MATHEMATICAL MODEL FOR SCREENING STORM WATER

CONTROL ALTERNATIVES

by

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Sc.B., Brown University
(1970)

Submitted in partial fulfillment of the
requirements for the degree of Master of
Science in Civil Engineering

at the

Massachusetts Institute of Technology

(September, 1972)

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ABSTRACT

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Submitted to the Department of Civil Engineering on August 14, 1972, in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

The general problem of pollution from combined sewer systems is discussed and control alternatives are described. A linear programming model for screening the sizes and operating policies of storage tanks, pipes, and treatment plants is formulated.

The thesis also discusses a storm water simulation model and shows how it can be used interactively with the screening model to plan for the control of combined sewer overflow and local flooding in the Bloody Run Drainage Basin, Cincinnati, Ohio. The results of this case study indicate that the screening model and the planning method are reliable.

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ACKNOWLEDGEMENTS

This thesis was written while the author was being supported by the U.S. Environmental Protection Agency through their traineeship program at M.I.T., Grant Number 1 P1-WP-303-01 (MIT DSR 73356). The MIT program director is Professor Donald R.F. Harleman.

Screening model computations were performed at the M.I.T. Information Processing Center. The simulation model computations were performed at the facilities of Metcalf & Eddy, Inc., Boston, Massachusetts.

The author wishes to express his gratitude to Professor David H. Marks, the supervisor of this thesis, for his guidance, interest and encouragement while the author has been at M.I.T.

Messrs. Jekabs Vittands, Richard Evans, and Richard Laramie, of Metcalf & Eddy, Inc., Boston, are to be acknowledged for their outstanding cooperation. Thanks are also extended to Ms. Erika Babcock, the thesis typist, for exceptional patience, skill and cheerfulness.

This thesis is dedicated to the thought that some day the Charles River and Boston Harbor will be suitable for swimming, and to my wife, Donna Starrak Kirshen, to whom enough thanks can not be given.

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CHAPTER I

Introduction

Combined and Separate Sewer Systems

During the last decade, environmental pollution has been recognized as a national problem and the process of identifying the major causes of pollution has begun. One of the major recognized causes of water pollution is the overflow from combined sewer systems (Suhre (1970)). A related, though less serious problem is the local flooding that sometimes occurs from combined sewers.

A combined sewer system is a wastewater collection system through which flows not only municipal sewage (municipal sewage is both domestic and industrial wastes) but also storm runoff during periods of precipitation. The alternative to combined sewer systems is separate sewer systems. In a separate sewer system, municipal wastes are transported in separate sanitary sewers while storm runoff is transported in separate storm sewers. The American Public Works Association (APWA)(1967) estimated that as of 1962, of the 125 million people served by sewer systems, approximately 54 million lived in areas served partially or totally by combined sewer systems and 36 million were served directly by combined sewers. Most of these people reside in older urban areas where combined sewers evolved from existing storm sewers. However, combined sewers are still being constructed today.

Combined sewers and sanitary sewers usually lead to interceptor pipes which carry the wastes to municipal wastewater treatment plants for treatment. Storm sewers lead usually directly to receiving waters. Sometimes, because of storm runoff, the capacities of the interceptors are exceeded and all or some of the combined sewage is diverted untreated directly into receiving waters. This is referred to as combined sewer overflow. This overflow contributes to water pollution because it contains both municipal wastes and storm runoff, which can also be high in pollutants as is discussed in Chapter II. The excess flows in combined sewer systems are diverted by devices called diversion works or regulators. Such devices include orifices, leaping weirs, float-operating valves and gates, siphons etc. The purpose of the overflow regulators is to prevent surcharging (i.e., the flows becoming pressurized) in the system. If the flows do become surcharged because of excessive flows, local flooding can occur as the flows may "backup" at inlets and manholes.

This thesis is concerned with preventing both combined sewer overflow pollution and controlling the local flooding.

Prevention of Combined Sewer Overflow Pollution

The first solution studied by engineers to prevent combined sewer overflow pollution was the reconstruction of all the combined systems to separate systems. However, Cywin and Rosenkranz (1971) state that this would have cost 48 billion dollars in 1967. Besides being too expensive, this solution would also be very disruptive as it would require the excavation and repaving of many streets, and it still would not prevent storm water pollution or local flooding from the storm sewers (storm sewers can also become surcharged and "backup").

The Environmental Protection Agency (EPA) as reported by Cywin and Rosenkranz (1971) is currently advocating the prevention of combined sewer overflow pollution by using storage to control runoff, and special wastewater treatment plants to treat the combined sewage. The special treatment plants may be used in conjunction with the storage facilities and with municipal wastewater treatment plants. The EPA views storage devices and special treatment plants as unit processes and plans to locate them where they determine them to be most effective based upon an area's water quality objectives, combined sewage characteristics, and receiving water characteristics.

Cywin and Rosenkranz (1971) report that using this "total systems approach" employing storage and special wastewater treatment plants, it may be possible to reduce the cost of control of overflows from 48 billion to 15 billion. The use of storage facilities can also be used to prevent local flooding.

Storm Water Simulation Model

To aid administrators and engineers in evaluating various control schemes employing these storage and special treatment facilities Lager et al. (1971) have developed a storm water simulation model under the sponsorship of the EPA, from whom the model is available.

This simulation model is a detailed mathematical model which determines the amount of runoff from a storm, routes the runoff through a combined (or separate) sewer system with user-specified storage and treatment devices and operating policies and finally into and through part of the receiving waters. The model also has the capabilities of determining the amounts and locations of local flooding as well as determining the water quality at various locations both in the system and the receiving waters.

Need for Storm Water Screening Model

While the simulation model does accurately simulate the flow processes through the storage and treatment facilities, it does not determine the "optimal" sizes, locations or operating policies of such facilities. "Optimal" usually means most economical given certain constraints. In this case, the constraints would be for water quality and water quantity control. What is needed (and presently does not exist) is another mathematical model to determine the optimal sizes,locations,and operating policies. Such a model is referred to as a screening model as it "screens" the various combinations of control facilities possible and selects the optimal combination. This means that the simulation model does not have to be used on an expensive and time-consuming trial and error basis until an optimal configuration is found. However, the simulation model is still needed because a screening model, so that it can perform optimization and be inexpensively solved, has to be a less detailed mathematical model of the physical system than is the simulation model. This means that the simulation model is needed to analyze in detail the control configuration suggested by the screening model and to determine if the suggested configuration actually meets its objectives. The two models work interactively. Such an approach has been successfully used in determining sizes, locations, and operating policies for reservoirs, power plants, and irrigation facilities in river basins (Maass et al. (1962), Grayman et al. (1971)).

Thesis Objective

The purpose of this thesis is to develop a screening model as described above and to show how it can be used with the storm water simulation model to plan for a desired level of combined sewer pollution control and flood control. The model as presently developed determines the sizes of pipes and the sizes and operating policies of storage and wastewater treatment facilities in a drainage basin such that construction and operation and maintenance costs are minimized and the constraints are met that all wastewater that enters the system receives a certain degree of treatment and that there is no excessive local flooding because of surcharging in the system.

The screening model that has been developed can also be used in planning to control pollution and flooding from separate storm sewers (as can the simulation model) but the emphasis in this thesis is upon controlling combined sewer overflow pollution and flooding.

Chapter Outline

Chapter II discusses the water quality effects of combined sewer overflows and separate storm sewer discharges. This includes more detail on why overflows and flooding occur and some examples of their exact water pollution effects.

In Chapter III the various types and uses of storage and special wastewater treatment plants are explained. Some other alternatives for pollution control besides complete separation are also discussed.

The storm water simulation model is described in detail in Chapter IV. The description includes what the model does, how the data is prepared for it, and its accuracy.

Chapter V is a literature review of mathematical programming applied to water quality management. Most of the models that have been developed deal with controlling water pollution in rivers.

The screening model is discussed in Chapter VI. This chapter describes in detail the formulation of the screening model and discusses some additional uses of the model besides screening.

Chapter VII shows how the simulation model and screening model can be used together to plan for combined sewer overflow pollution control and flood control. It also shows how data is prepared for the screening model. The Bloody Run Drainage Basin in Cincinnati, Ohio, is used as an example.

The last chapter, Chapter VIII, is a conclusion and summary of the thesis. It includes a section on suggested improvements in the screening model and directions for future research.

CHAPTER II

Water Quality Effects of Combined Sewer Overflows and Separate Storm Sewer Discharges

Reasons for Overflows and Local Flooding from Combined Sewers

Condon (1970) states that the major reasons that the volumes of combined sewers are exceeded during storms causing overflows and local flooding are: underestimation of runoff in design of the system , loss of system capacity due to infiltration, and the malfunctioning and deficiencies of regulators.

He reports that combined sewer systems are usually designed for 3 - 7 times the dry weather flow (DWF) rate while it is not uncommon to have 50 to 200 times the DWF rate in urban storm runoff. The underdesign is due partly to a lack of knowledge of the runoff process. Incidentally, not only are collector lines and interceptor pipes underdesigned, municipal wastewater treatment plants are also often underdesigned. Underdesign of the treatment plant means that the excessive flows that reach the plant are treated in a less efficient manner than the design flow. For example, Vilaret and Pyne (1971) report that if a 20 million gallon per day (MGD) treatment plant has a removal efficiency of 83% of the biological oxygen demand (BOD) of the influent, and its input is increased to 47 MGD, its removal efficiency drops to 50%.

Groundwater enters sewer systems via defects in the systems.

The water enters mainly through broken or offset joints. Pound (1971) states that in systems built since approximately 1960, infiltration is less of a problem as construction materials and methods have improved but that infiltration will remain a problem in older systems as it is not likely that these systems will be replaced for a long time. However, as repair techniques improve it may be possible to stop infiltration in these older systems.

The APWA (1970) found the main causes of failure of regulators to be clogging, silting, and sticking of parts due to lack of lubrication and power failure. Clogging was found to be the most common. However, even when the regulators are working properly, a water quality problem still exists as the excess flows are usually diverted to receiving waters by the regulators causing overflow pollution.

Other reasons for combined sewer overflows and flooding are the clogging of sewer pipelines by roots and accumulated debris and the direct connection of roof gutters, area drains and foundation drains to the combined sewers in many areas of the country. Overflows also occur in many older cities because these cities now treat the wastes of the surrounding communities. This additional sewage increases the dry weather flow of these older cities to the extent that storm water overflows and flooding are quite frequent. Another reason for overflows and flooding in many urban areas is because, as the urban areas grew, the increased urbanization of the drainage basins resulted in higher storm water runoff volumes and flow rates than were originally designed for. Lastly, overflows sometimes occur because the combined sanitary and storm sewage is harmful to the wastewater treatment plant process and has to be intentionally discharged to protect the plant. For example, Pound (1971) states that the combined storm and sanitary sewage entering the Oakland and Berkeley, California, wastewater treatment plant contains so much sand and silt that the sedimentation basin of the plant becomes blocked up and the sludge digestors must be thoroughly cleaned after such wastes are treated.

Waste Characteristics of Combined Sewer Overflows and Separate Storm Sewer Discharges

Shown in Table 2.1 are the waste characteristics of combined sewer overflows and separate storm water discharges gathered by Buckingham et al. (1970) for Washington, D.C., between May and September. The data are similar to those gathered by Preul and Papadakis (1970) for Cincinnati, Ohio, by Pound (1971) for Oakland and Berkeley, California, and Vilaret and Pyne (1971) for Atlanta, Georgia. Shown in Table 2.1 also are some of the same characteristics for domestic untreated sewage.

Some of the more important water quality parameters that were measured are biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), total phosphate, total nitrogen, and fecal coliform and streptococcus. BOD gives an indication of the amount of oxygen needed by biological activity to reduce organic matter to stable compounds. High BOD loadings can result in a decrease in dissolved oxygen (DO) in receiving waters. The DO level in receiving waters is a general indication of their water quality. COD measures the oxygen consumed by both inorganic and organic wastes in an oxidation-

Table 2.1

Characteristics of Combined Sewer Overflows, Separate Storm Water Discharges and Domestic Sewage

Constituent*	Combined Sewer Overflow+		Separate Storm Water+		Domestic Sewage++		
	Range	Mean	Range	Mean	Strong	Medium	Weak
Biochemical Oxygen Demand	10-470	71	3-90	19	300	200	100
Chemical Oxygen Demand	80-1,760	382	29-1,514	335	1,000	500	250
Total Solids	120-2,900	883	338-14,600	2,166	1,200	700	350
Total Suspended Solids	35-2,000	622	130-11,280	1,697	350	200	100
Total Volatile Suspended Solids	10-1,280	245	0-880	145	275	150	70
Total Phosphate	0.8-9.4	3.0	0.2-4.5	1.3	20	10	6
Total Nitrogen	1.0-16.5	3.5	0.5-6.5	2.1	85	40	20
Ammonia	0-4.7	1.5	-	-	50	25	12
Total Coliform (1000 Counts/100 mℓ)	420-5,800	2,800	120-3,200	600	-	-	-
Fecal Coliform (1000 Counts/100 mℓ)	240-5,040	2,400	40-1,300	310	-	-	-
Fecal Streptococcus (1000 counts/100 mℓ)	1-49	17.2	3-60	21	-	-	-

*all in concentrations of mg/liter unless otherwise specified

+from Buckingham et al. (1970)

++from Clark and Ungersma (1972)

reduction reaction. Therefore it is useful as a measure of inorganic as well as organic oxygen demand.

TSS are solids that either float on the surface of wastewater or are in suspension in wastewater. TSS are largely removable by filtering and are harmful to receiving waters because they exclude light essential for photosynthetic activity, impair the aestetics of the water and may be directly detrimental to aquatic life. Total phosphate is the total amount of dissolved and suspended phosphate in wastewater Phosphate is a nutrient and essential to the growth of organic material. Therefore, if phosphate is the limiting nutrient in an ecosystem, its input can often stimulate the growth of nuisance quantities of algae and other photosynthetic organisms. Total nitrogen is the total amount of nitrogen in wastewater. Nitrogen can be in the form of ammonia, nitrate, nitrite, and organic nitrogen. Of these forms, ammonia, nitrate and nitrite are essential nutrients, and, if either one is a limiting nutrient to an ecosystem, it can have the same effect as phosphate.

Fecal coliform and streptococcus are bacteria that inhabit the intestines of man and other warmblooded animals. If found in wastewater, they are generally evidence of fecal pollution and hence may indicate the presence of pathogenic bacteria.

As can be seen in Table 2.1, the mean concentrationsof BOD, total nitrogen, and total phosphate of combined sewer overflows are less than those of weak domestic sewage. This is to be expected because of the dilution of storm flows. Actually, as DeFilippi (1970) states, the BOD of the overflows may be higher because the high solids concentration in storm runoff hampers bacterial growth and thus delays biodegradabilities. Therefore the standard BOD test may not determine the full BOD of the wastes. The COD mean concentration of combined sewer overflows is between that of weak and medium domestic sewage. The mean concentration of TSS of combined sewage overflows is approximately double the concentration found in strong domestic sewage . This is undoubtedly because of the particles picked up by the storm runoff and because of the material that has settled in the sewer between storms because of low flow velocities and is now scoured out by the high storm velocities.

The BOD, total nitrogen and total phosphate mean concentration of separate storm sewer flows are considerably less than those of weak domestic sewage. However, as mentioned previously, the BOD of storm water discharges may be greater than measured due to the high solids concentration in the discharge. The COD mean concentration is between that of weak and medium domestic sewage. The TSS mean concentration of separate storm sewer discharges is considerably greater than that of strong domestic sewage. This is accountable to the storm water runoff.

The reasons why urban storm runoff is so high in pollutants is because, as reported by Sullivan (1970),the rain scavenges air pollutants out of the air and then flows across roofs and land often covered with insecticides and air pollution fallout. This land is also sometimes fertilized with nitrogen and phosphorus and covered with waste products from pets and birds, and with salts from snow and ice control and phenols from automobile exhaust. The runoff then flows through gutters which may average one pound of debris per day per 100 feet of curb and finally enters the sewer system through catchbasins. Catchbasins are devices built to remove heavy grit and detritus from runoff before it enters the sewer system. However, they also contain stagnate water in which the APWA (1969) found the BOD to vary from 35 mg/l to 225 mg/l. Therefore, catchbasins also contribute to the pollution of the entering runoff. Additional sources of pollution to separate storm sewers are illicit sanitary and industrial connections.

Water Quality Effects

The water quality effects of combined sewer overflows and storm water discharges vary according to location because both the amount of pollutants entering from these sources and the nature of the receiving waters themselves vary from location to location. However, the water quality effects can be quite severe because, as shown earlier, the concentration of some of the constituents of combined sewer overflows and storm sewer dischargesare equal to or greater than those of untreated weak to medium domestic sewage.

Actually, the effects of combined sewer overflows and storm sewer discharges may be more severe than those of domestic sewage discharges as domestic sewage is treated in many urban areas before discharge. Such treatment may result in 80% BOD removal, effluent TSS concentrations of 6 to 20 mg/l and, if chlorinated, effluents containing no pathogenic bacteria. The water quality effects of overflows and storm sewer discharges may also be more severe because since storm flow rates are higher than DWF rates, a greater mass of pollutants enter receiving waters during storms than during dry weather.

It has been determined nationwide by the APWA (1967) that overflows from combined sewers occur on the average of 28 times per year and that each overflow lasts for an average of 5 hours.

In addition, the APWA (1967) found that overflows flush into the nation's waters between 3 and 5 percent of all wastes that enter combined sewer systems, and that during storms as much as 95 percent of the sewage entering the system during that time is flushed into receiving waters. Therefore, while the total annual amount of sewage entering from combined sewers is small, the "spike" amounts are large. Sullivan (1970) reports that most overflows are near residentially or industrially zoned land and are into waters used for either limited body contact recreation or fishing. More specifically, the APWA (1967) found that 63.7% of the overflows enter streams, 26.7% enter tidewaters, 1.7% enter lakes and 1.7% enter normally dry water courses.

Examples of Combined Sewer Overflow Pollution

Buckingham et al. (1970) found that in the combined sewer area of Washington, D.C., overflows at various locations occurred 5 to 16.8 times per month in the summer and 3.8 to 4.7 times per month in the winter. The average duration of the summer overflows was 24 to 110 hours and of the winter overflows was 26 to 38 hours. It was also found that there were some overflows during dry weather. Buckingham et al. (1970) reports that these overflows have contributed extensively to the low DO, excessive algal growths, high concentrations of fecal bacteria and repulsive floating matter in the Potomac Estuary. In addition, they computed that the expected BOD that would enter the estuary from the 2 year, 24 hour storm is 160,000 lbs, which is nearly ten times the recommended maximum allowable daily BOD loading from all sources in the entire metropolitan Washington area if the area's water quality objectives are to be met.

Vilaret and Pyne (1971) determined that major sources of water pollution in Atlanta, Georgia, are discharges from separate storm sewers, combined sewer overflows and bypasses at the wastewater treatment plant. They calculated that the two week, 2 hour storm would cause anaerobic (no DO in the water) conditions 19 miles downstream of the study area and that the one year, 2 hour storm would cause the DO to be 1 mg/l. Mackenthun (1969) states that DO less than 3 mg/l is harmful to fish populations. The reason why the larger storm resulted in greater DO is because of the higher dilution and the greater velocities of flow and the deoxygeneration rate was lower during the larger storm. The DO at this point remained at these levels for brief periods of time.

As a last example, in the Providence, Rhode Island metropolitan area, the overflow problem is so severe that after 0.5 inches of rainfall in 24 hours the upper half of Narragansett Bay is closed to shellfishing for 7 days as the allowable coliform count is exceeded. If the rainfall exceeds one inch in 24 hours, then the upper half is closed for 10 days. While Wong (1972) realizes that other factors in addition to overflow may contribute to this condition, he feels that overflows are the main cause.

Time Characteristics of Combined Sewer Overflows

Hydrographs and pollutographs (graphs of pollutant concentration vs. time) of overflows have been collected for several cities in the U.S. One city is San Francisco, California, where Eckhoff et al. (1968) have gathered data on the Selby St. combined sewer drainage basin. A typical set of hydrographs and pollutographs for the basin is shown in Fig. 2.1. Eckhoff et al. (1968) qualitatively describe their data as follows,"as runoff commences, the mass of sewage in the downstream reaches of the sewage system is virtually forced as a plug to the overflow structure (the "first flush" effect). Consequently, the initial overflows generally have the characteristics of raw sewage...... If the flows are sufficient, the initial (sewage) phase is followed by a period of scour of materials from the sewer. A majority of the surface debris is also swept into the sewer system with the initial portions of intense runoff. Consequently, overflows during the second phase may be qualitatively the worst. It has been found that during this period the concentrations of the various constituents in the overflows rose to 150 to 200 percent of the average dry weather flow values. The levels of pollutants then decrease to steady-state values, which are in the range of 10 to 25 percent of dry weather flow values and most likely characteristic of surface runoff subsequent to the initial

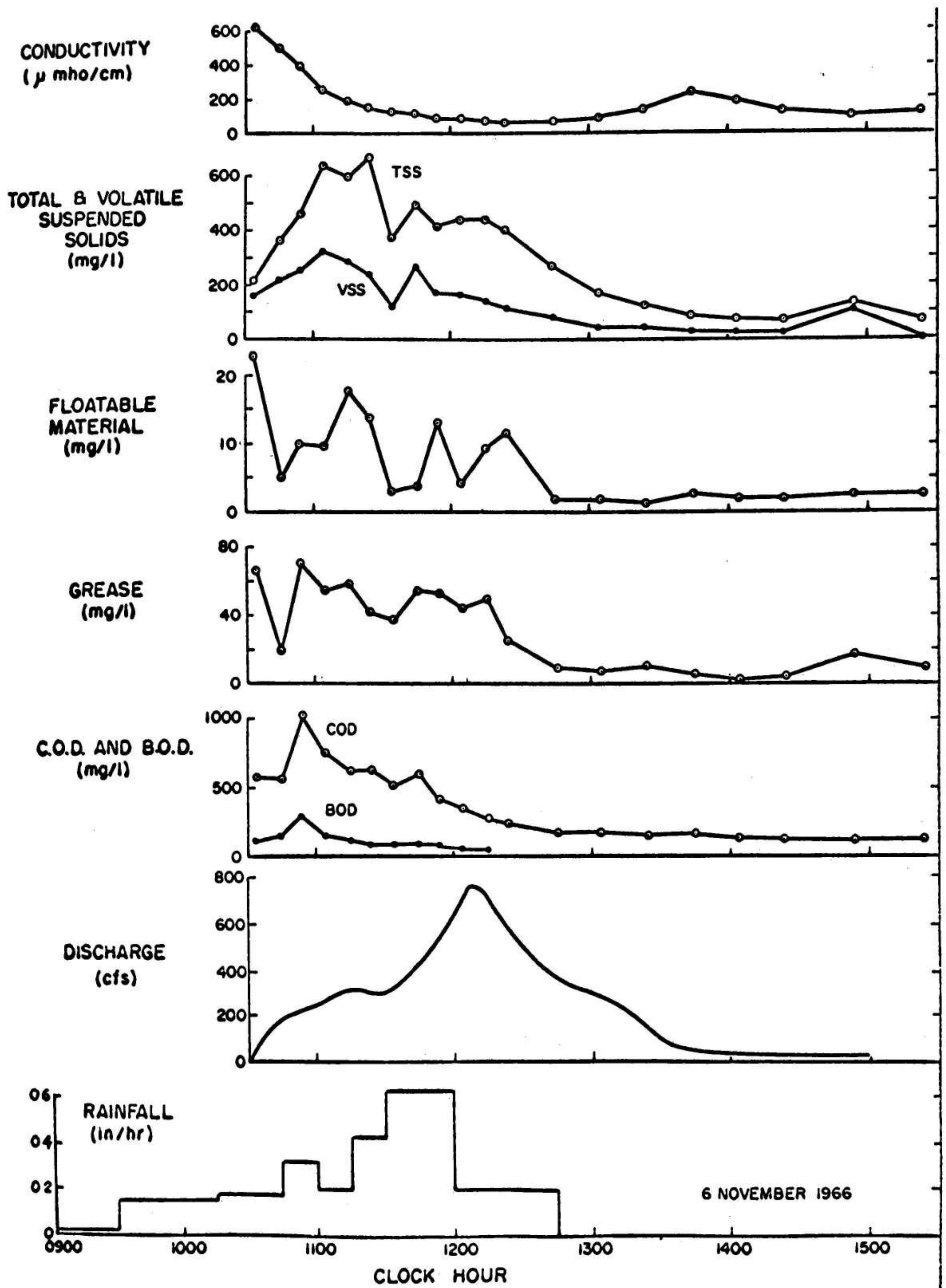

Figure 2.1 Time Characteristics of Combined Sewer Overflow

washing."[1] Data similar to this has been found by Buckingham et al. (1970) in Washington, D.C., for both combined and separate storm sewers and by Vilaret and Pyne (1971) in Atlanta, Georgia, for combined sewers. Eckhoff et al. (1968) also found that the mean concentrations of the pollutants were higher if the antecedent dry period was greater than one day.

However, not all pollutographs are of these shapes. As Buckingham et al. (1970) found in Washington, D.C., for short, intense storms, the concentration of the pollutants increased with discharge rate and concentrations remained significantly high during the monitoring period. Wright (1970) also states that the "first flush" effect can occur at any time during a storm. For example, the total suspended solids can range from a few mg/1 to 2000 to 5000 mg/1 during a storm. However, they all agree that a highly concentrated "slug" of pollution does enter receiving waters with initial overflows.

Local Flooding Effects

The local flooding in basements, underpasses and other low areas because of surcharging and "backups" in combined sewer systems is a problem as it causes inconveniences, property damage and health and sanitation menaces.

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[1] Eckhoff, David W., et al. "Characterization and Control of Overflows from Combined Sewers", Proc. of the Fourth American Water Resources Conference, New York, New York, Nov. 18-22, 1968, pgs. 73-74.

Summary

The main causes of combined sewer overflows and flooding are the underdesign of such systems, loss of capacity due to water infiltration, and maloperation of regulators. The concentrations of some of the constituents of such flows are equal to or greater than those of weak to medium untreated domestic sewage. Since these constituents enter receiving waters in such large amounts during storms, they have severe water quality effects as documented in Washington, D.C., Atlanta, Georgia, and Providence, Rhode Island. The "first flush" of sewer systems is always high in pollutants. Lastly, the local flooding from combined sewers is also a problem.

CHAPTER III

Control of Overflows and Flooding by Storage and Special Wastewater Treatment Facilities

As mentioned in the Introduction, the EPA is currently emphasizing preventing combined sewer pollution and flooding by controlling the runoff through storage, by treating the combined wastewater in specially constructed plants and by combinations of the two. This chapter discusses these alternatives and several others.

Storage

The purpose of the storage of storm runoff and excessive sewer flows is essentially to modulate the peak flow rates so that the combined sewage does not flow through the entire system all at once causing overflows and local flooding. There are basically two types of storage: insystem and offsystem.

Insystem storage makes use of pumps, valves, gates, inflatable dams, oversized conduits, and remote sensing and control to store combined flows in the existing sewer system. Condon (1970) reports that such systems are operating successfully in Milwaukee, Wisconsin, Detroit, Michigan, and Minneapolis and St. Paul, Minnesota. Suhre (1970) reports that the Detroit monitoring and remote control system cost 1.35 million dollars. The system has a retention ability of 150 million gallons. A supplemental retention basin would have cost 15 million dollars. For an additional 4 million dollars, another 100 million gallons insystem retention capacity could have been provided. To get this extra capacity using a supplemental retention basin would have cost 10 million dollars. Suhre (1970) reports that such a system can be used to retain 100 percent of the runoff from a small storm for later treatment, to retain all of the latter part of a large storm (when overflows and flooding are occurring) for later treatment (probably not too desirable as highest masses and concentrations of pollutants are in initial flows), to selectively intercept and store highly polluted flows from special areas during large storms, and to chlorinate stored wastes during large storms.

Offsystem storage is storage of combined sewer flows in large devices to prevent such flows either partially or totally from overflowing or flooding. There are several types of offsystem storage available. Holding tanks store the flows during the storm and then pump the stored flows back into the sewer system for regular treatment and discharge after the storm. Retention tanks are used to provide either short or long term retention of flows. The stored flows are discharged to receiving waters after sedimentation and disinfection. Holding and retention tanks can be located at either the points of overflow and flooding or "upsystem" where land may be more available or less expensive. The storage facilities themselves can be either concrete or earthlined tanks, or deep tunnels and mined caverns as described by Harza (1968) for Chicago, Illinois, parklands as proposed for Atlanta, Georgia, by Vilaret and Pyne (1971), urban lakes especially constructed for this purpose as Neijna et al. (1970) propose for Washington, D.C., or reinforced synthetic rubber underwater tanks as used by Underwater Storage, Inc., and Silver, Schwartz, Ltd. (1969) in Washington, D.C.. Condon (1970) also states that it may be possible to store storm water in geological areas of high permeability and void space.

Special Wastewater Treatment Plants

The special treatment plants constructed for treating storm water flows must be able to handle extremely high flow rates on an intermittent basis. Cywin and Rosenkranz (1971) state that the EPA in considering physical treatment techniques such as fine screening, microstrainers, dissolved air flotation, chemical techniques such as polyelectrolyte sedimentation aids, disinfection (chlorination and ozonization), physical-chemical techniques that include screening and dissolved air flotation with flotation aids, and biological techniques such as bio-adsorption and stabilization ponds. Cywin and Rosenkranz (1971) report that high-rate multi-media filtration and a combination of screening and dissolved air flotation presently seem to have the potential for producing good quality effluents. The first process has been found to remove up to 87% of TSS and an average of 35% of the BOD in a pilot plant.

The special treatment facilities can be located at either the points of overflow or flooding, as auxiliary facilities at municipal treatment plants, or next to storage facilities to treat stored water. Cywin and Rosenkranz (1971) suggest that special treatment plants could be used to further treat municipal wastewater during dry weather. The type and degree of treatment chosen depend upon the water quality objectives of the area, the characteristics of the wastewater, and the characteristics of the receiving waters.

Joint Use of Storage and Special Treatment Facilities

It appears that the most promising method to prevent combined sewer overflow pollution (and separate storm water pollution) and flooding is the use of both storage and special treatment facilities in a sewer system. This is because, as Cywin and Rosenkranz (1971) state, it is very unlikely that an economical treatment plant will ever be developed that can handle directly the instantaneous flows generated by storms. The flows will have to be retarded by storage. Furthermore, if no storage is provided, extra large conduits will have to be constructed to handle the peak storm flows. Special treatment plants are necessary because it is unlikely that storage tanks can be built large enough to store all the wastes until they can be treated at the municipal plant (if it is desired to treat all the stored wastes). Special treatment plants may also be necessary because, as described earlier, the increased solids concentration of storm flows may upset municipal wastewater treatment plants.

Other Alternatives for Combined Sewer Pollution Control

Other alternatives for controlling combined sewer pollution have been discussed by the American Society of Civil Engineers (1969), Lager et al. (1971), and Heaney and Sullivan (1971). These include placing pressurized pipes within existing combined sewers to carry to interceptor pipes the comminuted sewage from buildings, increasing the flow capacity by adding polymers to the sewage to reduce pipe friction, extending overflow points to locations where there are large bodies of dilution water, and the construction of relief sewers. Source controls that reduce the pollution levels of combined and separate storm flows have also been suggested. These controls include better catch-basin clearing and street sweeping, eliminating the use of home garbage grinders, and periodic flushing of the sewer system to flush out solids that settle between storms.

CHAPTER IV

Description of the Storm Water Simulation Model

General Description of the Model

The storm water simulation model developed by Lager et al. (1971) for the EPA is a detailed, mathematical, computer-based model that can determine the amount of runoff from a storm, route the runoff through a combined (or separate) sewer system with user-specified storage and treatment facilities and operating policies, and finally into and through part of the receiving waters. The model also has the capability of determining the amounts and locations of local flooding as well as determining the water quality at various locations both in the system and in the receiving waters. The model itself consists of over 10,000 FORTRAN statements. The printed output of the model can contain both tables and graphs of hydrographs and pollutographs of BOD, coliform and total suspended solids at various user-selected points in the system and in the receiving waters. This allows the user to evaluate the effectiveness of his control scheme. The actual computer simulation is done by 5 main groups of subroutines referred to as blocks. The results of each of the blocks are stored on computer storage devices and are used as part of the input to other blocks.

Executive Block

The Executive Block is always the first block used and is "in charge" of the rest of the model. It calls the other blocks when needed and all interfacing between blocks goes on through this block.

Watershed Block

The Watershed Block routes the rainfall over the drainage basin and through the smaller gutters and pipes of the sewer system into the main sewer pipes. This block also determines the pollution load of the runoff entering the system. To use this block, the user must input the time history of the hyetograph of the design storm and a discretization of the drainage basin. The basin is discretized into sub-basins of constant land form characteristics. The locations and characteristics of the gutters and pipes also have to be described. In addition, the user must input street cleaning frequency and catch-basin data as well as the land use and other features of the different areas of the basin.

Transport Block

The Transport Block routes the storm runoff (as determined by the Watershed Block), the dry weather flow, and the water that has infiltrated into the system through the main sewer pipes, and through a maximum of two optional "internal" storage tanks. The flows are routed to a maximum of 5 outlet points. If flows become surcharged, and "back-up" the flooding is assumed to occur at the closest upsystem manhole. In addition, this block determines DWF quality and quantity, the amount of water that infiltrates into the system, the water quality of the flows in the system, and calculates the capital, land, and operation and maintenance costs of the "internal" storage tanks. To model the system, this block requires that the sewer system be discretized into pipe segments of constant size, slope, and type joined by either manholes, control structures such as flow dividers, or "internal" storage tanks. An "internal" storage tank is described by its size, shape, outlet device, and unit cost. The outlet device can be either a pump specified to go on or off at a specified tank depth, a weir, or an orifice. The outlet device is used to specify the operating policy of the storage tank.

The DWF quality and quantity entering the sewer system are calculated by inputting to the model such parameters as daily and hourly pollution correction factors, land use and population of the subareas, and average market value of the dwellings in a subarea. If more exact data is available such as average BOD of flows, this can be used in place of some of the other data.

Infiltration is calculated by estimates of base dry weather infiltration and groundwater and rainwater infiltration, and such parameters as average joint distance. The use of the subroutines calculating DWF quality and quantity and infiltration is optional.

Storage Block

The Storage Block simulates the changes in the hydrographs and pollutographs of the sewage as the sewage flows through one optional special wastewater treatment faculty. The facility has to be located at one of the outfalls specified in the Transport Block.

The treatment process is chosen by the user to consist of a sequence chosen from the following unit processes: "external" storage (same as "internal" storage except that it is located adjacent to an outlet of the sewer system), bar racks, fine screens, dissolved air flotation, sedimentation tanks, microstrainers, high rate filters, effluent screens, and chlorinators and other chemical dispensers. The user can specify the sizes of the treatment processes or else can specify that the model is to select the sizes of the processes (except for "external" storage) such that a certain user-selected percentage of the peak flow receives treatment. The Storage Block also has the capability of calculating the capital, land, and operation and maintenance costs of the treatment processes chosen. The user has the option of either specifying the unit costs or using default values provided by the simulation model.

Receiving Water Block

The Receiving Water Block models the hydraulic and water quality effects of the effluent from the sewer system in a receiving body of water. Its flow input comes from either the Transport Block or the Storage Block, or both. To determine the resulting hydraulics, which determine the resulting water quality, the receiving body of water is discretized by the user to consist of a network of nodes connected by channels. Each channel is of constant surface area and cross-sectional area. Flow characteristics such as tidal periods can also be specified.

Accuracy of the Simulation Model

To determine the accuracy of the pollutographs and hydrographs produced by the simulation model, Lager et al. (1971) compared simulation results with measured values. To verify the other features of the model, Lager et al. (1971) had to rely upon engineering judgement.

Shown in Fig. 4.1 are some measured and computer hydrographs and pollutographs for a storm over the Bloody Run Drainage Basin in Cincinnati, Ohio. As can be seen, the model accurately predicts the shapes of the graphs but generally underestimates the amounts of runoff and pollutants. Lager et al. (1971) state that this is probably because the input hyetograph and flow measurements may have been inaccurate, no gutter pipes were modelled in the Watershed Block, and the drainage basin consisted of an unusually large amount of open land and parks which seem to contribute continuous amounts of pollutants which the model does not account for.

To verify the effect of storage upon pollutographs and hydrographs, Lager et al. (1971) had data available from the Selby St. Drainage Basin, San Francisco, California. They found what they considered to be generally exceptionally good results.

Since Lager et al. (1971) also judged the other features of the model to be adequate, it appears that the simulation model is accurate enough to determine the amounts and effects of combined sewer overflow and flooding and the effects of different control alternatives.

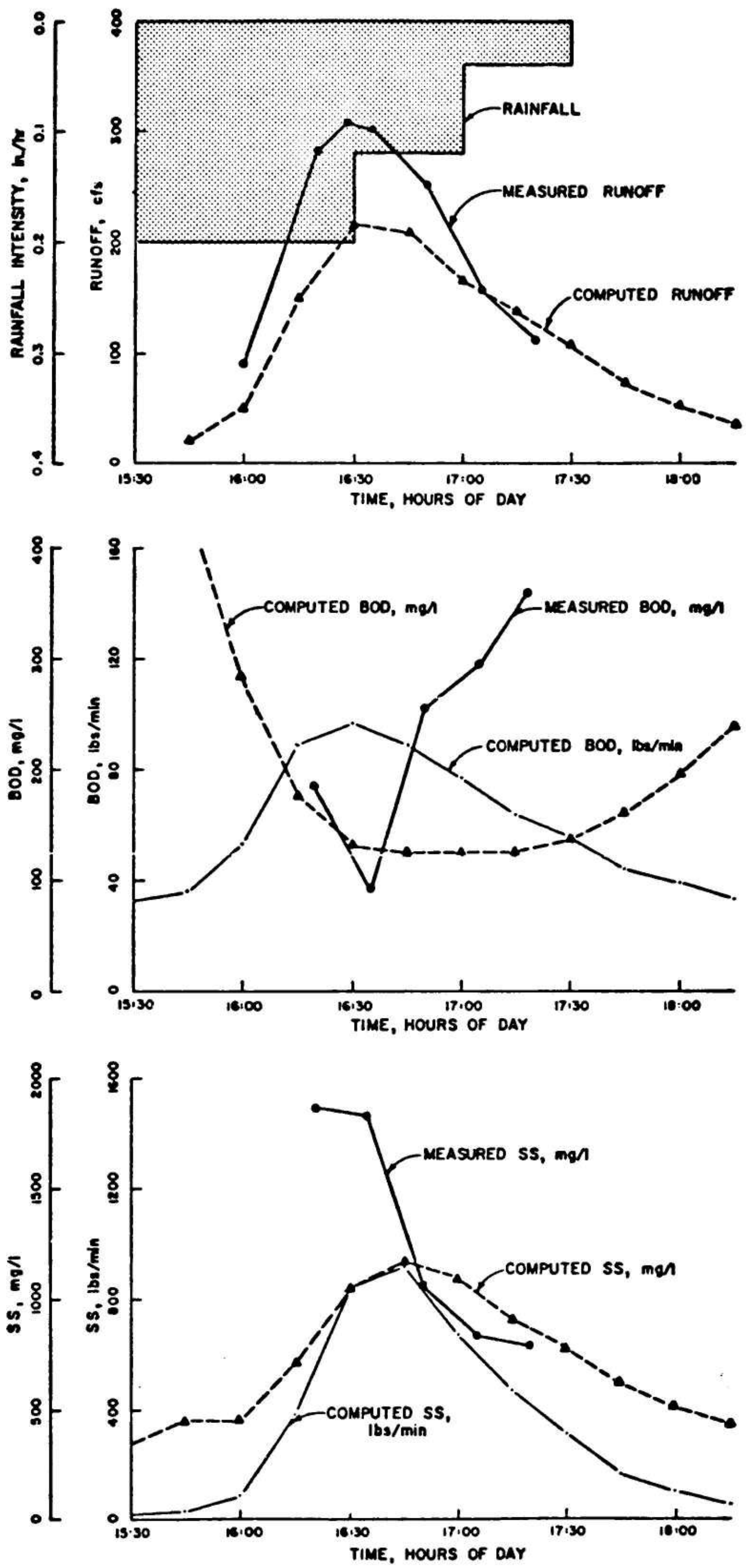

Figure 4.1 Accuracy of Simulation Model

CHAPTER V

Literature Review

While there is no literature on mathematical programming directly concerned with the control of combined sewer overflow and flooding, there is a considerable amount on mathematical programming applied to locating wastewater treatment facilities in river basins.

Thomann and Sobel (1964), Deininger (1965), Sobel (1965), Liebman and Lynn (1966), Revelle et al. (1968), and Camp, Dresser and McKee, Consulting Engineers (1969), all develop models that determine the specifications for treatment plants or other water pollution control facilities such that various stream standards are met. The plants or facilities are to be located along a river. Regionalization of treatment plants or facilities are not considered. Depending upon the paper, the objectives for choosing control levels include minimizing the total treatment costs, maximizing the benefit-cost ratio, minimizing dissolved oxygen variance, minimizing total pollution load to the river, maximizing the length of the river that meets stream standards, minimizing the expected value of the treatment cost, and minimizing the combined cost of wastewater treatment and water supply treatment. Both linear and nonlinear programming techniques are used.

Graves et al. (1969) illustrate that by piping effluents usually discharged into the reaches of low assimilative capacity of a stream to reaches of higher assimilative capacity, the general overall water quality of the stream, specifically the amount of dissolved oxygen, can be improved. The assimilative capacity of a certain reach could be low because of either heavy pollution, natural reasons, or else both. The problem is formulated as a large scale linear program where the number of variables greatly exceeds the number of constraints. The constraints are the dissolved oxygen requirements, and the objective function is to minimize the piping costs. The economies of scale of piping are taken into account by the cost coefficients so that the objective function can remain linear. A computationally efficient primal-dual algorithm is used to solve this problem. The algorithm uses a "truncated tableau" containing only a number of variables equal to the number of constraints plus one, the other elements being generated as needed. Graves et al. also suggest using the values of the dual variables to allocate the piping costs to the users.

Deacon and Giglio (1971) study whether or not having multiple outfalls from a treatment plant is a more economical technique of maintaining water quality than having only one outfall from a treatment plant of a higher efficiency. The reason why the former may be more economical is because it uses the natural assimilative capacity of the stream to aid in the waste treatment process more effectively than does one outfall. Of course, in the former, the piping costs are greater. Deacon and Giglio determine that having more than one outfall is more economical and, using an integer programming technique (implicit enumeration), they develop a method of locating outfalls and determining plant efficiencies such that the stream dissolved oxygen standard is always met and the costs of treatment and transport are minimized. The costs are taken as nonlinear.

Graves et al. (1970) consider the problem of determining the correct mix of regional treatment plants, source treatment plants and by-pass piping to achieve the dissolved oxygen standards in an estuary at a minimum cost. The costs of source treatment are taken to be piecewise linear. The costs of the regional treatment plants are fitted to equations approximating their cost curves, and the costs of piping are taken from the paper of Graves et al. (1969). The problem is formulated as a nonlinear program with a concave cost function. It is solved using essentially a gradient algorithm. Graves et al. apply this method to a 84 mile stretch of the Delaware River. They find that the cost of obtaining the dissolved oxygen goal is significantly less using a combination of regional treatment plants, source treatment plants, and by-pass piping instead of only using source treatment. They also discuss a possible method of charging the polluters for a regional system. The method is based upon the savings the polluters achieve using the regional system instead of only source treatment. They also suggest making the right to pollute saleable.

Sobel and Marks (1972) consider a problem similar to that of Graves et al. (1970). The problem is to determine the necessary increases in present treatment efficiencies, the locations, sizes and efficiencies of regional treatment plants, the placement of pipes, and the specification of flows such that total system costs are minimized and water quality objectives met. The method of solution chosen is a heuristic algorithm because of skepticism of the nonlinear programming algorithm of

Graves et al. (1970) to converge for moderate sized problems. The method of Sobel and Marks requires partitioning an area into subsets possibly served by regional treatment plants, determining the optimal treatment efficiencies and pumping and piping configurations within each subset and finally determining the flows of untreated wastewater between partitions. The application of the method to the Clear Lake and Clear Creek Drainage Basin in Texas is then discussed. The method could not be applied as sufficient data was not available.

From pipeline network cost equations developed for metropolitan areas and from treatment plant costs developed by the U.S. Public Health Service and others, Gemmell et al. (1971) observe that for a metropolitan area centralization of waste treatment plants may not be optimal. This is because most cities in the U.S. have low population densities for which the diseconomies of scale of pipeline networks (diseconomies of scale in this case means that the unit cost of a pipeline network increases as the size of a constant population density service area increases) outweigh the economies of scale of centralized treatment plants. This results in a least cost configuration of small service areas with a separate treatment plant instead of large service areas with centralized treatment plants. Gemmell et al. also find that for high density areas, the least cost service area is rather insensitive to decentralization or centralization of treatment facilities. They also mention that more than one plant may be optimal in terms of water quality because then the treated waste products could be dispersed into the receiving body of water or bodies of water from more than one outfall. However, the authors do add the consideration that centralized plants may be better because they are generally operated more efficiently. If this is the case, they suggest using a compromise solution such as a centralized treatment plant with multiple collection networks.

Wanielista and Bauer (1971) consider the problem of what should be the capacities of the existing treatment plants of a sewer system and of a possible regional treatment plant such that the total capacity will be large enough to handle the domestic and industrial levels projected for the next 20 years. The problem is formulated as a network. The nodes are the location of the existing treatment plants and the predetermined site of a possible regional treatment plant. The arcs are the pipelines. The decision variables are the future capacities at each site and the pipeline volumes. Apparently, it is assumed that the type of treatment at each site remains the same as before. It is not made explicit how the type of treatment at the regional plant is determined. However, limits are set on the amount of effluents that can be discharged at each site. This is done for water quality purposes. The costs of treatment and piping are considered to consist of fixed and variable costs. A mixed integer programming algorithm is used to solve the problem. When this method is applied to a 125 sq. mi. area in Florida, the least cost solution is that a large regional plant should be built and an existing plant expanded. Bauer and Wanielista report that this solution is about 10% less expensive than expanding each plant individually to meet the projected demands.

Bhalla and Rikkers (1971) study the problem of where should regional treatment plants be located in a region, when should they be constructed and what capacities should they have given the changing pollution loads of each community in the region and the objective of treating all wastes most economically and maintaining water quality. In addition, they study for how long should a community be served by a particular plant before being served by a larger plant. No expansion of the plants is allowed after they are built so that the capacity of a plant once it is constructed must be large enough for its maximum assigned load. The problem is formulated as a multi-time period facilities location model with fixed and variable costs for the plants and pipelines. In this type of formulation it is assumed that there are certain possible sites available for the facilities (the treatment plants) to serve the demand centers (the communities). The model is solved by a heuristic algorithm based upon known economic trade-offs to obtain a good solution (since it is not computationally feasible to obtain the optimal solution using a rigorous mathematical approach, a "good" solution, which is probably near optimal and computationally feasible, has to be determined instead). Water quality is considered by assuming that at each facility site a treatment plant of a certain efficiency exists, independent of the efficiencies of the other plants. If these efficiencies are found to be inadequate to maintain water quality, they are readjusted. In the test case involving the waste treatment system of the Lower Pioneer Valley Region in New England (a region of approximately 650 sq. miles containing 15 small towns), Bhalla and Rikkers find that the "best" solution is that all the wastes should be treated at a regional plant. Lastly, Bhalla and Rikkers point out that the problem of when should a plant be built, what should be the initial capacity of that plant and the subsequent expansions of that plant as the loads on it increase can be determined using an existing integer programming algorithm once the problem they have initially posed has been solved.

Chi (1970), also studies a dynamic problem. He idealizes a wastewater system to consist of n sources of pollution (either communities or industries) spaced along the banks of a river and connected by a common sewer line and receiving secondary treatment at the source. The secondary effluent can be either discharged directly to the river, receive tertiary treatment and then be discharged, or else be exported to another reach where it can be discharged directly or receive tertiary treatment. Chi's model tries to determine when, given a planning period with changing pollution loads, each plant and pipeline should be put into use or expanded such that the dissolved oxygen standard is met and the cost is minimized. The piping and treatment costs are taken from cost equations developed in the thesis and are nonlinear. Chi determines a near optimal or optimal solution to this problem by using a random search technique.

Value of the Literature to the Development of the Storm Water Screening Model

The major value of the literature to the development of the storm water screening model is the way the various authors meet the water quality objectives. Papers similar to that of Graves et al. (1969), and Revelle et al. (1968), use explicit techniques based upon either the work of Thomann (1963) or the Dobbins-Camp equation to meet the water quality objectives. The techniques make use of linear systems theory. As described by Sobel and Marks (1972), the methods assume that coefficients a_{ij} can be determined that when multiplied by a waste reduction at j, x_j, give the water quality improvement at i. If b_i is the required water quality improvement at i, and there are n waste discharges and m reaches of the river, then the constraints

$$\sum_{j=1}^{n} a_{ij} x_j \geq b_i \qquad i = 1...m \tag{5.1}$$

must be satisfied to achieve minimum water quality. The x_j's are decision variables in mathematical programming models and are related to the sizes and efficiencies of the treatment plants.

The other method used to meet water quality objectives is similar to that used by Bhalla and Rikkers (1971). In this implicit method, the efficiencies of the treatment plants are assumed, and their sizes determined. If the efficiencies turn out to be too low to meet the water quality objectives or too high, the problem is re-solved with adjusted treatment efficiencies.

The method of meeting water quality objectives chosen in this thesis is similar to this last method as is discussed in Chapters VI and VII.

CHAPTER VI

Storm Water Control Screening Model

The purpose of the storm water control screening model is to determine the sizes and operating policies of pipes and storage and wastewater treatment facilities in a sewer system such that the total cost of the pipes and facilities is minimized. The model specifies that all storm water runoff and dry weather flow that enters the system during the screening period receive treatment and that there is no excessive local flooding because of system surcharging. The storage facilities possible to size are offsystem facilities (such as tanks) and insystem storage pipes. The operating policy of a storage or treatment facility describes how flows should enter or leave the facility. The model is a linear programming model, chosen because the problem could easily be put in the form of a linear programming model and inexpensive computer codes are readily available to solve linear programming models.

Linear Programming Models

Linear programming models are mathematical optimization models. They can be used to determine the allocations of resources to a project such that the cost of employing such resources is minimized and the resource allocation meets certain constraints. The amounts of the resources allocated are referred to as decision variables. The function which determines the total cost of employing the resources is called the objective function. The allocation constraints form the constraint set. Both the objective function and the constraints have to be linear. To be physically feasible, the decision variables have to be non-negative. The optimal solution of a linear programming model is the set of values or activities of the decision variables that minimize the objective function subject to the constraints. Computer codes, such as IBM's Mathematical Programming System Extended (MPSX) (1971), can inexpensively and quickly solve linear programming models. A further description of linear programming can be found in Wagner (1969).

In the stormwater screening model, the decision variables are storage tank sizes, pipe sizes, treatment plant sizes, and flow amounts. The objective function is the total cost of the facilities. The constraint set contains constraints to show physical continuity, size facilities and specify water quality objectives.

Discretization of the Drainage Basin

To apply the screening model to a drainage basin, the drainage basin has to be divided into nonconduit and conduit elements. The nonconduit elements are storm water runoff and dry weather flow (DWF) inlets, feasible locations for storage tanks and special treatment plants, and existing locations of municipal treatment plants (Throughout the rest of the discussion, it is assumed that municipal treatment plants already exist in a drainage basin. If such plants do not exist or are not to be used during the storm, the appropriate variables should be set to zero). The conduit elements are the existing sewer pipes and potential pipes that connect nonconduit elements. The locations of the potential pipes and storage and treatment facilities are chosen by the user. For example, shown in Figure 6.1a is an existing sewer system. If it was desired to determine the optimal sizes and operating policies of storage tanks at locations C, E, and I and of a special treatment plant at B, the system might be discretized as in Figure 6.1b. Shown also is the modelling of a possible relief sewer between E and C.

Division of the Screening Time Period

The screening model formulation requires that the total screening time period be divided into equal time intervals. The length of the total screening time period is usually the time it takes for all the storm water runoff to enter the sewer system from the design storm. The inflows from the storm runoff and dry weather flow that enter the system during each time interval must be specified by the user. The initial flow conditions are assumed to be DWF conditions. The model determines the optimal amounts of flow, storage and treatment during each time interval, i.e., the optimal operating policies of the different parts of the system.

Continuity Constraints

The continuity constraints insure that all the flows that

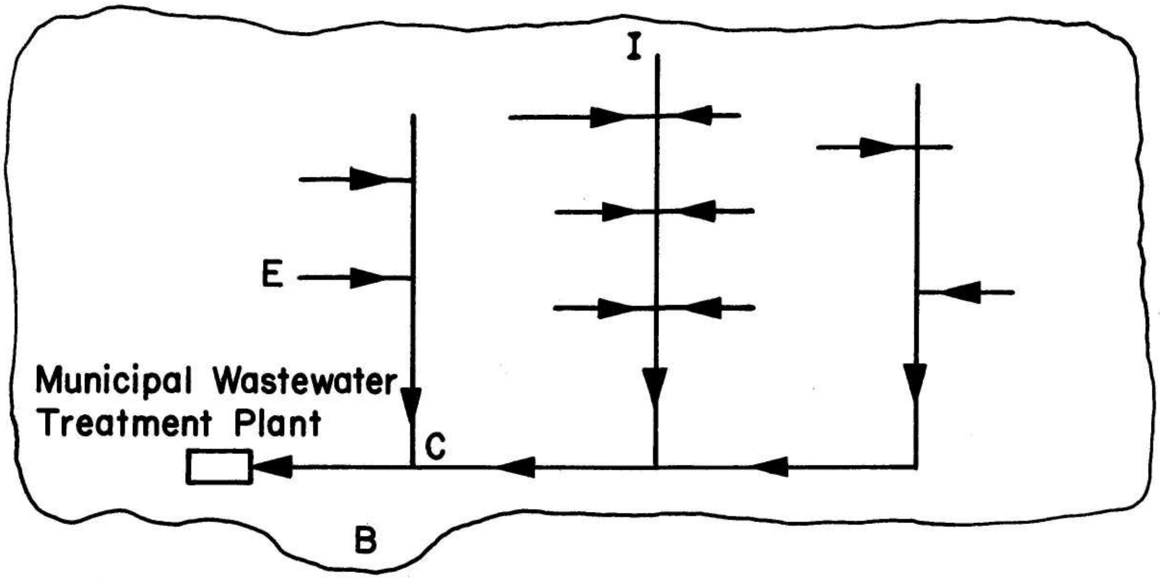

Figure 6.1a Example Sewer System

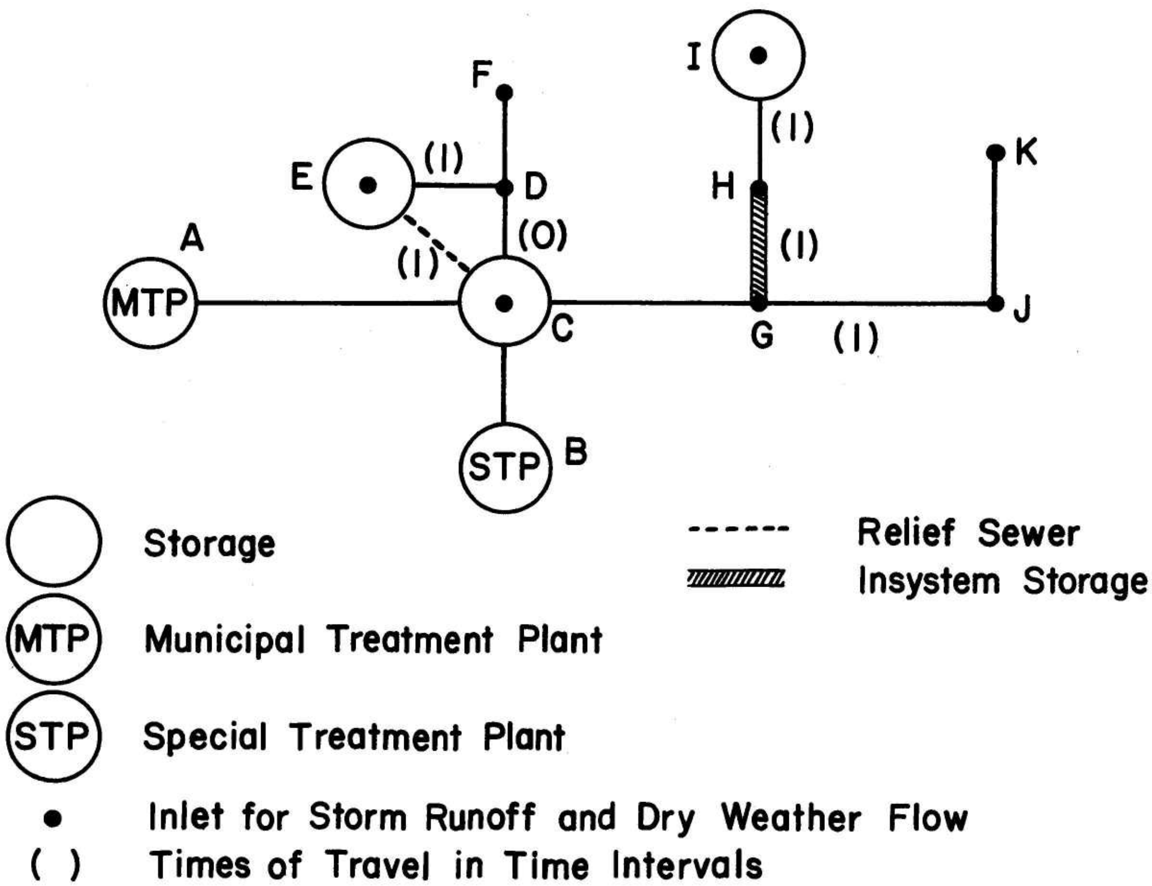

Figure 6.1b Example Discretization

enter either a nonconduit element (hereafter referred to as a node) or an insystem storage pipe either leave that node or pipe or are stored at that node or pipe. A continuity constraint is written for each node in the system, for each location where there is an insystem storage pipe, and for each time interval of the screening period.

The continuity constraint written for each node i is of the form

$$Q_{it} + S_{i,t-1} + \sum_j F_{ji,t-\tau_{ji}} = \sum_k F_{ikt} + S_{it} \qquad (6.1)$$

This equation says that the inflow into node i during time interval t, Q_{it}, plus the previous amount of flow stored at i during the last time interval , $S_{i,t-1}$, plus the sum of the inflows from upsystem that enter i during t from nodes j , $\sum_j F_{ji,t-\tau_{ji}}$, must equal the sum of the flows that leave node i for nodes k during t plus the amount of flow stored at i during t. τ_{ji} is the time of travel of the flow from j to i in time intervals. The flow can be entering or leaving from an existing pipe, or a potential relief sewer or insystem storage pipe. The outflow could also be to a special or municipal treatment plant. The units of all the variables in Equation (6.1) are cubic feet.

To clarify this continuity constraint, shown below is how it would be written for node C in Figure 6.1b for time interval two, i.e., $t = 2$. The times of travel in time intervals between D and C, E and C, and G and C are 0, 1, 1, time intervals respectively (i.e., $\tau_{DC} = 0, \tau_{EC} = \tau_{GC} = 1$).

$$Q_{C2} + S_{C1} + F_{DC2} + F_{GC1} + F_{EC1} = F_{CA2} + F_{CB2} + S_{C2} \qquad (6.2)$$

The constraint written for an insystem storage pipe between nodes k and i (i is upsystem of k) is,

$$FI_{ikt} + SI_{ik,t-1} = FD_{ikt} + SI_{ikt} \qquad (6.3)$$

This constraint requires that the flow that enters the pipe between i and k from i during t, FI_{ikt}, plus the amount of flow stored in the pipe during the previous time interval, $SI_{ik,t-1}$, equal the amount of flow that flows through the pipe during t, FD_{ikt}, plus the amount of flow that is stored in the pipe during t, SI_{ikt}. The units of these variables are cubic feet.

As an example, suppose in Figure 6.1b it was desired to model insystem storage in the pipe between nodes H and G. Assume the time of travel of the flow between I and H is 1 time interval and the constraint in being written for time interval 2. For node H the continuity equation would be written

$$Q_{H2} + F_{IH1} = FI_{HG2} \qquad (6.4)$$

For the pipe between H and G the continuity equation would be

$$FI_{HG2} + SI_{HG1} = FD_{HG2} + SI_{HG2} \qquad (6.5)$$

The continuity equation for node G would be, if the times of travel between H and G, and J and G are one time interval,

$$Q_{G2} + F_{JG1} + FD_{HG1} = F_{GC2} \tag{6.6}$$

Sizing Constraints

The purpose of the sizing constraints is to insure that the volumes that flow through the system or are stored in the system do not exceed the capacities of the pipes and of the treatment and storage facilities in the system. In addition, the sizing constraints allow the optimal capacities of the pipes and treatment and storage facilities to be found if these capacities are decision variables. These constraints are written for every pipe and treatment and storage facility in the system and for each time interval of the screening period.

For an existing pipe between i and j, the sizing constraint is

$$\frac{c\,F_{ijt}}{\Delta t} \leq FMAX_{ij} \tag{6.7}$$

$FMAX_{ij}$ is the discharge in cubic feet per second (cfs) in the pipe between i and j when the pipe is flowing full but is not surcharged. It is a constant. When this discharge is exceeded, backups can result. Therefore the flow value should not exceed this amount.[1] According to

[1] Actually, the maximum discharge in a circular pipe under open channel flow conditions is $1.066\ FMAX_{ij}$ and occurs when the water depth to pipe diameter ratio is 0.90. However, for our purposes, we can assume the maximum discharge possible is $FMAX_{ij}$.

Fair, Geyer, and Okun (1966), this value can be found from Manning's formula,

$$FMAX_{ij} = Q = A \frac{1.49}{n} r^{2/3} S^{1/2} \qquad (6.8)$$

where Q is the discharge (cfs) of the full pipe, A is the cross-sectional area of the pipe (ft^2), n is the coefficient of roughness, r is the hydraulic radius (feet) and S is the slope.

To make the units compatible in Equation (6.7), F_{ijt}, the flow from i to j during t in cubic feet, has to be divided by the length of the time interval in seconds, Δt, to obtain the average pipe discharge during t. c is a pipe flow coefficient. It is usually the peak to average discharge ratio (i.e. greater than 1.0). Such a coefficient is necessary in this case because $\frac{F_{ijt}}{\Delta t}$ is the average flow between i and j during t, and, if it is required that there is no local flooding, the peak flow that occurs during t also has to be contained within the pipe. If it is assumed that during a storm the sewer pipes flow on the average 70 percent full, (i.e. the water depth to pipe diameter ratio is 0.70) then, according to Fair, Geyer and Okun (1966), the ratio of the average pipe discharge to the full pipe discharge would be .838 or the peak to average flow factor would be $\frac{1.000}{.838}$ or 1.19. However, the coefficient c can be set at a value less than 1.0 if a user decides that some minor local flooding is acceptable. For such a case, the value of c could be set to 0.8.

If the flow between two nodes is to be in a pipe that is to be constructed, the constraints

$$\frac{cF_{ijt}}{\Delta t} \leq CAPAC_{ij} \tag{6.9}$$

$$CAPAC_{ij} \leq CAPMAX_{ij} \tag{6.10}$$

are written. $CAPAC_{ij}$ is a decision variable and is the capacity needed in cfs of the pipe to be built between i and j. The constraint, Equation (6.9), "sizes" the pipe capacity by choosing that value that is the maximum of the adjusted flows through the pipe , $\frac{cF_{ijt}}{\Delta t}$. Equation (6.10) insures that the capacity chosen is less than or equal to the maximum physical feasible size, $CAPMAX_{ij}$, which is a user-determined constant.

To model increasing the capacity between the nodes already connected by a pipe by laying an additional pipe between i and j the constraints needed are:

$$\frac{cF_{ijt}}{\Delta t} \leq CAPAC_{ij} \tag{6.11}$$

$$CAPAC_{ij} \leq FMAX_{ij} + ADDCAP_{ij} \tag{6.12}$$

$$ADDCAP_{ij} \leq CAPMAX_{ij} \tag{6.13}$$

Equation (6.12) specifies that the maximum adjusted flow between i and j has to be less than or equal to the existing capacity between i and j, $FMAX_{ij}$, plus the capacity of the additional pipe to be constructed next to the existing pipe, $ADDCAP_{ij}$, a decision variable. This equation also "sizes" the additional pipe. Equation (6.13) specifies that $ADDCAP_{ij}$ has to be less than or equal to the maximum additional capacity it is possible to physically construct between i and j, $CAPMAX_{ij}$, a user determined constant.

The sizing constraint written for an insystem storage pipe between i and j requires that the volume stored in the pipe during t in cu.ft., SI_{ijt}, plus the corrected volume that flows through the pipe during t, $cFD_{ijt/\Delta t}$, be less than or equal to the volume of the pipe. This constraint is written below:

$$SI_{it} + \frac{cFD_{ijt}}{\Delta t} \leq L_{ij} \cdot A_{ij} \qquad (6.14)$$

L_{ij} and A_{ij} are the length and cross-sectional area of the insystem storage pipe respectively. A_{ij} is a decision variable. The following constraints are also needed for an insystem storage pipe.

$$\frac{cFD_{ijt}}{\Delta t} \leq A_{ij} \cdot V_{ij} \qquad (6.15)$$

$$\frac{cFD_{ijt}}{\Delta t} \leq FMAXDS_{ij} \qquad (6.16)$$

Equation (6.15) says that the corrected discharge between i and j , $\frac{cFD_{ijt}}{\Delta t}$, can not exceed the capacity available, $A_{ij} \cdot V_{ij}$, where V_{ij} is the flow velocity between i and j when the pipe is full. $FMAXDS_{ij}$ is the capacity in cfs of the pipe immediately downsystem of the insystem storage pipe. Therefore Equation (6.16) says that the corrected flow rate through and from the storage pipe can not exceed this downsystem pipe capacity. If the insystem storage is to be provided in a potential conduit between points or by replacing the existing pipe between i and j by a new larger one, the constraint

$$A_{ij} \leq AMAX_{ij} \tag{6.17}$$

is needed where $AMAX_{ij}$ is cross-sectional area of the largest pipe it is possible to construct between i and j and is a user-determined constant. However, if the storage capacity is to be provided by constructing a new pipe parallel to the existing pipe and storing in both pipes, the constraints

$$A_{ij} = AEXIS_{ij} + AADD_{ij} \tag{6.18}$$

$$AADD_{ij} \leq AMAX_{ij} \tag{6.19}$$

are needed. Equation (6.18) says that the total cross-sectional area needed, A_{ij}, equals the cross-sectional area of the existing pipe, $AEXIS_{ij}$, plus the cross-sectional area of the additional pipe to be constructed between i and j, $AADD_{ij}$. The other equation insures that the cross-sectional area of the new pipe does not exceed the cross-sectional area of the largest pipe possible to construct between i and j, $AMAX_{ij}$. In this case V_{ij} in Equation (6.15) is the average of the two pipe velocities when the two pipes are full. This is a valid assumption to make because most combined sewers are designed to flow full at approximately 10 feet per second (Fair, Geyer, Okun (1966)).

The sizing constraint for the flow from j to a special treatment plant i is written

$$\frac{c \sum_j F_{jit}}{\Delta t} \leq STP_i \tag{6.20}$$

where $\frac{c \sum_j F_{jit}}{\Delta t}$ is the adjusted flow in cfs to the special treatment plant i from nodes j during t and STP_i is the plant capacity to be constructed in cfs. STP_i is a decision variable and has an upper physical bound of $STPMAX_i$, a user-determined constant. This last constraint is written,

$$STP_i \leq STPMAX_i \tag{6.21}$$

For flow to a municipal treatment plant i, the constraints are

$$\frac{c \sum_j F_{jit}}{\Delta t} \leq MTP_i \tag{6.22}$$

$$MTP_i \leq MTPMAX_i \tag{6.23}$$

Equation (6.22) determines the maximum flow that enters the treatment plant during the storm, MTP_i, and Equation (6.23) specifies that this amount can not exceed the plant's capacity, $MTPMAX_i$.

The storage sizing constraint written at i is

$$S_{it} \leq SCAP_i \tag{6.24}$$

where $SCAP_i$ is a decision variable and is the amount of storage that should be built at i. The constraint says that all the flow stored at i during t have to be less than or equal to the storage to be constructed there. To insure that the amount of storage to be constructed at a node is physically feasible, the constraint

$$SCAP_i \leq SMAX_i \tag{6.25}$$

also has to be written. It says that the amount constructed at a site i has to be less than or equal to some maximum feasible amount, $SMAX_i$. Both $SCAP_i$ and $SMAX_i$ are in cubic feet units.

Water Quality Constraint

The water quality constraint requires that all the flows that are stored in the system during the storm receive wastewater treatment after the storm. The constraint is written as

$$\frac{\sum_{i} S_{it_f} + \sum_{k,j} SI_{kjt_f} + DWF}{TE}$$

$$\leq \quad \sum_{\ell} STP_{\ell} + \sum_{m} MTPMAX_{m} \tag{6.26}$$

where TE is a constant and is the time allowed to pump the tanks and insystem storage pipes dry after a storm in preparation for the next storm. TE is in units of seconds. DWF is the average dry weather flow in the system in cfs. $\sum_{i} S_{it_f} + \sum_{k,j} SI_{kjt_f}$ is the sum of the amounts of flow stored in the storage tanks and insystem storage pipes during the last time interval of the screening period, t_f. When this sum is divided by TE it gives the average rate of flow in cfs for the transfer of the stored wastes after the storm if they are to receive treatment. Therefore the sum of this value and DWF is the total amount of flow in cfs that requires treatment during TE. Thus this constraint says that the total capacity of the special treatment plants and the municipal treatment plants in the system, $\sum_{\ell} STP_{\ell} + \sum_{m} MTPMAX_{m}$, must be large enough to handle this flow rate. The level of treatment given to the stored wastewater (and to the flows during the storm) is chosen by the user and is reflected in the costs of treatment in the objective function. This is discussed more in Chapter VII, the case study.

Objective Function

The objective function of the screening model is to minimize the total cost of the system. The costs are incurred in the construction and operation and maintenance of sewer pipes, wastewater treatment plants, and storage facilities. The construction costs include land costs. The objective function is gotten by multiplying the sizes of the pipes, treatment plants and storage facilities by their unit costs as shown in Equation (6.27).

$$\text{Min } Z = \sum_{i,j} \alpha_{ij} \cdot \text{CAPAC}_{ij}$$

(new piping)

$$+ \sum_{k,l} \beta_{kl} \cdot \text{ADDCAP}_{kl} \tag{6.27}$$

(additional capacity by constructing pipes next to existing pipes)

$$+ \sum_{m,n} \gamma_{mn} \cdot A_{mn}$$

(insystem storage by constructing new pipes or by replacing existing pipes by larger pipes)

$$+ \sum_{p,q} \delta_{pq} \cdot \text{AADD}_{pq}$$

(insystem storage by constructing additional pipes next to existing pipes)

$$+ \quad \sum_s \varepsilon_s \cdot STP_s$$

(special treatment)

$$+ \quad \sum_t \mu_t \cdot MTP_t$$

(municipal treatment)

$$+ \quad \sum_u \nu_u \cdot SCAP_u$$

(storage tanks)

The units of the unit costs for all piping and treatment except for insystem storage piping are dollars/cfs. The units of insystem storage piping costs and storage costs are dollars/cross-sectional area and dollars/cu.ft. respectively.

To determine the unit costs, the cost curves of the pipes and facilities must be available. Sources of such curves are discussed in Chapter VII. Once the curves have been obtained, they must be linearized within the feasible capacity ranges of the pipes and the facilities. Figure 6.2 shows how a hypothetical cost curve might be linearized within the range AB. The slope of the straight line is the unit cost.

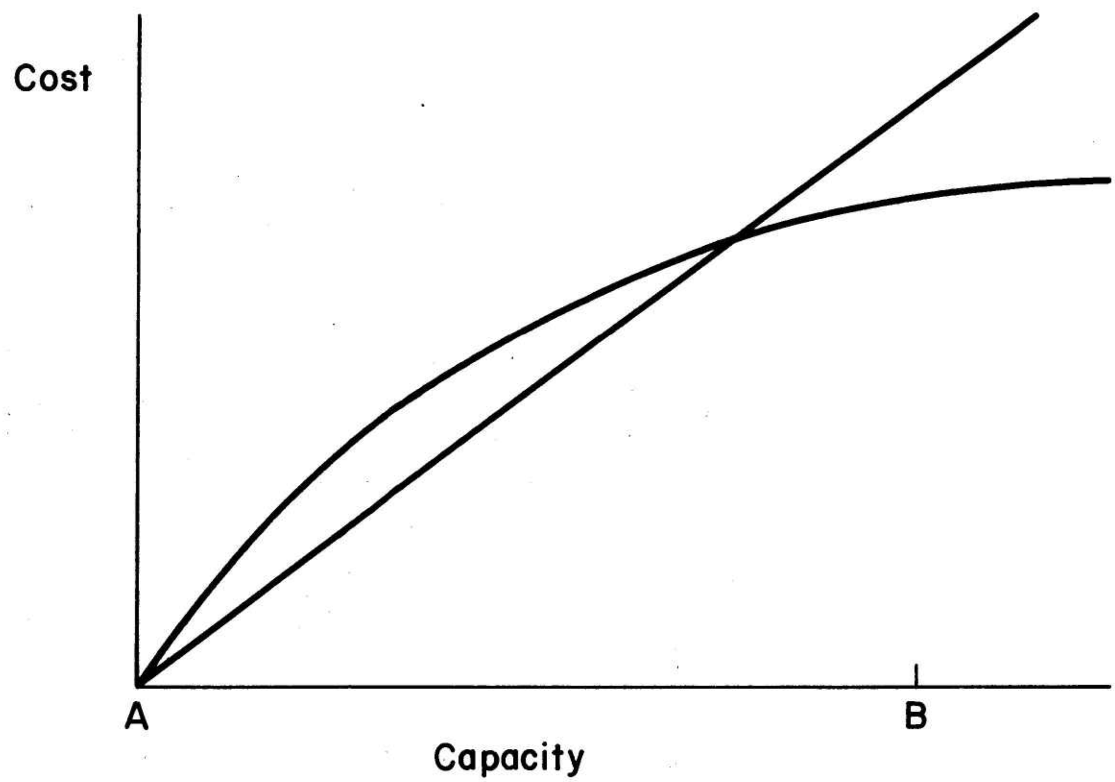

Figure 6.2 Linearized Cost Curve

Screening Model Uses

The major use of the screening model is to "screen" the various control alternatives and determine the optimal combination. However, the model, especially since it is a linear programming model, is particularly useful for studying how sensitive the optimal solution is to the input data. This process is called sensitivity analysis and is important to carry out for data that is either doubtful or significant. Linear programming packages such as MPSX(1971) allow the user to determine the range of a variable coefficient in the objective function such that the solution remains optimal, or to parametrically vary a set of column coefficients and see the new optimal solution that results for each variation. Possible uses of sensitivity analysis for the storm water screening model are changing the storm runoff inputs (i.e., changing the design storm) and seeing how this changes the previous optimal operating policies and sizes, and decreasing the upper bounds on pipes and storage and treatment facilities to investigate, for example, how social constraints on storage tank sizes and locations effect the solution. Using sensitivity analysis it is also possible to determine how sensitive the solution is to the water quality objectives of the region. This can be done two ways. The first way is to parametrically vary the costs of special treatment and municipal treatment because these costs determine the removal efficiencies of the plants.

In this way one could determine the trade-offs between total system cost and receiving water quality because it is possible to approximate the receiving water quality given treatment levels, and influent and receiving water characteristics. The second way to examine how sensitive the solution is to an area's water quality objectives is to parametrically vary the time allowed to pump out to treatment the wastes stored by the system during a storm. If it turns out that this constraint (Equation(6.26)) requires additional treatment capacity to be built, one can examine the trade-offs between requiring all the stored wastes to be treated before the next storm and treating some of the stored wastes but also discharging some directly to receiving waters.

Table 6.1

Variable Listing

Variable	Definition	Units	D=decision variable C=constant
A_{ij}	Total cross-sectional area of insystem storage pipe or pipes between i and j	ft^2	D
$AADD_{ij}$	Cross-sectional area of additional insystem storage pipe between i and j	ft^2	D
$ADDCAP_{ij}$	Capacity of pipe constructed between i and j for additional capacity between i and j	cfs	D
$AEXIS_{ij}$	Cross-sectional area of existing pipe between i and j	ft^2	C
$AMAX_{ij}$	Cross-sectional area of the largest pipe possible to construct between i and j	ft^2	C

Variable	Definition	Units	D=decision variable C=constant
c	Pipe flow coefficient		C
$CAPAC_{ij}$	Capacity of pipe to be built between i and j	cfs	D
$CAPMAX_{ij}$	Capacity of the largest pipe possible to construct between i and j	cfs	C
DWF	Average dry weather flow in a drainage basin	cfs	C
F_{ijt}	Flow that leaves i for j during t	ft^3	D
FD_{ijt}	Flow through the insystem storage pipe between i and j that leaves i during t	ft^3	D
FI_{ijt}	Flow entering the insystem storage pipe between i and j from i during t	ft^3	D

Variable	Definition	Units	D=decision variable C=constant
$FMAX_{ij}$	Capacity of existing pipe between i and j	cfs	C
$FMAXDS_{ij}$	Capacity of the existing pipe downsystem of the insystem storage pipe between i and j	cfs	C
L_{ij}	Length of the insystem storage pipe between i and j	ft.	C
MTP_i	Maximum flow through municipal treatment plant i during study time	cfs	D
$MTPMAX_i$	Capacity of the municipal treatment plant i	cfs	C
Q_{it}	Storm runoff and dry weather flow that enters i during t	ft^3	C
S_{it}	Storage at i during t	ft^3	D
$SCAP_i$	Capacity of the storage to be built at i	ft^3	D

Variable	Definition	Units	D=decision variable C=constant
SI_{ijt}	Insystem storage in the pipe between i and j during t	ft^3	D
$SMAX_i$	Maximum storage capacity it is possible to construct at i	ft^3	C
STP_i	Capacity of special treatment plant to be built at i	cfs	D
$STPMAX_i$	Maximum special treatment plant capacity it is possible to construct at i	cfs	C
t	Time interval number		C
TE	Time during which the stored flows have to be pumped out of the sewer system after a storm	sec.	C
t_f	Final time interval number		C

Variable	Definition	Units	D=decision variable C=constant
V_{ij}	Flow velocity in the pipe (or pipes) between i and j when the pipe is flowing full but not surcharged	ft/sec.	C
α_{ij}	Unit cost of constructing a pipe between i and j	\$/cfs	C
β_{ij}	Unit cost of constructing a pipe for additional capacity between i and j	\$/cfs	C
γ_{ij}	Unit cost of constructing insystem storage pipe between i and j	\$/ft^2	C
δ_{ij}	Unit cost of constructing a pipe between i and j for additional insystem storage capacity	\$/ft^2	C
Δt	Duration of a time interval t	sec	C

Variable	Definition	Units	D=decision variable C=constant
ε_i	Unit cost of special treatment at i	\$/cfs	C
μ_i	Unit cost of municipal treatment at i	\$/cfs	C
ν_i	Unit cost of storage at i	\$/ft^3	C
τ_{ij}	Time of travel of flow from i to j in number of time intervals		C

Table 6.2

Summary of Formulation

Continuity

$$Q_{it} + S_{i,t-1} + \sum_j F_{ji,t-\tau_{ji}} = \sum_k F_{ikt} + S_{it} \quad \forall_{i,t} \tag{6.1}$$

$$FI_{ikt} + SI_{ik,t-1} = FD_{ikt} + SI_{ikt} \quad \forall_{i,k,t} \tag{6.3}$$

Existing Piping

$$\frac{cF_{ijt}}{\Delta t} \leq FMAX_{ij} \quad \forall_{i,j,t} \tag{6.7}$$

Potential Piping

$$\frac{cF_{ijt}}{\Delta t} \leq CAPAC_{ij} \quad \forall_{i,j,t} \tag{6.9}$$

$$CAPAC_{ij} \leq CAPMAX_{ij} \quad \forall_{i,j} \tag{6.10}$$

Potential Additional Capacity

$$\frac{cF_{ijt}}{\Delta t} \leq CAPAC_{ij} \quad \forall_{i,j,t} \tag{6.11}$$

$$CAPAC_{ij} \leq FMAX_{ij} + ADDCAP_{ij} \quad \forall_{i,j} \tag{6.12}$$

$$ADDCAP_{ij} \leq CAPMAX_{ij} \qquad \forall_{i,j} \tag{6.13}$$

Potential Insystem Storage

$$SI_{ijt} + \frac{cFD_{ijt}}{\Delta t} \leq L_{ij} \cdot A_{ij} \qquad \forall_{i,j,t} \tag{6.14}$$

$$\frac{cFD_{ijt}}{\Delta t} \leq A_{ij} \cdot V_{ij} \qquad \forall_{i,j,t} \tag{6.15}$$

$$\frac{cFD_{ijt}}{\Delta t} \leq FMAXDS_{ij} \qquad \forall_{i,j,t} \tag{6.16}$$

$$A_{ij} \leq AMAX_{ij} \qquad \forall_{i,j} \tag{6.17}$$

or

$$A_{ij} = AEXIS_{ij} + AADD_{ij} \qquad \forall_{i,j} \tag{6.18}$$

$$AADD_{ij} \leq AMAX_{ij} \qquad \forall_{i,j} \tag{6.19}$$

Potential Special Treatment

$$\frac{c \sum_j F_{jit}}{\Delta t} \leq STP_i \qquad \forall_{i,t} \tag{6.20}$$

$$STP_i \leq STPMAX_i \qquad \forall_i \tag{6.21}$$

Municipal Treatment

$$\frac{c \sum_j F_{jit}}{\Delta t} \leq MTP_i \qquad \forall_{i,t} \qquad (6.22)$$

$$MTP_i \leq MTPMAX_i \qquad \forall_i \qquad (6.23)$$

Potential Storage

$$S_{it} \leq SCAP_i \qquad \forall_{i,t} \qquad (6.24)$$

$$SCAP_i \leq SMAX_i \qquad \forall_i \qquad (6.25)$$

Water Quality

$$\frac{(\sum_i S_{it_f} + \sum_{k,j} SI_{kj\,t_f})}{TE} + DWF$$

$$\leq \sum_l STP_l + \sum_m MTPMAX_m \qquad (6.26)$$

Objective Function

$$\text{Min } Z = \sum_{i,j} \alpha_{ij} \cdot CAPAC_{ij} + \sum_{k,l} \beta_{kl} \cdot ADDCAP_{kl}$$

$$+ \sum_{m,n} \gamma_{mn} \cdot A_{mn} + \sum_{p,q} \delta_{pq} \cdot AADD_{pq}$$

$$+ \sum_{s} \varepsilon_{s} \cdot STP_{s} + \sum_{t} \mu_{t} \cdot MTP_{t}$$

$$+ \sum_{u} \nu_{u} \cdot SCAP_{u}$$

CHAPTER VII

Case Study - Bloody Run Drainage Basin

The purpose of this chapter is to show how the simulation model and the screening model can be used interactively to control combined sewer overflows and flooding. This is done by applying the method to the Bloody Run Drainage Basin in Cincinnati, Ohio. The Bloody Run Drainage Basin was chosen because it had been studied previously by Lager et al. (1971) in the verification of the simulation model and the data was available.

The Bloody Run Drainage Basin

As described by Lager et al. (1971), the drainage basin is approximately 2,380 acres of hilly land serving a population of approximately 26,000. 55 percent of the area is residential, 17 percent is commercial, 5 percent is industrial, and 22 percent is open land or parks. As shown in Figure 7.1, the main feature of the combined sewer network is a trunk sewer that splits into three branches which run down the valleys of the test area. Commercial and industrial establishments are located in the valleys. Residential housing is on the ridges. The outfall is located at point A. The outfall discharges to an interceptor to the Mill Creek Wastewater Treatment Plant. During storms, overflows are diverted directly to Mill Creek, a tributary of the Ohio River.

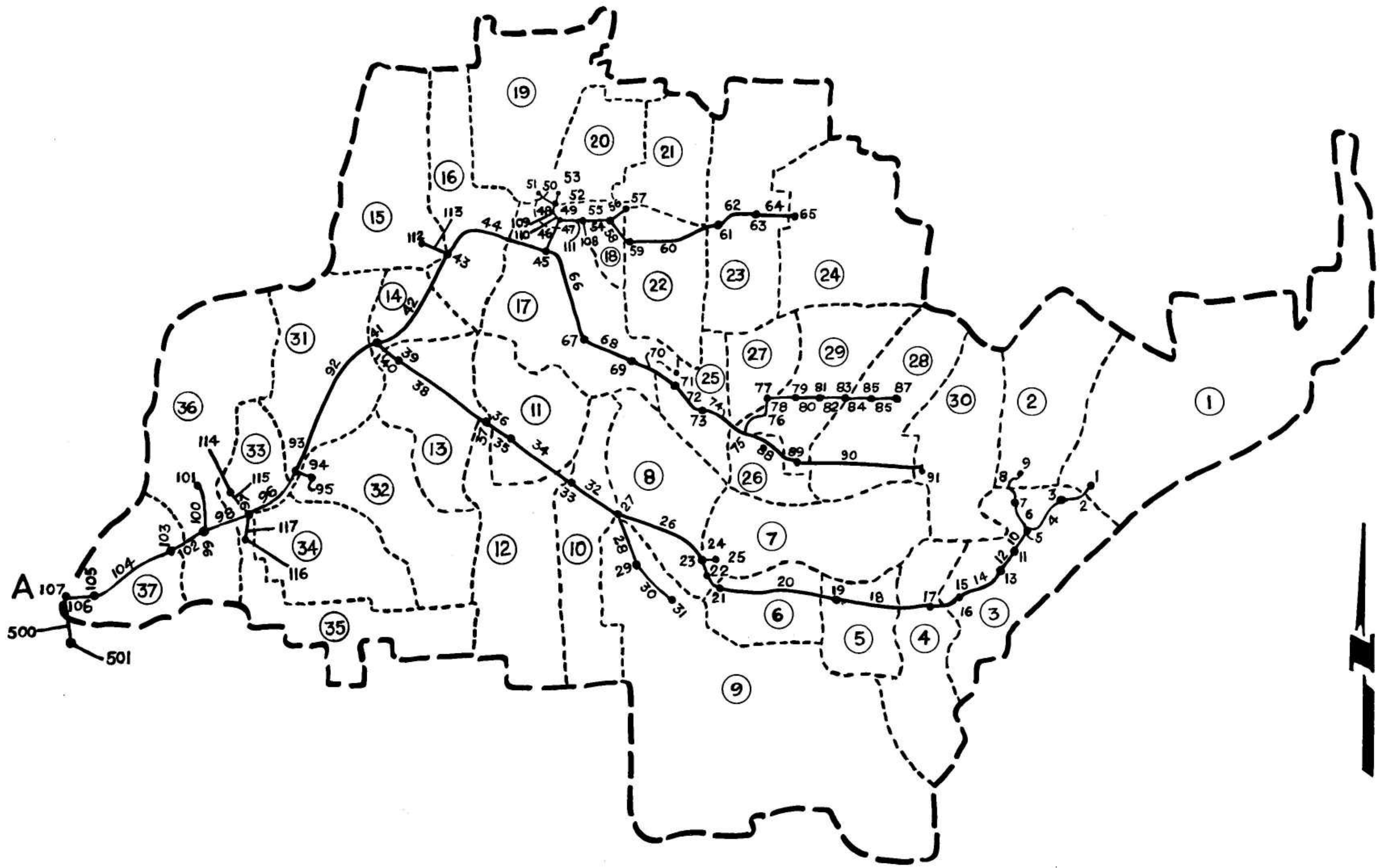

Figure 7.1 Plan of the Bloody Run Drainage Basin Sewer System and Discretization for Simulation without Control

Simulation Without Control Alternatives

The first step in planning for storm water control in the drainage basin was to determine the areas and magnitudes of local flooding and the magnitude of the overflow for a design storm. This was done by running the simulation model with the design storm. The basin was discretized as shown in Figure 7.1. This is the discretization for which the data was supplied by Metcalf & Eddy, Inc., Palo Alto, California. The data is essentially the same as that used by Lager et al. (1971) in their verification run with this drainage basin. Changes and additions made to the data were the addition of pipe 500 and manhole 501, and the specification of element 107 to be a flow divider with the undiverted flow entering pipe 500 and the excess overflowing into Mill Creek from element 107. Pipe 500 represents the interceptor to the Mill Creek Wastewater Treatment Plant. The flow amount that is to be undiverted, i.e., the design flow for the interceptor, is 45 cfs. This value is three times the approximate DWF of the basin, 15 cfs, as measured by Preul and Papadakis (1970), who did the actual data gathering for Lager et al. (1971). The reason why the interceptor is so designed is to provide enough capacity to carry away some of the infiltration and storm runoff that enters the system in addition to DWF. Three times the average DWF is the value that Fair, Geyer, Okun (1966) found to be a common design factor in North America. The actual capacity of the interceptor was unknown.

The choice of a design storm is a very important and complicated process. For this drainage basin, the ten year, 2-hour design was synthesized from rainfall intensity-duration-frequency curves published by the U.S. Weather Bureau (1955). The hyetograph of the design storm is shown in Figure 7.2.

The Executive, Watershed and Transport Blocks of the simulation model were then run for 140 time steps of 3 minutes each for a total simulation time of seven hours. This was to allow all the runoff to enter the system and all the system flow values to approach DWF conditions. The pipes which surcharged were then tabulated along with the maximum amounts stored at the upstream nodes, and the time periods of the start and end of the storage at the upstream nodes. This is shown in Table 7.1. Storage at a manhole due to surcharging physically represents local flooding. The general water quality of the flooding was also noted. The hydrograph and pollutograph of the overflow at node 107 in Figure 7.1 were also examined.

As shown in Table 7.1 there is considerable surcharging and flooding in the system as a result of this design storm. To make the decision of what flooding to control requires inputs from city administrators and engineers as to how much flooding they are willing to tolerate. It was assumed in the case study that only the most harmful flooding had to be controlled as actual information was not available.

To determine what was harmful flooding, the amounts of surcharging and storage were ranked by time period of initial flooding,

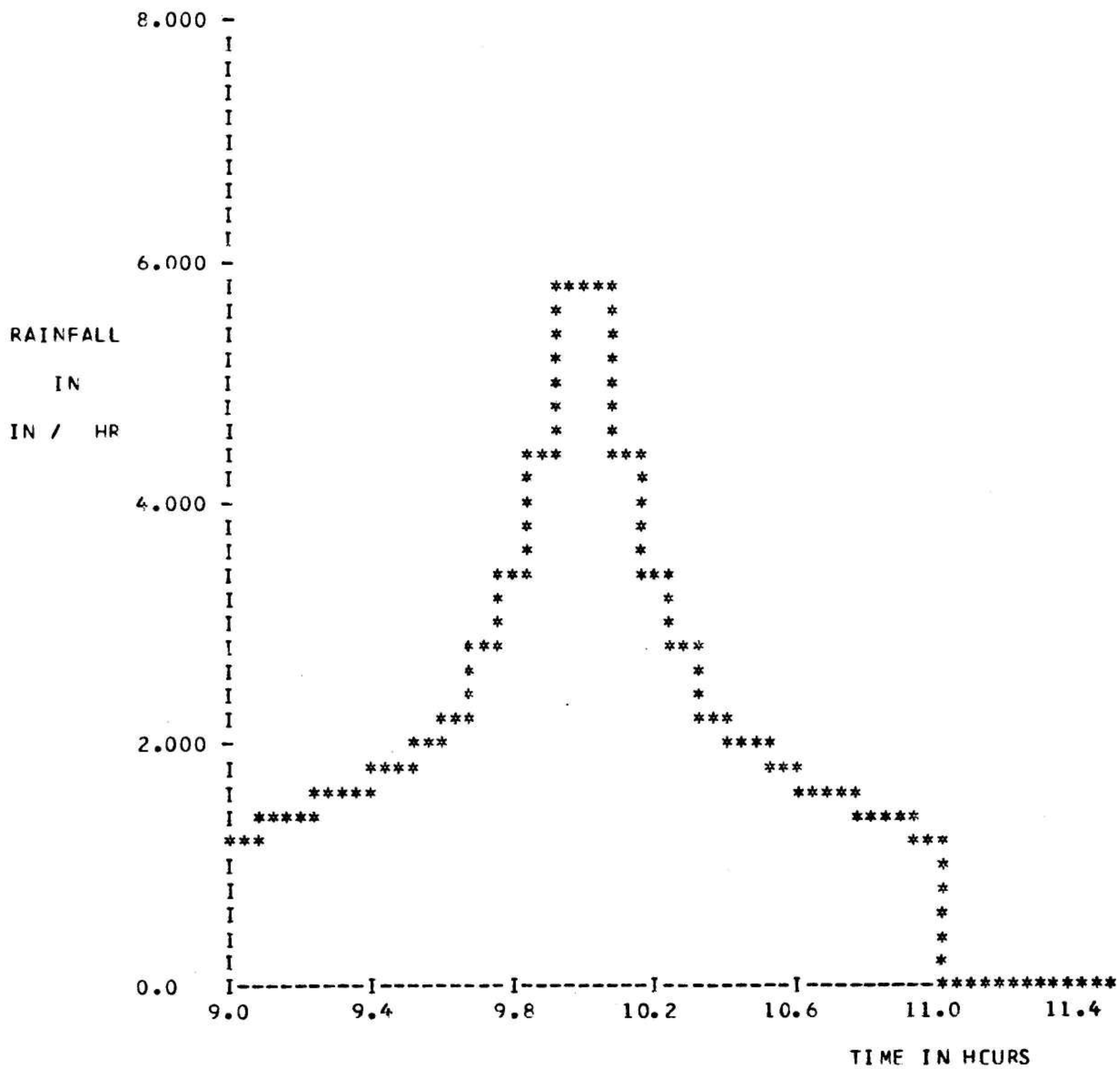

Figure 7.2 Design Storm Hyetograph for Bloody River Drainage Basin

Table 7.1

AMOUNTS AND DURATION OF LOCAL FLOODING BEFORE CONTROL

Pipe Number	Maximum Amount of Flooding at the Upsystem Manhole (Millions of cubic feet)	Start of Flooding (Time Period)	End of Flooding (Time Period)	Duration of Flooding (Time Periods)
2	.090	20	37	18
4				
6				
8				
10	.19	20	45	26
12				
14				
16	.26	19	50	32
18	.32	19	54	36
20	.31	19	58	40
22	.04	22	40	19
24	.03	22	29	8
26	.10	21	39	19
28				
30	1.30	14	83	70
34	.44	20	48	29
36	.50	19	56	38
38	.81	20	63	44
40	.48	20	67	48
42	1.76	18	65	48
44	.56	20	54	35
46	.20	21	52	32
48	.12	19	44	26
50	.04	20	35	16
52	.01	21	30	10
54	.25	18	52	35
56	.004	22	24	3
58	.090	20	47	28
60	.43	19	48	30
62				
64	.07	20	36	17
66				
68	.05	22	43	22
70				
72				(contin.)

(Continuation of Table 7.1)

Pipe Number	Maximum Amount of Flooding at Upsystem Manhole (Mill. of cubic feet)	Start of Flooding (Time Period)	End of Flooding (Time Period)	Duration of Flooding (Time-Period)
74	.23	19	50	32
76				
78				
80				
82				
84				
86	.50	8	116	109
88	.15	19	47	29
90	.06	20	41	22
92	5.63	17	89	73
94				
96				
98				
100	.85	14	82	69
102				
104				
106	1.80	22	96	75
110				
111				
113				
115				
117				
500				

maximum amounts and duration. The water quality of the flooding was also noted. It was found that in all cases except three surcharging started between time periods 17 to 22. Therefore this factor was not considered important because it just indicated that most flooding started at approximately the same time, i.e., within 20 minutes of each other. The 7 areas of maximum flooding were at nodes 41 (5.63×10^6 cu.ft.), 105 (1.80×10^6 cu.ft.), 43 (1.76×10^6 cu.ft.), 31 (1.30×10^6 cu.ft.), 101 ($.85 \times 10^6$ cu.ft.), 37 ($.81 \times 10^6$ cu.ft.), and 45 ($.56 \times 10^6$ cu.ft.). The 7 nodes which remained flooded the longest were nodes 87 (109 time periods), 105 (75 time periods), 41 (73 time periods), 31 (70 time periods), 101 (69 time periods), 39 (48 time periods), and 43 (48 time periods). Except for BOD, the water quality characteristics of the flooded areas were not significantly different enough to effect the decision on what were the most harmful areas of flooding. Therefore it was decided to control only the flooding at nodes where there was still flooding after 80 time steps or 4 hours after the storm had started and to tolerate the other flooding as it was "short term". This means that flooding at nodes 105, 101, 41, 31, and 87 would be entirely stopped. This would eliminate four of the top five areas of flooding and all five of the top five areas with flooding of the longest duration. Before a decision such as this is finalized, the land characteristics and uses of the flooded areas should be studied in detail to determine if even minor amounts of flooding at certain areas are harmful or if major amounts of flooding are tolerable at certain areas. It was not done for this case study as the detailed information was not available.

The amount of flow diverted into the receiving waters at element 107 during the 7 hour period varied from 0 cfs to 2153 cfs. The mean value was 1168 cfs. The BOD value varied from 0 mg/1 to 368 mg/1g with a mean of 41 mg/1. TSS varied from 0 mg/1 to 566 mg/1 with a mean of 102 mg/1. Coliform varied from 0 MPN/100 ml to 4.82×10^7 MPN/100 ml. Even though the characteristics of the receiving waters were not available, it may be stated that such amounts of BOD, TSS, and coliform entering a stream that flows through an urban area and is tributary to a major river, the Ohio River, are harmful. Therefore, it may be assumed that the pollutants entering from this overflow cause the stream standards to be exceeded and the entire overflow must be prevented.

Control Alternatives

To control the flooding at nodes 105, 101, 41, 31, and 87, and the overflows at 107, the following control alternatives were considered as feasible (see Figure 7.3): insystem pipe storage between nodes 87 and 77, relief sewers between 87 and 45, 31 and 41, and 101 and 107, increased pipe capacity between 87 and 77, 101 and 99, and 99 and 107, storage tanks at 87, 45, 31, 41, and 107, and a special treatment plant adjacent to 107. The treatment provided at the special treatment plant was microstraining and chlorination. The

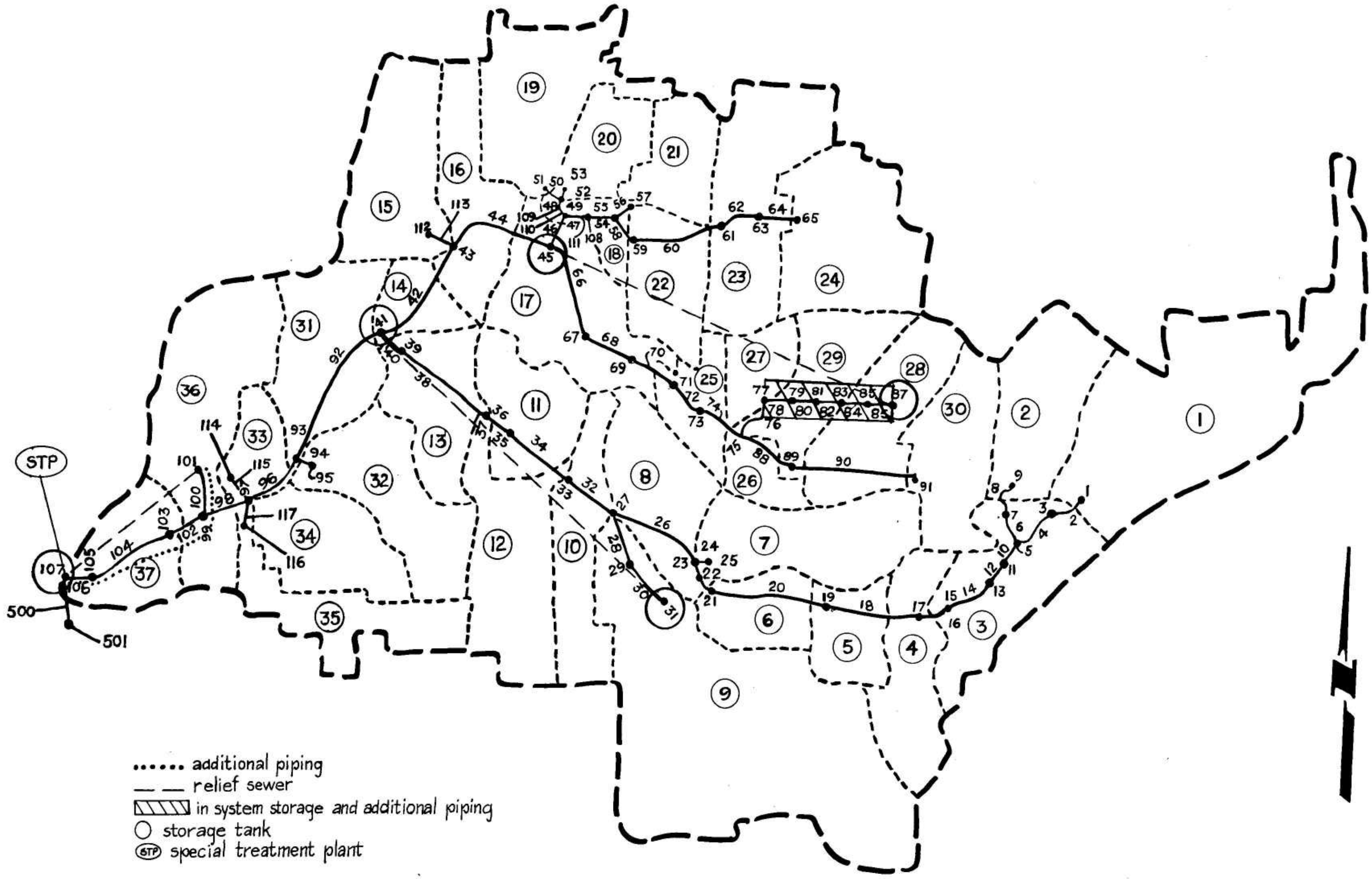

Figure 7.3 Proposed Control Alternatives for Bloody River Drainage Basin

choice of the treatment requires an estimate of the water quality characteristics of the combined sewer flows entering the plant after the flows have been controlled, and knowledge of the area's water quality objectives and the receiving water characteristics. Once these three factors have been estimated or determined, the treatment process can be selected that will remove the correct amount of pollutants from the flows that enter the plant such that when the treated effluent enters the receiving water, the water quality standards for the stream will at least be met. This process was not carried out for the case study as the required information was not available. If it turns out that after screening or simulation of the drainage basin with the control alternatives the quantity (and quality, in the case of simulation) characteristics of the combined flows entering the plant are not as expected, and sensitivity analysis indicates that the cost of the treatment change necessary will change the present optimal configuration, the screening model with the new treatment cost should be rerun to determine the new optimal configuration. This procedure should be repeated until the influent estimate approaches the screening model and simulation model results.

Discretization of the Drainage Basin for the Screening Model

The drainage basin was discretized as shown in Figure 7.4. As can be seen, as few as possible nodes were used. This was done to limit the number of constraint equations. The nodes were lettered A-K. At all the nodes, except for A and C, there was storm water runoff and

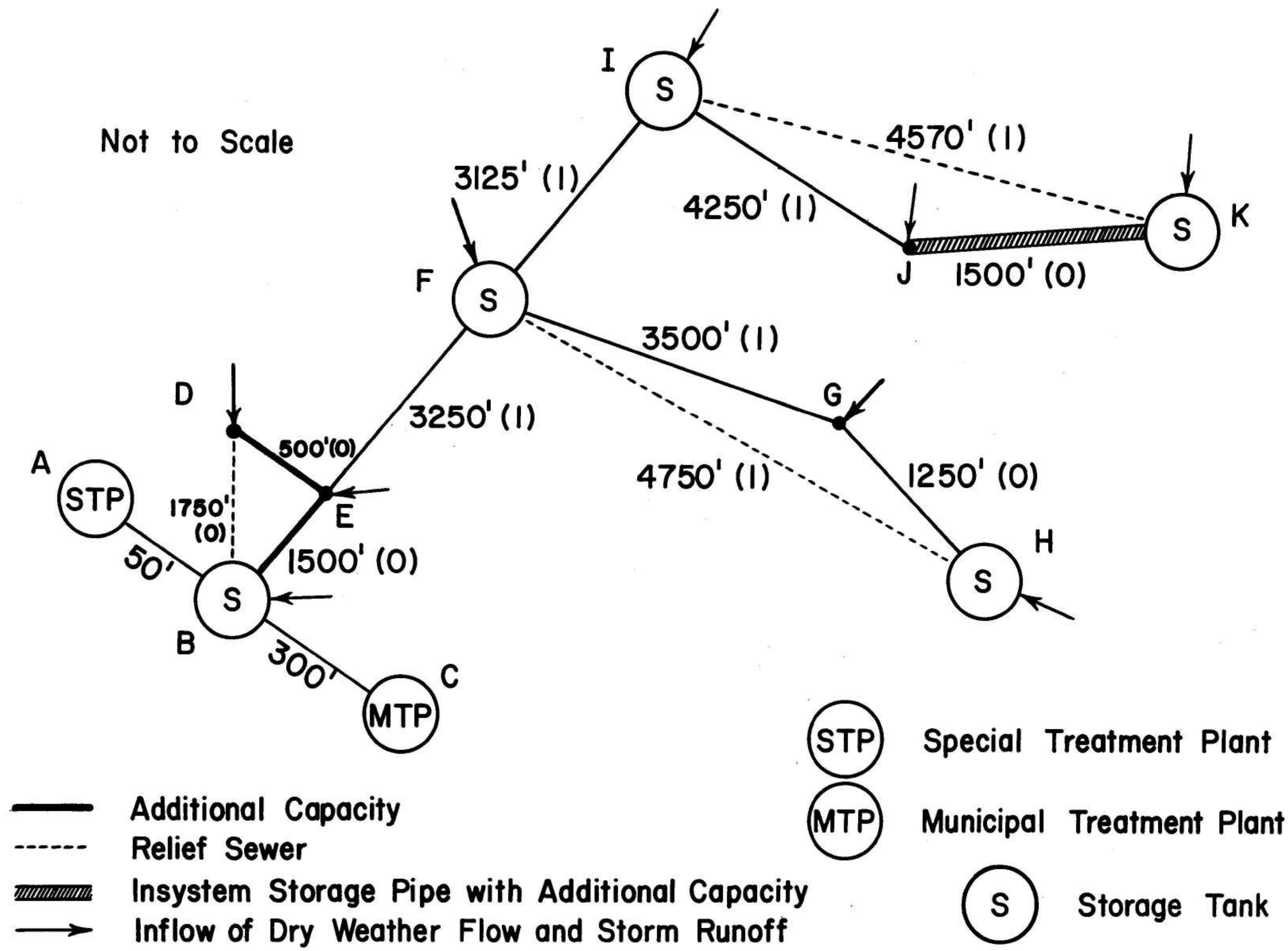

Figure 7.4 Discretization of Bloody Run Drainage Basin for Screening Model

DWF inputs. At nodes B, F, H, I, K, there is tank storage. A municipal treatment plant is at C and a special treatment plant is at A. The insystem storage pipe is from K to J. Relief sewers are from H to F, K to I, and from D to B. Increased capacity is being modelled between E and B, K and I, and D and E. The distances between nodes are labeled in feet.

Choice of Screening Time Intervals and Total Screening Time Period

The choice of the screening time interval depended upon the time of travel of the flows between the nodes. Since the flows are routed by time intervals, the length of a time interval must be short enough so that the routing is accurate enough. However, the length should not be so short that an excessive number of constraints have to be written as each constraint for each node is written for each time interval. To determine the length of a time interval, the times of travel of the flows between nodes must first be determined. This was done by determining the approximate velocities of flow in the modelled pipes of the system when they were full. This can be determined from Manning's formula, Equation (6.8). For the Bloody Run Drainage Basin the full velocities varied from 10 ft/sec. to 14 ft/sec. 10 ft/sec. was taken to be the full flow velocity for all modelled pipes. It was assumed that during the screening period the pipes would flow on the average 70 percent full. Therefore since Fair, Geyer, Okun (1966) calculate that in this situation the velocity is 1.12 times the full flow velocity, the average pipe flow velocity is 11.2 ft/sec. Therefore the time of travel between two nodes can be determined by dividing the distance between the two points by 11.2 ft/sec.

If one was to determine the length of a time interval by deciding to separate all the nodes by at least one time interval of travel, the length of a time interval would be the minimum distance between nodes where for at least one node flow is routed into as well as out of divided by the average velocity of flow. This would be 500 ft. divided by 11.2 ft/sec. or .75 minutes. This is obviously too short an interval. For this case study it was decided to determine the time of travel between nodes I and F and use this value to determine the time interval length. The time of travel between I and F is 4.66 minutes. Therefore, if the length of a time interval was 9 minutes, the flow would take one time interval to travel from I to F as 4.66 minutes is greater than one half a time interval. Thus, the time interval length was chosen to be 9 minutes. The figures in parenthesis in Figure 7.4 are the times of travel in time intervals between the nodes.

To determine the total number of time intervals to be modelled, i.e. the total screening time period, enough time intervals have to be modelled such that most of the runoff has entered the system and has been routed through the system. From the hydrographs of the runoff and of the system flows produced by the simulation model, it was possible to determine that the significant amounts of flow had ended at the end of 31 time intervals or approximately 4 1/2 hours. Therefore 31 time intervals of 9 minutes each were used in the screening model.

Determination of Inflows

To determine the inflows to the nodes, the inflow and outflow hydrographs produced by the simulation model were used. These were used in essentially two ways to generate the inflows. The first way is straightforward and requires using the inlet hydrograph generated by the simulation model. This was done for a node such as K. The input here was taken from the inlet hydrograph produced for node 87 in Figure 7.1. The other method is more complicated and is necessary for such nodes as G. The inflow here was determined by subtracting the runoff and DWF inflow to node 31 from the outflow from node 27 to get the inflow contribution to G from the sewer stem between node 27 and nodes 1 and 9. To this was added the runoff and DWF inflow to node 33. The runoff inflows to nodes 35, 37, 39 were assumed to enter at node F. If the nodes are far apart, for example, nodes 31 and 27, the "lag time" of the inflow to 31 and its outflow from node 27 had to be accounted for in the calculation of the inflows to the screening model.

Selection of Pipe Flow Factors

To select the pipe flow factor, c, used in the sizing constraints in Chapter VI, it first has to be determined whether or not flooding is permissible along a sewer stem. If it is not, then the c is the ratio of the maximum flow to the average flow. Since for circular pipes with a flow depth of 70% of the diameter, Fair, Geyer, Okun (1966) calculate that the maximum flow is 1.19 times the average flow, c equals 1.19. However, for sewer stems in which minor flooding is to be tolerated, for example stem IF in Figure 7.4, c should be less than 1.0. The value of .80 was chosen for c in these cases.

Upper Bounds on Pipe Capacities

It was decided that for relief sewers and additional pipes, the maximum size piping it would be feasible to construct would be that of ten feet diameter. This would allow a maximum discharge of approximately 780 cfs. The upper diameter bound set on the size of the additional piping to be laid between K and J for insystem storage was twenty feet. This would allow a maximum discharge of approximately 3142 cfs if there was no storage in the pipe.

Upper Bound on Special Treatment Plant Capacity

The upper bound on the capacity of the special treatment plant at A was set at 60.0 cfs. This value seemed a realistic upper bound as for an area of 4,200 acres in Washington, D.C., Neijna et al. (1970) are proposing constructing a plant of 77 cfs capacity.

Upper Bound on Municipal Treatment Plant Capacity

This upper bound was set at 45 cfs, the capacity of the interceptor. This means that the treatment plant has capacity not only to treat DWF but also infiltration and some storm runoff.

Upper Bounds on Storage Tank Capacities.

The upper bounds of all the storage tank capacities were 20 million cubic feet. This value is a feasible limit as Neijna et al. (1970) are proposing to build an underground storage tank of 23.5 million cubic feet for a 4,200 acre area in Washington, D.C. The tanks would be approximately 40 feet deep. In an actual situation, an area's land use and geological formations should be taken into account in setting upper bounds on the tank sizes (as well as other control facilities).

Selection of Pumping Out Time

The time allowed to pump the tanks dry after the storm was set at 10 days. Due to the nature of constraint Equation (6.26), this means that the tanks would be half full after 5 days. It was judged that half the system capacity would be adequate to handle a storm that might follow in 5 days. More detailed analysis based upon rainfall recurrence intervals should go into this decision in an actual situation.

Costs

The unit costs of the variables in the objective function were determined from cost curves as described later. When applicable, the costs included construction costs (including land), and operation and maintenance costs. The costs calculated were put in the form of present values based upon a lifetime of 25 years, an interest rate of 7% and an Engineering News Record (ENR) construction cost index of 1712, that of May, 1972.

Piping Costs

Since all the piping, including insystem storage piping, was piping that just had to be purchased and installed, i.e. no piping was to be replaced, the cost equation of Linaweaver and Clark (1964) was used. This equation, based upon regression analysis, gives the cost/ft. of piping of diameter d. The cost covers pipe line construction, right-of-way, and maintenance. The equation is

$$\text{cost (\$/ft)} = 0.358\ d^{1.29} \tag{7.1}$$

It is based upon an ENR construction cost index of 877. To convert the cost to dollars/cfs for each pipe except insystem storage piping, a plot was prepared of dollars/ft. *vs.* capacity in cfs. The pipe capacity was figured by multiplying the pipe area by the pipe velocity when the pipe is full, 10 ft/sec. Once the plot was done, which covered a range from 31.42 cfs (2 ft. diameter) to 3142 cfs (20 ft. diameter), the curve was linearized as in Figure 6.2.

The slope came out to be \$.342/cfs-ft. The length of each pipe was then multiplied by the slope to obtain the unit construction cost for that pipe in dollars/cfs. For insystem storage piping, the slope of \$.342/cfs-ft. was multiplied by the insystem storage pipe length and the velocity in the pipe when no flow was stored in it to obtain the unit cost of \$5,130/ft^2 of cross-sectional area.

Special Treatment Plant Costs

Lager et al. (1971) have developed cost equations for the capital costs of many special treatment processes, including microstrainers, chlorinators, and the associated pumping, the processes selected for this case study. In addition they report that the irreducible maintenance costs are 2 percent per year for these processes. The per storm maintenance costs were found to be negligible. Estimated land costs were added to the capital and maintenance costs to obtain the total cost. A curve of cost vs. capacity was plotted in the range from 0 cfs to 62 cfs. and linearized to obtain the unit cost. This value was found to equal .067 million dollars/cfs.

Municipal Treatment Plant Costs

In cases where only one drainage basin is being modelled and a municipal treatment plant already exists, the unit cost of municipal treatment only includes operation and maintenance costs. However, in cases where more than one drainage basin that uses an existing plant is being modelled, the total unit cost of the treatment plant should be used. This is because this value would be the actual "opportunity cost" of the treatment plant and would reflect the fact that if all or some of the treatment capacity was not available to one drainage basin, that basin may have to consider building such capacity.

Since only one drainage basin was being modelled here, the operation and maintenance costs were used. The unit operation and maintenance costs were determined from the estimate of Fair, Geyer, Okun (1966) that for a secondary treatment plant such costs average $.88/person annually. Using the area's design population, design flow rate, and discounting methods, this cost was converted to $21,000/cfs.

Storage Tank Costs

Buckingham et al. (1970) have capital cost curves for underground, multi-cell, concrete tanks in the volume range from 26,000 cu.ft. to 4,020,000 cu.ft. This range was extended to 20 million cu.ft. by assuming that several tanks could be built at a site to store this volume. For example, five 4-million cubic feet tanks could be combined to store 20 million cubic feet. This procedure is rather on the expensive side as it ignores economies of scale so that the storage costs are probably overestimated. Added to these capital costs were estimated land, pumping, and operation and maintenance costs. The final unit cost was $4.70 per cu.ft.

Results from Initial Screening

The initial screening process actually consisted of solving five problems. The first problem solved was with storage possible at all six sites, B, F, H, I, K, and insystem between K and J as shown in Figure 7.4. However, since the simulation model can only model a maximum of two upsystem storage sites and one storage-treatment site at the outlet (node B), the problem was resolved four additional times with storage possible at B plus two additional sites upsystem. The combinations of two upsystem storage sites that are feasible are I and H, I and F, F and K, and F and insystem between K and J. This procedure is relatively inexpensive to do as MPSX (1971) allows the user to start the next solution procedure from the previous optimal solution. This reduces the number of iterations necessary to determine the next optimal solution. The results of the 5 solutions are shown in Table 7.2. As can be seen in the table, the least cost solution is Solution I, the one where all six storage sites are available. This solution calls for building an additional pipe between D and E (in Figure 7.4) of 475.6 cfs capacity, building an additional pipe between K and J of 314.2 cfs capacity to use for insystem storage along with the existing pipe between K and J, not using a special treatment facility at A, using the maximum capacity of the interceptor to the municipal treatment plant at C of 45 cfs, and providing storage at B of 3,906,000 cu.ft., at F of 14,200,000 cu.ft., at H of 1,568,000 cu.ft., at I of 8,389,000 cu.ft., and

Table 7.2

Results from Initial Screening

Solution	I	II	III	IV	V
Cost (10^6 dollars)	134.972	136.330	137.062	136.674	136.043
Variable Description					
Cross-sectional area of insystem pipe between K and J (ft^2)	314.2	-	-	-	314.2
Capacity of relief sewer between K and I (cfs)	-	236.7	236.7	-	16.1
Capacity of additional pipe between D and E (cfs)	475.6	-	475.6	475.6	475.6
Capacity of addition pipe between E and B (cfs)	-	212.5	-	-	-
Capacity of relief sewer between D and B (cfs)	-	475.6	-	-	-
Capacity of relief sewer between H and F (cfs)	-	-	636.9	636.9	636.9
Capacity of special treatment plant at A (cfs)	-	-	-	-	-
Maximum amount of municipal treatment at C used	45.0	45.0	45.0	45.0	45.0
Storage at K (16^6 ft^3)	.09	-	-	.70	-
Storage at I (10^6 ft^3)	8.39	8.03	9.33	-	-
Storage at F (10^6 ft^3)	14.20	-	3.28	9.43	9.65
Storage at H (10^6 ft^3)	1.57	1.71	-	-	-
Storage at B (10^6 ft^3)	3.91	18.90	16.04	18.51	18.51

at K of 93,000 cu.ft. The total cost of this would be 134.972 million dollars.

The least cost solution using only two upsystem storage sites is Solution V with a total cost of 136.043 million dollars, only slightly more expensive than Solution I. This solution requires the construction of relief sewers between H and F and K and I of 636.9 cfs and 16.1 cfs respectively. It also requires additional piping between D and E and K and J of 475.6 cfs and 314.2 cfs respectively as does Solution I. The additional capacity between K and J is necessary for insystem storage. This solution also requires full use of the interceptor to the municipal treatment plant at C. Lastly, storage is to be built at F of 9,646,000 cu.ft. and at B of 18,510,000 cu.ft. Therefore this solution calls for insystem pipe storage between K and J, and tank storage at B and F. Since Solution V with only two upsystem storage sites (same as the internal storage sites in the simulation model) can be simulated directly on the simulation model, only the solution it presents was analyzed in detail in the simulation model. However, it should be noted that it would be possible to analyze Solution I on the simulation model by determining the outflow hydrograph from storage at K and storage between K and J, inputting this into storage at I and determining the resulting outflow hydrograph, and lastly, inputting this hydrograph into the system with storage at F, H, and B.

Determination of Operating Policy

As described in Chapter VI, besides determining the optimal sizes of storage tanks and pipes in the system, the screening model also determines the optimal flow amounts between the nodes for each time period. In order to simulate a screening solution, methods must be determined by which the simulation model will cause approximately the same flow amounts to occur. The features that can be used on the simulation model to do this include flow dividers and the outlet devices of the storage tanks (orifices, weirs, and on-off pumping). Since these features will not allow the user to simulate exactly the optimal flow amounts specified in the screening model, the optimal flow amounts must be altered somewhat so that they can be simulated on the simulation model. From studying the screening solution and knowing the features available in the simulation model and by hand-simulation it is possible to judge what features in the simulation model can be used to approximate the screening solution. However, how much such a fixed operating policy will change the optimal screening solution and its associated configuration is unknown. There is also doubt whether or not such an operating policy is feasible. Therefore, the fixed operating policy should be run on the screening model to settle these questions. This can be done by determining what some of the flow decision variables' values would be if such a fixed operating policy was in effect and fixing those decision variables to these values. For example, if under an operating policy the flow between two points is to be a constant value, the flow variable for the flow between these points should be set at this value for all time periods. Such a procedure was carried out for Solution V of the screening model. This operating policy called for relief sewers from H to F and from K to I with flow dividers in front of them, holding tanks at F and B, and insystem storage between K and J with an orifice outlet device with an area of 1.67 sq.ft. and a coefficient of discharge of 0.60. In order to run the screening model with such an operating policy, the upper bound on the additional pipe between K and J was increased because hand-simulation had determined that under the operating policy its present bound would be exceeded. In addition, the flows between F and B, K and I, and K and J were specified.

Screening with Operating Policy

The solution to the screening model run with the operating policy, Solution Va, is compared to Solution V in Table 7.3. As can be seen, both solutions require the same relief capacities from K to I and from H and F and the same additional piping capacities between D and E. Both also require the same use of municipal treatment. However, Solution Va requires additional piping between K and J of 354.6 sq.ft. cross-sectional area instead of 314.2 sq.ft. as specified in Solution V. Solution Va also specifies storage at F of 13,511,000 cu.ft. opposed to 9,646,000 as specified in Solution V. However, Solution Va specifies less storage required at B than does Solution V,

Table 7.3

Comparison of Screening Model Results with and without Operating Policy Built-In

Solution	V (no built-in operating policy)	Va (built-in operating policy)
Cost (10^6 dollars)	136.043	136.941
Decision Variable Description		
Cross-sectional area of in-system storage pipe between K and J (ft^2)	314.2	354.6
Capacity of relief sewer between K and I (cfs)	16.1	16.1
Capacity of additional pipe between D and E (cfs)	475.6	475.6
Capacity of additional pipe between E and B (cfs)	-	-
Capacity of relief sewer between D and B (cfs)	-	-
Capacity of relief sewer between H and F (cfs)	636.9	636.9
Capacity of special treatment plant at A (cfs)	-	-
Maximum amount of municipal treatment at C used	45.0	45.0
Storage at K (10^6 ft^3)	-	-
Storage at I (10^6 ft^3)	-	-
Storage at F (10^6 ft^3)	9.65	13.51
Storage at H (10^6 ft^3)	-	-
Storage at B (10^6 ft^3)	18.51	14.79

14,793,000 cu.ft. versus 18,510,000 cu. ft. The cost of Solution Va is only slightly greater than Solution V at 136.941 million dollars versus 136.043 million dollars. Therefore, the operating policy determined for the drainage basin for this storm appears feasible and close to optimal.

Simulation Modelling

Having determined a feasible operating policy for the optimal screening model solution, this solution was then analyzed in detail on the simulation model with this operating policy. To allow margin for error, the depths of the storage tanks at B and at F and the storage pipe between K and J were increased so that they would not flood during the storm. Since the simulation model prints out the maximum water depths reached in the tanks and the storage pipe during the simulation period, those depths were then used to size the tanks exactly. It was also decided to model the insystem storage between 87 and 77 as one pipe instead of two. This is because the cross-sectional area of the existing pipe, 7.7 sq.ft., is insignificant compared to the additional pipe cross-sectional area, 354.6 sq. ft. This would make the cross-sectional area now 362.3 sq.ft. Therefore the insystem storage pipe has really become a storage tunnel. This raises the question whether the resulting cost change will change the optimal solution configuration. This should be investigated using sensitivity analysis.

Shown in Figure 7.5 is the way the system was discretized for the simulation modelling. Elements 502 through 520 are the new elements that are needed to model the control system process. The main features of the control system are:

1. A storage holding tank at 502 of base area 425,000 sq.ft. and depth 45 ft. that stores the flows that enter it during the storm from element 516 for treatment after the storm. Since no outflow from this tank is desirable during the storm and an outlet device must be specified, an on-off pump was specified that would start at a water depth of 45 feet. Since this water depth would never occur in the tank during this storm, there would be no outflow from the tank during the storm as is desired.

2. A flow divider at 516 that diverts that portion of flows in excess of 45 cfs into storage at 502. The undiverted flows go to the municipal treatment plant interceptor, pipe 500. Element 107 is now just a manhole.

3. A flow divider at 503 that diverts that portion of the inflows into 101 in excess of 115 cfs into pipe 504. The undiverted flows enter pipe 100. 115 cfs is the capacity of pipe 100.

4. A pipe (element 504) from 503 to 99 of capacity 475.6 cfs.

5. A flow divider at 505 that diverts that portion of the inflows into 41 in excess of 970 cfs into storage at 506. The undiverted flows enter pipe 92.

6. A storage holding tank at 506 of base area 400,000 sq.ft. and

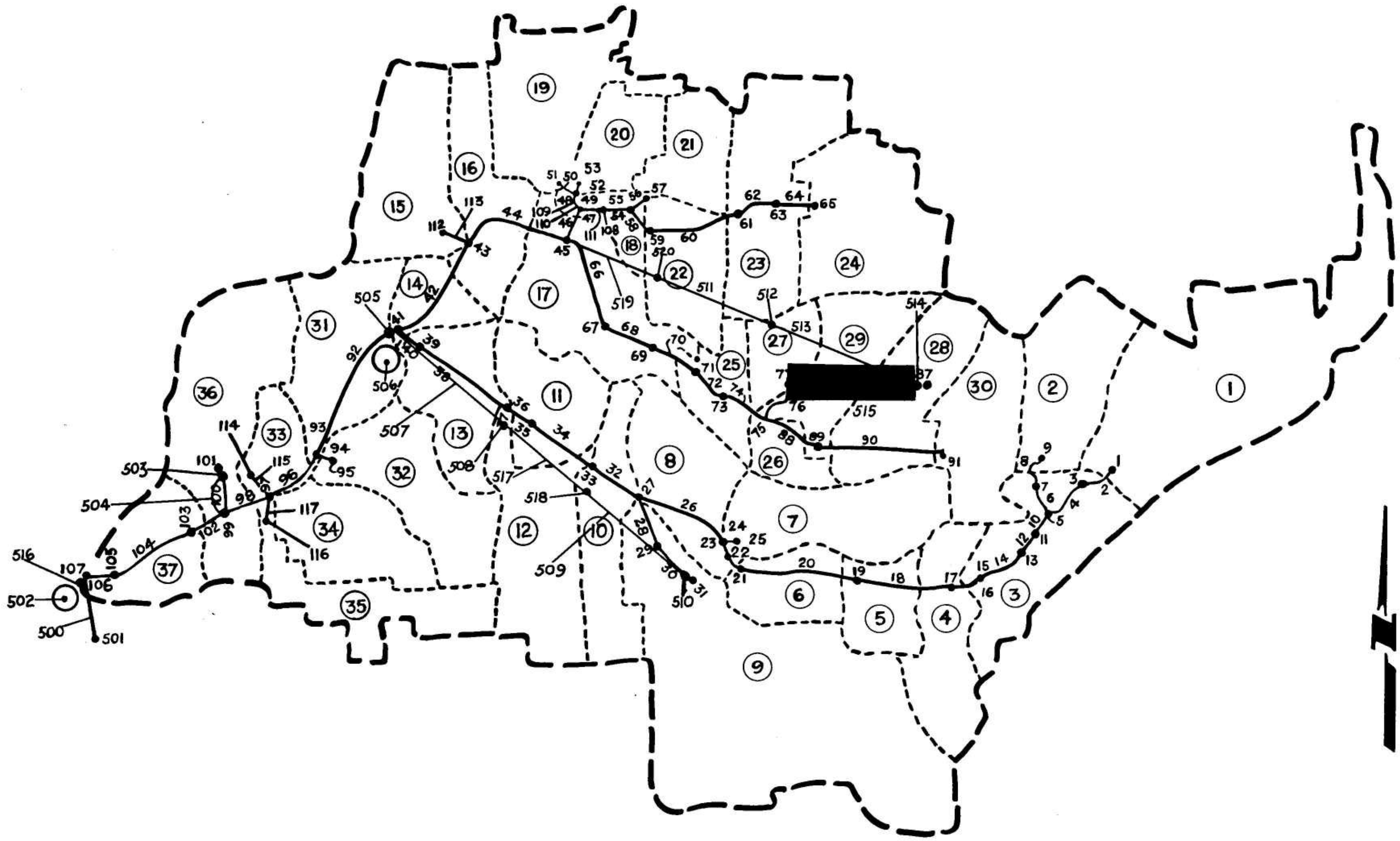

Figure 7.5 Discretization of Bloody Run Drainage Basin for Simulation with Storm Water Control

height 45 feet that stores the flows diverted at 505. This tank operates the same way as does that at 502, i.e., there is to be no outflow from the tank during the storm.

7. A relief sewer from 510 to 41 composed of elements 507-509, 517, and 518. The capacity is 637 cfs.

8. A flow divider at 510 that diverts that portion of inflows into 30 in excess of 238 cfs into pipe 509. The undiverted flows enter pipe 30. 238 cfs in the capacity of pipe 30.

9. A relief sewer from 514 to 45 composed of elements 511-513, 519, and 520 of capacity 16.1 cfs.

10. A flow divider at 514 that diverts that portion of inflows into 87 in excess of 16.1 cfs into insystem storage between 514 and 77. The undiverted flows enter pipe 513.

11. An insystem storage tunnel at 515 between 514 and 77 that is 1500 feet long, 23 feet wide and 20 feet high. The outlet device is an orifice flush with the tunnel bottom that lets the outflow enter node 77. The orifice has a cross-sectional area of 1.67 sq.ft. and a coefficient of discharge of 0.60.

Results from Simulation with Control

Shown in Table 7.4 is a listing of the pipes. Each pipe number is followed by the maximum amount of local flooding caused by that pipe surcharging, the start and end period of the flooding,

Table 7.4

AMOUNTS AND DURATION OF LOCAL FLOODING AFTER CONTROL

Pipe Number	Max. Amount of Flooding at the Upsystem Man-hole (mill.of cu.ft.)	Start of Flooding (Time Period)	End of Flooding (Time Period)	Duration of Floodg. (Time Periods)	Change N=No Y=Yes
2	.089	20	37	18	N
4					N
6					N
8					N
10	.19	20	45	26	N
12					N
14					N
16	.26	19	50	32	N
18	.32	19	54	36	N
20	.31	19	58	40	N
22	.04	22	40	19	N
24	.03	22	29	8	N
26	.10	21	39	19	N
28					N
30					Y (MAJOR)
32					N
34	.49	20	48	29	Y (MINOR)
36	.49	19	54	36	Y (MINOR)
38	.80	20	61	42	Y (MINOR)
40	.46	20	64	45	Y (MINOR)
42	1.76	19	65	47	Y (MINOR)
44	.63	20	55	36	Y (MINOR)
46	.20	21	52	32	N
48	.12	19	44	26	N
50	.04	20	35	16	N
52	.01	21	30	10	N
54	.25	19	52	35	N
56	.004	22	24	3	N
58	.09	20	47	28	N
60	.43	19	48	30	N
62					
64	.07	20	36	17	N
66					N
68	.05	21	43		Y (MINOR)
70					N
72					N
74	.19	19	49	31	Y (MINOR)

(Contin.)

(Continuation of Table 7.4)

Pipe Number	Max. Amount of Flooding at the Upsystem manhole (mill.of cu.ft.)	Start of Flooding (Time Period)	End of Flooding (Time-Period)	Duration of Flooding (Time Periods)	Change N=No Y=Yes
76					N
78					N
80					N
82					N
84					N
86					Y (MAJOR)
88	.15	19	47	29	N
90	.06	20	41	22	N
92					Y (MAJOR)
94					N
96					N
98					N
100					Y (MAJOR)
102					N
104					N
106					Y (MAJOR)
110					N
111					N
113					N
115					N
117					N
500					N
504					
507					
509					
511					
513					
517					
519					

the duration of the flooding, and the change of the flood description compared to the system with no controls as seen in Table 7.1. As can be seen, the goals of no flooding at nodes 105, 101, 41, 31, and 87 have been achieved. The changes in the amounts and duration of flooding at other points are slight, in most cases being a decrease. However, it had been decided previously that the flooding at these other points was tolerable. In addition, because of the control scheme, there were no overflows entering the receiving water. That portion of the flow entering node 107 that exceeds 45 cfs is now stored at 502 instead of being discharged into Mill Creek.

The maximum water depth reached in the storage tunnel during the storm was 18.25 feet. This means that a storage tunnel of volume 629,000 cu.ft. would be needed here. The maximum water depth reached in the storage tank at 506 during the storm was 33.89 ft, which means that a tank of volume 13,557,000 cu.ft. would be needed here. 34.56 ft was the maximum water depth obtained in the tank at 502. Therefore the total volume of this tank should be 14,686,000 cu.ft. These values and the times of peak depth agree remarkably well with the screening results from Solution Va as shown in Table 7.5.

Sensitivity Analysis

Having shown that the screening model results are agreeable with the simulation model, a user of the screening model can now afford to have confidence in its results and use the screening model

Table 7.5

Comparison of Simulation Model and Screening Model Results

	Simulation	Screening Solution Va
Volume Required at 502 (millions of cu.ft.)	14.686	14.793*
Time Peak Depth Reached at 502	291 minutes	279 minutes
Volume Required at 506 (millions of cu.ft.)	13.557	13.511
Time Peak Depth Reached at 506	201 minutes	189 minutes
Insystem Volume Required at 515	.629	.523
Time Peak Depth Reached at 515	129 minutes	126 minutes

*This was the amount of water stored at the end of the screening time period (279 minutes). The storage amount was still increasing slowly.

for sensitivity analysis as described in Chapter VI. The only sensitivity analysis done for the case study was to employ the RANGE command of MPSX(1971).

RANGE outputs the unit cost range of each variable such that as long as the unit cost of that variable remains within that range and there are no other changes made in the model, the activities of that variable and the other variables remain the same and optimal. The RANGE outputs for Solution I and Solution V of Table 7.2 are shown in Table 7.6.

As an example of the value of RANGE, it can be seen in Table 7.6 that if the unit cost of special treatment at A in Solution V was less than \$63,983/cfs, it would be optimal to use some amount of special treatment at A and the resulting total system cost would be less. However, it should be realized this would mean that the effluent from the plant would be of lower quality and perhaps cause a violation of the receiving water standards.

The reason why sensitivity analysis was done on the screening model without the operating policy built-in is because many of the decision variables in the screening model with the built-in operating policy are either directly or indirectly fixed and the sensitivity analysis done produces meaningless results. For example, RANGE was commanded in the screening model with the operating policy built-in that produces Solution Va in Table 7.3. The output indicated that the unit cost range of the additional insystem storage piping between K and J is $-\infty$ to $+\infty$. This is relatively meaningless and could

Table 7.6

RANGE Output

		Solution I	Solution V
Variable Description	Present Unit Cost	Unit Cost Range	Unit Cost Range
Special treatment at A (\$/cfs)	67,000	$63,983 \rightarrow \infty$	$63,983 \rightarrow \infty$
Municipal treatment at C (\$/cfs)	21,000	$-\infty \rightarrow 63,983$	$-\infty \rightarrow 63,983$
Relief sewer from H to F (\$/cfs)	1,640	$0 \rightarrow \infty$	$0 \rightarrow \infty$
Relief sewer from D to B (\$/cfs)	598	$171 \rightarrow \infty$	$171 \rightarrow \infty$
Additional Capacity from E to B (\$/cfs)	513	$0 \rightarrow \infty$	$0 \rightarrow \infty$
Additional Capacity from D to E (\$/cfs)	171	$0 \rightarrow 598$	$0 \rightarrow 598$
Relief sewer from K to I (\$/cfs)	1,640	$0 \rightarrow \infty$	$1640 \rightarrow 1640$
Insystem storage between K and J (\$/ft^2)	5,130	$-\infty \rightarrow 7,050$	$-\infty 7,494$
Storage at K (\$/ft^3)	4.7	$4.7 \rightarrow 4.7$	$-\infty \rightarrow 5.0$
Storage at I (\$/ft^3)	4.7	$4.7 \rightarrow 4.7$	$4.7 \rightarrow \infty$
Storage at F (\$/ft^3)	4.7	$4.7 \rightarrow 4.7$	$3.12 \rightarrow 4.7$
Storage at H (\$/ft^3)	4.7	$4.7 \rightarrow 4.7$	$-\infty \rightarrow 8.31$
Storage at B (\$/ft^3)	4.7	$4.7 \rightarrow 4.7$	$4.7 \rightarrow 4.9$

have been predicted because the activity of this variable is determined by decision variables that have been fixed.

Proposed Planning Process

The purpose of this section is to formally outline a procedure to plan for the control of combined sewer overflows and local flooding. It is based upon the experience gained in the case study and essentially summarizes the procedure followed there.

I. Use the simulation model to determine what are the areas of local flooding and what are the magnitudes and water quality characteristics of the local flooding and of the overflows. An appropriate design storm should be chosen. A preliminary decision should be made on how much the flooding and overflows are to be controlled.

II. Decide upon the control alternatives possible and build the screening model. The bounds on the control alternatives should reflect physical and social-economical-political factors. Solve the screening model and check to make sure the treatment process is satisfactory (i.e., the influent quantity is as expected). If it is, determine a feasible operating policy for the simulation model.

III. Run the screening model with the operating policy built-in to determine if the operating policy is feasible and close to optimal. This run will also adjust the sizes of the control facilities to fit the operating policy.

IV. Once a satisfactory operating policy has been found, analyze the control configuration suggested by Step III in detail on the simulation model. Again determine if the treatment process selected in Step II is satisfactory (i.e., the influent characteristics, particularly quality, are as estimated). If the treatment process is not satisfactory, select a new treatment process and start again at Step II.

V. If the results of the simulation analysis are satisfactory, giving confidence in the screening model results, conduct sensitivity analysis on data that is either doubtful or significant as described in Chapter VI. One very important possible use of sensitivity analysis at this point might be to study more closely the initial decision on what flooding and overflows to control in the basin. It may be that for a little more cost, a large improvement could be realized or that for a large decrease in cost, there would only be a slight decline in control. As discussed previously, sensitivity analysis should be done using the original screening model, not the screening model with the operating policy built-in. Other especially significant sensitivity analysis that should be done is on the cost data and the design storm.

VI. Determine the final configuration from the screening model and the sensitivity analysis results. Analyze this configuration in detail on the simulation model to make certain it controls as effectively as desired. This analysis should include the system's effectiveness under different design storms than the one originally chosen. Be certain to take into account the inaccuracies of the simulation model. Make any final adjustments in the control facilities and then begin planning for actual construction.

CHAPTER VIII

Summary, Suggested Improvements and Further Research

Summary

This thesis has presented the development of a storm water control screening model and a case study illustrating its possible use with an existing simulation model to plan for the control of combined sewer overflows and flooding in an urban area.

The screening model is a linear programming model that determines the minimum cost sizes and operating policies of control facilities in a sewer system such that there are no overflows or extensive local flooding. The major control alternatives modelled are storage tanks, insystem storage pipes, and special and municipal treatment plants. The water quality goals of an area are reflected in the choice of the type of treatment the wastes are to receive before discharge. The screening model is inexpensively and conveniently solved using the IBM MPSX(1971) package, which can also perform useful sensitivity analysis.

The simulation model is a large-scale, computer-based, mathematical model that can model with good accuracy the hydrographs and pollutographs produced in an urban sewer system during and after a storm. This includes modelling the effects of two upsystem storage facilities and one storage-treatment facility located at a system outfall. The model also has the capability of modelling the surface runoff of an area during a storm and the effects of discharges from the sewer system on receiving waters. The accuracy and capabilities of the simulation model exceed that of the screening model. However, the simulation model cannot perform optimization.

The proposed planning method for combined sewer overflow and flooding control as illustrated partially in the case study makes use of both models. The simulation model is first used to determine the major areas of the local flooding and the magnitudes of the flooding and overflows for a design storm. A preliminary decision is then made on what flooding and overflows to control, and then control alternatives are proposed. The control alternatives are then screened on the screening model. The results from this first screening are the approximate sizes of the facilities and a system operating policy. The system operating policy is then adjusted so that it can be run on the simulation model. The screening model with the adjusted operating policy built-in is then run to determine if the adjusted operating policy is near optimal and to re-size the control facilities for this operating policy.

Simulation modelling is then done with the control alternatives suggested by the screening model with the built-in operating policy. If the results are satisfactory (indicating confidence in the screening model), sensitivity analysis can be done using the screening model. Finally, the configuration suggested as "best" by the screening model and the sensitivity analysis results is analyzed in detail on the simulation model, and if control is satisfactory, plans are started for actual construction of the facilities. The case study done on the Bloody Run Drainage Basin, Cincinnati, Ohio, only carried out the planning procedure up to the point of showing that the screening model results are reasonable and of conducting some sensitivity analysis. However, it can be concluded from this experience with the case study that the modelling techniques of the screening model are sufficient for its purpose and that the proposed planning method is feasible.

Suggested Improvements and Further Research

Presently the screening model meets water quality objectives in an implicit manner. Treatment efficiencies are estimated that will meet the objectives and then revised if the objectives are not met. However, it appears that it may be possible to meet the water quality objectives in an explicit manner using the linear systems theory formulation discussed in the Literature Review, Chapter V. If this could be done, it would certainly improve the screening model as it would eliminate the estimation of treatment plant efficiencies.

Another improvement in the screening model would be made if the model could account for the nonlinearities of the cost functions of the control facilities. Separable programming techniques were tried in which the cost functions were broken up into linear segments, but local optima were found so that the results were difficult to analyze. Perhaps this problem can be remedied by more experience in using separable programming or by using another solution algorithm besides linear programming.

Another possible weakness in the screening model is that, while it is formulated for the general case, it has to be constructed for each case study specifically. What is needed is some computer software that can accept input data for any discretized sewer system and produce the specific screening model for that sewer system. If linear programming remains the solution algorithm, such software could produce the input card deck for MPSX (1971).

With or without these suggested improvements or other changes, there is still some further research that can be done on the screening model. One is to try using the screening model on a large urban area, for example, the size of metropolitan Boston, to determine if the screening model can produce realistic "optimal" control configurations for so large a system. Such research will probably require the model to handle nonlinear costs if its results are to be reasonable. This is because for large areas economies of scale of control facilities significantly influence final "optimal" configurations. Incidentally, some improvements may have to be made in the simulation model if a large area with many control facilities is to be simulated.

An additional area of further research may be on the routing method of the screening model. Presently routing is accomplished by assuming that all the flow that enters a pipe segment during a time interval either flows through the pipe segment in that time interval or else takes one or more time intervals to flow through. This method does not account for any storage effects in the pipe segment or the effects of upsystem or downsystem flows. A routing method that does account for such effects is the Muskingum Method (Henderson (1966)) which has been used to model river flows which like most sewer flows is open channel flow. The basic equation of the Muskingum method is

$$O_2 = aI_2 + bI_1 + cO_1 \tag{8.1}$$

where O_i is the outflow from the pipe in time period i, I_i is the inflow to the pipe segment in time period i, and a, b, c are constants that depend upon the length of the routing period, the kinematic wave response time of the channel, and the channel geometry. A routing scheme of this type would probably be more accurate than the present method and could also be adapted to route flow through insystem storage pipes and storage tanks.

A last area of further research has to do with the planning method. There are several questions that need answers here. The first is how valid is the sensitivity analysis done on the screening model without the fixed operating policy. This is a problem because some of the operating policies suggested by the screening model are physically meaningless (i.e., no control system could be realistically built to control that way), and because it is not always possible to simulate a physically feasible operating policy suggested by the screening model exactly on the simulation model because of simulation model limitations. It would be convenient if the sensitivity analysis could be done on the screening model with the fixed operating policy but as discussed in Chapter VII, sometimes its sensitivity results are meaningless because of the "fixed" nature of the model. Another question is how to choose the design storm and how should it be varied for sensitivity analysis. Lastly, the question arises of how much screening, sensitivity analysis, and simulation should be done until there is enough value in the results to actually start construction planning. This last question and the one dealing with the design storm will always require a certain amount of judgement to answer, but we want to know if it is possible to improve this judgement quantitatively.

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