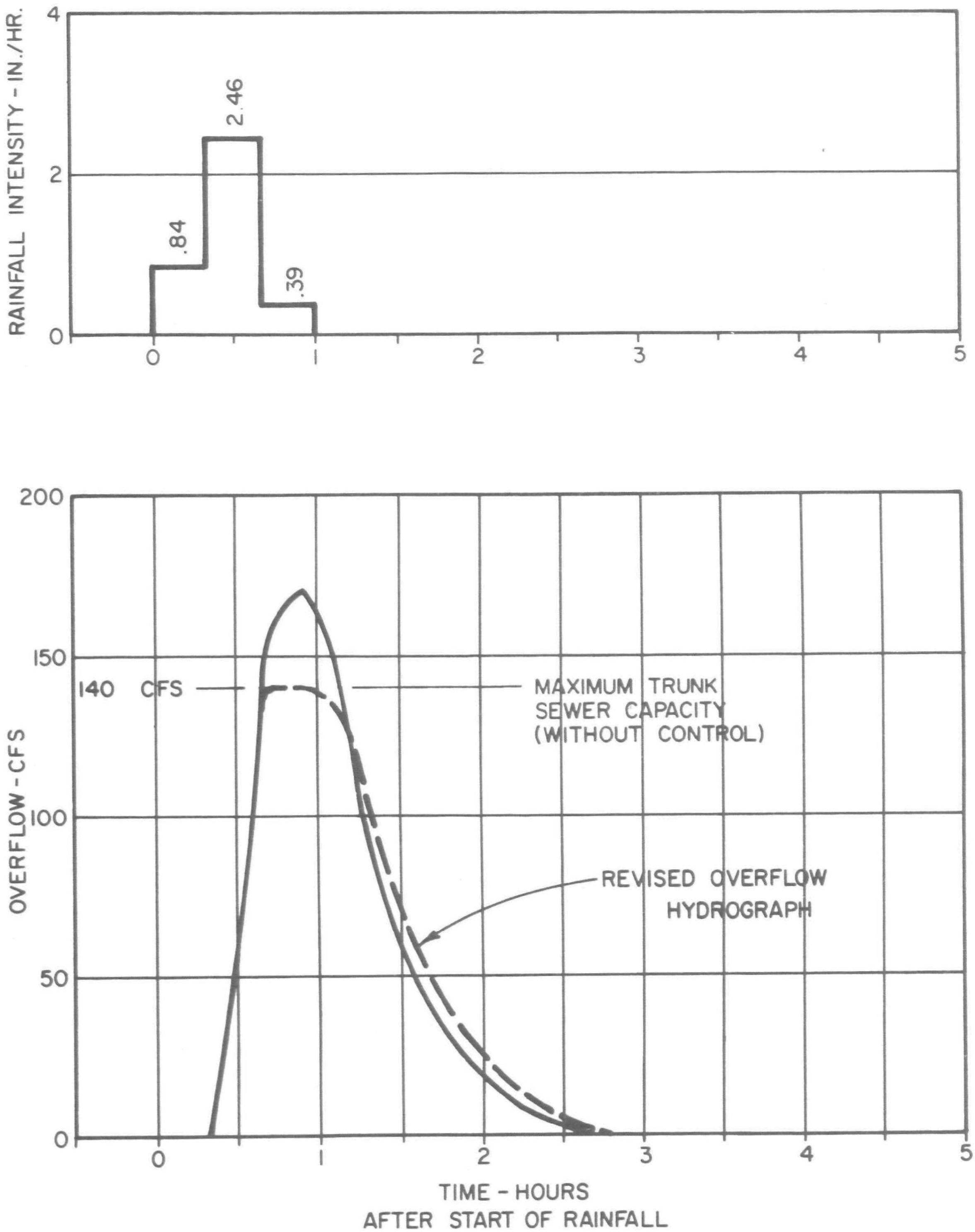


Figure No. 61
Rainfall and Overflow
Two Year, One Hour Storm
No. 17 Overflow

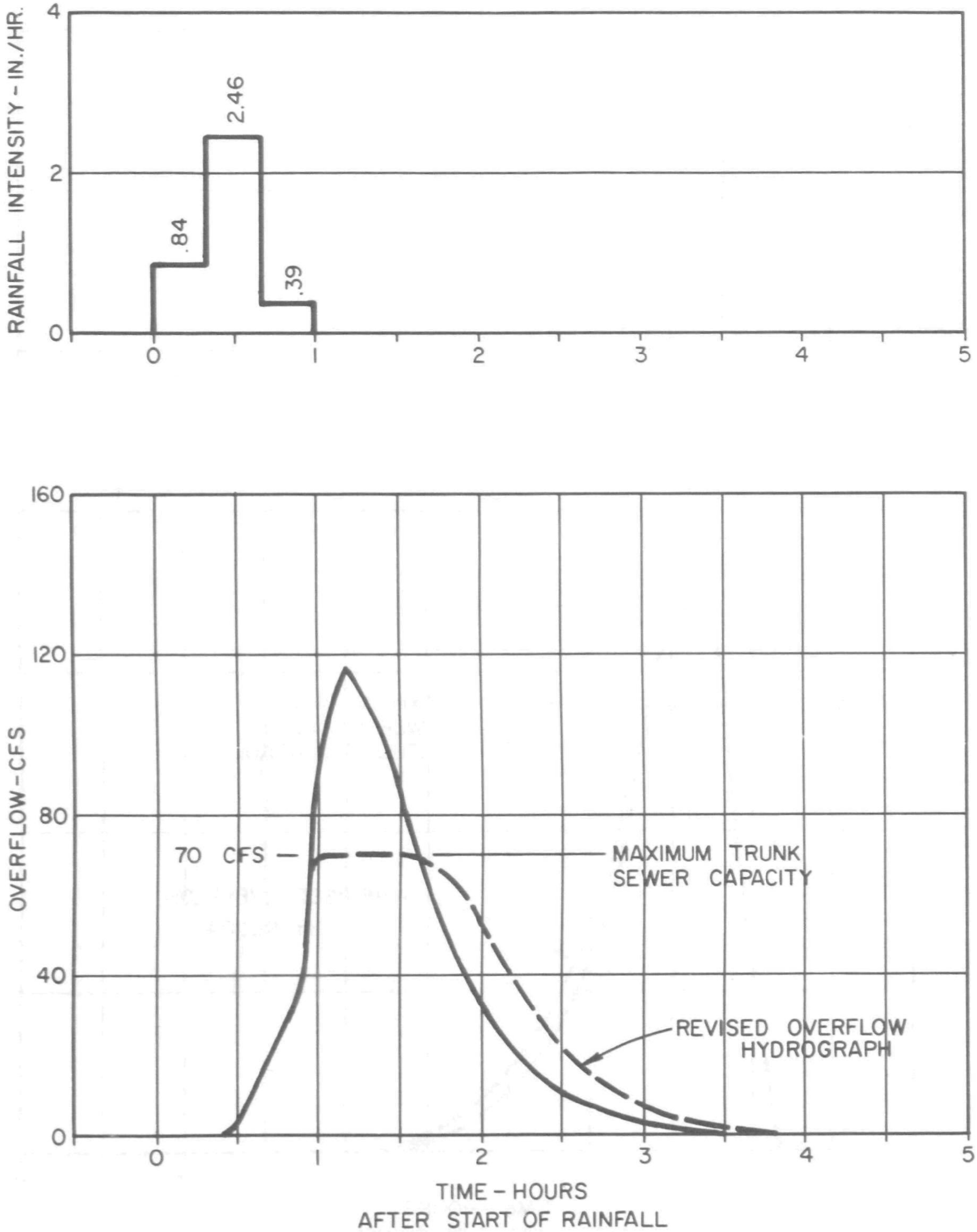
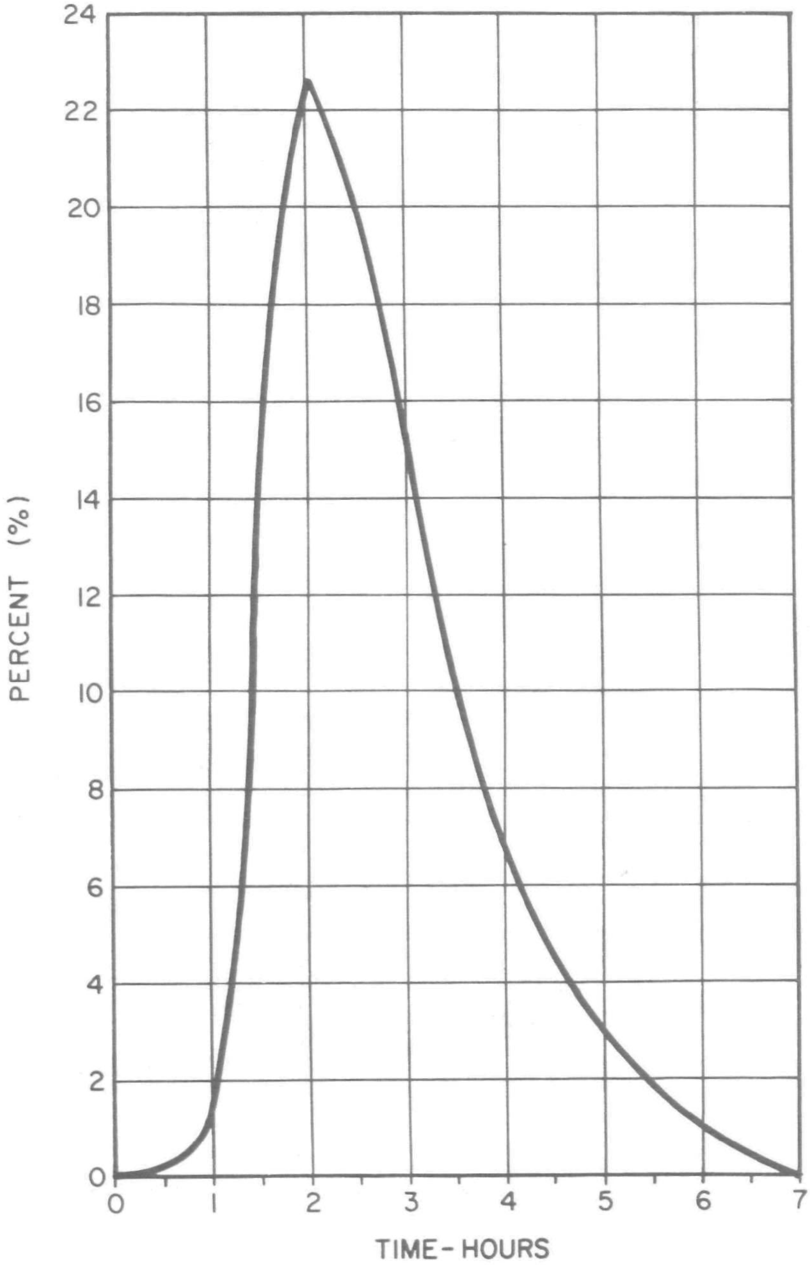
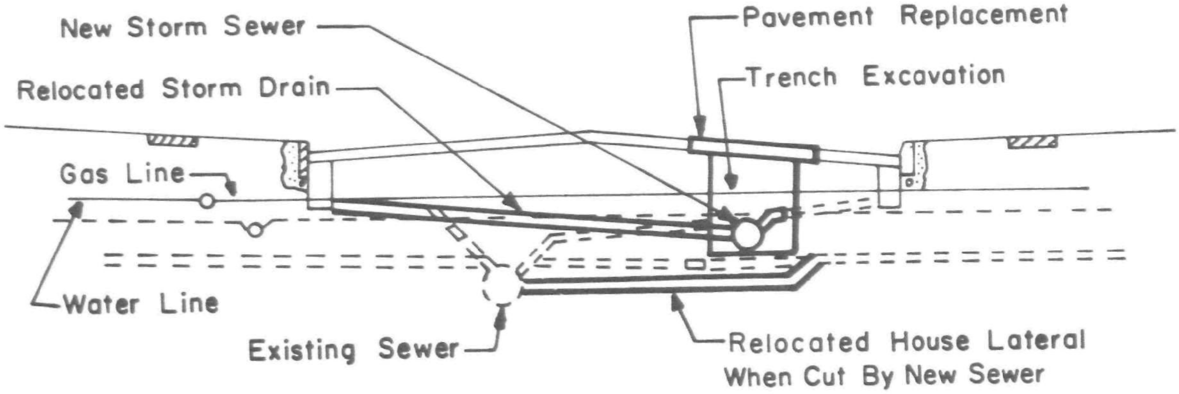


Figure No. 62
Rainfall and Overflow
Two Year, One Hour Storm
No. 23 Overflow

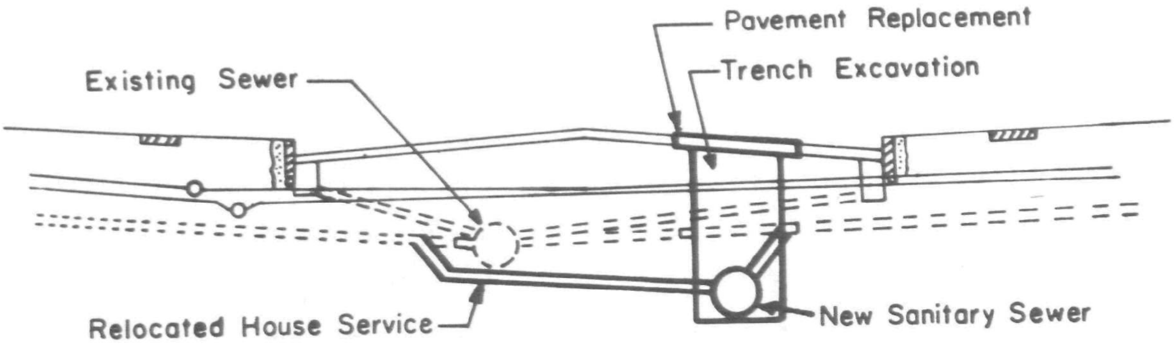


(PERCENT BASED ON 30 MINUTE TIME INTERVALS)

Figure No. 63
Distribution Graph for Urban Runoff
Downstream Gauge



NEW STORM SEWER SYSTEM



NEW SANITARY SEWER SYSTEM

Figure No. 64

SEPARATION OF SANITARY & STORM SEWERS
TYPICAL CROSS SECTIONS

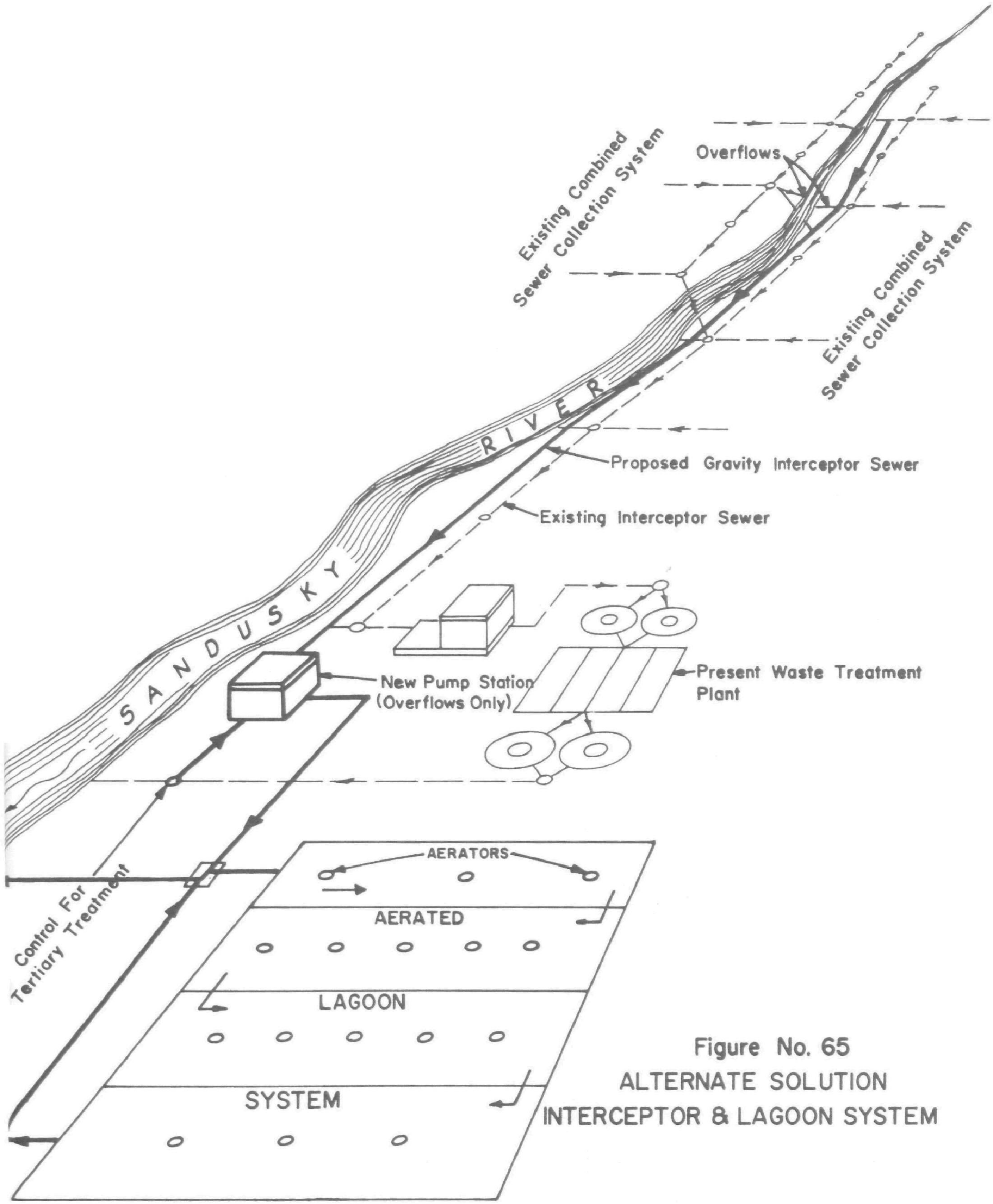


Figure No. 65
ALTERNATE SOLUTION
INTERCEPTOR & LAGOON SYSTEM

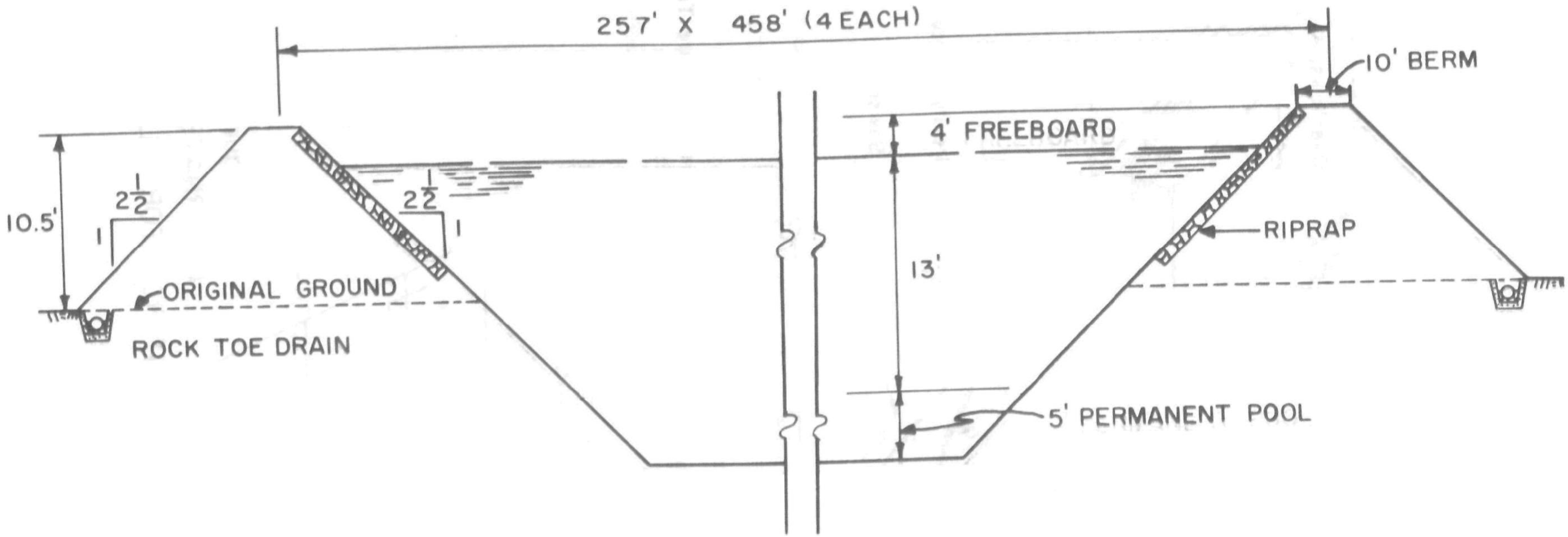
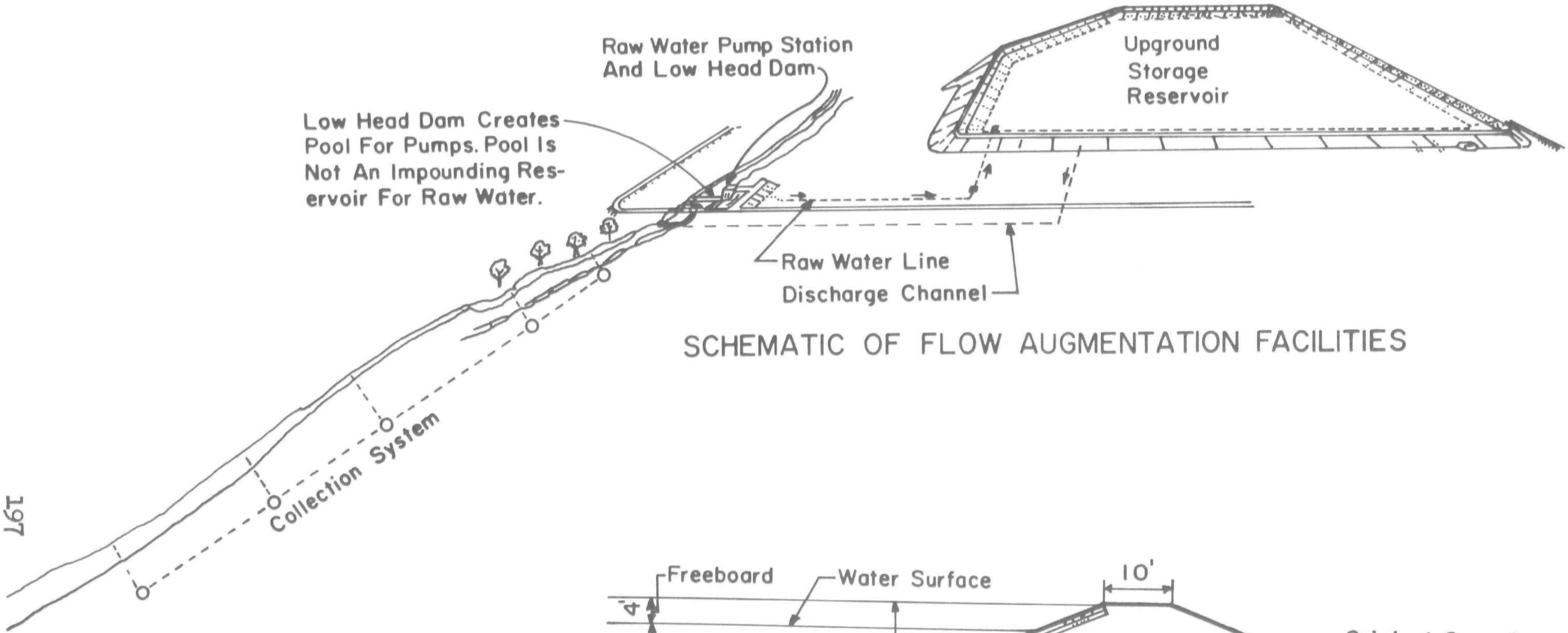
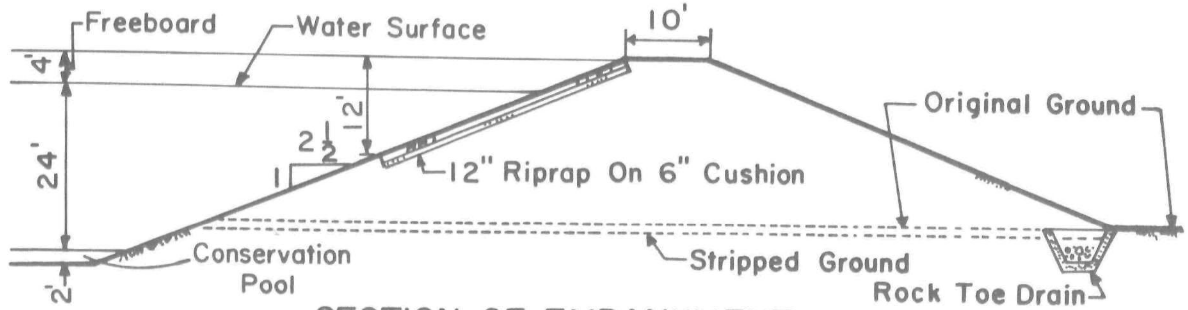


Figure No. 66
TYPICAL CROSS SECTION
OF
AERATED LAGOON



SCHEMATIC OF FLOW AUGMENTATION FACILITIES



SECTION OF EMBANKMENT

Figure No. 67

FLOW AUGMENTATION

UPGROUND STORAGE RESERVOIR

BIBLIOGRAPHIC:
Burgess & Niple, Limited. Stream Pollution and Abatement From Combined Sewer Overflows. FWPCA Publication No. DAST-32 November 1969

ACCESSION NO.

KEY WORDS:

Combined Sewers
Drainage
Hydrographs
Infiltration
Interceptor Sewer
Overflows
Runoff
Stream Flow
Urban Runoff
Wastewater Analysis

ABSTRACT:
This report contains the results of a detailed engineering investigation and comprehensive technical study to evaluate the pollutional effects from combined sewer overflows on the Sandusky River at Bucyrus, Ohio and to evaluate the benefits, economics and feasibility of alternate plans for pollution abatement from the combined sewer overflows. The City of Bucyrus is located near the upper end of the Sandusky River Basin which is tributary to Lake Erie. Bucyrus has an incorporated area of about 2,340 acres, a population of 13,000, and a combined sewer system with an average dry weather wastewater flow of 2.2 million gallons per day. A year long detailed sampling and laboratory analysis program was conducted on the combined sewer overflows in which the overflows were measured and sampled at 3 locations comprising 64% of the City's sewered area and the river flow was measured and sampled above and below Bucyrus.

The results of the study show that any 20 minute rainfall greater than 0.05 of an inch will produce an overflow. The combined sewers will overflow about 73 times each year discharging an estimated annual volume of 350 million gallons containing 350,000 pounds of BOD and 1,400,000 pounds of suspended solids. The combined sewer overflows had an average BOD of 120 mg/l, suspended solids of 470 mg/l, total coliforms of 11,000,000 per 100 ml and fecal coliforms of 1,600,000 per 100 ml. The BOD concentration of the Sandusky River, immediately downstream from Bucyrus, varied from an average of 6 mg/l during dry weather to a high of 51 mg/l during overflow discharges. The suspended solids varied from an average of 49 mg/l during dry weather to a high of 960 mg/l during overflow discharges. The total coliforms varied from an average of 400,000 per 100 ml during dry weather to a high of 8,800,000 per 100 ml during overflow discharges.

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Various methods of controlling the pollution from combined sewer overflows are presented along with their degree of protection, advantages, disadvantages and estimates of cost. The methods presented include (1) complete separation, (2) interceptor sewer and lagoon system, (3) stream flow augmentation, (4) primary treatment, (5) chlorination, and (6) offstream treatment. It was concluded that the most economical method of providing a high degree of protection to the Sandusky River is by collecting the combined sewer overflows with a large interceptor and using an aerated lagoon system to treat the waste loads from the overflows.

This report was submitted in fulfillment of Contract 14-12-401 between the Federal Water Pollution Control Administration and Burgess & Niple, Limited.

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