1975

Stormwater Management for Urban Areas

Gerald L. Chancellor
University of Central Florida, chance69@hughes.net

STORMWATER MANAGEMENT FOR URBAN AREAS

BY

GERALD L. CHANCELLOR
B.S.E., Florida Technological University, 1972

THESIS

Submitted in partial fulfillment of the requirements
for the degree of Master of Science in Engineering
in the Graduate Studies Program of
Florida Technological University

Orlando, Florida
1975

ACKNOWLEDGEMENTS

I would like to express my gratitude to the following members of my committee:

Dr. Martin P. Wanielista
Dr. David L. Block
Dr. Waldron M. McLellon
Dr. David E. Clapp

I am also grateful for data supplied by the City of Orlando Engineering Department and their personnel.

Also, special thanks and appreciation to Mrs. Micki Davis for both the patience and the excellent typing which she performed for this research paper.

Finally, my sincere gratitude to Dr. M. P. Wanielista and Ms. Rosemary Chapman for their contributions and backing. This work was partially funded by a Department of Pollution Control Research grant.

STORMWATER MANAGEMENT FOR URBAN AREAS

by
GERALD L. CHANCELLOR

A B S T R A C T

Stormwater management in urban areas is a major concern today. The problem of disposing of this stormwater runoff in a satisfactory manner is very difficult indeed. Both the quantity and quality aspects of the runoff must be dealt with to obtain a solution of this problem. The water quality of the runoff can vary depending upon the different land uses of the drainage basin. The quantity of the stormwater runoff also depends upon the land uses, the rainfall intensity and duration of the storm.

The traditional methods available for determining the quantity of the stormwater runoff are numerous. These traditional methods and recently developed mathematical simulation models are discussed in this paper. Prediction of the water quality of stormwater runoff is in its infancy. Several of the mathematical models have the capabilities of quality simulation, however, the simulation results are usually inconsistent with actual quality data. Of the simulation models currently in use, the EPA Storm Water Management Model is one of the most comprehensive models.

Application and verification of these newly developed models is limited. The EPA Model was chosen to simulate the quantity and quality of a small urban drainage area. The study area chosen was an urban commercial section of the Lake Eola drainage basin. Physical data

of the study area, such as ground slopes, storm sewer sizes and locations and slopes were determined. This data was then utilized for simulations of actual rainfall events. Verification of the quantity and quality simulation results was performed with actual quantity and quality data obtained for these rainfall events. Quantity simulation was considered successful with good correlation between the simulated and actual runoff. Quality simulation was successful to a lesser degree, the conclusion being that further quality calibration of the Model was required. Correlation between actual and simulated stormwater quality was achieved to some extent. The lack of correlation was felt to be due to calibration of the Model.

Martin P. Wanielista
Director of Thesis

TABLE OF CONTENTS

LIST OF TABLES

LIST OF ILLUSTRATIONS

CHAPTER I

INTRODUCTION

Stormwater runoff can be defined as that wastewater resulting from precipitation which flows across the ground during and after a storm. The purpose of storm drainage systems is to guide this runoff from areas where it cannot be discharged to a discharge area. The object being to prevent damage from flooding, protect the environment, and minimize costs. This routing must be done to avoid property and human damage which indicates sufficient drainage capacity must be provided. The determination of this capacity can be difficult in some cases. Several factors must be considered, one of the most important being precipitation itself. The calculation for the amounts and frequencies of stormwater runoff can be done in many ways. Several traditional methods are used and will be discussed further in this paper. In addition, many new methods and techniques, based for the most part on traditional methods, have been developed recently. These will also be discussed in this paper.

Stormwater runoff has become a controversial topic in recent years. In the past, the quality of this runoff was not of much concern. The emphasis was on control and disposal. Now it is realized, that the quality of the stormwater is not the same as the precipitation quality. Therefore, much research and work has been undertaken in the field of stormwater pollution control. The question must be answered, though, what are the quantities?

The continental United States receives an average precipitation of about thirty inches per year. This totals about 4400 U.S. billion gallons per day (BGD).[1] In the thirty inches, on the average, twenty-one and one-half inches is evaporated; the other eight and one-half inches is available for use. This eight and one-half inches, which is available, is still a large quantity of water. Available meaning that portion available for human use and consumption. Of this eight and one-half inches, the human population utilized in 1960 approximately two inches, of which one-half inch was lost to evaporation.[2] The used portions were discharged to the environment. The quantity not evaporated, approximately eight inches or 1165 BGD, eventually becomes contaminated to some degree.

It should be noted that precipitation is widely distributed and may vary greatly from one locale to the next. Likewise, the storm runoff of 1165 BGD is spread in varying amounts over the United States, the largest amounts occurring in the Mississippi and Columbia River basins of 400 and 223 BGD respectively. The design and operation of storm drainage systems depends upon the specific area with precipitation data playing an important role in the design. The State of Florida receives an average rainfall of 52 inches per year. Naturally, the various aspects of runoff, such as the quantity and quality, depend heavily upon the topographic characteristics, soil conditions, and land use of the area. There are large scale differences in runoff in both quality and quantity between urban and rural drainage basins.

Urban runoff can cause disruptment of sewer and catchbasin functions, overloading of treatment facilities (in combined systems),

and the pollution of receiving waters. Urban runoff quality can consist of pollutants, normally not found in rural runoff. The quality of urban runoff can also vary depending on the land uses of the area contained in the urban basin. The toxicity of urban runoff pollutants, on the whole, generally are greater than those pollutants contained in urban runoff.

Urban runoff can, in most cases, be classified as a nonpoint source of water pollution. A nonpoint source being defined as "a location or land use at which pollutants are released at an uncontrolled rate."[3] A point source being one where the rate is controlled. Urban areas consist of different types of land uses. These land uses are usually classified as:

1. Commercial
2. Residential
3. Industrial
4. Parkland (undeveloped)

Detailed analyses of urban runoff from commercial areas are limited.

Objectives

A comprehensive analysis and simulation of the storm water runoff resulting from an urban commercial area utilizing the U.S. Environmental Protection Agency Storm Water Management Model was undertaken. The objectives of this report were as follows:

1. Review past and present methods of stormwater runoff determination including mathematical models.

2. Application of a mathematical model to an urban drainage basin, in one locale, consisting of commercial land use.

3. Analysis and comparison of the model simulation to actual data.

CHAPTER II

STORM WATER QUALITY

Background

Stormwater quality has its initial beginnings in the rainfall from which it is comprised. It has been found that rainwater has definite quality aspects which may change from one area to the next. The chloride contained in rainfall has its source from sea salt. Analyses of other ions, such as Na^+, Ca^{2+}, Mg^{2+}, and NO_3^-, contained in rainwater have been done. First, it is apparent that rainfall contributes significant amounts of chemical substances to southeastern streams, which must, therefore, be considered in any serious "ionic" balance or geochemical studies involving dilute waters.[4] A major study on the chemical composition of rainfall in southeast Virginia and North Carolina by A. W. Gambell in 1965 is one study which has been done. These studies have pointed out the fact, that precipitation cannot be ignored as a source of nutrients. One feature of the Gambell study is that one-half of the dissolved solids in the streams was found to have been contributed by rainfall.

The distribution of the various chemical parameters contained in rainwater varies widely over the United States. Chloride ion concentrations are highest along the coasts and decrease farther inland. There are seasonal changes also, which can be attributed to the higher frequencies of storms in the winter. Sodium and magnesium contents increase somewhat in the winter also. Calcium reaches a peak in the summer

most likely because of agricultural activities. A survey of ammonium and nitrate concentrations in rainwater by C. E. Junge in 1958 had interesting results. Low concentrations were found along the coastlines and increased inland. Rainfall is not "pure" but imparts several ions and nutrients to the runoff, no matter what the land use. However, land use has an effect and runoff can be generally classified as rural and urban stormwater runoff.

Rural Runoff

Rural runoff is a major source of water pollutants in the United States. In the midwestern states, where large areas are devoted to agricultural activities, high concentrations of pollutants can be discovered. These pollutants have their origins in the fertilizers, pesticides and other agricultural chemicals employed by our nation's farmers. The natural reaction is to cry for a halt to their use, such as the case for DDT was. But, unless suitable alternatives are furnished which will maintain the current level of production, this method of halting their use cannot be undertaken. Because of the large areas of land involved, the control and monitoring of rural land runoff proves to be very difficult. Several factors affect the runoff quality. Among these are soil types, climates, type of land-use, cover crops, and topography of the area.

Rural land uses are usually either agricultural or woodlands in nature. The major portion of stormwater runoff in rural areas originates in these agricultural areas. The various agricultural land uses are:

1. cultivation
2. pasture and feedlots
3. groves and orchards

The runoff quality depends on the type of land use and crop cover. Recent studies have begun to document the characteristics and transport mechanisms associated with "rural" runoff.

A study has been done by Wallace and Dague[5] on the dissolved oxygen content of rivers. The purpose of the study was to develop a computer mathematical model to simulate the effects of agricultural land uses on river dissolved oxygen concentrations. The authors had difficulty in verifying their model due to the limited quantities of water quality data available. Conclusions reached in the study were as follows:

1. Agricultural land runoff causes low dissolved oxygen values during higher river flows each summer.

2. Typical runoffs can cause dissolved oxygen concentrations below five mg/l to occur.

3. Land runoff is the only cause of low dissolved oxygen values in the Iowa River.

Crawford and Donigan[6] developed a model for predicting the transport of pesticides over agricultural land. Pesticide is a general term for all forms of insecticides, herbicides, fungicides, fumigants, nematocides, algacides, rodenticides, etc. Depending on the types or forms of pesticides used, the effects they have can be either short or long term. Some will have detrimental effects for long periods of time, even years. Among these long term effects are birth defects, genetic changes and total extinction. However, pesticides will continue to be used until a suitable alternative is found.

The quantities of pesticides produced yearly is staggering. The total production of these chemicals in 1971 amounted to 600,000 tons.[7] The use of such large quantities necessitated the need for state and

federal regulations to control these pesticides. The pesticide runoff model simulates the transport of pesticides in the agricultural environment. The model takes into account the various mechanisms of transport such as surface runoff, sediment loss, and the time dependent decomposition of the pesticide. Prediction of the quantitites of pesticides in surface runoff from agricultural land is important. The work done by Crawford and Donigan[8] provides a basis for this prediction. The model could be the prototype for a model which would simulate other pollutants such as fertilizers.

Various parameters, such as BOD_5, suspended solids, nitrogen and phosphorus, have been shown to vary according to the agricultural land use. Another model which simulates these parameters is discussed later in this paper. The effects of animal wastes on land runoff has been studied by the North Carolina State University.[9]

This study considered the pollutant loadings and characteristics of four types of animals with these being:

1. swine
2. dairy
3. beef
4. poultry

production and feed areas. Several parameters were investigated, including BOD_5, Total Organic Carbon, nitrogen, phosphate and bacteria. The pollutants associated with the various areas were found to differ. This was found to depend on land characteristics, animals and waste disposal techniques. The extent of water pollution caused by farm animal production units is more dependent on production and waste management practices than on the volume of wastes involved.[10] Table 1 demonstrates the variations in pollutants, for different animal production operations,

TABLE 1

AVERAGE AMOUNTS OF WASTES IN LAND RUNOFF FROM REPRESENTATIVE ANIMAL-GROWING OPERATIONS[12]

	FC	BOD_5	TOC	N	PO_4
	10^6 col/day /animal	lb/day/animal			
Swine					
Direct discharge[1]	30,000	0.176	0.128	0.032	0.017
Anaerobic lagoons[2]	910	0.154	0.156	0.009	0.005
Land spreading[3]	630	0.003	0.007	0.002	0.001
Dairy[1]	60,000	0.190	0.230	0.034	0.027
Beef[3]	430	0.030	0.100	0.008	0.003
Poultry[3]	0.6	0.0005	0.0007	0.0002	0.00003

1 Sampling station downstream from point of discharge with intervening section of stream providing some stabilization and settling.

2 Lagoon effluent.

3 Sampling station at base of watershed immediately below site.

contained in land runoff.

One problem the authors had in the analysis of the wastes was the interference of heavy metals and antibiotics contained in the animal feeds. The difficulty was with the BOD_5 tests as test results varied above 60 mg/l BOD_5, necessitating the concurrent use of TOC (or COD) for the estimation of degradable organics and oxygen demand at BOD_5 levels above 60 mg/l.[11] One suitable method for the prevention of receiving water pollution from the surface runoff is land spreading of the animal wastes. This technique would be similar to that employed in the land spreading of sewage sludge. The only disadvantage being the additional costs for the land, equipment and manpower for the land spreading operation.

Rural runoff is generated from several different land uses. Accordingly, the amounts and types of pollutants contained in this runoff will vary widely. Generally, the greater the extent of mankind's utilization of the rural land, the greater the amounts of pollutants introduced. These pollutants, whether they are fertilizers, pesticides or animal wastes, diffuse over the surface of the land and eventually enter the aquatic environment. The transport processes, including surface runoff, are varied. In the past few years, investigations and studies have been undertaken to determine these processes, the pollutant levels occurring and their subsequent interactions. Another source of water pollution and perhaps more detrimental than rural land runoff is urban stormwater runoff.

Urban Runoff

Urban runoff has the potential to contain many polluting materials.

The sources of these pollutants are widely varied, ranging from the "city" birds, such as pigeons, to vehicle tires. Pollutants can consist of solid waste litter, chemicals used by the public, air deposited substances, and vehicle pollutants. In urban areas the sources of pollution in the runoff are: precipitation, vehicles, animals, buildings, humans, lawns and others. These result in street and lawn contaminants.

Runoff from urban streets is usually highly contaminated and can contain several toxic materials. This polluting material will vary widely in both quantity and distribution on the street surfaces. Street surface contaminants consist of several types: heavy metals, nutrients, pesticides, bacteria, and dirt (including dust). Inorganic materials, specifically dirt and dust, make up the major portion of the street surface contaminants. Table 2 summarizes the contaminants of street surfaces found by Sartor, Boyd, and Agardy.[13] The organic content is a rather small fraction of the total.

Significant amounts of heavy metals were detected in the contaminant materials collected from street surfaces.[14] When the metals found in urban runoff are compared with the metals content of sanitary sewage, loadings of 10 to 100 times the concentration (mg/l) of sanitary sewage metals is observed. The most prevalent metals found by Sartor, Boyd, and Agardy[15] were zinc and lead. However, Pitt and Amy[16] observed the highest concentrations of metals were iron, lead, manganese and zinc given in Tables 3 and 4. The metals concentration varied from one street surface to another, which could be expected. Industrial land use areas have high loadings of heavy metals with residential second. Commercial areas have the least loadings, but except for the industrial land use,

TABLE 2

QUANTITY AND CHARACTERISTICS OF CONTAMINANTS FOUND ON STREET SURFACES[17]

Measured Constituents	Weighted Means for All Samples (lb/curb mile)
TS	1,400
Oxygen demand	
BOD_5	13.5
COD	95
VS	100
Algal nutrients	
Phosphates	1.1
Nitrates	0.094
Kjeldahl Nitrogen	2.2
Heavy metals	
Zinc	0.65
Copper	0.20
Lead	0.57
Nickel	0.05
Mercury	0.073
Chromium	0.11
Pesticides	
p,p-DDD	67×10^{-6}
p,p-DDT	61×10^{-6}
Dieldrin	24×10^{-6}
PCB	$1,100 \times 10^{-6}$
Bacteriological	
Total Coliforms* (organisms/curb mile)	99×10^9
Fecal coliforms* (organisms/curb mile)	5.6×10^9

* Number of observed organisms/mile
Note: Lb x 0.454 = kg; mile x 1.61 = km

TABLE 3

ELEMENTS HAVING SUBSTANTIAL (10 TIMES) STRENGTH DIFFERENCES BETWEEN DIFFERENT LAND-USE SAMPLES[18] (mg/kg)

Element	Residential	Industrial	Commercial
Beryllium	0.2	2	0.2
Fluorine	1	5	0.5
Strontium	1000	200	100
Uranium	2	5	0.5
Vanadium	5	50	50

TABLE 4

ELEMENTS HAVING SUBSTANTIAL (10 TIMES) LOADING DIFFERENCES BETWEEN DIFFERENT LAND-USE SAMPLES[19]

Element	lb/curb mile		
	Residential	Industrial	Commercial
Antimony	0.002	0.014	0.001
Barium	0.240	0.56	0.058
Chromium	0.240	1.4	0.029
Cobalt	0.006	0.014	0.001
Fluorine	0.001	0.014	0.001
Hofnium	0.006	0.028	0.001
Lead	2.4	14	1.4
Lithium	0.006	0.014	0.001
Molybdenium	0.024	0.056	0.001
Nickel	0.12	0.28	0.015
Scandium	0.006	0.056	0.001
Strontium	1.2	0.56	0.029
Sulfur	0.60	1.4	0.14
Uranium	0.002	0.014	0.001
Zirconium	0.60	2.8	0.058

the highest strengths measured as mg/kg. The milligrams per kilogram unit being the milligrams of specific pollutant contained in one kilogram of street surface contaminants.

Cities with high particulate loadings were also found to have high metal loadings.[20] This would seem reasonable considering most cities with high particulate loadings have high concentrations of industry which would yield the high metal loadings. Of the metals, copper, cadmium, lead and zinc are sufficiently soluble to produce toxic effects to some aquatic life with the necessary conditions present. It could be assumed that under certain conditions the concentrations of heavy metals could build up over time to reach toxicity levels. These conditions, unfortunately, can be found to exist in many cases. The most dramatic toxic effects of metals most likely occur when runoff is discharged into quiescent water where it is allowed to accumulate to toxic concentrations.[21] The problem of heavy metals contained in urban runoff is serious as the reduction of these metals at their sources, would be an almost impossible task. One major source of these metals is the automobile. The exhaust gases produced by the internal combustion engine contain the various heavy metals. Efforts and techniques to remove these metals have just begun. Lead-free gas is one technique employed to remove the lead from exhaust gases. Other heavy metals are still present and will have to be dealt with eventually.

Nutrients in street surface contaminants are basically of four types. These organic materials can be classified as:

1. Greases and oils from vehicles (including exhaust hydrocarbons)

2. Bird and other animal wastes

3. Food litter

4. Organic materials consisting of wood, leaves, grasses, and other vegetation wastes

These organic materials can produce high biological oxygen demand in receiving waters. This decreases the oxygen levels in these waters, which can lead to the death of aquatic life. Greases and oils were the most prevalent organic materials. These pollutants, like the high heavy metal concentrations, have as their main source the vehicles utilizing the streets. All vehicles deposit greases and oils to some extent on the street surfaces. The quantities deposited vary according to the individual vehicle. Pitt and Amy[22] found grease and oil loadings ranging from 32.8 lb/curb mile for industrial areas to 4.90 lb/curb mile for commercial areas. Residential areas had a reported loading of 18.6 lb/curb mile.

A total oxygen demand of 208.5 lb/curb mile was reported by Sartor, Boyd and Agardy[23]. This includes BOD_5, COD and volatile solids. This oxygen demand is an average for the cities which were studied. Algal nutrients were found consisting of phosphates, nitrates, and Kjeldahl nitrogen. Kjeldahl nitrogen had the highest loading according to Sartor, Boyd, and Agardy.[24] This agreed with the findings of Pitt and Amy[25] for city streets. Table 5 gives the findings of Pitt and Amy[26] for three types of roadways. The city street samples have the highest reported values of BOD_5, COD, NO_3^-, N, Cr, Fe, Pb, and Zn. The city streets were commercial area streets with high volumes of traffic. It is interesting to note that Pitt and Amy[28] report that rural roads have high phosphate loadings with Kjeldahl nitrogen second highest. The question arises as to whether this is due to the increased use of fertilizers

TABLE 5

COMPARISON OF LOADINGS OF DIFFERENT TYPES OF ROADWAYS FOR COMMON POLLUTION PARAMETERS AND CERTAIN HEAVY METALS[27]

Parameter	lb/curb mile		
	City Street	Rural Road	Highway
BOD_5	18	2.4	15
COD	95	77	299
$PO_4^{\equiv}$	1.1	3.0	1.32
NO_3^-	0.043	0.22	0.23
N	2.4	0.79	4.22
Cd	0.0037	0	0.058
Cr	0.231	0.34	1.20
Cu	0.129	0.06	0.26
Fe	24.4	36	136
Mn	0.468	1.35	2.39
Ni	0.040	0.16	0.68
Pb	1.66	0.10	3.17
Sr	0.022	0.078	0.32
Zn	0.409	0.11	1.24

in rural areas?

Increases in nitrogen and phosphorus loadings have been observed in the Spring and Fall. This would seem to indicate that fertilization was contributing to these increases. However, studies reported by Kleusner and Lee[29] using artificially generated precipitation indicated that runoff from established lawns in the Madison, Wisconsin, area would occur only under unusually heavy rainfall conditions. It would appear that fertilization is not a contributing source of phosphates and nitrogen to residential runoff. Under the appropriate conditions, fertilizing could contribute to the water pollution problem. These conditions include high precipitation rates and possibly high fertilization rates.

Pesticides are found in street surface contaminants to some extent. Those detected were p,p - DDD, p,p - DDT, Dieidrin, and PCB. These are organic pesticides which will decompose in the environment. PCB is not a pesticide as such but has many pesticide properties. The interpretation of observed pesticide levels is difficult indeed.[30] The levels, which are acceptable for various pesticides, are difficult to determine and the subject of much controversy. Pesticide concentrations show no variations with land use. Data reported by Sartor and Boyd[31] showed no recognizable patterns of pesticide distribution with land use. There has not been much study on pesticides effects on water environments. Much study has been done on pesticides effects in soil environments. The effects of pesticides in aquatic environments are largely a matter of speculation. Certainly, at some (undetermined as of now) levels, detrimental effects do occur. Pesticides are present throughout our eco-

systems and most certainly are contained in storm water runoff to some extent.

Bacteria content of street surface contaminants has been studied. Pathogens are difficult to determine in natural waters. For this reason, tests are run on the groups of bacteria known as coliforms. Certain coliforms are associated with the presence of pathogenic bacteria. Two terms commonly used in describing the bacteriologic quality of water are "total coliforms" and "fecal coliforms." Fecal coliforms are those whose presence indicates the presence of pathogenic bacteria also. Total coliforms include the fecal coliforms and those coliforms occurring naturally in the soil and elsewhere. Drinking water standards established by the Public Health Service set limits on the number of total colifroms acceptable in drinking water sources. The use of water with high total coliform counts has been shown to cause several varieties of diseases. The bacterial loadings found by Sartor, Boyd, and Agardy[32] are shown in Table 2.

Total coliforms were found to be higher in industrial areas than commercial areas by Sartor and Boyd.[33] Residential areas had the lowest total coliform loadings. The quantities of coliform bacteria contained in urban runoff has not been satisfactorily determined. Several factors may affect the concentrations of bacteria. These will be discussed later. The coliform levels in receiving waters also cannot be estimated using the references sited as a basis. Coliforms usually die off quickly when they are introduced into receiving waters.

Precipitation has been discussed earlier in this paper. The quality of rainfall varies from one location to the next. Kluesener and

Lee[34] found that rainfall contributed anywhere between 20 to 90 percent of the nitrate-N loading in urban residential runoff.

Thus, most of the ammonia-N and about one-third of the nitrate-N in residential runoff seem to originate from rainfall itself.[35] Precipitation will contribute varying amounts of "pollutants," which occur in it through natural processes. These amounts depend upon many factors and may vary greatly.

The nutrients provided by the leaching of vegetation from precipitation are small in quantity. Not much data is available on the subject, but it can be said that some nutrients are added to the storm runoff. The amounts added, in this manner, are approximately the same amounts which would have been added by the rainfall, which does not reach the ground surface. Leaching of the vegetation, therefore, does not increase or decrease the total amount of nutrients that reach the ground surface. The nutrient increase from leaching is the same as what is contained in the rainfall which is not allowed to reach the surface of the ground.

There are many factors affecting the amounts of pollutants introduced into storm water runoff. Most notably is the type of land use with which the runoff is associated. There are three basic land uses associated with urban areas. These are commercial, residential and industrial. These land uses are demonstrated in Figure 1. A wide variation in their characteristics can be noted. Industrial areas have higher loadings of street surface contaminants than others. This probably is because they are not swept as frequently and production of pollutants is high. On the other hand, commercial areas have the lowest

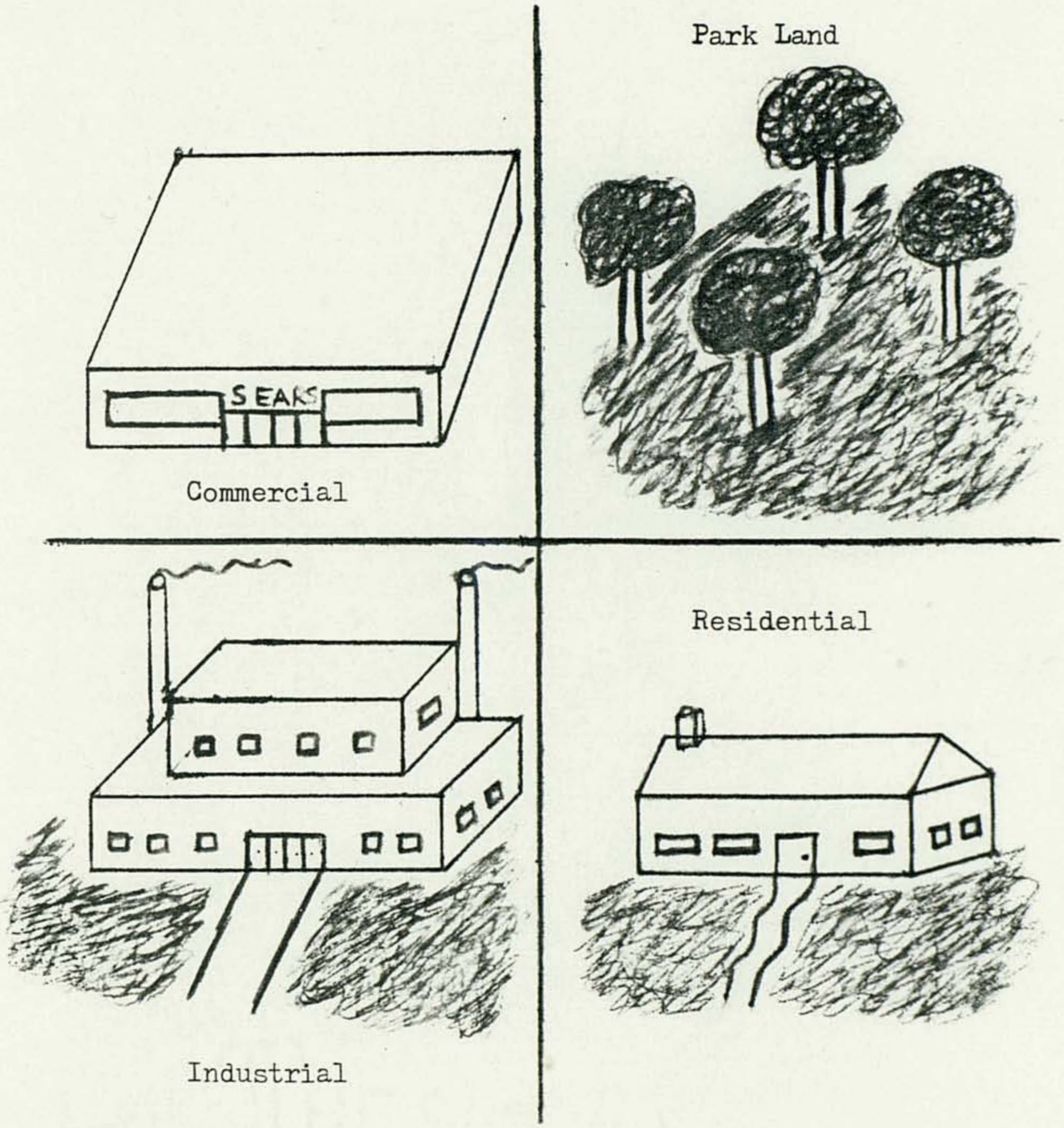


Fig. 1. Land Use Classifications

loadings of street contaminants. They are swept much more than other areas. The practice of street sweeping, mainly its frequency and how good a job is done, greatly affects the amounts of contaminants on the streets. A basic problem with street sweeping, is the presence of parked cars. With vehicles parked along the streets, the gutters and a large portion of the street cannot be swept. This adds to the quantities of pollutants on the street surfaces. Street cleaning efforts required to achieve a greater removal effectiveness of the dust and dirt fraction of street surface contaminants is several times the effort normally expended in sweeping operations.[36]

The volume and type of traffic moving over the streets has an effect on the contaminants present also. Much more contaminants will be present on crowded city streets than rural roads. Heavy metal concentrations are especially high as can be seen in Table 2. The material a street is constructed of is also important. Different materials impart varied surface characteristics to the street. Sartor, Boyd, and Agardy[37] found that asphalt surfaces had loadings up to 80 percent more than concrete surfaces. The physical condition of the street surfaces is also important. Poor surfaces generally have higher contaminant loadings than good street surfaces. The time of year also has a role in the amounts of street surface contaminants present. In northern areas road salt and other chemicals may be utilized as needed. These also enter into the storm water runoff. Other factors associated with quantities present are the amounts and previous occurrences of past rainfall, public works department practices and quantities of air pollution fallout.

An interesting fact noted by Sartor and Boyd[38] is that a large portion of the pollutants present were concentrated in the fine solids of the street surface contaminants. Further, these fines account for only a minor portion of the total loading on street surfaces.[39] In other words, the fine solids were a minor portion of the total volume of solids present. This presents a major problem to the public works director. If he is to decrease the quantities of pollutants present on the street surfaces, he must remove these fine solids. However, in order to do that, a much greater effort must be expended. Conventional street cleaning techniques remove only a small percentage of these fine materials. New techniques will have to be developed or existing techniques improved upon. Catch basins were found to retain only large debris and coarse grained solids. Fine solids and most organic matter is allowed to pass through by the catch basins. Improved designs to entrap these particles may be one solution to the pollution of the receiving waters. The most promising solution appears to be renovating street cleaning techniques to remove pollutant materials.

Florida Storm Water Quality

Florida receives an average annual rainfall of approximately 52 inches per year. This rainfall produces storm water runoff which is affected by all of the factors discussed previously. The quantities of pollutants contained in this runoff have been studied. One study is being done by Florida Technological University[40] for the Florida Department of Pollution Control. The loading rates of different pollutants determined by the University for various land uses are given in Table 6.

TABLE 6

STORM FLOW* LOADINGS OF NITROGEN, PHOSPHORUS, BOD_5, AND SUSPENDED SOLIDS IN FLORIDA[41]

Land Use	BOD_5**	Phosphorus**	Nitrogen**	Suspended Solids**
Urban	75.0	2.0	8.5	1700.0
Citrus	None	0.18	22.4	None
Pasture	11.0	0.30	5.3	840.0
Cultivated	18.0	1.05	26.0	4213.0
Woodland	5.0	0.10	3.1	98.0
Atmospheric	None	0.10	2.0	100.0

* For 52 inches of rainfall

** kg/ha/year

Urban land uses were found to have the highest loadings of BOD and phosphorus. Urban land use had the third highest loadings of nitrogen and the second highest loadings of suspended solids. Cultivated land was found to have the highest suspended solids loadings, almost three times as great as the urban land. Cultivated land also had the highest nitrogen loadings, with citrus land having the second highest.

It should be noted that the BOD_5 loading of urban land was more than four times as great as that of cultivated land. The phosphorus loadings of urban land also were about twice as great as that of the cultivated land. Of course, it can be seen that the more involved man is with a land area the higher the pollutant loadings. Woodlands have the lowest loadings for all of the pollutants except nitrogen. The only area having a lower loading for nitrogen is the atmospheric.

Another study that was published in December of 1974 was by A. G. Lamonds.[42] This study was done over a two year period from 1971 through 1973. The storm water runoff entering Lake Dicie in Eustis, Florida, is primarily from the Southeastern urban area of Eustis. This is a typical urban area consisting of streets, roofs, parking lots and other impervious areas. The quality of the storm water runoff was found to vary depending upon the rainfall intensity, antecedent dry period, and amount of rainfall.

The lake quality and runoff quality were both sampled throughout the study. One of the most obvious differences between the quality of runoff and the quality of the lake was in the phosphorus concentration.[43] The average concentration of phosphorus in the runoff was over eight times as much as the phosphorus concentration in the Lake. Table 7 illustrates

TABLE 7

CHEMICAL AND PHYSICAL CHARACTERISTICS* OF LAKE DICIE AND STORMWATER RUNOFF[44]

Constituent	Lake Dicie**	Urban Stormwater Runoff**
Arsenic	5.0	8.0
Chromium, dissolved	0	0
Copper, dissolved	10.0	20.0
Iron, dissolved	10.0	40.0
Lead, dissolved	1.0	56.0
Total Nitrogen as N	1.47	2.32
Total Phosphorus as P	0.03	0.26
Total Organic Carbon	10.0	32.0
Oil and Grease	10.0	16.0
Biochemical Oxygen Demand	3.6	7.2
Suspended Solids	-----	99.0
Turbidity (JTU)	10.0	24.0
Color	10.0	65.0

* Concentrations in mg/l

** Averages of all samples taken from April 1971 through April 1973

some of the chemical and physical parameters of the Lake and the storm water runoff. A significant difference can be noticed between the Lake and runoff qualities for different constituents. The iron, lead, total phosphorus and total organic carbon concentrations of the runoff are several times that of the Lake.

The Lake bottom sediments adsorb much of the pollutants in the runoff. The question remains, however, what quantities are adsorbed by the sediments and what quantities are utilized by various aquatic plants? More than likely, varying amounts of the nutrients are utilized by the aquatic environment, enhancing eutrophication and aging of the Lake. The quantities of pollutants entering any lake depends on the quantity of storm water runoff available to that lake. This runoff can vary greatly in quantity and there are several methods of estimating this quantity.

From the data given in Table 7, loading rates in terms of kg/ha/yr can be calculated for the various constituents. Table 8 illustrates these storm flow loadings for the urban storm water runoff. These loading rates when compared with those in Table 6 are reasonable. BOD_5 and nitrogen are higher at Eustis and suspended solids are lower.

TABLE 8

STORM FLOW LOADINGS OF NITROGEN, PHOSPHORUS, BOD_5 AND SUSPENDED SOLIDS FOR URBAN RUNOFF AT EUSTIS, FLORIDA

Constituent	Loading* (kg/ha/yr)
BOD_5	94.70
Phosphorus	3.43
Nitrogen	30.7
Suspended Solids	1300.0

* Based on an average rainfall of 52 inches/ year

CHAPTER III

METHODS FOR DETERMINING STORMWATER RUNOFF

Background

Urban storm drainage system design has been based, in the past, upon empirical methods, experience, and engineering intuition. However, with costs of labor and materials increasing steadily, it has become a necessity to utilize better methods of design. Public concern and the need for stormwater pollution control have stimulated much research interest in the area of urban hydrology in recent years.[45] Because of this, many new methods have been developed in recent years for the design engineer.

Since the turn of the century, methods for determining runoff have been many in number. Recently, new techniques utilizing mathematical model simulations have been appearing. Simulation models to describe and predict the quantity and quality characteristics of stormwater runoff more adequately. These methods formulate the hydraulic and quality relationships better and thus save costs associated with stormwater systems design and protect the environment. Old methods usually foretold only very extreme runoff events and therefore cause systems to be overdesigned. The question is which method or model should the engineer depend or base his designs upon? That, of course, will vary depending upon the economic and social factors with which he is working.

Rational Method

Empirical formulas are the oldest and still most universally used methods for urban runoff calculations. The "rational" method is still the most widely used technique. It was developed initially in the United States in 1889 by Kuichling. While some improvement in the method has been accomplished the procedure is still basically the same as that presented by Kuichling.[46] The rational method formula is:

$$Q = C\,i\,A \tag{1}$$

where: Q = peak runoff rate in cubic feet per second (cfs)

C = a dimensionless runoff coefficient

i = average rainfall intensity in inches per hour (in/hr)

A = drainage area (acres)

The runoff coefficient, "C," is difficult to obtain as it accumulates several characteristics of the drainage area. These include infiltration rates, ground cover and slope, surface retention and the length of the antecedent dry period. Various values of the runoff coefficient, "C," can be seen in Table 9.

The fact that the rational method is based on three assumptions should be mentioned. These assumptions consist of:[48]

1. The peak rate of runoff at any point is a direct function of the average rainfall intensity during the time of concentration to that point.

2. The frequency of the peak discharge is the same as the frequency of the average rainfall intensity.

3. The time of concentration is the time required for the runoff to become established and flow from the most remote part of the drainage area to the point under design.

The rational method is usually limited to small urban drainage basins, less than five square miles. If applied to large areas, surface retention

TABLE 9

TYPICAL VALUES OF THE RUNOFF COEFFICIENT "C"[47]

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single Family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemetries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

Character of Surface	Runoff Coefficients
Pavement	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

and groundwater seepage cause the rational method to overestimate the quantity of runoff. With experience and a knowledge of the area, the rational method can provide a useful design technique for the engineer.

Unit Hydrograph

The second method is known as the unit hydrograph. The basic theory of the unit hydrograph appears to have been suggested first by Folse (1929).[49] In 1932, Sherman[50] provided the basic theory and concept of the unit hydrograph method. The method which Sherman conceived has become one of the most overwhelmingly received techniques in use today for the determination of the runoff hydrograph from the effective rainfall. The concepts of the unit hydrograph are as follows for a given drainage area (Morgan and Johnson):[51]

1. The time base of surface-runoff hydrographs resulting from similar storms of equal duration are the same regardless of the intensity of rainfall.
2. The ordinates of the surface-runoff hydrographs from similar storms of equal duration are proportional to the volume of surface runoff.
3. The time distribution of surface runoff from a particular storm period is independent of that produced by any other storm period.

The observation should be made that the unit hydrograph assumes that the runoff hydrograph from one inch of effective rainfall, which is produced uniformly over the drainage area in a unit time period is a characteristic of the drainage area. Horner and Flynt[52] in 1936 were first to report the application of the unit hydrograph method to urban sewer systems. The Corps of Engineers used the theory extensively in the 1940's and early 1950's. To develop a unit hydrograph for an urban basin, observations must be made of the actual rainfall and runoff. The

unit hydrograph technique is useful in predicting flood flows and flood crests during storms. The unit hydrograph is also not limited to the finding of the peak flow alone but yields the complete hydrograph resulting from a storm. Very useful results are obtained from short records on rainfall data.

Eagleson,[53] in 1962, applied the unit hydrograph concept to several small urban basins in Louisville, Kentucky. The data for his studies were originally obtained by the Corps of Engineers in the late 1940's. His analysis correlated the peak discharge and lag time of the unit hydrograph to the drainage area characteristics. Snyder[54] developed a synthetic unit hydrograph method which has been utilized extensively. A synthetic unit hydrograph is not based upon data taken at the location being studied. Instead many unit hydrographs for similar areas would be used to develop a synthetic unit hydrograph.

Of the two methods discussed so far, some drawbacks to their use has been found. These faults were proposed by Watkins[55] (1962) after evaluating 286 storms on 12 drainage areas in England. His findings were that "the rational method is unsatisfactory for all but the smallest areas." For sewer systems with pipes larger than 24 inches in diameter, intolerable errors arose. As for the unit hydrograph method, his statements were to the effect that it was "unsuitable in the design of urban sewer systems owing to the difficulty of determining the shapes of the unit hydrographs."

RRL Method

Watkins[56] developed, in 1962, what is known as the Road Research Laboratory method. This method works with only the impervious areas of

a drainage basin that are connected to the storm sewer system. The rest of the drainage basin area is ignored. The RRL method is based upon the unit hydrograph method but assumes that any pervious areas do not produce runoff. But impervious areas are just that, 100% impervious; all stormwater is runoff. The RRL method achieves a runoff hydrograph which is named the virtually runoff hydrograph. Good results have been reached using the RRL method (Terstriep and Stall).[57] The RRL method can be applied to drainage basins before urbanization takes place. An example of a unit hydrograph can be found in Figure 2. A comparison of a hydrograph of an urban area with a rural area is in Figure 3.

Other Methods

Several other methods for determining stormwater runoff have been developed. A few are based on extensive hydrologic studies of specific areas. The Chicago Hydrograph Method, developed in 1960, is one of these methods. A detailed explanation of the use of this method is given by A. L. Tholin and C. J. Keifer.[58] A synopsis of the method and a comparison of actual and computed runoffs is given by S. W. Jens and M. B. McPherson.[59] Wide ranges of similar land uses were studied and their various characteristics tabulated. A unit area of ten acres was typically used for the hydrograph analysis. A rainfall hyetograph is selected and applied to a test area. Based upon the characteristics of the area a sewer flow hydrograph is developed and routed through the test area sewer system. A weak point of this method is the assumption of the routing time as being eight minutes. The work done, using a basin in Chicago, can be applied to other urban areas which have the same similar factors such as land use, rainfall, topography and soil conditions.

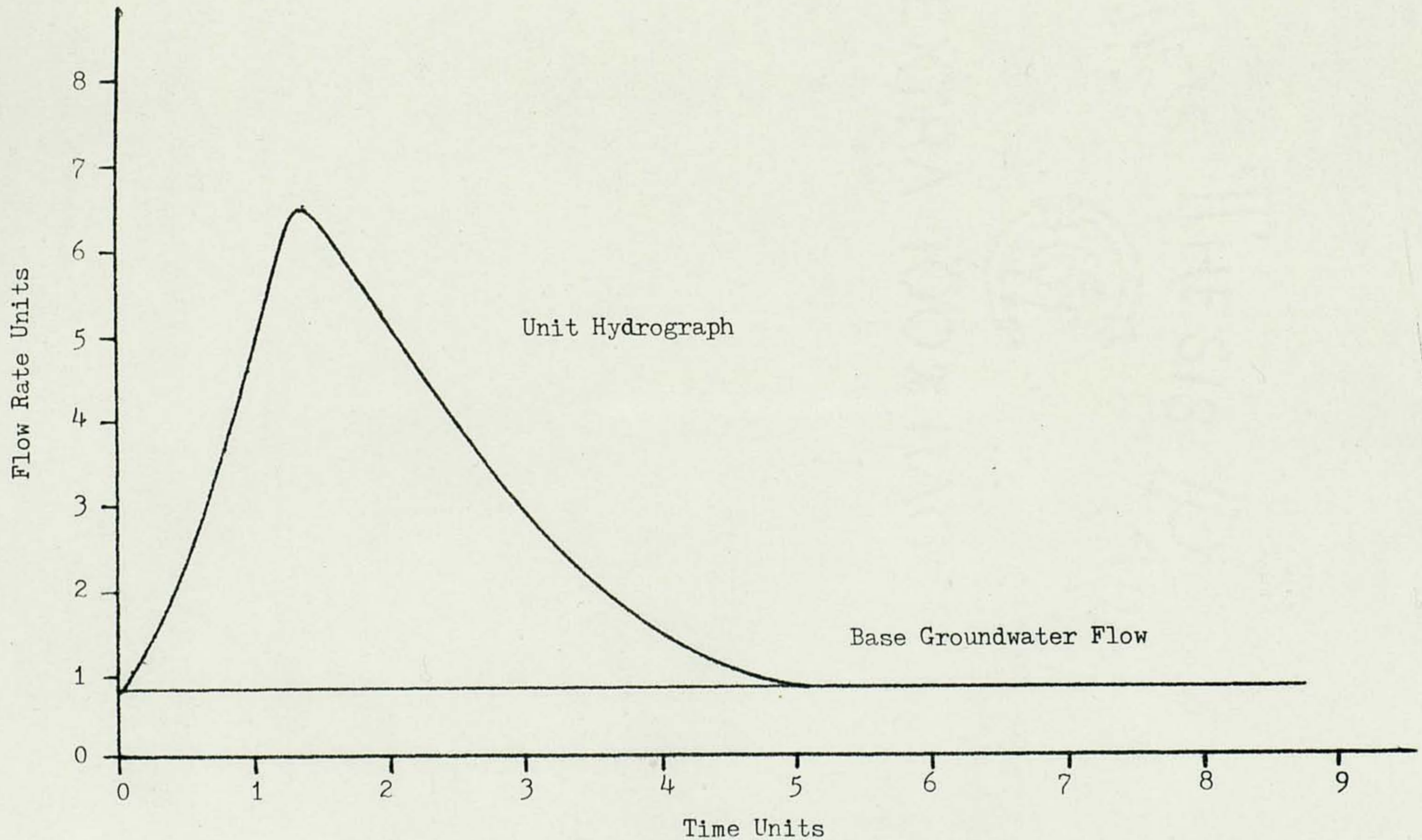


Fig. 2. Unit Hydrograph

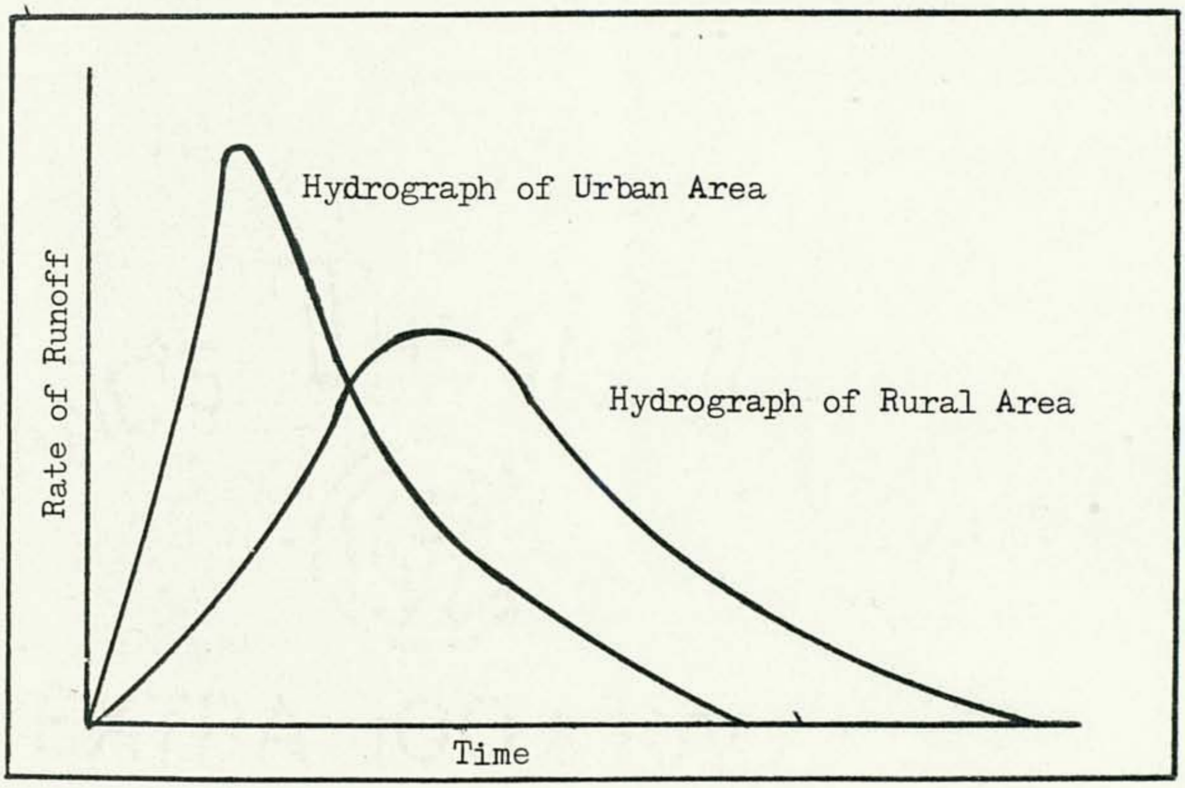


Fig. 3. Comparison of Urban and Rural Runoff Hydrographs

Another method developed in the late 1950's and early 1960's is the Inlet method. This method was originally studied at the John Hopkins University and the New Mexico State University. The Inlet method was developed further by Kaltenbach[60] in 1963. The first step is the determination of the peak runoff rate to each inlet in the storm sewer system. This is accomplished through the use of the rational formula. These inlet hydrographs are triangular in shape and assumed to have a height of the peak runoff rate and a base of twice the time from the beginning of heavy rainfall to the end of the maximum rainfall intensity. A subarea peak flow, which consists of the sum of the inlet hydrographs in that subarea, is modeled as flowing down its respective sewer pipe. The total number of subarea peak flows are then summed to form the total peak flow at the point in question. This is the total hydrograph for the design point. The method yields better results when applied in impervious areas.[61] The ranges of land size and use to which this method can be applied have not yet been determined completely. But, the extent of its use has been suggested limited to inlet subarea of three acres or less and total drainage areas of one square mile or less. Limitations of imperviousness over a range of 30% to 60% are also imposed.

The city of Los Angeles, California, has been using a hydrograph method for the computation of runoff for approximately thirty years. This method, called the Los Angeles Hydrograph Method, was initially developed by W. I. Hicks[62] in 1944. The method is based upon experimental work done in the area on soil infiltration rates and overland flow rates over various soils in the area with different slopes. The method is used by some areas adjacent to Los Angeles, but is hardly applicable

to other sections of the United States, unless the soil types and other factors, which the method is based upon, correspond exactly. The techniques used to develop the Los Angeles method could be used by other regions to develop their own methodologies; but with the methods available today, this type of large scale undertaking would be unwarranted.

Several other variations of the methods described have been done. Notably, one of these is Izzard's Method, which simulates overland flow. This method was developed by C. F. Izzard[63] in 1946. Results of this method do agree with those obtained by other methods. Work has been done on other areas, including Baltimore, St. Louis, Houston, and Austin. Ven Te Chow[64] did extensive work in Illinois in the early 1960's. He formulated a method for determining the runoff hydrograph from the unit hydrograph. The literature contains other methods which will not be reviewed in this paper.

Mathematical Models for Urban Stormwater Simulation

Until recent years, the methods discussed earlier were the tools utilized in the design and analysis of urban stormwater systems. With the advent of the computer age, new methods were brought out for these purposes. The methods were mathematical simulations using computers for the design and analysis of urban stormwater systems. Several of the more prominent mathematical models will be discussed. The discussion will include the Environmental Protection Agency Storm Water Management Model, a model used for analysis in this report.

There are mathematical models available which will handle any or all factors associated with catchment runoff, water quality, flow routing,

realtime control and design. These models differ greatly in many aspects such as the size of the system which can be handled, input data required, and computer output. The use of a particular model will depend heavily upon the size and characteristics of the system to be simulated. Care must be taken also in the usage of particular models. Some of the models have been tested and verified with actual and experimental data to a great extent. Others have been developed but not yet verified, and some have not been applied to existing systems at this time. Accordingly, the ways and means of testing and verification of the models has also varied widely. This is because no standards or criteria for evaluating the models are currently in existence. One of the earliest models developed was the British Road Research Laboratory Model.

Road Research Laboratory Model

The computer model termed the Road Research Laboratory Model was formulated from the Road Research Laboratory method. The model was developed in 1963 after initial work was performed by Watkins[65] in 1962. The RRL model computes surface runoff from individual storms. It does not feature any water quality output, realtime control or design parameters. In addition, pervious areas are ignored and only runoff from impervious areas is computed by the RRL model. Manning's equation is used extensively in flow computations.

Some advantages of the model is that a minimum amount of input data is required and it has been tested extensively. For small urban basins, with a storm frequency of less than 20 years, good results can be obtained. It provides a reasonably accurate model for the computation of surface runoff from the impervious areas of an urban drainage

system. The program is written in Fortran IV and can be run on an IBM 360. The model was operational in the early 1960's. An excellent evaluation of the RRL method applied to the various drainage basins in the U.S. is provided by Terstriep and Stall.[66]

Stanford Watershed Model

The Stanford watershed model was one of the first extensive mathematical models of drainage basins. The model was developed by Crawford and Linsley[67] in 1966 at Stanford University. It was the first model to be based on physical concepts such as infiltration rates and soil conditions. The original model has been updated and is now known as the Hydrocomp Simulation Program. Another program dealing with water quality has since been developed and is now part of the hydrologic program. The quality program simulates seventeen water quality parameters including their interrelationships in receiving waters. The model can be used to simulate urban and rural areas, i.e., sewered and nonsewered. When an area of large size is modeled assumptions have to be made about specific local conditions. One assumption made concerning infiltration is that it is a linear relationship. The Stanford Watershed Model recognizes this by assuming a linear variation of infiltration rate as precipitation continues.[68]

Input data required for the model are quite extensive. Precipitation records and other meteorological data are needed as well as dry weather flow and quality data. Physical data on the sewer system and its elements are also required as are several empirical coefficients. The model does not handle any wastewater treatment, cost calculations, design or realtime control. The model was developed originally for rural areas

and then modified to handle urban basins also. One advantage is the ability to simulate water flow and quality continuously in rather complex networks. Program output consists of soil moisture data, water velocities, discharges and quality for the various channel and storage elements, and also volumes of storage.[69]

Testing of the hydrologic portion of the model has been fairly extensive. This portion of the model has been applied to urban and non-urban drainage areas. The water quality section of the model has not been thoroughly tested yet. The program is written in PL/1 language and can be run on either of IBM's 360 or 370 computers.

University of Cincinnati Urban Runoff Model

This mathematical model was developed in 1970 by Papadakis and Preul.[70] The UCUR model consists of:

1. infiltration
2. surface retention
3. overland flow
4. gutter flow
5. routing through the sewer system

With these five submodels, a complete system can be simulated. Input data required are mainly physical describing the drainage basin and the sewer system. Only one rainfall hyetograph can be input at once for the drainage basin. The model has been simplified to the extent that its accuracy and applications seem to be affected.[71] Further testing may tend to prove this one way or another. Testing done by the model developers has yielded good results. These tests were done on a small

and large drainage area. Others utilizing the model have reported large variations between actual runoff and computed runoff.

Some of the assumptions which have been incorporated into the model are:

1. rainfall is uniform over the entire area
2. dry weather flows are neglected
3. infiltration is calculated from rainfall instead of overland flow depth
4. drainage area moisture conditions are not used in the calculation of infiltration

The model tends to produce higher peak flows than those computed by other models.[72] Water quality, realtime control and design are not features of the models. The program is written in Fortran IV for an IBM 360. The output consists mainly of runoff discharges at points in the system selected by the user.

Massachusetts Institute of Technology Urban Watershed Model

This model is similar to the University of Cincinnati model in that the runoff of several catchments and a converging sewer and open channel network is simulated. The MIT model computes the least-cost combination of sewers, storage and treatment facilities required to eliminate flooding and surcharging. This could be considered an optimal solution based on a least-cost decision; linear programming is used to determine this solution. Another exceptional feature is the capability of the user to choose between four methods for the calculation of run-off. Most models use only one method. These four methods are:

1. Horton's equation

2. Holtan's equation

3. a U.S. Soil Conservation Service method

4. a runoff coefficient method

Data input consists of the usual rainfall hyetograph and physical data describing the various catchments. The use of the model could prove to be difficult. Model documentation is scattered through several project reports and a draft user's manual was not released.[73] Resource Analysis, Inc., of Cambridge, Massachusetts, has utilized the model and could possibly provide further information. The program was written for an IBM 360/67 computer.

Computer output consists of rainfall data, overland, catchment and channel discharge and depth. When the design option is used, the volume and duration of flooding for each sewer, along with the associated costs and sizes for the sewers, storage basins and treatment facilities is outputed. Testing was done on a small 23 acre basin in Maryland and good results were obtained. Computed and actual runoff valves compared reasonably well. Additional testing on the design/cost option is being done. One important asset of the model is that it is based on physical principles, assumptions and approximations were held to a minimum.

<u>University of Illinois Storm Sewer System Simulation Model</u>

This mathematical simulation model deals with the computation of nonsteady flows based upon the dynamic wave equations. This model differs from the previous models discussed in that inflow hydrographs to the sewers are needed for data input. The model does not consider

water quality, costs or realtime control. Since inflow hydrographs are used, dry weather flows from land use or runoff from rainfall data are computed. The model is still being developed with a submodel dealing with urban hydrology now underway.

One disadvantage of the model is the computer time required to run it. Continuous simulation is available but the cost is quite high. The simulation is usually restricted to single design events. The model has not yet been tested or applied to real data or systems. Initial testing done with experimental data produced very accurate results. If and when the complete model is available, it will be one of the more accurate models available.

Computer output consists of inflow hydrograph plots, depth and discharge plots, and sewer diameters from the design option. The program is written in PL/1, for the use of an IBM 360/75 computer system.

SOGREAH

A model developed by a French consulting firm known as SOGREAH (Societe Grenobloise d'Etudes et d'Application Hydraulics) is based on a river basin model developed earlier. This model simulates the runoff of combined sewage systems of several catchments and a sewer and open channel system. The model is quite extensive, modeling the hydraulics of most elements found in closed conduit and open channel networks. Pollution parameters were recently added but realtime control, design and cost factors are not included. The creation of runoff from precipitation is limited to single precipitation events. The flow routing submodel can be used for continuous simulation. Dynamic wave equations are utilized for flow routing in the various systems.

Input data consists of physical catchment data and dry weather flow. The model does not include a provision to input rainfall data but computes design rainfall excess hyetographs of specified frequency for each catchment using formulas developed by Caquot.[74] One serious limitation of applying the model to other areas is that these formulas require ten empirical coefficients. The formulas are therefore based on French hydrologic data. Another problem is the amount of computer time required for the flow routing computations. A different method of computing runoff hydrographs would also be more suitable.

As was stated before, the model routing subroutine is based on a river basin model developed previously. The river basin model was verified satisfactorily earlier and the sewer model was therefore not verified with urban data. The main program is made up of five subprograms written in Fortran IV. An IBM 360/65 or equivalent system is necessary to run the model. Program output consists of tables and plots of water depth, velocity and discharge at each specified point. The program utilizes metric units for input and output.

S.T.O.R.M.

The Corps of Engineers has developed a Storage, Treatment, and Overflow Model, commonly known as STORM. The models main purpose was to evaluate the stormwater storage and treatment required to meet specific limits. The model does not utilize a sewer or channel network in its program. All calculations are based upon the entire drainage area. The hourly stormwater runoff and its quality can be simulated for several years for one drainage basin.

The runoff is computed from hourly precipitation data. Other

input data consists of the physical catchment data description and runoff coefficients. The coefficients used in quality calculations are contained internally in the program. Five different water quality parameters are calculated - biochemical oxygen demand, nitrogen, phosphorus, suspended and settlable solids. These quality parameters depend upon several factors, such as street sweeping methods and the number of days between storms. The storage submodel does not modify the water quality during storage.

Verification of the model is still under way and application to several drainage basins has been accomplished. The accuracy of the model has not been determined. The model does not consider cost, dry-weather flow or quality of the dry-weather flow. Therefore, its application is limited to stormwater drainage systems. Output of the program consists of the overflow, storage and treatment flow volumes, and the quality data for each storm. Input precipitation data is also printed. The program is written in Fortran IV for a UNIVAC 1108 or equivalent system.

Water Resources Engineers, Inc. Storm Water Management Model

Water Resources Engineers, Inc., has developed a modified version of the EPA's Storm Water Management Model. This model computes dry-weather and stormwater quality for twenty-three parameters, including oil and grease. The model is capable of simulating the runoff from several catchments and the disposal network. The dynamic wave equations form the basis for the flow routing.

Input data consists of precipitation data, physical catchment data and runoff coefficients. Dry-weather flow rates and quality must

also be input. The water quality is routed through the sewer system by pure advection. The model is very extensive as it considers sewer system flow routing and quality routing. One of its disadvantages is the amount of computer time required for running. Since the model is very comprehensive, it requires long computer run times.

The model was tested on small and large drainage areas. Good results were obtained with actual and computed runoff values correlating fairly well. Water quality computations were in the correct range, but further verification of quality is needed. Program output includes plots of water depth, discharge and quality for the subcatchments, system outfall, and specified sewer system points. The program has three main subprograms and can be used on an IBM 360/65. The model does not include realtime control, design and cost computations.[75] It is also limited to the simulation of individual runoff events.

Environmental Protection Agency Stormwater Management Model

The Environmental Protection Agency Stormwater Management Model is one of the most comprehensive models to date. The model was developed by Metcalf and Eddy, Inc., the University of Florida and Water Resources Engineers, Inc. Originally developed in 1971, the model has been updated since then. Four volumes were published on the model. These were:

1. Volume 1 - Final Report[76]
2. Volume 2 - Verification and Testing[77]
3. Volume 3 - User's Manual[78]
4. Volume 4 - Program Listing[79]

In 1975, an updated Volume 3 was made available by EPA. This updated

version contains several new and different concepts. One new aspect is the addition of another program block. This is the combine block which can be used to combine and/or collate the output of the other blocks. The model now consists of six main program blocks - executive, combine, runoff, transport, storage, and receiving. The capability of running any or all of these blocks exists.

Demonstration and verification runs on selected catchments, varying in size from 180 to 5,400 acres in four U.S. cities (approximately 20 storm events, total) were used to test and verify the model.[80] The model has been tested and good results were obtained for the runoff and flow computations. The water quality calculations have not been proven accurate at this time. Input data required to run the complete program is quite extensive.

The input data consists of physical description data of the subcatchments, sewer system and land uses. Precipitation data in the form of rainfall hyetographs is also required. The collection and application of the various input data required for the different program blocks can be a major undertaking. Questions can arise as to the use and meaning of some data required.

Treatment processes, internal storage and costs are some of the features of the model. Water quality is output for several parameters; notably, biochemical oxygen demand, suspended solids, total coliforms, and dissolved oxygen. Output of the program also includes tables and line printer plots of rainfall intensities, discharges and water quality for each of the subcatchments and sewer system elements. Cost and treatment performance is output, too. The program is written in Fortran IV

and can be run on an IBM 360 and 370. Due to the size of the program, overlay procedures are required. A computer programmer would be helpful in the initial set up of the program. The user should be cautioned that the model is complex and quite time consuming in input data preparation. The SWMM model is utilized in the remaining chapters of this report.

CHAPTER IV

DESCRIPTION OF LAKE EOLA AND DRAINAGE BASIN

Lake Eola

Lake Eola is a small land-locked lake, located in the City of Orlando. The Lake is in the heart of the city and receives stormwater runoff from the commercial areas of the central business district and from residential areas. The location of the Lake is shown in Figure 4. Orange Avenue and Magnolia Avenue are the main south and north routes respectively through downtown Orlando. These streets handle high volumes of traffic daily.

The Lake has a surface area of approximately 28.75 acres and a volume of 100 million gallons. A view of the Lake from the south shore and a view from the commercial area are shown in Figures 5 and 6 respectively. Figure 5 is a view of the Lake from the park land area on the south shore. The fountain in the Lake can be seen quite clearly. Figure 6 is a view of the Lake from the commercial area.

The primary sources of pollution are the several storm sewers which drain into Lake Eola. At present, there are eleven active street drains into the Lake. The locations of these and the fountain are shown in Figure 4. The Lake does not have any surface drainage to other lakes. The level of the Lake is controlled by two drainage wells. Their locations are also shown in Figure 4. These wells lead into the artesian aquifer and can be used to raise or lower the Lake level. Since the

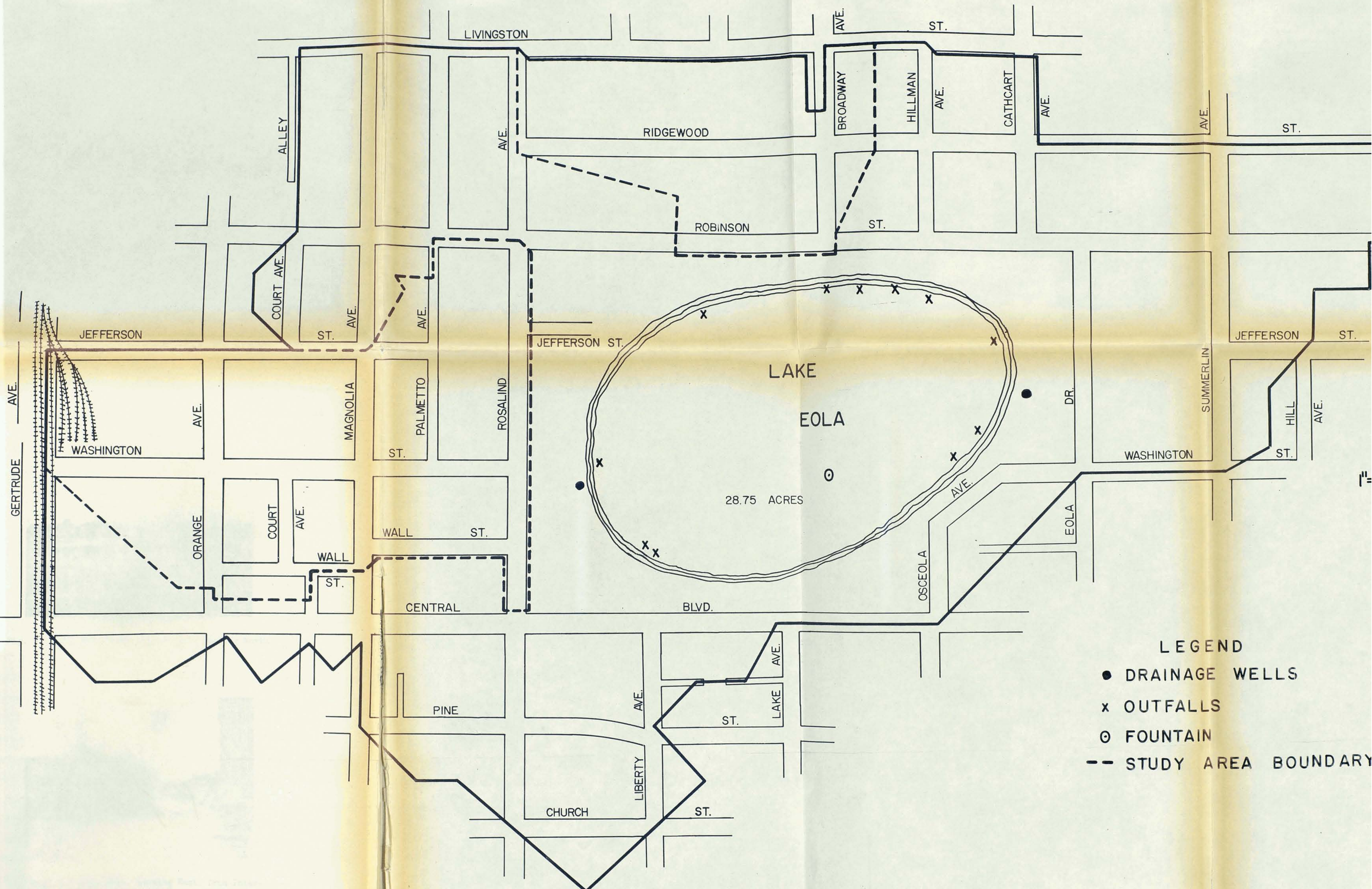


FIGURE 4

LAKE EOLA DRAINAGE BASIN

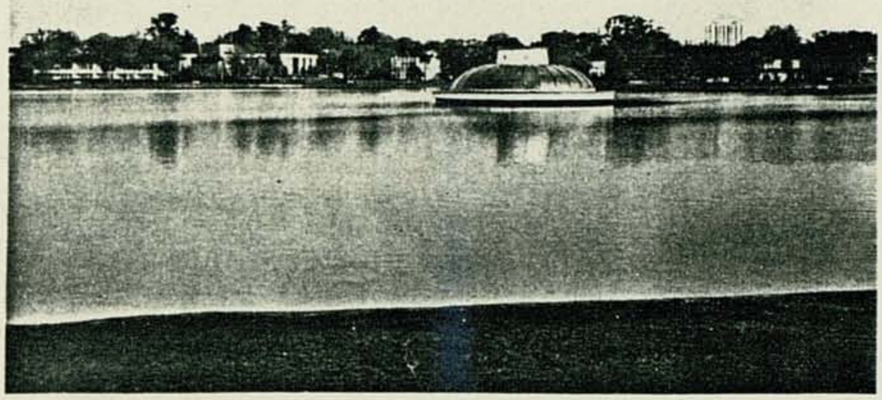


Fig. 5. Lake Eola, from South Shore, Looking North

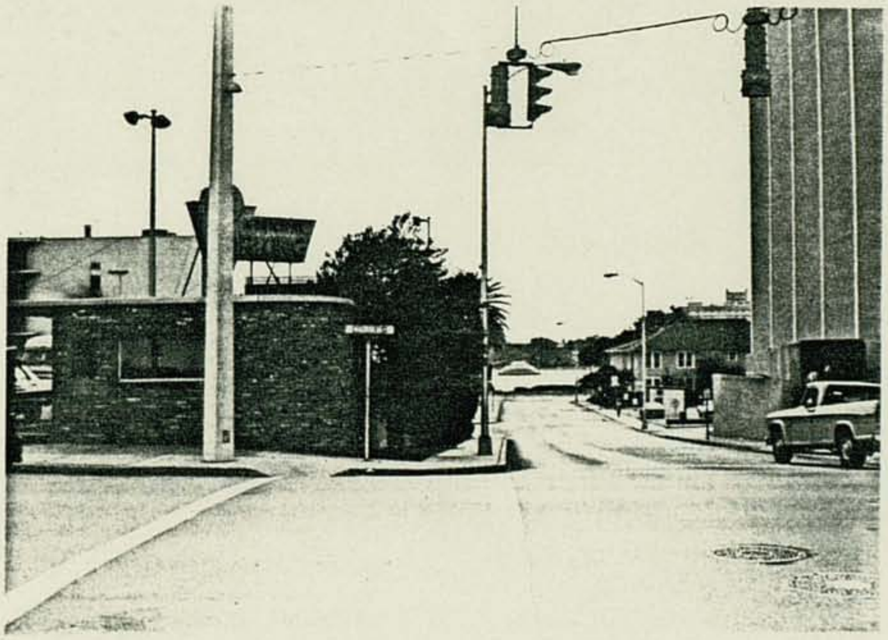


Fig. 6. Lake Eola, Looking East, from Intersection of Magnolia Avenue and Washington Street

Lake level is higher than the piezometric surface, water will flow naturally down the wells. Pumping of these wells will raise the level of the lake. The level is usually between 87.0 and 88.5 feet above sea level. The piezometric surface is at approximately 57.0 feet above sea level. The ability of a well to take or to yield water is dependent on the permeability of the aquifer into which it is drilled and the difference between the level of the lake and the piezometric surface.[81] The water depth is shown in Figure 7. This figure also illustrates the fountain which is in the Lake. The Lake has a maximum water depth of about 22 feet, gradually decreasing to the shore.

The level of a lake is influenced by several factors. These consist of rainfall, evaporation, surface inflow and outflow, and underground inflow and outflow. Each of these factors depend on their individual characteristics associated with them. Two of the primary factors are rainfall and evaporation. Rainfall can vary greatly from year to year. This variation is the main reason lake levels fluctuate. The average annual rainfall of Orlando is approximately 52 inches. This annual rainfall has been as low as 33.75 inches and as high as 68.74 inches. Over half of this rainfall occurs during the rainy season, which is from June through September. Unfortunately, evaporation does not vary with rainfall.

The average annual evaportation from lakes in Orange County was computed to be 51.07 inches based on the normal pan evaporation reported by the U.S. Weather Bureau at Orlando.[82] This leaves the lakes with an average of .93 inches of rainfall per year that is not evaporated. Dependent upon the other factors affecting lake levels, the maintaining

FIGURE 7
DEPTH MAP OF LAKE EOLA

of a "normal" lake level can be difficult.

Lake Eola also experiences these fluctuations in lake levels. However, because of its location, the level of the Lake must be maintained. Excessive rainfall leads to high Lake levels which are lowered through the use of the drainage wells. The purpose of lowering being to prevent flooding and property damage in the surrounding area. During drought or "dry" periods, the Lake level must be kept at the minimum level acceptable. This is primarily for aesthetic reasons. Maintenance of the Lake at these levels necessitates pumping out of the wells into the Lake. This puts an increased demand upon the aquifer, thereby lowering the piezometric surface. Of course, during "wet" seasons this peizometric surface is raised from the drainage of the Lake's excess water.

A restoration of Lake Eola was undertaken in 1972. This project was undertaken because of the poor water quality and bottom appearance. The Lake was undergoing eutrophication with the aquatic environment deteroirating rapidly. This restoration project has been reported by Boyter[83] and others. The Lake was lowered exposing about 40 percent of the bottom. The bottom was cleaned, storm drains extended into the lake and sand placed on the bottom. The Lake was then refilled from the drainage wells. Now, three years later, the quality of Lake Eola is again being questioned.

Lake Eola Drainage Basin

The Lake Eola drainage basin consists of approximately 350 acres. Of this area, a large portion is drained by drainage wells located in the northern section of the basin. These wells primarily drain the areas

north of Livingston Street. The remainder of the basin drains directly into Lake Eola. This area includes Lake Eola and is approximately 169 acres. This acreage consists of:

Lake Eola	28.75 acres
Impervious	85.00 acres
Pervious	56.00 acres

An impervious area being defined as any area such as buildings, parking lots, streets and driveways which do not allow natural infiltration to occur. Pervious areas are characterized as those areas which permit the natural infiltration to occur. These areas would include lawns and parkland. Building roof areas are also considered pervious if drainage onto a pervious area, such as a lawn, is allowed to occur.

From this 169 acres a study area of approximately 28.0 acres was chosen. This study area had a commercial land use and is located in the western portion of the drainage basin. There are three major land uses in the Lake Eola drainage basin - commercial, residential, and park land. The commercial land use area being to the West of Lake Eola and the residential areas to the North and East. The park land is the area surrounding the Lake. The following figures describe the study area and the other land uses of the Basin. Park land consists of approximately 14 acres surrounding the Lake. Figures 8 and 9 show some of the park areas. Figure 8 is a view of the Southeast corner of the park where a children's playground is located. Figure 9 illustrates the park land on the Eastern shore of the Lake. The park land around the Lake is well-maintained and landscaped by the City of Orlando. On the Western shore is a bandshell and concert area. A small concession facility and boat rental are located on the Northwest shore. These park

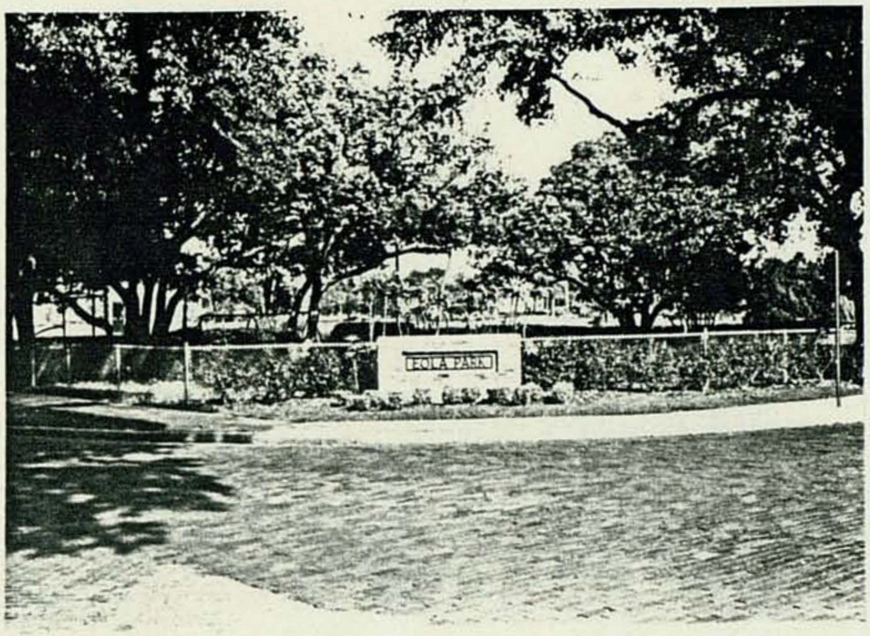


Fig. 8. Southeast Corner of Eola Park

Fig. 9. Eastern Shore of Lake Eola

areas are used and enjoyed by many of the residents of Orlando and the tourists who visit the area. However, if the park and Lake areas are to be enjoyed, the quality of Lake Eola must be maintained.

The commercial areas in the drainage basin are highly developed consisting of churches, high-rise office buildings, parking lots and various businesses. The commercial study area is on the Western shore of the Lake. Figure 10 shows the boundary of the commercial study area, the boundary of the proposed residential study area and the land uses of the drainage basin. The complete Figure is of the 169 acres which drains into the Lake. Figures 11, 12, 13, and 14 illustrate commercial land uses and parking lot areas in the commercial study area.

Figure 11 is of the central business district included in the commercial study area. This was taken early in the morning and traffic was already heavy. Figure 12 is another view of the commercial study area. The ground slope is about five feet per hundred feet and the area is highly impervious. Figure 13 is of one of the large parking lots contained in the study area. These parking lots are on the West border of the drainage basin. Figure 14 is of the commercial and parking areas in the vicinity of the Orange County Courthouse. These areas are quite busy during the daytime. In the Lake Eola drainage basin there are approximately 36 acres of streets and 18.5 acres of parking lots. The commercial study area has a total area of 28.0 acres. This total area includes the street and parking lot areas. The other proposed study area was a residential section on the north side of Lake Eola.

This proposed residential study area was comprised of approximately 16.1 acres. The area is essentially a multi-family residential

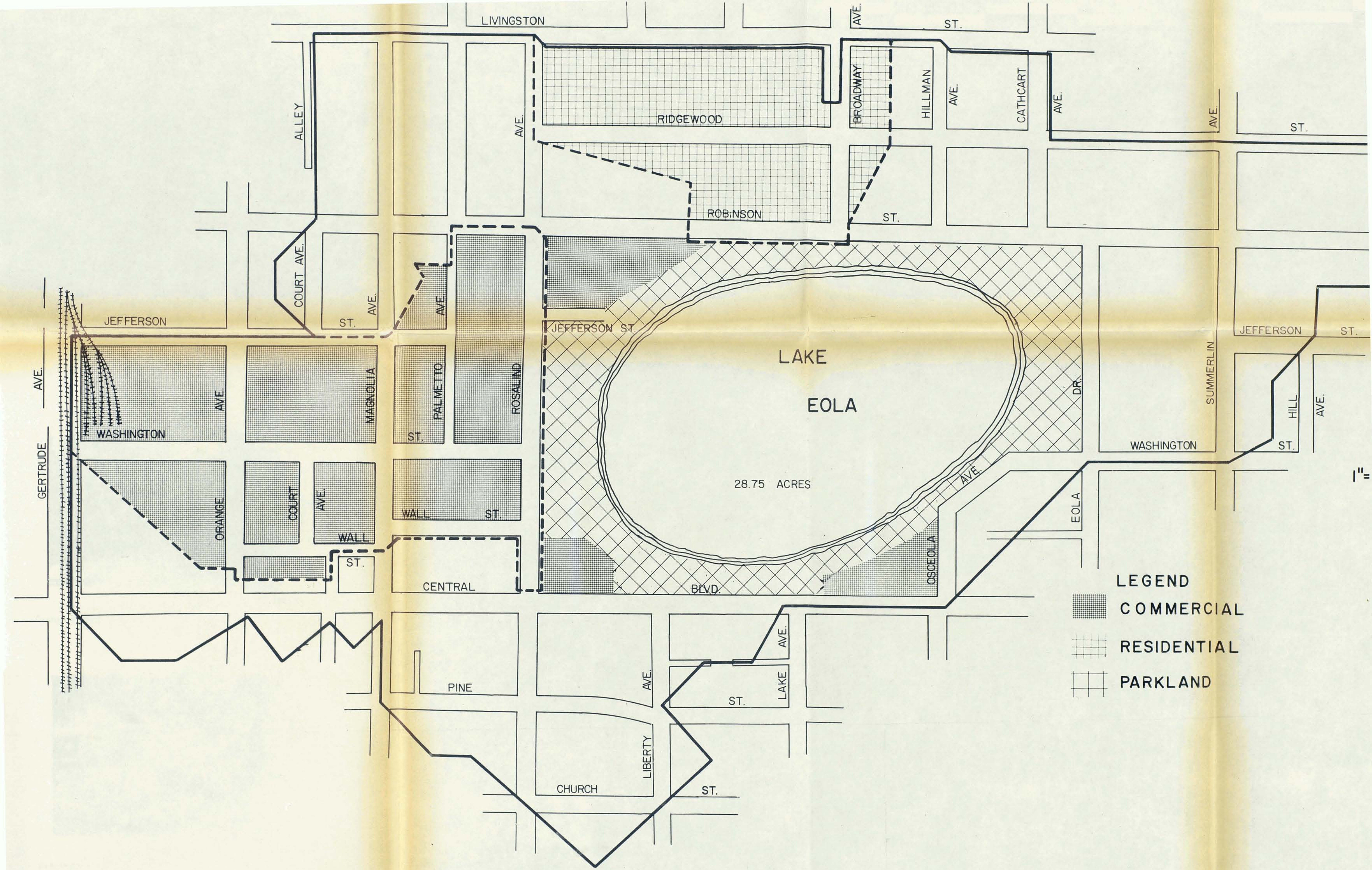


FIGURE 10

LAKE EOLA DRAINAGE BASIN LAND USES

Fig. 11. Corner of Washington Street and Orange Avenue, Looking North

Fig. 12. Intersection of Palmetto Avenue and Jefferson Street, Looking Southwest

Fig. 13. Parking Lot Area, Looking from Jefferson Street, South to Washington Street

Fig. 14. Intersection of Court Avenue and Washington Street, Looking South

section consisting of large older homes. These homes are divided into apartments and rooms which are used by separate households or individuals.

Figure 15 is of the residential area, looking west, down Ridgewood Street. Large oaks abound in the area and overhang most of the streets. Figure 16 is a good view of one of these older multi-family dwellings. Several vehicles belonging to the residents can be seen on the West side of the dwelling. The streets in the residential area are fairly wide, as can be noted in Figure 16. Parking is available on either side of most of the streets. This practice hampers street sweeping operations but is unavoidable.

The pollutants entering Lake Eola come from several sources, which have different characteristics. These sources and their characteristics will be discussed in the following pages.

Pollutant Sources of the Lake Eola Drainage Basin

Because of its unique location and various land uses, Lake Eola is subjected to several pollutant sources. These sources are of four categories:

1. Solid waste discarded into the Lake
2. Precipitation
3. Natural wildlife and domestic fowl
4. Stormwater runoff

The control of these sources (except precipitation) is perhaps possible, to some extent. But the costs and techniques must still be determined.

The discarding of solid waste such as cans, cups, papers and candy wrappers should be halted. Proper disposal containers are provided

Fig. 15. Intersection of Hillman Avenue and Ridgewood Street, Looking West

Fig. 16. Intersection of Ridgewood Street and Broadway Avenue, Looking Northwest

by the City and must be used. However, a certain segment of the population insists on abstaining from the use of these containers. For this reason, extra manhours are expended by the City workers to maintain the sanitation of the park. Littering ordinances do exist and need to be enforced. Besides creating an eyesore, the solid waste further decreases the water quality of the Lake.

Precipitation adds nutrients to the Lake in small quantities. These amounts are small when compared to other sources and have been discussed earlier. Phosphates and nitrogen are the primary constituents in the rainfall. The percentages of pollutants contributed by rainfall to runoff has also been discussed.[84]

Another natural source is the water fowl, both domestic and wild, which frequent Lake Eola.[85] Migratory fowl increase the population on the Lake in winter months; during the remaining months, domestic fowl are present. Depending upon the season, large numbers of seagulls, pigeons and ducks are present. These water fowl contribute very little nutrients to the Lake (about 1% of the total).[86] The amounts of bacteria and organics are high compared to others. Of the four sources, stormwater runoff is the major contributant to the pollution of Lake Eola.

Stormwater runoff into the Lake comes from three sources or areas - the residential, commercial and park lands in the drainage basin. These land uses have been discussed previously. The commercial and residential sections comprise the major areas in the Lake Eola drainage basin. They also contribute the most stormwater runoff into the Lake.

The quantity of stormwater available as runoff depends upon the various characteristics of the land surface. These characteristics in-

clude such factors as the topography of the basin, the permeability of the surface materials, type of surface cover, existing soil moisture and others. The topography of the Lake Eola drainage basin is fairly uniform. On the North, South, and East sides of the Lake the land slopes gently at about two feet per thousand feet away from the Lake. On the Western shore, the land surface rises at approximately two feet per hundred feet in the main commercial area. Much of the runoff from this area is from streets, parking lots, roofs and other impervious areas.

These areas, especially the streets and parking lots, contribute many of the pollutants discussed earlier to the Lake waters. A mathematical model which has the capabilities of predicting both the quantities and quality of the stormwater runoff will be applied to the study area. This study area will consist of that portion of the Lake Eola drainage basin illustrated previously in Figure 10. This Figure also demonstrates the land uses of the study area. The model utilized will be the Environmental Protection Agency Storm Water Management Model (SWMM).

CHAPTER V

EXPLANATION OF THE E.P.A. STORM WATER MANAGEMENT MODEL

Background and Purpose

The EPA Storm Water Management Model furnishes the user with important quality and quantity parameters which can be utilized in the design and evaluation of urban drainage basins.

Simulation of individual rainfall events by the model is accomplished. This simulation procedure takes into account the physical characteristics and storm sewer system of the drainage basin. The Storm Water Management Model has been tested and verified on several drainage basins in the United States. It is a natural extension to use it on the Lake Eola Drainage Basin; however, it requires extensive data on the basin, ranging from the street sweeping practices to the topography of the area. These characteristics vary according to the different land uses.

Verification of the results of the model simulation was done with data collected in the field and with the appropriate lab work. The computer model is comprehensive and requires a significant amount of data before it can be initiated. This data is of varied nature and has several sources. Initial data collected for the commercial study area that drains directly into the Lake was analyzed on the computer. This was successful and the capability of simulating the entire drainage basin was proven. For the purposes of this study, one specific area was studied - the commercial area.

The commercial study area is labeled as study area 84. The 84 is simply the number assigned to the outfall of the commercial storm sewer system. The computer simulation requires the numbering of all sewer elements in the system. This will be discussed later in this paper.

Model Description

The mathematical model consists of six main programs. These are:

1. Executive
2. Combine
3. Runoff
4. Transport
5. Storage/Treatment
6. Receiving

The programs are termed "blocks" and may be run in several ways. All the blocks do not have to be utilized in a single simulation. For instance, the Executive, Runoff and Transport blocks could be run and the simulation ended. The Executive block must always be utilized.

The Executive block is not a computational block, but controls the operations of the other blocks. A detailed description of this block is given in Volume III[87] on pages 21 to 34. The Executive block assigns input and output tapes for the following blocks and also scratch tapes for the blocks. The remainder of the Executive block is simply the blocks which will be used in a given simulation. This includes the data cards and other information required of the computational blocks. Therefore, the Executive block could be visualized as consisting of only two cards - those describing the input/output tape assignments and scratch tape assignments.

The Combine block gives the capability of simulating large drainage basins. With this block, the output from two or more of the same computational blocks (Runoff, Treatment, or Storage/Treatment) can be combined, collated, or combined and collated. For example, two Transport blocks for one drainage basin could be modeled and their output combined and input to the Storage/Treatment block. The Combine block can be used in a number of different ways and now gives the Storm Water Management Model the capability of simulating the largest and most diverse cities.[88] Volume III[89] on pages 35-40 describes the Combine block in detail. Its utilization would be necessary for the modeling of large, complex systems.

The first of the computational blocks is the Runoff block. The Runoff block simulates the quality and quantity of runoff occurring from an individual storm for the entire drainage basin. The drainage basin is divided into small subcatchments to accomplish the simulation. A maximum of 200 subcatchments can comprise one drainage basin. These subcatchments usually consist of small areas which have the same physical characteristics, such as land use or ground cover and topography. For simplification, one subcatchment is conceived as having one inlet manhole for drainage. This inlet manhole is connected to a part of the main storm sewer system.

An inlet manhole in the Runoff block simulation, therefore, is an inlet manhole in the Transport block. The Transport block is limited to only 70 inlets. So, unless more than one Transport block is going to be utilized, the user is limited to having 70 inlets and hence 70 subcatchments. Of course, more than one Transport block can

be utilized for one drainage basin, which allows the basin to be subdivided into more than 70 subcatchments.

Another limitation is that only 160 sewer elements can be defined in one Transport block. A sewer element being a conduit, flow divider, manhole or lift station. Each inlet manhole counts as a sewer element. If 70 inlet manholes are utilized, there can only be a total of 90 additional sewer elements in the simulation, the sum being 160 which is the maximum handled by the Transport block. But the use of more than one Transport block can overcome this limitation. Caution must be exercised in the initial system simulation to incorporate these limitations in a satisfactory manner. Again, Volume III[90] on pages 41 to 107 describes the Runoff block in detail.

The second computational block is the Transport block. This block takes the stormwater runoff generated by the Runoff block and routes it through the storm sewer system. The stormwater quantity and quality parameters are routed through the system considering infiltration parameters. The Fortran program is about 4100 cards long, consisting of 25 subroutines and functions.[91] The majority of the data required by the Transport block is physical data describing the storm sewer system. One feature of the Transport block is a subroutine which will increase the conduit size if surcharging occurs. The pipe is increased in standard sizes until a size which will handle the flow is achieved. This information is then output for the conduits which are resized.

The Transport block is programmed to handle 13 different conduit shapes and 7 different non-condiut elements. The capability exists

for the user to also input three conduit shapes which may not be provided in the model. The non-conduit elements that are handled are manholes, lift stations, backwater elements, storage units and three types of flow dividers. It will usually not be possible to model every conduit or non-conduit element in a drainage basin. Of course, several Transport blocks could be developed for the drainage basin to accomplish a complete simulation of all the elements. Volume III[92] on pages 108 to 216 gives a detailed description and instructions on the use of the Transport Model.

The third computational block is the Storage/Treatment block. This block is about 3700 cards and consists of 16 subroutines. The Storage model allows man-made or natural storage basins to be simulated. Quantity and quality of the runoff as it moves through the storage unit are modeled. This movement may be simulated either as plug flow or complete mixing. Inlet and outlet control can be by pumping, weirs, or orifice.

The input data required is basically a physical description of the storage unit. The various treatment options desired and associated costs also must be input. These costs consist of chemical, land and energy costs. If none are input, the model will utilize default values. The treatment options available are:

1. Bar Racks
2. Swirl Concentrations
3. Fine Screens
4. Dissolved Air Flotation
5. Sedimentation
6. Microstraining
7. Filtration (high-rate)
8. Chlorination

A biological treatment process is also included in the model, but the exact type could not be determined. Volume III[93] on pages 217 to 268 gives a detailed description of the Storage/Treatment block and instructions on its use.

The fourth and last computational block is the Receiving Water block. This block models the behavior of natural receiving waters, such as lakes, rivers and estuaries. The simulation is accomplished by dividing the receiving water system into a series of individual one- and two-dimensional areas (node elements). Each node is connected to another node. In this work, the velocity of flow is assumed constant with depth. One-dimensional elements represent rivers and specific channels.[94]

The Receiving Water block consists of three main subroutines:

1. RECEIV
2. SWFLOW
3. SWQUAL

Subroutine RECEIV is used for interactions with the Executive block and calls either SWFLOW, SWQUAL or both as necessary. SWFLOW is the hydraulic subroutine which simulates quantity parameters. SWQUAL simulates the quality parameters. Several smaller subroutines are utilized in SWQUAL and SWFLOW.

The data required for utilizing the Receiving Water block is primarily a description of the physical system. Some of the other input data required are rainfall data, whether the system in tidally influenced, evaporation rates, and initial pollutant values if available. A detailed description of the input data and instructions on the use of the Receiving Water block are available in Volume III,[95]

pages 269 to 337.

The Storm Water Management Model is a large mathematical model. Figure 16A illustrates the data deck setup for the complete model. A graph subroutine is available for plotting of the outputs of the various blocks. It requires very little data input, merely the titles assigned to the different plots. Utilization of all the various blocks is a major undertaking.

For this study, three of the blocks will be used. These will be the Executive, Runoff, and Transport blocks. The major data input required is for the Runoff and Transport blocks. The following section describes the data needed for these two blocks. It should be noted that this is for a storm sewer system simulation, only not a combined (storm and sanitary) system.

Physical System Data Required

The physical data required for the model is of two basic types - storm sewer system data and subcatchment area data. Sewer system data consists of all the physical parameters associated with the system. Subcatchment data is the various characteristics of the areas contained in the drainage basin. Sewer system data requirements for conduits are:

1. Length of individual conduits between non-conduit sewer elements in feet.
2. Invert slope of the conduit, feet per 100 feet.
3. Manning's roughness coefficient for the conduit.
4. Diameter of conduit in feet.

It should be noted that if the conduit is not circular but some other

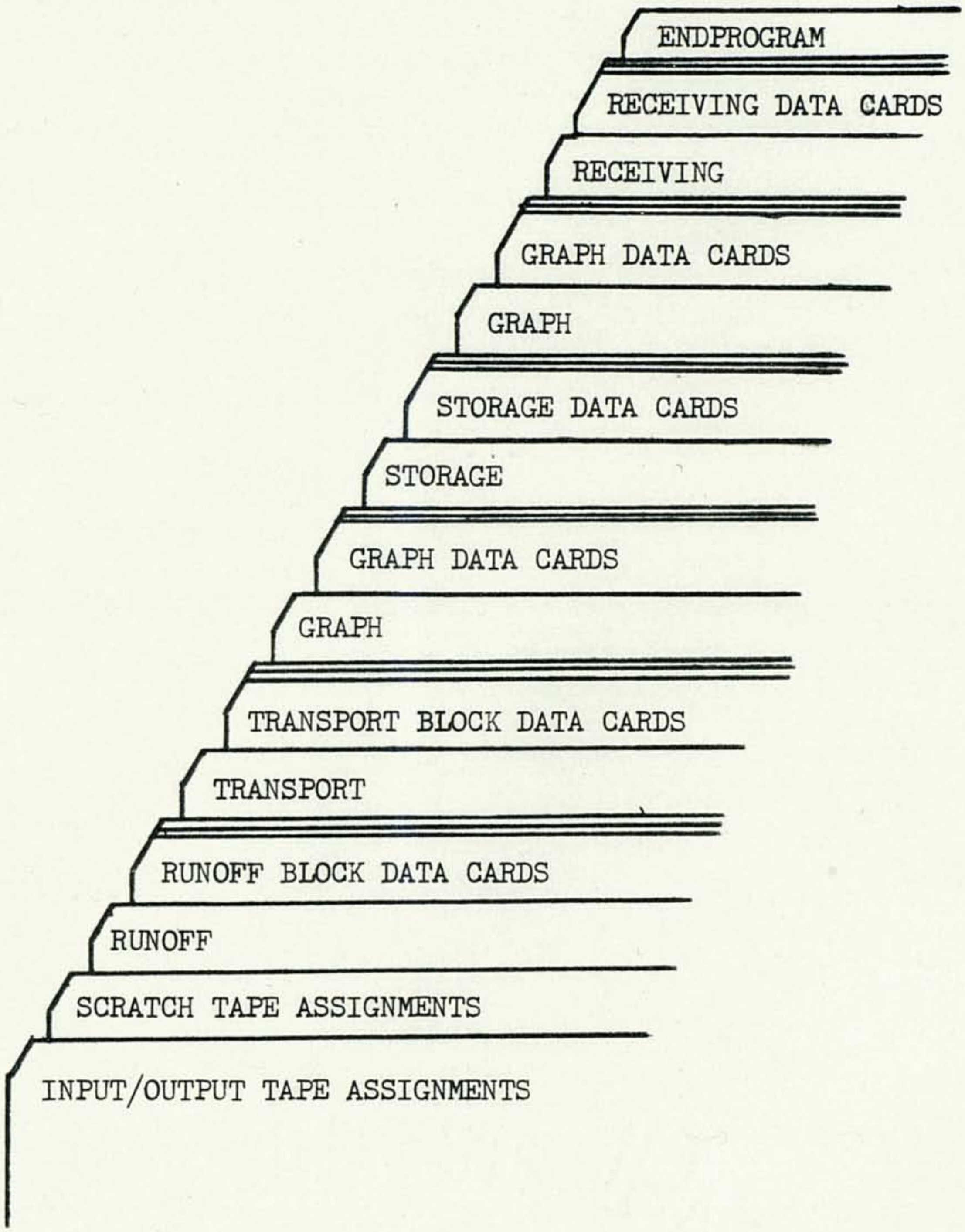


Fig. 16A. Data Deck Setup for SWMM

shape, those characteristic dimensions must be obtained. For example, a rectangular conduit would have the characteristic dimensions of height and width instead of diameter.

For this study, the above information was gathered from storm sewer drawings obtained from the City of Orlando Engineering Department. Their cooperation was essential for this study. A complete set of data on the sewer system is necessary for the model simulation. This data should include the locations, sizes, and types of all the storm sewer elements, including manholes and catchbasins. The data may be obtained from several sources depending upon the specific area.

If infiltration into the sewer system is to be estimated, several factors must be determined. The three types of infiltration that can occur are:

1. Dry weather infiltration
2. Groundwater infiltration
3. Rainwater infiltration

To simulate infiltration, some estimates of these in gallons per minute for the total sewer system must be made. This data can be from previous flow records, estimates from local engineering professionals, or generalized estimates based upon similar systems and areas.

Dry weather infiltration is that occurring continuously in the system from ground moisture. Groundwater infiltration is infiltration that occurs when the groundwater table is above the sewer invert. Rain water infiltration is that occurring from antecedent precipitation over the previous nine days. For this to be estimated, the rainfall data for the previous nine days must be available. Using this data, an equation, by linear regression, is developed yielding the infiltra-

tion rate in gallons per minute. In Volume III,[96] pages 138 and 139, example equations for several areas are given. If the ground water table is above the sewer invert continually, all infiltration is assumed to come from ground water infiltration. But, if the opposite is the case, all infiltration is assumed to originate from dry weather and rainfall (antecedent) infiltration.

Subcatchment area data consists of the various physical parameters which describe the drainage basin. The drainage basin studied is subdivided into individual subcatchments for the model simulation. Subcatchments represent idealized runoff areas with uniform slope.[97] The division of the drainage basin into subcatchments can be done by using various parameters such as land use, topography, and roughness. This subdivision can be as "fine" as desired, even to utilizing individual roof areas as subcatchments. The data required for the subcatchments is as follows:

1. Width of subcatchment
2. Area of subcatchment in acres
3. Percent imperviousness of the subcatchment
4. Ground slope in feet per feet
5. Manning's n for the impervious area
6. Manning's n for the pervious area
7. Impervious area, retention storage, inches
8. Pervious area, retention storage, inches
9. Maximum infiltration rate, inches per hour
10. Minimum infiltration rate, inches per hour
11. Decay rate of infiltration, 1/sec

The width of the subcatchment is twice the length of the longest drainage gutter through it. Volume III,[98] page 76, may be helpful in the interpretation of this characteristic.

For surface quality to be simulated, additional input model data is needed for each subcatchment. These data are:

1. Land use
2. Number of catch basins in each subcatchment
3. Total length of all gutters in subcatchment, hundreds of feet
4. Street cleaning frequency in days
5. Number of street sweeper passes
6. Catch basin storage volume, in cubic feet
7. BOD concentration in the catch basin water
8. Number of dry days before the modeled storm in which the summation of rainfall is less than one inch

Land use can be classified as either single family residential, multiple family residential, commercial, industrial, or park lands. BOD concentration will require lab samples.

The street cleaning frequency and number of passes were obtained from the city street department. Rainfall data is used to supply the number of dry days. Other data describing the subcatchments can be obtained from the topography maps and street maps of the drainage basin. The various coefficients and factors needed can be found in the literature or obtained from local professionals.

The only additional data requirements for the model simulation is the rainfall hyetograph and the percent of impervious area with no detention. The rainfall data for each storm should come from a gage or gages set up in the drainage basin. If this cannot be done, data from the nearest or most reliable rainfall recording station should be utilized. These rainfall data are input in inches per hour with a minimum value input of 0.01 inches per hour. The number of data points for the individual hyetograph can vary up to a maximum of 200.

The percent of impervious area with zero detention (immediate runoff) can be calculated as the total area of streets and parking lots in the drainage basin divided by the total impervious area in the basin

times one hundred percent. This assumes that streets and parking lots have zero detention, but that other impervious areas such as roofs will have some detention. For the Lake Eola drainage basin, the percent impervious area with zero detention was found to be about 48%. The 48% value is calculated by dividing the summation of the parking lot and street areas for the basin by the total impervious area of the basin. When the above data have been obtained, the Storm Water Management Model can be executed.

CHAPTER VI

APPLICATION AND CALIBRATION OF THE E.P.A. STORM WATER MANAGEMENT MODEL

Commercial Study Area Physical Data

The Commercial Study Area is 28.0 acres in size with a high degree of imperviousness. The initial collection of data was performed by visiting Mr. Bob Sevrens at the City of Orlando Engineering Department. Mr. Sevrens provided storm sewer system maps of the complete Lake Eola drainage basin. From these drawings, the Commercial Study Area storm sewer system was isolated.

Each system sewer element was numbered and listed as to the type of element. Three varieties of sewer elements were utilized - sewer conduits, catch basin manholes, and system manholes. Sewer conduits are numbered from 324 through 351 for a total of 28 conduits. Catch basin manholes are numbered from 217 through 235 for a total of 19. System manholes, which connect two conduits and are not catch basins, are numbered from 10 through 19 for a total of 10 system manholes.

The next step is to assign each conduit its corresponding characteristics. These characteristics consist of the slope, dimensions, length, and associated upstream manhole. For the Commercial Study Area, all conduits are circular and therefore the diameter is the only dimension required. If any other shapes such as rectangular had been present the appropriate dimensions such as height and width

would have been required. The length of each conduit and the invert slope are obtained from the storm sewer drawings. Manning's coefficient of roughness may also be specified for each conduit. If not specified, the SWMM utilizes a roughness coefficient value of 0.013. This value is for vitrified clay or concrete pipe in good condition or corrugated steel in very good condition. The value of 0.013 is utilized in the commercial study area simulation.

Table 10 lists the data for the storm sewer system of the commercial study area. Pipe lengths ranged from 6 feet to 250 feet, pipe diameters from 1.0 feet to 2.0 feet. Slopes are in feet per 100 feet and ranged from 10.0 to 0.5 feet per 100 feet. The manhole upstream from each conduit is also listed. The storm sewers are concrete and in good condition. Table 10 is the Transport block data required for the SWMM simulation of the sewer system.

Table 11 is the card groups required for the Transport block simulation. These groups are required for a storm water simulation only and are used in the order shown. The number of cards in Groups 15, 29 and 30 vary depending on the size of the storm sewer system. Group 15 requires one data card for each sewer element such as a conduit or manhole.

Figure 17 illustrates the commercial study area storm sewer system. The numbering sequence and system arrangement can be seen as well as the location of the outfall. The farthest upstream catch basin manhole is number 220 and the nearest catch basin manhole to the outfall is number 233; number 233 being a double catch basin arrangement. This double catch basin is shown in Figure 18. This is also the site

TABLE 10

TRANSPORT BLOCK DATA FOR STORM
SEWER SYSTEM 84

Pipe #	Slope (ft/100 ft)	Length (ft)	Diameter (ft)	Upstream Manhole #
324	2.3	35	1.0	217
325	3.08	50	1.0	218
326	9.6	25	1.0	219
327	0.5	40	2.0	10
328	0.5	80	1.0	220
329	0.45	250	1.0	221
330	10.0	30	1.0	222
331	2.0	200	2.0	11
332	1.0	135	1.75	235.233
333	2.0	25	1.0	15
334	2.0	40	1.0	234
335	0.5	6	1.5	223
336	2.0	20	1.0	224
337	2.0	30	1.0	226
338	0.51	190	1.0	14
339	8.2	35	1.0	225
340	0.5	82	1.25	13
341	2.0	260	2.0	12
342	2.0	45	1.0	227
343	2.0	50	1.0	229
344	5.2	187	2.0	16
345	5.0	37	1.25	17
346	5.0	38	1.0	230
347	1.0	225	2.0	18
348	2.0	30	1.0	232
350	1.1	22	2.0	19
351	0.11	20	1.0	228

TABLE 11

TRANSPORT BLOCK CARD GROUPS

Group #	# of Cards	Type of Cards
1	1	Control
11	1	Title
12	1	Execution Control
13	1	Execution Control
14	1	Execution Control
15	Variable*	Sewer Element Data
27	1	Outfall
29	Variable*	Print Control
30	Variable*	Print Control
31	1	Estimated Infiltration
32	1	Infiltration Control
33	1	Monthly Degree Days

* Depending upon storm sewer system size.

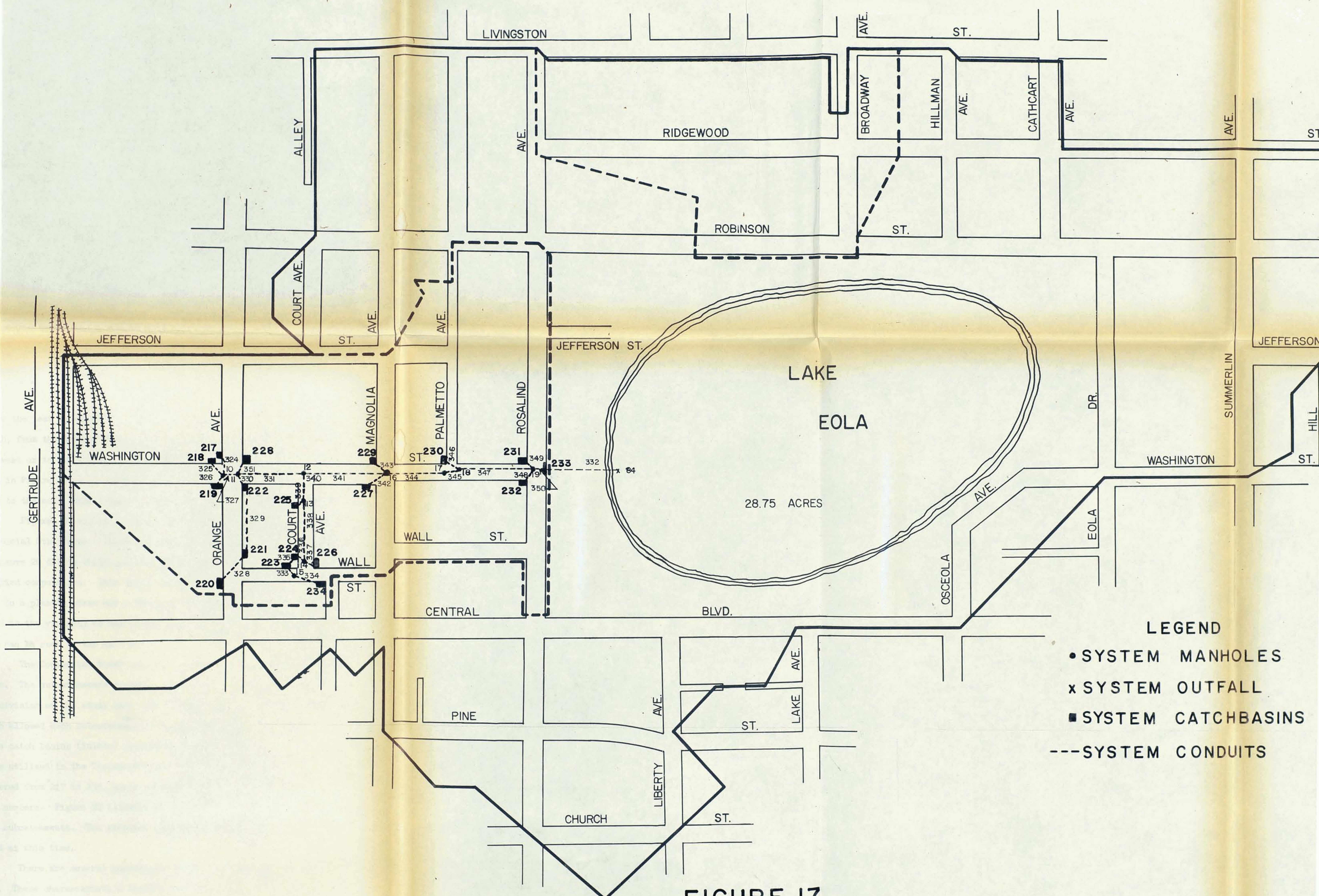


FIGURE 17

COMMERCIAL STORM SEWER SYSTEM

where the test equipment is located. Figure 19 is a view, looking north, from the double catch basin. The Commercial Study Area is to the west or to the right in the photograph. To the east or the the left in Figure 19 can be seen the parkland which surrounds Lake Eola. Also to the east, approximately 135 feet, is outfall 84 in Lake Eola.

Figures 20 and 21 illustrate two of the catch basins in the Commercial Study area. These are catch basins 228 and 226 respectively. In Figure 20 debris which has collected in the gutter can be seen with a varied composition. This debris ranges from cigarette butts and dirt to a plastic straw and a plastic comb. Figure 21 is of a catch basin in the vicinity of the Orange County Court House. A parking area can be seen to the left of it.

The Commercial Study area was subdivided into 19 subcatchment areas. The subcatchments ranged in size from 0.2 acres to 4.0 acres. The division of the study area into subcatchments was done in a manner which allowed each subcatchment one catch basin manhole or inlet. These catch basins (inlets) correspond to the same catch basin manholes utilized in the Transport block simulation. The subcatchments are numbered from 217 to 235, again corresponding to the catch basin manhole numbers. Figure 22 illustrates the division of the study area into subcatchments. The proposed residential study area is not numbered at this time.

There are several parameters needed to describe each subcatchment. These characteristics include the area, ground slope, total gutter length, percent imperviousness and land use. Table 12 lists the data required for the Runoff block simulation. A 95% imperviousness

Fig. 18. Catchbasin 233, Western Shore of Lake Eola

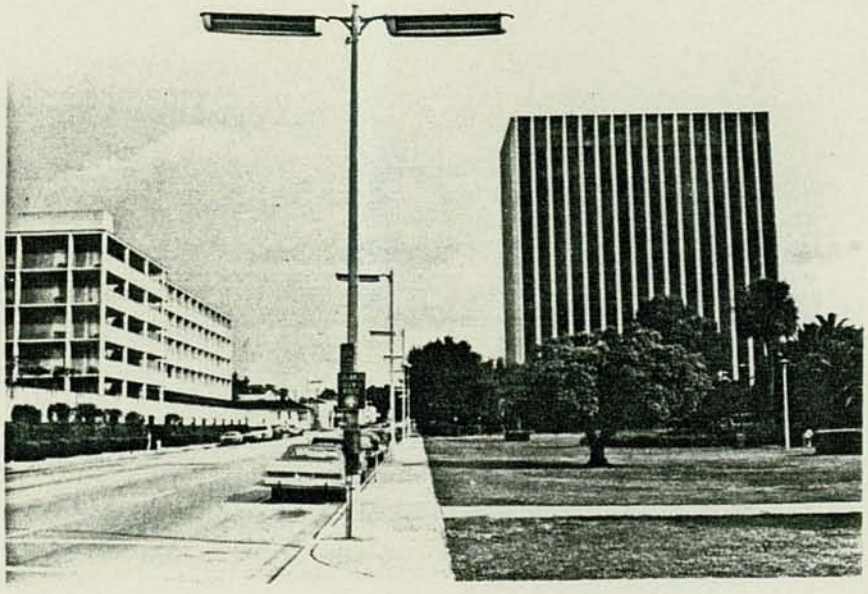


Fig. 19. Looking North from Catchbasin 233

Fig. 20. Catchbasin 228, Northeast Corner of Intersection of Orange Avenue and Wall Street

Fig. 21. Catchbasin 226, Northeast Corner of Wall Street and Court Avenue

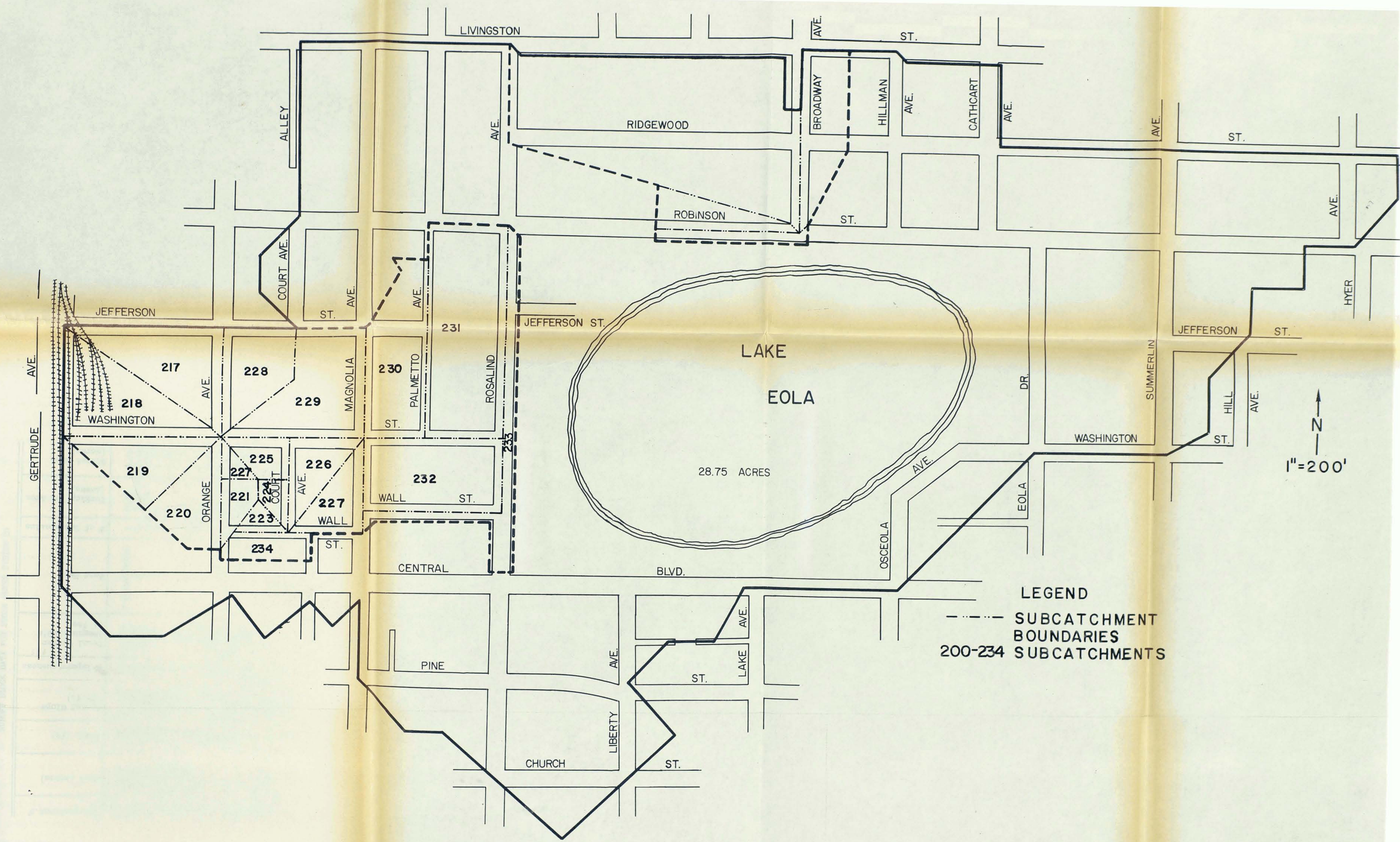


FIGURE 22
COMMERCIAL AREA SUBCATCHMENTS

TABLE 12

RUNOFF BLOCK DATA FOR STORM SEWER SYSTEM 84

Subcatchment #	Area (acres)	Width (ft)	Ground Slope (ft/ft)	% Imperviousness	Total Gutter Length, 100's of feet	Land Use	# of Catchbasins	Parking Lot Area (acres)	Street Area (acres)
217	2.336	480	0.008	95	14.89	3 (commercial)	1	0.39	0.56
218	2.336	480	0.008	95	34.97	3	1	1.5443	0.56
219	1.579	350	0.005	95	19.59	3	1	0.85	0.331
220	1.351	350	0.004	95	3.5	3	1	0	0.241
221	0.4252	170	0.003	95	1.7	3	1	0	0.117
222	0.2066	110	0.003	95	1.2	3	1	0	0.09
223	0.2754	160	0.003	95	1.6	3	1	0	0.11
224	0.3156	170	0.003	95	1.7	3	1	0	0.117
225	0.6711	180	0.003	95	3.57	3	1	0.05	0.186
226	1.20	280	0.004	95	7.3	3	1	0.26	0.192
227	1.25	280	0.004	95	4.19	3	1	0.12	0.145
228	1.57	340	0.007	95	5.5	3	1	0	0.379
229	2.772	385	0.01	95	15.4	3	1	0.3213	0.675
230	2.62	650	0.0466	95	24.42	3	1	0.5785	0.988
231	4.01	1440	0.022	95	19.1	3	1	0.3443	1.32
232	2.984	890	0.029	95	19.1	3	1	0	0.9
233	0.63	700	0.009	95	7.5	3	1	0	0.63
234	0.65	200	0.0035	95	0.5	3	1	0	0.04
235	0.82	500	0.009	95	13.0	3	1	0	0.82

value was used for the subcatchments. All are commercial land use areas and consist of buildings, streets, parking lots, etc. There is very little, if any, pervious subareas contained in the commercial study area.

The total gutter length in hundreds of feet also includes additional gutter footage for the parking lot areas. From Volume II,[99] 1740 feet of additional gutter length is suggested for each acre of parking lot. The parking lot area of each subcatchment is calculated and entered in Table 12. The total gutter length is the sum of this additional gutter footage and the actual gutter footage in the subcatchment.

Other factors required for the Runoff block simulation include Manning's n for the impervious and pervious surface areas and a decay rate of infiltration in Horton's equation. These and other coefficients utilized by the SWMM are given in Table 13. Table 13 lists the default values utilized by the Runoff block for the various factors. If these default values are not utilized, other values may be used.

In addition to these parameters, others must be determined which depend on the drainage basin being simulated. For the Commercial Study area, a street cleaning frequency of four days and one street sweeper pass are used. This information was obtained from Mr. W. R. Lawson of the City of Orlando Street Department. Two additional surface quality variables which are needed are the catch basin storage volume and the BOD concentration of the stored water in the catch basin. For the commercial study area, these were taken as 16.044 cubic feet and 100 mg/l respectively. The BOD concentration is a suggested value

TABLE 13

REQUIRED RUNOFF BLOCK VARIABLES

Variable	Default Value
Manning's n, Impervious area	0.013
Manning's n, Pervious area	0.250
Retention storage, Impervious area	0.062 (in)
Retention storage, Pervious area	0.184 (in)
Maximum Infiltration Rate	3.00 (in/hr)
Minimum Infiltration Rate	0.52 (in/hr)
Decay Rate of Infil- tration in Horton's Equation, 1/sec	0.00115

by the EPA. The default values listed in Table 13 were also utilized in the initial commercial study area simulation.

The card groups required for the Runoff block are given in Table 14. These card groups must be included in a storm water simulation. With these card groups, a description of the drainage basin or area is supplied to the Model. The card groups shown allow simulation of surface quality and quantity but erosion is not modeled. To model erosion, card group 10 would have to be included. The blank cards listed in Table 14 must be included in the data deck. They simply signal the termination of previous card groups.

The physical data describing the study area was obtained from engineering drawings, discussions with public officials and field work. The data gathering and reduction consumes a considerable amount of time. During the same time period, an effort was being made to utilize the EPA, SWMM program. After the Commercial Study area data was obtained and used in the SWMM Model, a test storm of September 27, 1973, was simulated.

Initial SWMM Simulation

The storm of September 27, 1973, was of ten hours duration and had a total rainfall of about 1.27 inches. The rainfall data of Table 15 for this storm was obtained from Local Climatological Data published by the U.S. Department of Commerce.[100] Table 15 lists the rainfall data used in the initial simulation run.

As can be expected when a "new" program use is attempted, some "bugs" will occur. Several problems which arose and prevented the program from running were in the various control cards. Some which

TABLE 14

RUNOFF BLOCK CARD GROUPS

Group #	# of Cards	Type of Cards
1	2	Title
2	1	Control
3	1	Rainfall Control
4	Variable*	Rainfall Data
6	1	Blank
7	Variable**	Subcatchment
8	1	Blank
9	1	Surface Quality Control
11	Variable**	Surface Quality
12	1	Print Control
13	Variable**	Inlet Print

* Depends on duration of storm.

** Depends on number of subcatchments.

TABLE 15

RAINFALL DATA FOR SEPTEMBER 27, 1973, STORM OF 10 HOURS DURATION

Hour Ending at:*	Rainfall, in/hr
700	0.02
800	0.89
900	0.04
1000	0.14
1100	0.01
1200	0.09
1300	0.01
1400	0.02
1500	0.01
1600	0.04

* Start of storm at 600 hours.

were supposed to have been left out were needed by the program. These problems were handled as they arose and a successful simulation run was achieved on the 29th of April, 1975. The results of this test simulation are shown in Figures 23, 24 and 25. These are the plotted flow, BOD and suspended solids graphs for the storm of September 27, 1973. Figure 23 is the runoff hydrograph occuring at the outfall of the Commercial Study area. Peak runoff is approximately 25.0 cfs. Figure 24 is the BOD flow in lbs/minute. The peak five-day BOD flow is about 2.45 lbs/minute. Figure 25 is the suspended solids, also in lbs/minute. Suspended solids peak flow is about 31.5 lbs/minute. All these maximum or peak values occur about one and a half hours after the start of the storm. The peak values occurred around 7:30 a.m. and ended at approximately 9:00 a.m. These dots are for the outfall of the Commercial Study area.

The computer simulation had a CPU time of 85 seconds and an I/O time of 129 seconds with a cost of $44. The program was run on an IBM 360 and had a turn around time of about 24 hours. The simulation required 390k bytes of memory. The results of the simulation of the September 27, 1973, storm could not be compared with actual quantity and quality data since none were available. The next goal was to obtain flow and quality data occurring from actual rainstorms, calibrate the SWMM Model and determine pollutant loading rates.

Quantity and Quality Measurement Techniques

In order to calibrate and have actual data with which to compare computer simulations, it is necessary to perform quantity and

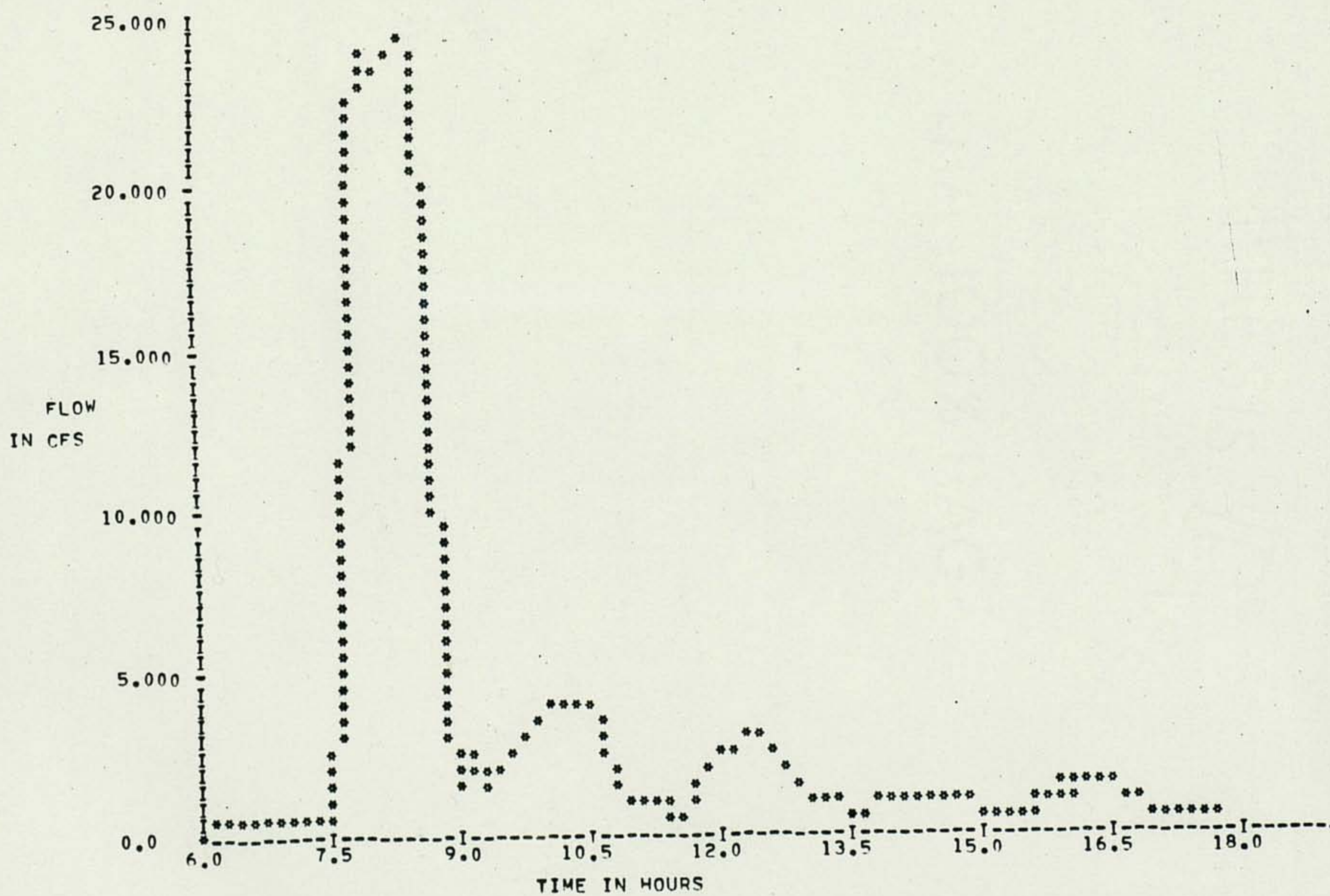


Fig. 23. Stormwater Runoff Hydrograph at Commercial Study Area Outfall

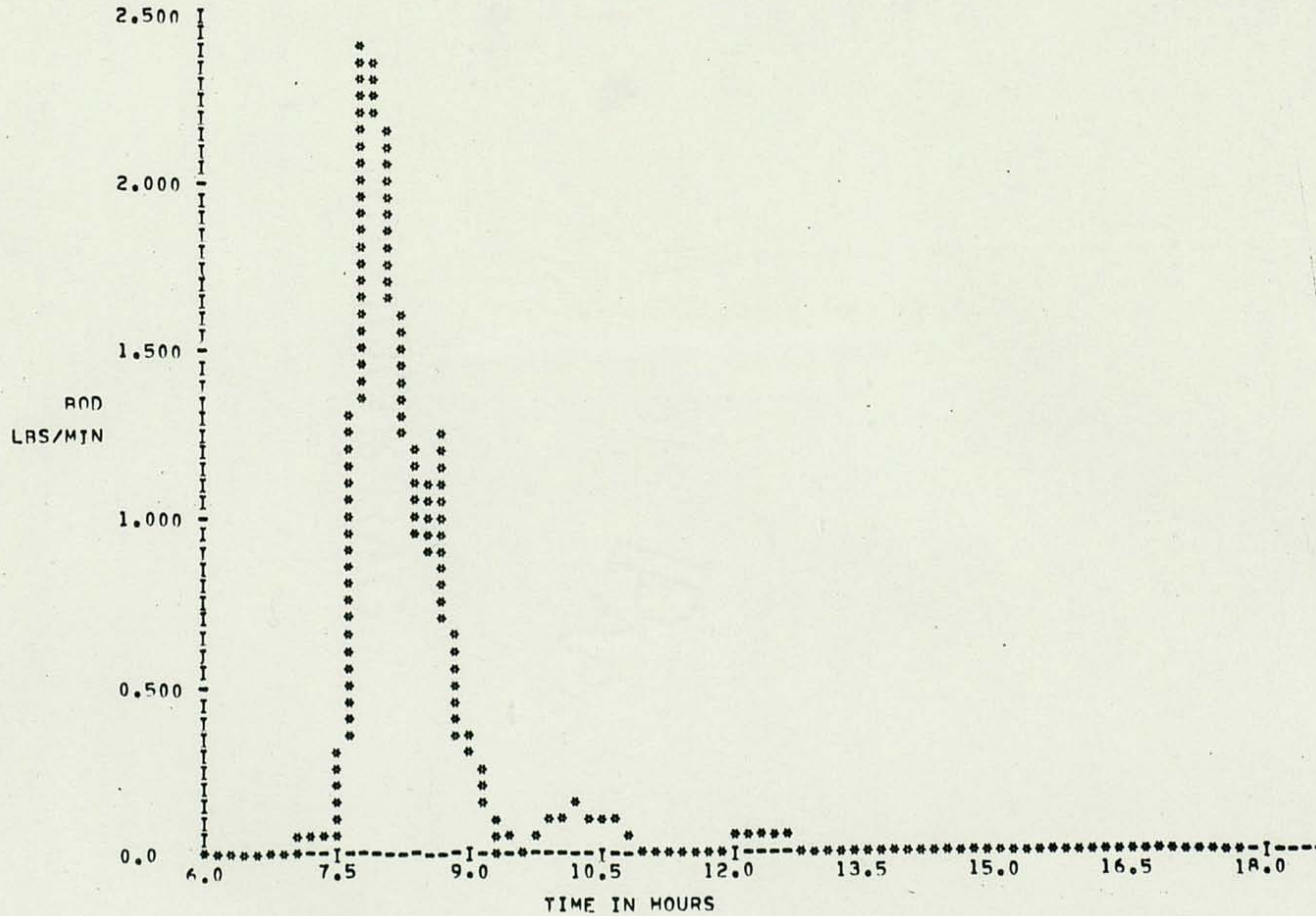


Fig. 24. Pollutant Flow of 5-Day BOD at Commercial Study Area Outfall

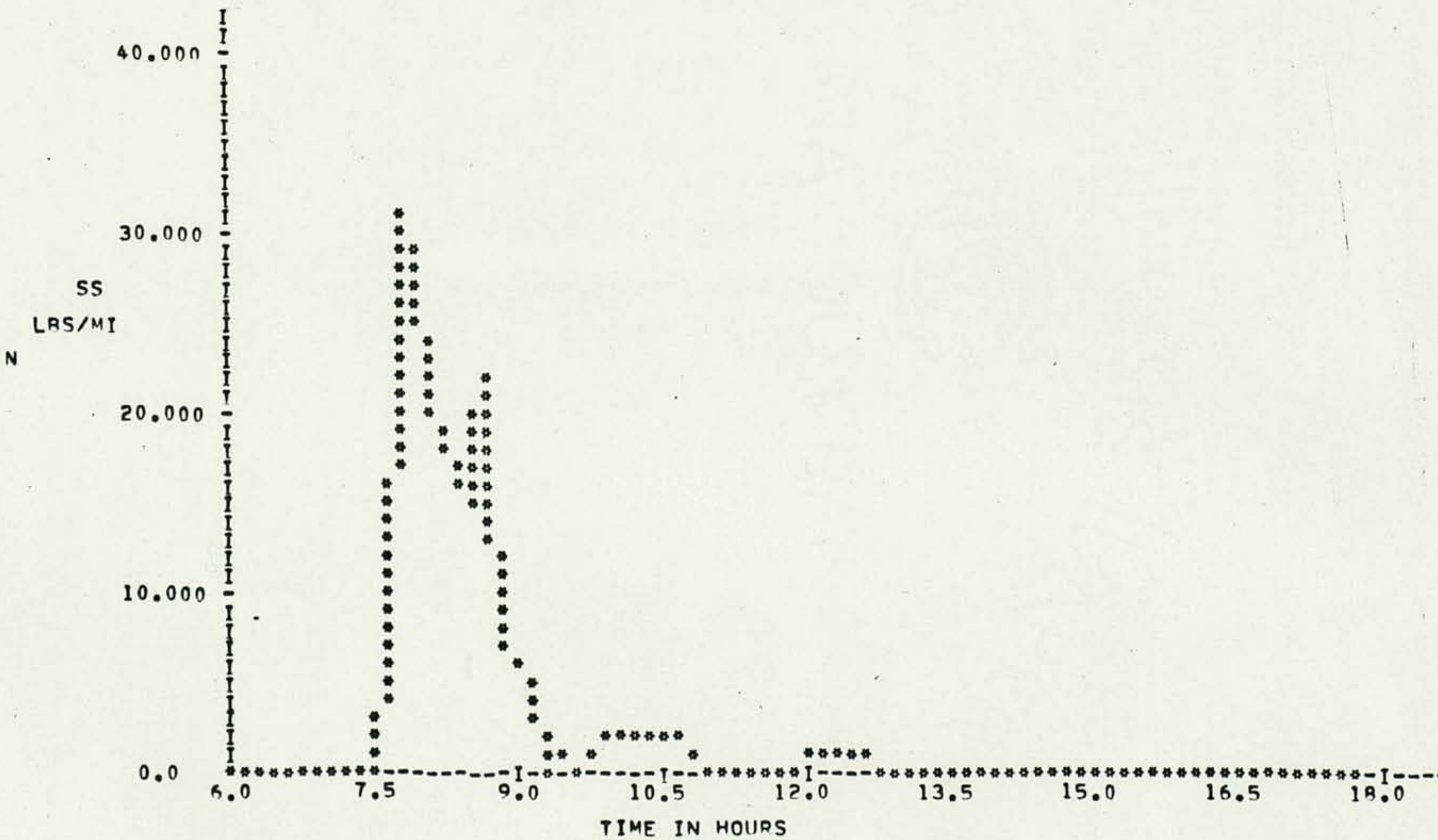


Fig. 25. Pollutant Flow of Suspended Solids at Commercial Study Area Outfall

quality analyzes of actual rainfall events. To do this, a quality sampler constructed at FTU was positioned at the farthest downstream manhole of the commercial storm sewer system. The Sequential Sampling Technology (SST) as it is called, has the capabilities of taking several types of quality and rainfall quantity data.

The unit provides a continuous rainfall record by utilizing a Texas Instruments Rainfall Recording System, Model R2-1014. This model gives total rainfall in hundredths of an inch and a rainfall record of up to 31 consecutive days. This equipment is coupled with a N-Con Systems Company Sequential Composite Sampler. This sampler takes composite samples of the storm sewer flow at chosen time intervals. Composite samples of one liter can be taken in 30 minutes. For the commercial study areas, composites of one liter in 30 minutes or one liter in one hour were utilized. Large composites of 6 liters were also collected over 3 and 6 hour cycles. In addition to these composite samples, grab samples were taken during each rainstorm.

Flow measurements are performed at the site by measuring the depth of the flow in the sewer conduit. Thus, knowing the slope, diameter, and roughness of the conduit, the flow can be determined from depth measurements. In this manner, actual runoff hydrographs can be constructed for individual rainfall events. These will be compared with the SWMM computer simulations. The computer modeling can be calibrated for the actual runoff hydrographs.

The SST was set up at the test site the morning of the 16th of April, 1975. Needless to say, it rained early in the morning of the 16th and began to break off at about 10:00 a.m. The Orlando area

subsequently experienced a drought or dry period for eighteen days. On the 19th day, May 5, 1975, a rainfall event of 0.03 inches occurred, ending the dry period. During this dry period, dry weather flow quantity and quality measurements were obtained.

Dry Weather Flow, Quantity and Quality

Dry weather flow is approximately 0.33 cfs or about 150.0 gpm. This dry weather flow occurred on the 20th of April, 1975. At the end of the dry period, dry weather flow was approximately 0.27 cfs or 123.0 gpm. This dry weather flow measurement was taken on the 5th of May, 1975, before the rainfall event of that same day. It was impossible to determine if this was really dry weather flow because the groundwater elevation was unknown. The dry weather flow remained fairly constant which is usually not the case with wet weather flow. This dry weather flow was assumed to be infiltration resulting from ground moisture.

Sampling of the dry weather flow was performed by both grab and composite sampling. A grab sample of 3 liters was taken on the 16th of April and analyzed. Three more grab samples were on the 17th at 10:30 a.m. These were taken at the test site manhole, outfall 84, and of the Lake itself. The results of these grab samples are given in Table 16.

For the test manhole (233) a composite sampling sequence of three consecutive days was performed. The sequence started at 4:00 a.m. or 0400 hours on the 18th of April and ended at 10:00 a.m. or 1000 hours on the 20th of April. During this time, twelve 3 hour composite samples of one liter each and two 18 hour composite samples of 6 liters each

TABLE 16

DRY WEATHER FLOW, GRAB SAMPLE ANALYSIS

Date	Time	Turbidity (JTU)	pH	Conductivity (mho/cm)	Alkalinity (mg/l $CaCO_3$)	Hardness (mg/l $CaCO_3$)	Carbon (mg/l)		Nitrogen (mg/l)			Phosphorus (mg/l)		Total Nitrogen (mg/l)
							IC	TOC	NH_3-N	Org-N	NO_3-N	OP	TP	
4/16	1030	1.1	8.27	340	151.2	156	27.3	4.5	0	.27	.36	.52	.82	.63
4/17	1030	1.7	8.32	345	144.8	158	28.8	3.9	0	0	.18	.59	.88	.19
4/17	1030	1.3	8.33	340	146.6	160	29.2	3.5	0	0	.19	.59	.90	.12
*4/17	1030	1.3	7.97	630	94.2	154	18.3	5.7	0	.01	.92	.12	.40	.93

*Lake sample

were taken. The results of the chemical analysis of these samples is shown in Table 17. These values show little variation over the three day period.

Averaging the fourteen composite sample values yields the following dry weather flow characteristics:

Turbidity	1.50 (JTU)
pH	8.29
Conductivity	349.30 (mho/cm)
Alkalinity	150.20 (mg/l as $CaCO_3$)
Hardness	160.58 (mg/l as $CaCO_3$)
Inorganic Carbon	29.05 (mg/l)
Total Organic Carbon	5.35 (mg/l)
TKN	0.43 (mg/l)
NO_3-N	0.15 (mg/l)
Total Nitrogen	0.58 (mg/l)
Organic Phosphorus	0.518 (mg/l)
Total Phosphorus	0.83 (mg/l)

Comparing these averaged values with the two grab samples, very little variation is apparent. The two grab samples listed first in Table 16 are from the test site, the third from the outfall into the Lake and the fourth is from Lake Eola.

Comparing the storm sewer dry weather flow average results to that of the Lake Eola grab sample is interesting. The Lake has a slightly lower pH, much higher conductivity, lower alkalinity, higher nitrogen and much lower phosphorus content than the dry weather flow. Phosphorus is the most important parameter concerned with the Lake's water quality. It has been determined by Boyter,[101] that phosphorus is the deciding parameter for Lake Eola. In other words, if high levels of phosphorus are entering the Lake, the water quality and environment will suffer.

An organic phosphorus level in the dry weather flow of 0.518 mg/l and a total phosphorus level of 0.83 mg/l was determined. The

TABLE 17

DRY WEATHER FLOW, COMPOSITE SAMPLE ANALYSIS

Date	Time	Turbidity (JTU)	pH	Conductivity (mho/cm)	Alkalinity (mg/l $CaCO_3$)	Hardness (mg/l $CaCO_3$)	Carbon (mg/l)		Nitrogen (mg/l)			Phosphorus (mg/l)		Total Nitrogen (mg/l)
							IC	TOC	NH_3-N	Org-N	NO_3-N	CP	TP	
4/18	0700	1.1	8.36	340	148.9	172.0	29.6	5.4	.95	---	.15	.46	.70	1.10
4/18	1000	0.9	8.39	340	153.3	16.20	29.3	3.0	---	---	---	.46	.78	----
4/18	1300	1.4	8.30	340	148.9	136.0	29.6	4.1	---	---	---	.47	.76	----
4/18	1600	2.2	8.08	360	146.6	158.0	30.5	5.2	.54	---	.18	.60	1.08	0.72
4/18	1900	1.6	8.26	340	151.1	152.0	29.6	2.1	---	---	---	.44	.74	----
4/18	2200	1.7	8.22	360	144.5	164.0	31.5	5.1	---	---	---	.66	1.03	----
4.19	0100	2.8	8.20	360	155.5	164.0	30.1	8.3	.41	---	.14	.68	.98	0.55
4/19	0400	2.1	8.30	360	151.1	164.0	30.0	6.6	---	---	---	.72	.98	----
4/19	0700	1.4	8.34	350	155.5	164.0	29.6	5.2	---	---	---	.54	.94	----
4/19	1000	1.4	8.35	350	153.3	164.0	29.6	4.5	---	.41	.15	.50	.80	0.56
4/19	1300	1.2	8.35	350	148.9	166.0	27.9	5.5	---	---	---	.42	.68	----
4/19	1600	1.2	8.35	340	148.9	162.0	24.7	8.3	---	---	---	.37	.62	----
4/18	0400	1.2	8.18	345	144.5	160.0	26.8	5.6	.14	---	.14	.48	.80	0.28
4/20	1000	1.0	8.34	355	151.1	160.0	28.0	6.0	---	.14	.13	.45	.72	0.27

Lake grab sample had an organic phosphorus concentration and a total phosphorus concentration of 0.12 and 0.40 mg/l, respectively. The increased phosphorus levels could be occurring naturally in the infiltration or resulting from human sources. These human sources could include cooling tower water or parking lot washing by various hotels in the area. The "dry" period ended May 5 with 0.03 inches of rainfall.

Calibration Storm Selection

In order to calibrate the SWMM model a single rainfall event must be utilized. Using the actual rainfall and runoff data for this calibration storm, the appropriate coefficients of the model are adjusted to obtain a reasonably good computer "fit." That is, upon completion of the calibration, the Model should accurately predict both quantity and quality parameters which compare with the actual quantity and quality parameters.

Four rainfall events occurred during the study period. These were as follows:

May 5, 1975	0.03 inches
May 10, 1975 (a.m.)	0.02 inches
May 10, 1975 (p.m.)	0.48 inches
May 12, 1975	0.70 inches

Table 18 is the rainfall occurrences for 1972 and 1973 categorized according to their intensities. The past rainfall data for the two years (1972 and 1973) was analyzed and the following data obtained. Approximately 214 storms occurred in this two year period. The number of storms with an intensity of 0.50 inches/hour or greater was 29. A storm of this intensity or greater is likely to occur about 13% of the time. In other words, out of 100 storms, 13 will probably have an

TABLE 18

1972 AND 1973 STORM INTENSITIES

Month	Storms of Less Than 0.5 inch/hr Intensity		Storms of More Than 0.5 inch/hr intensity	
	1972	1973	1972	1973
Jan	7	8	0	0
Feb	11	5	0	1
Mar	10	6	1	1
Apr	5	4	1	1
May	14	7	2	2
June	7	14	1	2
July	13	15	1	3
Aug	16	15	3	3
Sept	4	13	0	3
Oct	7	11	1	1
Nov	9	1	0	0
Dec	4	9	0	2

Total number of storms = 214
Total number greater than 0.5 in/hr intensity = 29

intensity of about 0.500 inches per hour. This storm would have a frequency of about 10 percent and give a 90 percent design frequency. The storm chosen for the Model calibration is the May 10, 1975 (p.m.) storm.

Quantity Calibration of the SWMM Model

After selection of the calibration storm, an initial simulation was undertaken. This simulation utilized most of the default values of the Model except for the percent impervious area with zero detention and the impervious area surface storage coefficients. These were set at 48% and 0.074 inches respectively.

The rainfall hyetograph for the storm is contained in Figure 26. This hyetograph is developed by the Model from actual rainfall input data. This actual data is obtained from the rainfall recorder at the test site and given in Table 19. The storm duration is 51 minutes with high intensities occurring initially.

Actual runoff was measured from the commercial study area at manhole 233. This is the farthest downstream manhole of the storm sewer system. The runoff flow data is given in Table 20. The total flow includes the estimated flow from a conduit adjacent to manhole 233 which drains a small street area. This estimated flow is given in Table 21 and was obtained at the site during the rainfall event. The estimated flow is a small portion of the total flow. It is felt that the flow was more than likely underestimated. This fact should be kept in mind.

The first calibration run resulted in the runoff hydrograph of Figure 27. The actual runoff hydrograph is also plotted in Figure 27.

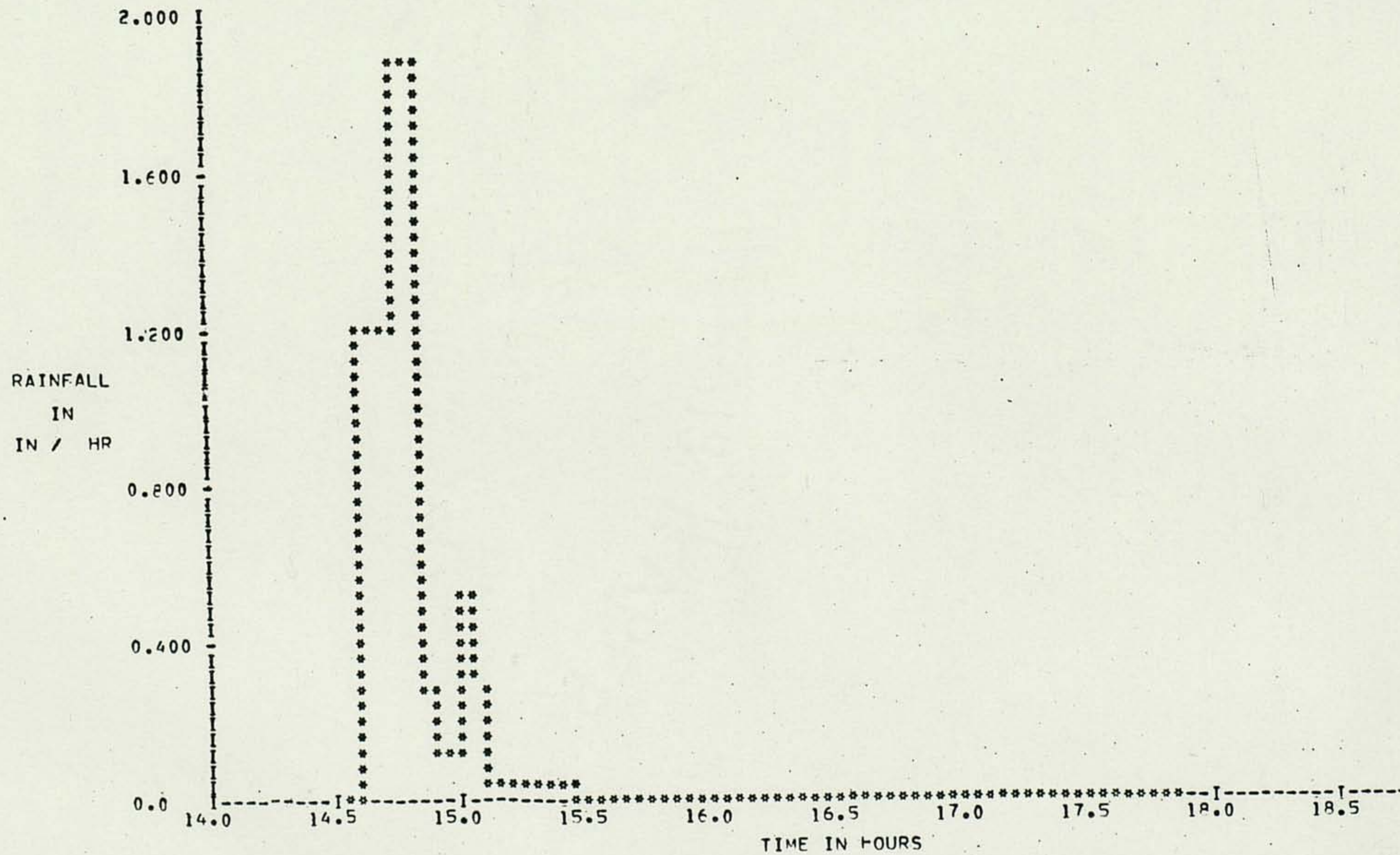


Fig. 26. Rainfall Hyetograph for Storm of May 10, 1975

TABLE 19

RAINFALL DATA, STORM OF MAY 10, 1975

Rainfall Intensity (in/hr)	Duration (min)
1.20	8
1.89	7
0.30	3
0.12	5
0.54	5
0.03	23

TABLE 20

FLOW DATA, STORM OF MAY 10, 1975

Time	Depth (in)	Flow (cfs)	Total Flow (cfs)
1437	1.75	0.268	0.268
1440	24.00	22.620	23.120
1445	24.00	22.620	23.870
1450	20.00	22.900	23.750
1500	15.00	16.160	16.760
1505	12.00	11.330	11.830
1510	9.75	7.820	8.120
1517	7.50	4.760	5.060
1520	6.00	3.010	3.210
1527	5.50	2.620	2.720
1530	5.00	2.120	2.120
1535	4.50	1.700	1.700
1538	4.50	1.700	1.700
1544	3.50	1.030	1.030
1550	3.45	1.000	1.000
1600	3.00	0.770	0.770
1608	2.85	0.680	0.680
1645	2.75	0.630	0.630
1925	1.75	0.268	0.268

Pipe Diameter = 24 inches
Slope = 1.0 ft/100 ft
Manning's n = 0.013

TABLE 21

ESTIMATED FLOW, MAY 10, 1975

Time	Flow (cfs)
1437	0.00
1440	0.60
1445	1.25
1450	0.85
1500	0.60
1505	0.50
1510	0.40
1517	0.30
1520	0.20
1527	0.10
1530	0.00

Pipe of 36" diameter which drains small street areas to north of test site.

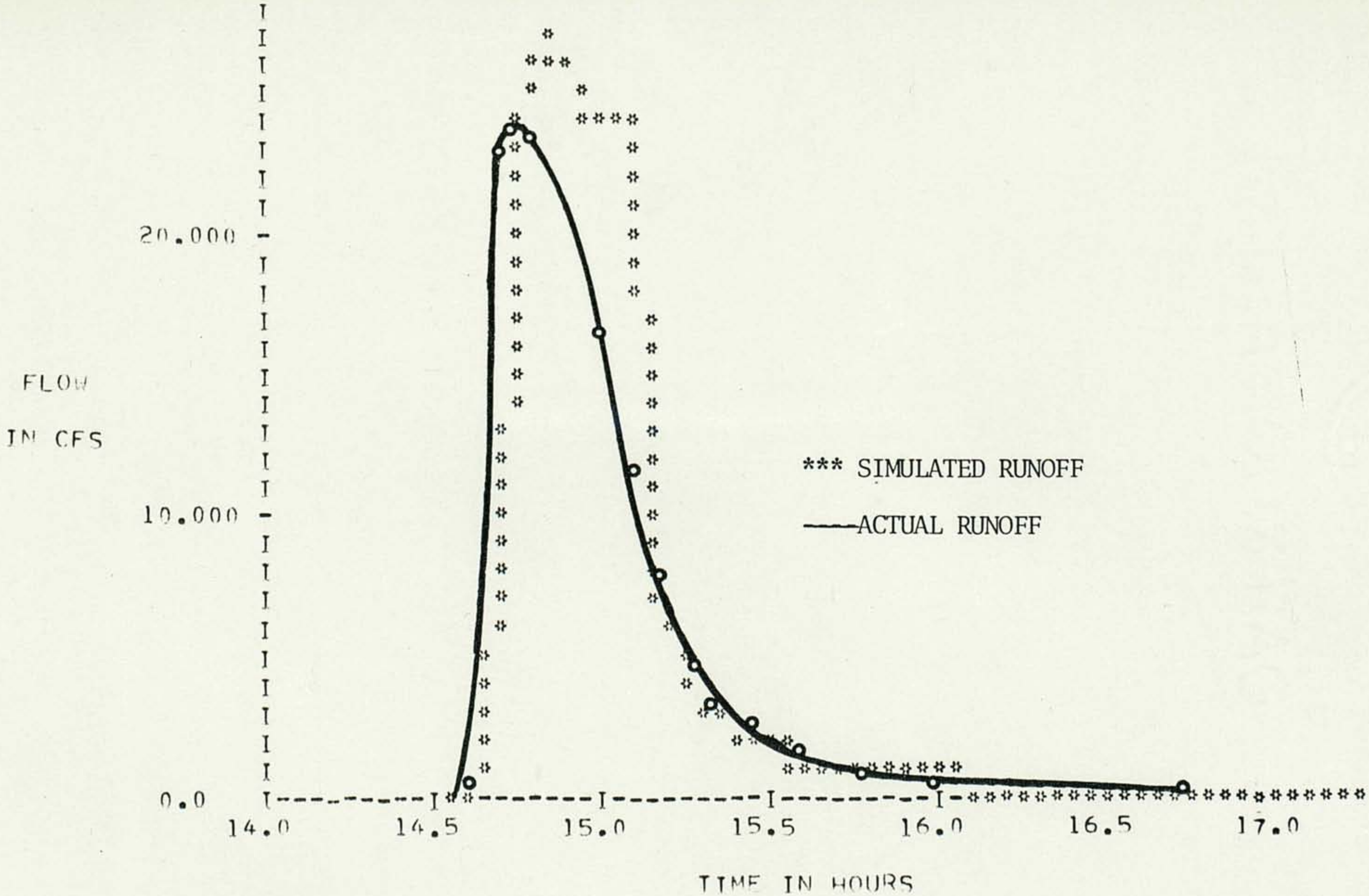


Fig. 27. Actual and Simulated Runoff, Calibration Storm of May 10, 1975

For the initial calibration run, the actual peak runoff flow was less than that produced by the Model simulation. The actual runoff occurred slightly quicker than the simulated runoff. Total actual runoff volume was about 40,050 cubic feet. Actual surface storage and infiltration was approximately 8,750 cubic feet and computed infiltration and surface storage was 4,856 cubic feet.

Actual runoff volume was calculated from the hydrograph plotted from the flow data. The total actual rainfall falling on the study area was calculated as:

$$0.48\ (1/12)\ (43{,}560)\ (28) = 48{,}800 \text{ cubic feet.}$$

This equation is simply the total rainfall (inches) multiplied by 1 foot per 12 inches, multiplied by 43,560 square feet per acre, and finally multiplied by the study area of 28 acres. This yields 48,800 cubic feet of water available for runoff. The Model simulation calculates this volume of water as being 48,990 cubic feet.

The simulation of the calibration storm contained what were considered small errors in the quantity portion. Approximately 4,000 cubic feet more runoff was produced in the Model simulation. The simulated peak runoff was slightly more and the simulated runoff occurred a few minutes later than in actuality. In order to decrease the simulated runoff volume and increase surface storage in the impervious areas, it was decided to increase the impervious area surface storage from 0.074 inches to 0.130 inches. Increasing this factor would yield the storage of the 4,000 cubic feet of excess simulated runoff.

The shifting of the simulation hydrograph to the left in order to coincide with the actual hydrograph was another problem. It was

felt that this could be accomplished by decreasing the coefficient of resistance for the impervious area. The value currently being used was 0.013. This value was decreased to 0.012. The amount of time that the hydrograph needed to be moved to the left was about 4.0 minutes. Changing the resistance factor will shift the hydrograph a certain amount which will possibly be the amount of time desired.

The simulated runoff data which was utilized by the computer in plotting the runoff hydrograph is shown in Table 22. Peak simulated runoff is 26.491 cfs, the actual peak runoff occurring is 23.870 cfs. The difference being about 2.62 cfs. After adjustment of the coefficients discussed previously, another calibration run was undertaken. This run was very successful with actual runoff volume being only 66 cubic feet less than the simulated runoff volume. The results of this Model run are given in Chapter VII. The quantity calibration was considered accurate and simulation of the remaining storms was undertaken.

Quality Calibration of the SWMM Model

The quality simulation of the SWMM Model depends on several factors which were discussed previously. Twenty-five dry weather days were utilized in the calibration, the dry weather days being the preceding period of time in which there was a cumulative total of less than 1.0 inches of rainfall. A BOD concentration of 25.0 mg/l was utilized for the catchbasin stored volume of water. These were essentially the only quality parameters varied in the Model simulation.

The BOD concentration chosen was $\frac{1}{4}$ of the suggested value of 100.0 mg/l. However, initial runs with 100.0 mg/l appeared to yield high values of BOD concentration through the storm sewer system. Ini-

TABLE 22

STORMWATER RUNOFF DATA FOR STORM OF MAY 10, 1975

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
233	0.267	0.267	0.926	4.240	9.463	16.058	24.308	26.300	26.437	26.491
	25.607	24.561	24.233	24.105	24.187	24.384	24.289	14.727	7.569	6.471
	5.032	4.065	3.120	2.873	2.574	2.288	2.105	1.958	1.779	1.619
	1.424	1.284	1.145	1.045	0.958	0.884	0.819	0.763	0.717	0.675
	0.640	0.607	0.579	0.553	0.531	0.510	0.493	0.476	0.462	0.449
	0.438	0.427	0.417	0.408	0.399	0.391	0.384	0.377	0.371	0.368
	0.363	0.355	0.354	0.350	0.343	0.342	0.337	0.334	0.331	0.328
	0.326	0.323	0.321	0.318	0.317	0.314	0.313	0.311	0.309	0.308
	0.306	0.305	0.303	0.302	0.301	0.300	0.299	0.298	0.298	0.297
	0.296	0.295	0.294	0.293	0.292	0.291	0.290	0.289	0.289	0.288

tial calibration therefore was undertaken with a concentration of 25.0 mg/l.

Table 23 is the pollutant concentrations calculated by the Model for the test manhole. The table is read from left to right and then down the rows as time progresses. Each value in the table represents the concentration of each pollutant occurring at a particular instant of time. Each time step is of two minutes with a total of 100 time steps. This yields an overall simulation time of 200 minutes or 3 hours and 20 minutes.

This data is taken by the Model, converted to flow rates and plotted for the duration of the pollutant flow. Figures 28 and 29 are the pollutographs of BOD and Suspended solids, respectively. The peak pollutant flows occur during the peak runoff flows and decay at about the same time. It should be mentioned that the infiltration flow is included in the runoff hydrograph. However, the infiltration is assumed by the Model to contain no pollutants whatsoever. The pollutant tables and pollutographs are the amounts contained in the runoff flow only. Table 24 contains the pollutant flow data which is calculated by the Model and plotted in Figures 28 and 29.

Composite and grab samples were obtained of the actual runoff occurring for the storm of May 10, 1975. The grab samples were analyzed and a comparison made with the simulated pollutant data. The grab samples were analyzed for BOD (5-day) and suspended solids. The following is the results of the grab samples:

Time Taken	BOD_5 (mg/l)	SS (mg/l)
1445	37.20*	372.0
1500	8.85*	112.0
1530	6.70*	26.0

TABLE 23

POLLUTANT CONCENTRATIONS FOR TEST SITE MANHOLE

SELECTED OUTFLOW POLLUTOGRAPHS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
					*** BOD IN MG/L ****					
233	0.0	0.0	18.525	46.415	49.913	45.743	35.294	23.093	21.366	17.154
	15.355	17.193	19.312	23.626	28.245	33.064	39.962	27.612	53.411	45.217
	45.811	39.887	40.155	30.013	34.851	22.424	27.459	26.622	15.265	29.655
	7.625	32.513	0.138	30.780	0.097	22.434	0.075	16.423	0.061	12.168
	0.051	9.185	0.000	7.154	0.000	5.692	0.000	4.615	0.002	3.808
	0.102	3.130	0.181	2.569	0.203	2.104	0.230	1.723	0.266	1.411
	0.314	1.193	0.330	0.994	0.355	0.858	0.346	0.748	0.289	0.647
	0.210	0.545	0.143	0.454	0.083	0.367	0.017	0.283	0.000	0.162
	0.000	0.106	0.013	0.084	0.024	0.069	0.029	0.059	0.026	0.051
	0.000	0.039	0.000	0.004	0.000	0.000	0.000	0.0	0.000	0.0
					*** SUSPENDED SOLIDS IN MG/L ***					
233	0.0	0.0	135.566	658.380	685.981	574.569	378.351	199.118	154.018	89.687
	66.689	98.545	154.410	242.090	364.417	504.434	688.289	528.681	919.736	884.811
	773.929	807.935	648.493	616.549	549.979	460.571	426.740	359.731	340.906	343.333
	269.766	408.672	75.693	429.090	0.511	342.849	0.282	238.100	0.166	165.107
	0.102	115.976	0.0	82.709	0.000	59.712	0.000	43.580	0.000	32.195
	0.000	24.216	0.000	18.357	0.000	13.967	0.000	10.565	0.000	8.049
	0.081	6.357	0.170	4.927	0.311	3.897	0.389	3.107	0.245	2.449
	0.082	1.896	0.000	1.429	0.000	1.034	0.000	0.713	0.000	0.419
	0.000	0.297	0.020	0.229	0.037	0.183	0.045	0.149	0.047	0.123
	0.000	0.076	0.000	0.015	0.000	0.006	0.001	0.005	0.001	0.004

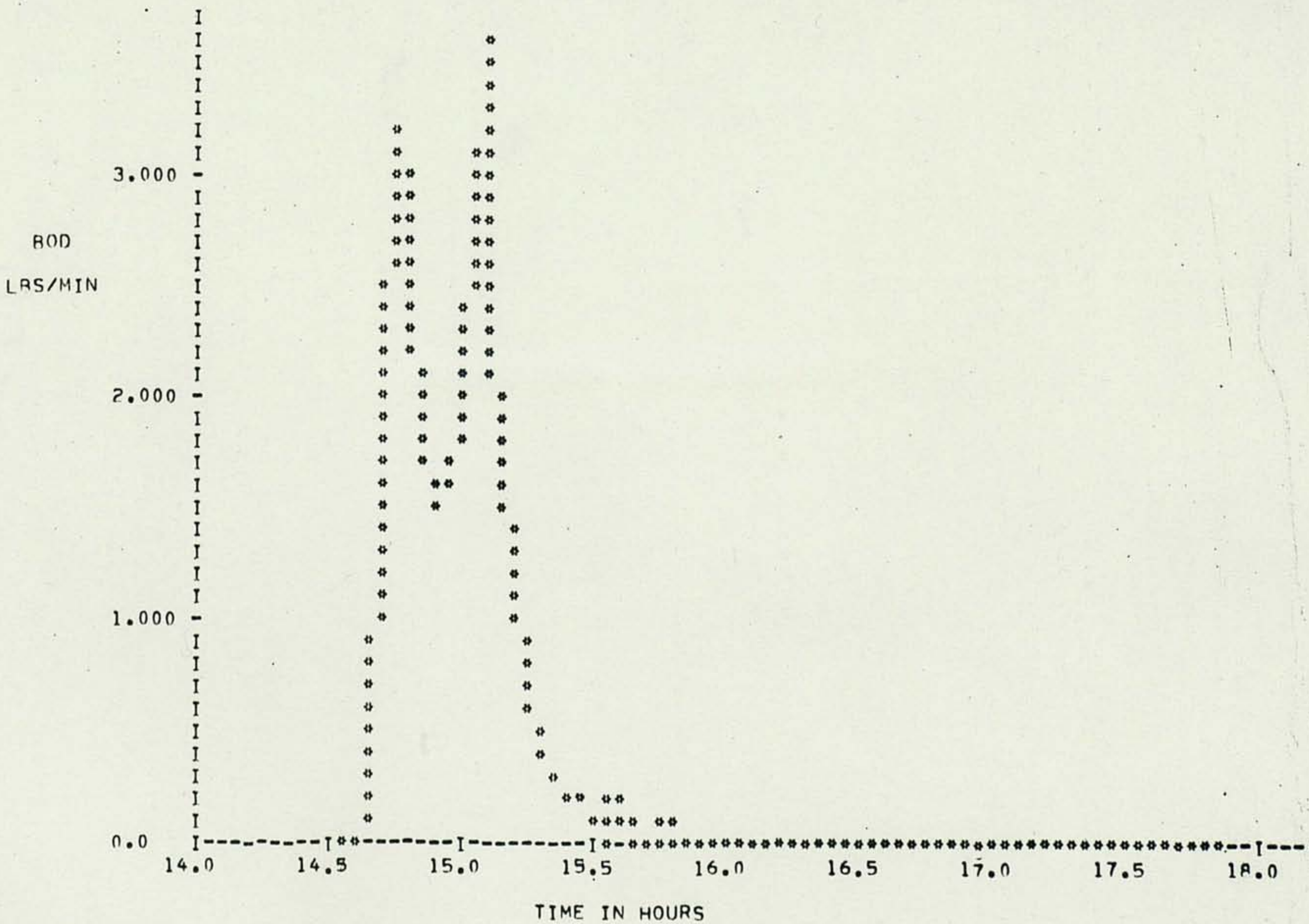


Fig. 28. 5-Day BOD Flow (lb/min) Versus Time

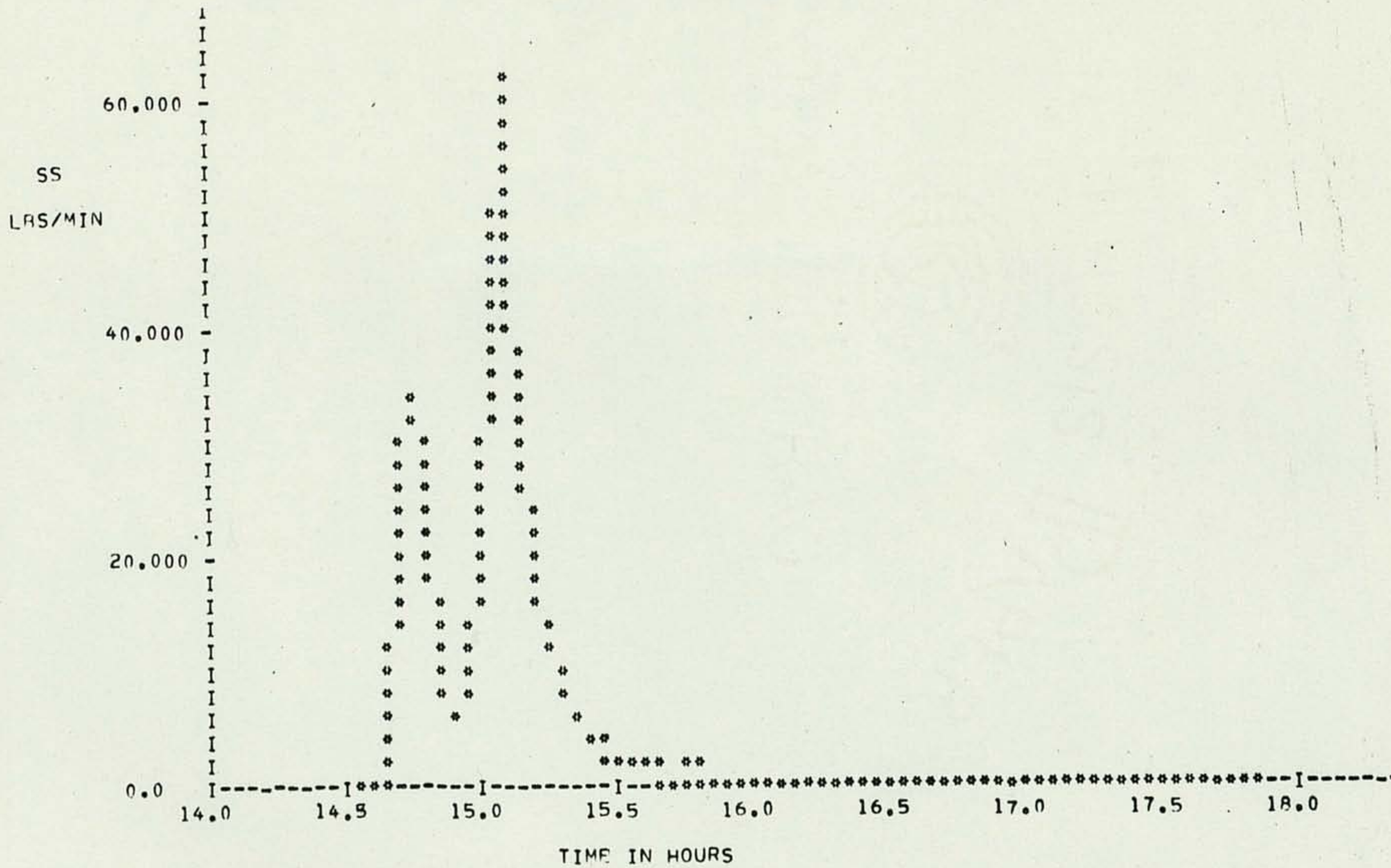


Fig. 29. Suspended Solids Flow (lb/min) Versus Time

TABLE 24

POLLUTANT FLOW DATA (lb/min) FOR STORM OF MAY 10, 1975

SELECTED OUTFLOW POLLUTOGRAPHS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
				*** BOD IN LBS/MIN ***						
233	0.0	0.0	0.064	0.736	1.766	2.746	3.207	2.270	2.112	1.699
	1.470	1.579	1.750	2.129	2.554	3.014	3.629	1.520	1.511	1.094
	0.862	0.606	0.468	0.322	0.335	0.192	0.216	0.195	0.102	0.179
	0.041	0.156	0.001	0.120	0.000	0.074	0.000	0.047	0.000	0.031
	0.000	0.021	0.000	0.015	0.000	0.011	0.000	0.008	0.000	0.006
	0.000	0.005	0.000	0.004	0.000	0.003	0.000	0.002	0.000	0.002
	0.000	0.002	0.000	0.001	0.000	0.001	0.000	0.001	0.000	0.001
	0.000	0.001	0.000	0.001	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.0	0.000	0.0
				** SUSPENDED SOLIDS IN LBS/MIN **						
233	0.0	0.0	0.469	10.437	24.267	34.492	34.381	19.577	15.221	8.882
	6.384	9.048	13.988	21.815	32.951	45.982	62.497	29.106	26.025	21.405
	14.560	12.277	7.565	6.621	5.292	3.940	3.358	2.633	2.268	2.077
	1.436	1.962	0.324	1.677	0.002	1.133	0.001	0.679	0.000	0.417
	0.000	0.263	0.0	0.171	0.000	0.114	0.000	0.078	0.000	0.054
	0.000	0.039	0.000	0.028	0.000	0.020	0.000	0.015	0.000	0.011
	0.000	0.008	0.000	0.006	0.000	0.005	0.000	0.004	0.000	0.003
	0.000	0.002	0.000	0.002	0.000	0.001	0.000	0.001	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

The asterisk indicates the samples tested twice; the number represents the average of these two tests.

The initial simulation run values contained in Table 23 for the above times were as follows:

Time	BOD_5 (mg/l)	SS (mg/l)
1445	27.00	289.50
1500	28.24	364.41
1530	29.65	343.33

After adjustment of the surface storage and surface resistance coefficients, another simulation was undertaken. The results of the second quality simulation were as follows:

Time	BOD_5 (mg/l)	SS (mg/l)
1445	39.31	467.73
1500	33.54	456.17
1530	40.32	753.51

The actual BOD_5 and suspended solids concentrations compare reasonably only at 1445. Actual BOD_5 concentration at 1445 is found to be 37.20 mg/l and the second simulation run had a concentration of 39.31 mg/l. Suspended solids (actual) at 1445 was about 372.00 mg/l and the Model had a simulated value of 467.73 mg/l.

The actual and simulated values for BOD_5 and suspended solids at 1500 and 1530 did not compare at all. There could be several reasons, most dealing with the SWMM Model. The simulated pollutant loadings are based on research done in Chicago. If pollutant loadings were utilized that were from the Orlando area, better results would probably be obtained. There are also two methods available to SWMM users for the calculation and simulation of suspended solids. One produces high initial suspended solids concentrations and terminates quicker than the other method. The other method produces initial concentrations of about

half the first method and will continue to produce suspended solids when the first method has ceased to do so. The first method is the one utilized in this study. It has initial and terminating times comparable to what is actually occurring but the concentrations are too high.

The simulated BOD_5 concentrations were also higher than the measured concentrations. The reason could be that various contaminants such as heavy metals are interfering in the measured samples. These would tend to cause measured BOD_5 concentrations to be questioned.

The quality simulation yielded results which were expected. Until considerable research and use of the Model is completed, inaccuracies will result in the quality simulation. For this study, the results obtained were considered successful. To perform extensive quality calibrations, several storms should be sampled and pollutant loadings for each land use developed for the study area. This would also require large amounts of computer time. The data cards required for the simulation of the storm of May 10 are given in Appendix A.

CHAPTER VII

ANALYSIS AND RESULTS OF STORM SIMULATIONS USING THE CALIBRATED SWMM MODEL

Storm of May 10, 1975, Quantity Analysis and Results

A total of four rainfall events occurred in the period from May 5 to May 12, 1975. Of these four, the third event, which took place the morning of May 10, was not simulated. The reasons being the small quantity of rainfall and it was not felt that the storm was constant over the entire commercial study area. The remaining three storms were simulated and the results analyzed.

The afternoon storm of May 10 was utilized for calibration purposes. Upon completion of the calibration, the storm was simulated with the SWMM Model. The rainfall data was input to the Model in one minute intervals. The resulting rainfall hyetograph was illustrated previously in Figure 26. Peak intensity was 1.89 inches per hour with a duration of seven minutes. The storm was simulated over 100 time steps of 2 minutes each or approximately 3 hours and 20 minutes.

The actual rainfall volume was calculated as 48,800 cubic feet; the simulated rainfall volume was 48,991 cubic feet. A difference of only 191 cubic feet. The measured runoff volume was 40,050 cubic feet; the calibrated Model runoff volume was 40,116 cubic feet. The error between the actual and simulated was very small, only 66 cubic feet. This was approximately a .16% error in runoff volume. Figure 30 illu-

strates the simulated runoff and actual runoff hydrographs. Table 20 contains the data for the actual runoff hydrograph. Peak simulated runoff occurs at 1848 with a value of 27.82 cfs. Actual peak runoff occurs at about 1845 with a value of 23.87 cfs. Changing the coefficient of resistance from 0.013 to 0.012 resulted in shifting the runoff hydrograph 1 minute to the left as was desired. The initial simulated runoff occurs at almost the same time as the actual runoff. Even though the peaks do not correspond exactly, the beginnings and endings of the two hydrographs are approximately the same. Increasing the surface storage from 0.074 inches to 0.130 inches resulted in an increased storage of 3944 cubic feet. This was very close to the 4000 cubic feet increase which was desired.

The simulated infiltration and storage was 8819.0 cubic feet. The actual infiltration and storage was about 8750.0 cubic feet. Again, the difference in the actual and simulated values was very slight. Minor adjustments in the Model could be made but the accuracy was considered sufficient for simulation purposes.

The storm of May 10 was discussed previously when it was utilized for calibration. Two additional storms were simulated - those occurring on May 5 and May 12, 1975.

Storm of May 5, 1975, Quantity Analysis and Results

The storm of May 5 had a duration of 25 minutes and a total rainfall of 0.03 inches. The rainfall hyetograph was plotted by the computer and is shown in Figure 31. The rainfall data obtained from the rainfall recorder is in Table 25. From these data the following

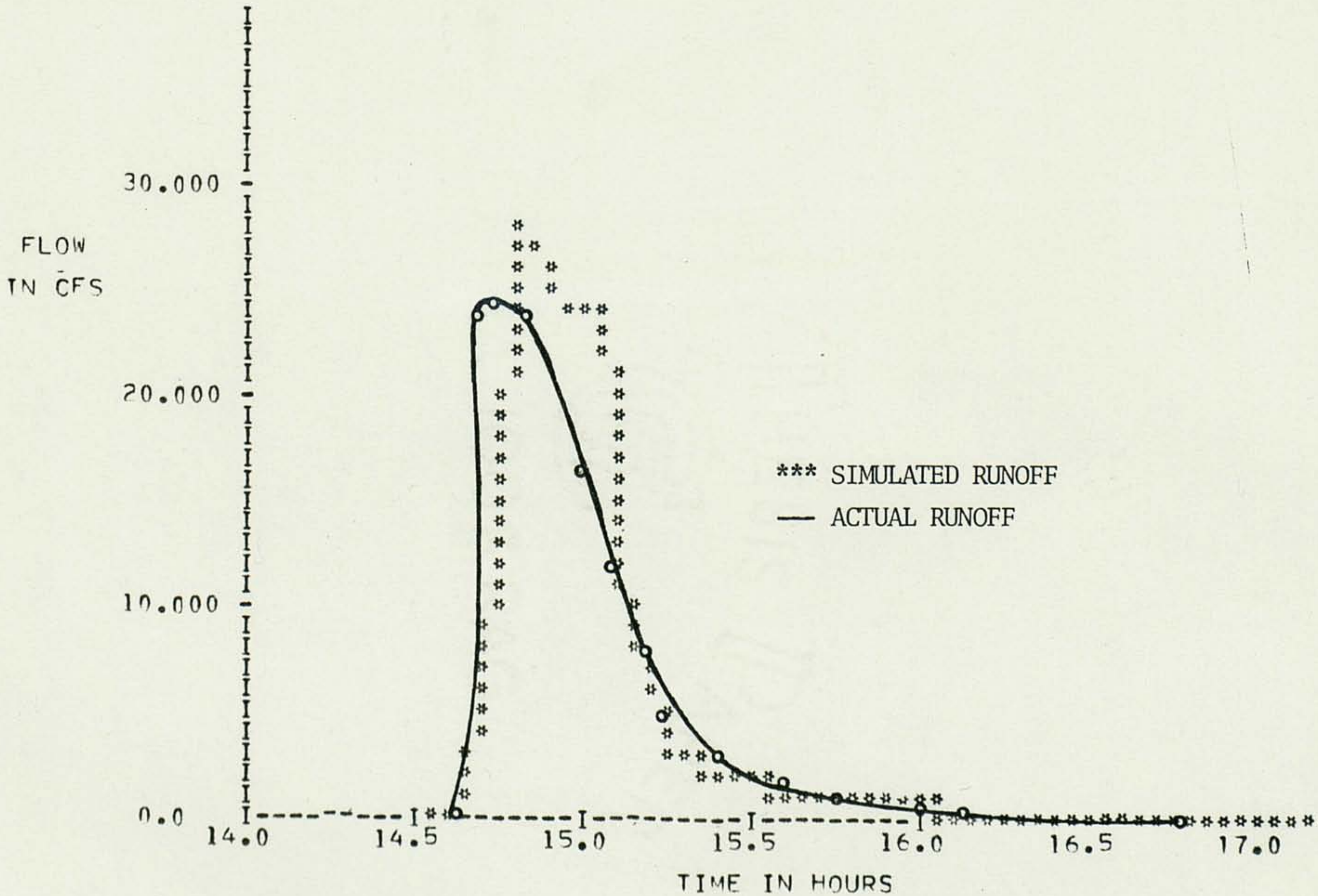


Fig. 30. Simulated and Actual Runoff Hydrograph for Storm of May 10, 1975

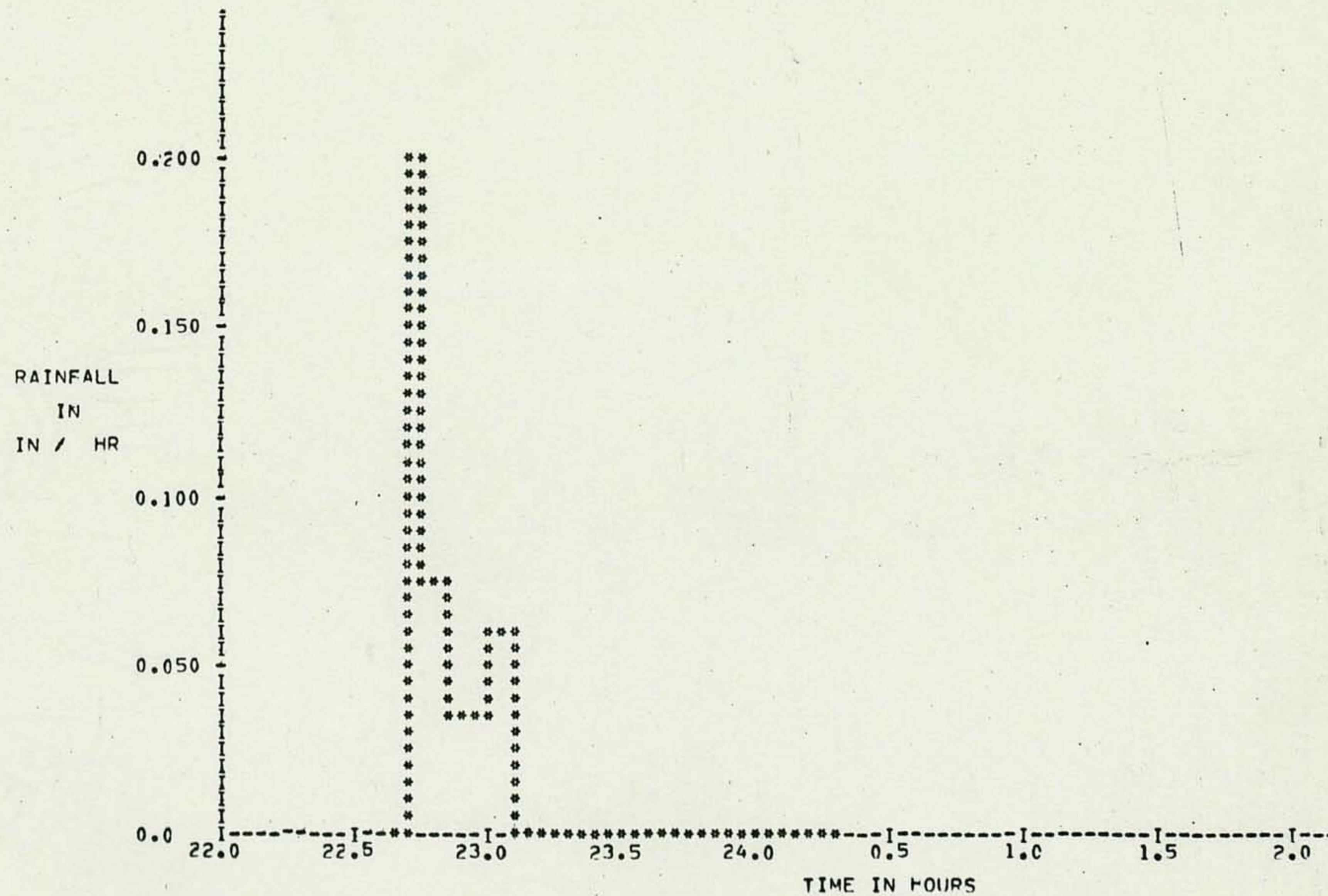


Fig. 31. Rainfall Hyetograph for Storm of May 5, 1975

TABLE 25

RAINFALL DATA FOR THE STORM OF MAY 5, 1975

Time Period (minutes)	Total Rainfall (inches)
2237 - 2240	0.010
2241 - 2245	0.020
2246 - 2256	0.025
2257 - 2303	0.030

rainfall intensities and durations can be calculated.

Intensity	Duration
0.150 in/hr	9.0 minutes
0.025 in/hr	10.0 minutes
0.042 in/hr	6.0 minutes

The above data was input to the SWMM Model in 1 minute intervals for a total time period of 25 minutes.

The actual flow data measured for the storm is contained in Table 26. Initial dry weather flow was approximately 0.268 cfs measured at 2233. Runoff occurred almost immediately after the storm start at 2238. Peak runoff flow was approximately 2.40 cfs at 2251. This is eighteen minutes after the start of the storm. The major portion of the runoff was between 2240 and 2310. This is a time period of only 30 minutes duration. The actual flow data is plotted on a different graph as then the computer-plotted simulated flow data. The actual hydrograph can be seen in Figure 32. The simulated hydrograph is in Figure 33. The actual hydrograph was not plotted on the simulated because of the large differences in the amounts of runoff and the computer scales of Figure 33. The storm was simulated for 100 time steps of two minutes each. A return to dry weather flow actually occurs about 0010 on the 6th of May. The simulation flow also returns to dry weather flow at approximately the same time.

The simulated peak runoff was only about 0.810 cfs while the actual was around 2.40 cfs. The total duration of runoff is approximately the same for both the actual and simulated hydrographs. Calculated runoff volume from the actual plotted hydrograph was 2310.00 cubic feet. The available quantity of water for runoff was 3050.00

TABLE 26

FLOW DATA, STORM OF MAY 5, 1975

Time	Depth (in)	Flow (cfs)
2233	1.75	0.268
2234	1.75	0.268
2238	2.50	0.477
2243	3.50	0.513
2248	5.00	2.125
2250	5.25	2.388
2255	4.60	1.866
2300	3.80	1.177
2305	3.25	0.889
2310	2.75	0.630
2315	2.60	0.485
2320	2.50	0.477
2325	2.25	0.420
2330	2.10	0.350
2335	2.00	0.310

Pipe Diameter = 24 inches
Slope = 1.0 ft/100 ft
Manning's n = 0.013

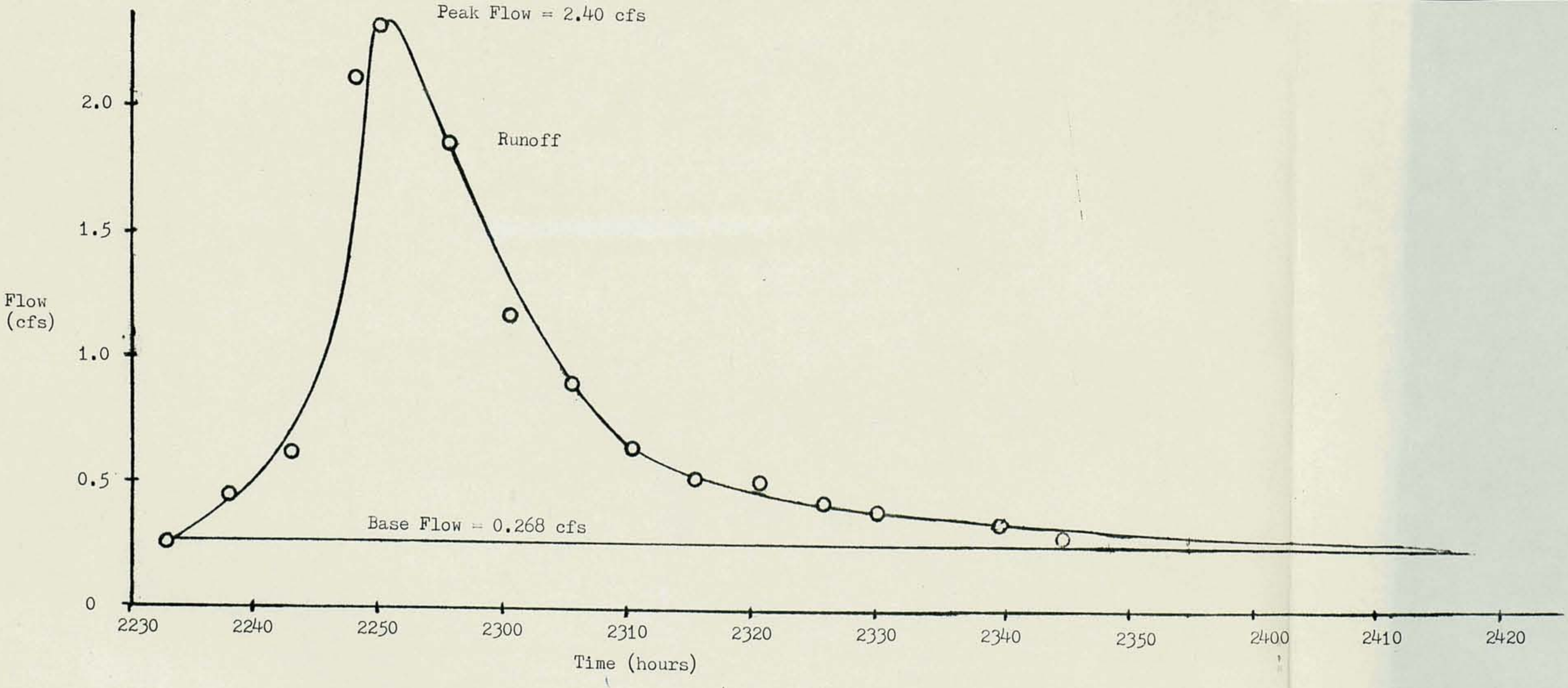


Fig. 32. Actual Runoff Hydrograph for the Storm of May 5, 1975

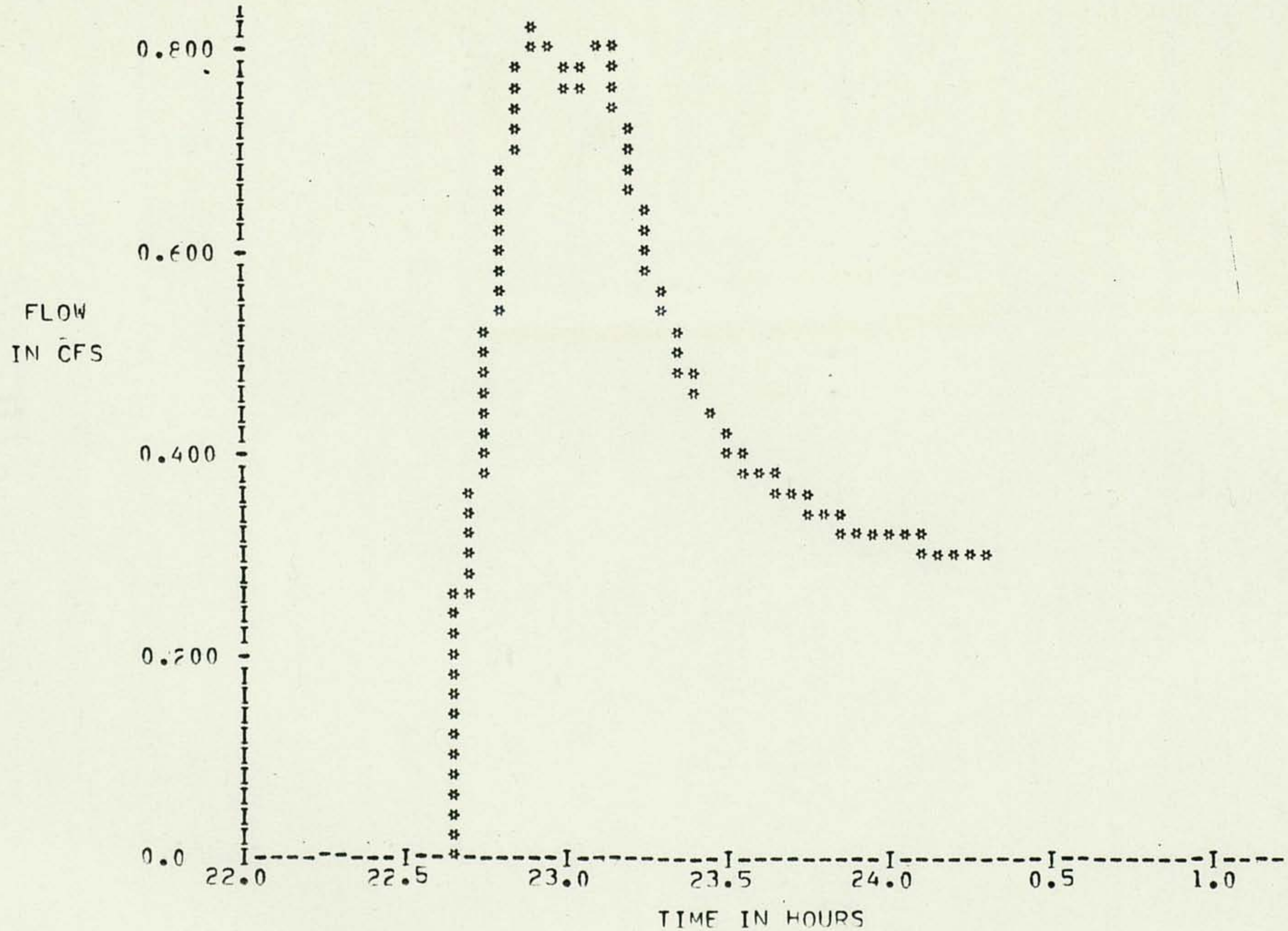


Fig. 33. Simulated Runoff Hydrograph for the Storm of May 5, 1975

cubic feet. Surface storage and infiltration was 740.00 cubic feet. The Model simulation calculated a total runoff volume of 1270.00 cubic feet and 1656.00 cubic feet for surface storage and infiltration.

The discrepancies in volumes which occurred for the storm of May 5 were fairly large. The actual volume of rainfall compares fairly well with the simulated volume of rainfall, 3050.00 to 3044.00 cubic feet, respectively. However, the calculated runoff volume is about 1040.00 cubic feet more than the simulated runoff volume. Also the simulated surface storage and infiltration volume is about 916.00 cubic feet more than the actual surface storage and infiltration.

Actual runoff was about 76% of the actual rainfall volume and the remaining 24% was surface storage and infiltration. The Model simulation calculated a simulated runoff to rainfall percentage of about 42% and the remaining 58% was simulated surface storage and infiltration. The Model of course takes the rainfall as occurring uniformly over the entire commercial study area. In actuality, this may or may not be the case. For large, intense storms the rainfall is more than likely uniform where small areas such as the study area are involved. The brief shower of May 5 which had a total rainfall of 0.03 inches at the test site was apparently not uniform over the entire study area. It was decided, therefore, that this probably occurred during the storm of May 5 and accounts for the discrepancies between the actual and simulated volumes.

Storm of May 12, 1975, Quantity Analysis and Results

The storm of May 12 had a duration of 1 hour and 25 minutes and

a total rainfall of 0.70 inches. The rainfall data from the continuous rainfall recorder is in Table 27. These data are then converted to the rainfall intensities and durations also shown in Table 27. The intensities were input to the Model in one minute intervals. The storm was simulated for 150 time steps of two minutes each. This is a total simulation time of 300 minutes or 5 hours. The rainfall hyetograph plotted by the computer is contained in Figure 34.

The actual flow data is listed in Table 28. Table 29 contains the estimated flow in the conduit that drains a small area of the study area. It is felt that these estimates are probably lower than what actually occurred. Peak flow occurs at approximately 1930 with a value of 23.633 cfs. Smaller peaks also occur at 2000 and 1910. The actual flow data is plotted on the simulated runoff data of Figure 35. The simulated runoff peaks at 26.96 cfs at about 1925. The actual measured peak occurs at 23.63 cfs at around 1930. The actual runoff occurs at about the same time as the start of the simulated runoff. The ending of the simulated also occurs at about the same time as the actual.

The actual rainfall available for runoff, surface storage and infiltration was calculated as 71,150 cubic feet. The Model simulation calculates the available rainfall as 73,280 cubic feet. A difference of about 2130 cubic feet results. The actual runoff volume calculated from the hydrograph, was 63,500 cubic feet. The simulated runoff volume was 63,020 cubic feet. A discrepancy of 480 cubic feet did occur. This was a very small error in the runoff volumes when compared to the amount of runoff. The Model calculated a surface storage and infiltration volume of 10,120 cubic feet. Actual surface storage

TABLE 27

RAINFALL DATA FOR THE STORM OF MAY 12, 1975

Time Period (minutes)	Total Rainfall (inches)
1845 - 1900	0.15
1901 - 1915	0.28
1915 - 1923	0.43
1924 - 1930	0.47
1931 - 1945	0.47
1946 - 2000	0.67
2001 - 2010	0.70

Rainfall Intensity (in/hr)	Duration (minutes)
0.600	15
0.520	15
1.125	8
0.343	7
0.000	15
0.800	15
0.180	10

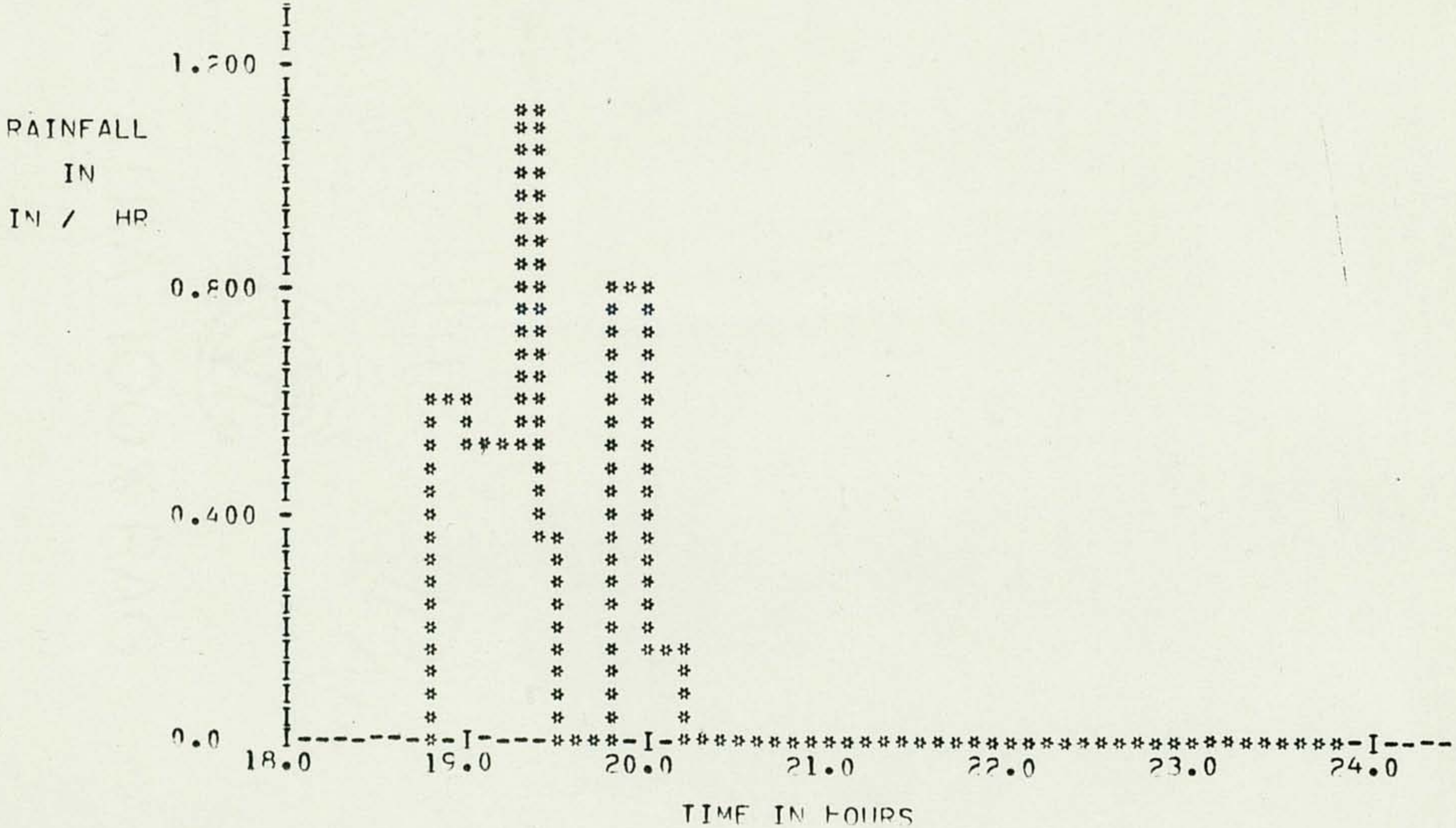


Fig. 34. Rainfall Hyetograph for Storm of May 12, 1975

TABLE 28

FLOW DATA FOR STORM OF MAY 12, 1975

Time	Depth (in)	Flow (cfs)	Total Flow (cfs)
1848	1.75	0.268	0.268
1850	2.00	0.310	0.460
1855	2.50	0.477	0.727
1857	3.75	1.150	1.400
1900	10.50	8.914	9.414
1905	13.50	13.775	14.425
1910	14.00	14.507	15.257
1913	12.00	11.300	11.880
1915	10.00	8.206	8.656
1920	11.25	10.062	10.712
1925	13.50	13.775	14.775
1930	19.00	21.883	23.633
1940	8.25	6.100	7.100
1945	6.00	3.010	3.760
1950	10.00	8.206	9.106
1955	15.00	16.160	17.410
2000	17.50	19.98	21.480
2004	14.50	15.044	16.294
2020	5.00	2.120	2.620
2040	3.25	0.889	0.889
2050	2.75	0.630	0.630
2100	2.40	0.452	0.452
2110	2.00	0.310	0.310

TABLE 29

FLOW DATA, STORM OF MAY 12, 1975

Time	Flow (cfs)
1848	0.00
1850	0.15
1855	0.25
1900	0.50
1905	0.65
1910	0.75
1913	0.55
1915	0.45
1920	0.65
1925	1.00
1930	1.75
1940	1.00
1945	0.75
1950	0.90
1955	1.25
2000	1.50
2004	1.25
2020	0.50
2040	0.00

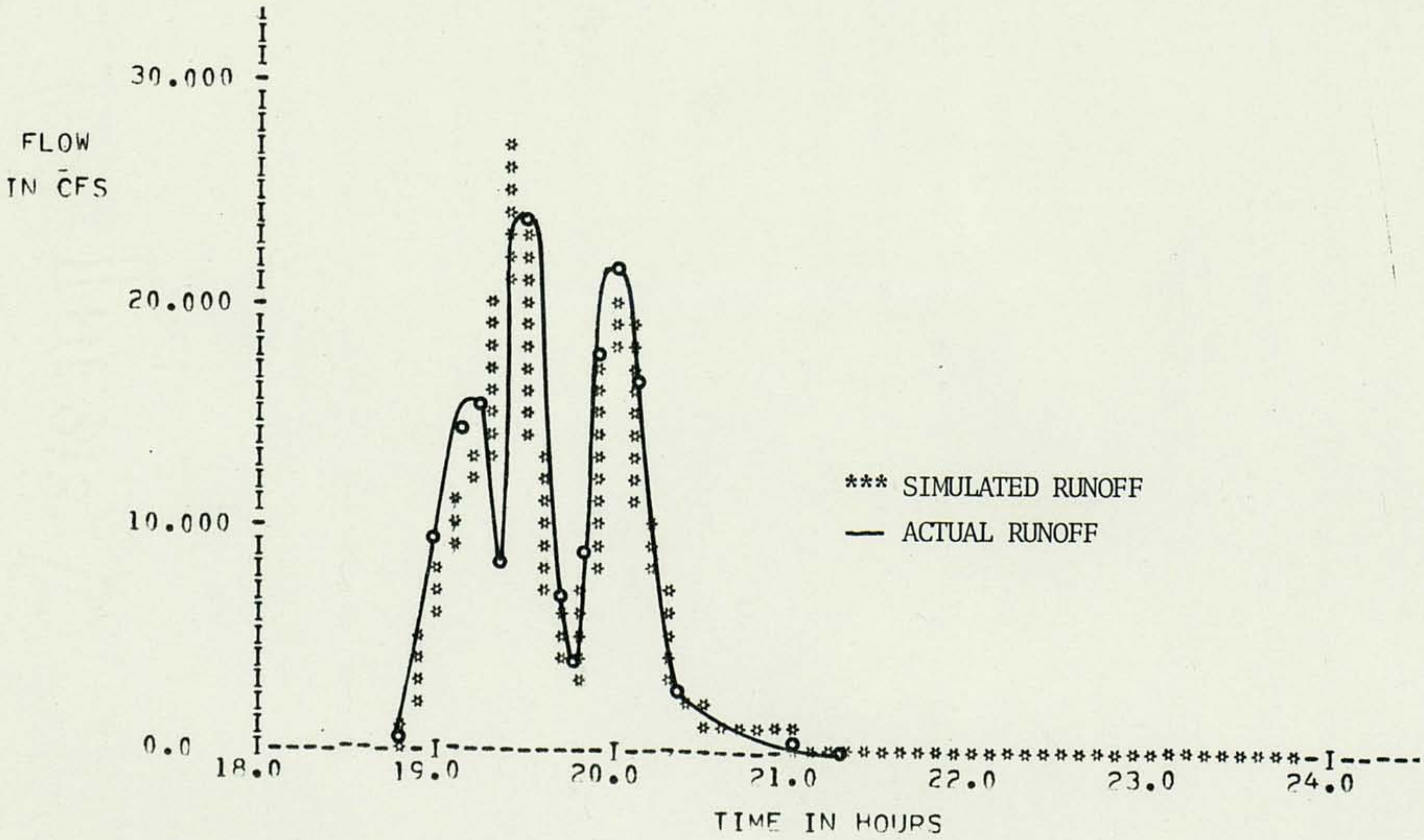


Fig. 35. Actual and Simulated Runoff Hydrographs for the Storm of May 12, 1975

and infiltration was calculated as 7650 cubic feet. Most of the error can be accounted for as the difference in available rainfall volumes of 2130 cubic feet. The errors are considered tolerable for simulation purposes. The simulated quality results will now be discussed.

Storm of May 10, 1975, Quality Results

The quality results of the stormwater simulations were considered fairly accurate for initial values of BOD_5 and suspended solids. For the storm of May 10, peak loading rates of BOD_5 and suspended solids were 3.627 and 60.437 lbs/min respectively. These peaks occurred at 1504 which was during the peak runoff flow. Figures 36 and 37 are the BOD and suspended solids pollutographs respectively for the storm of May 10.

It can be observed from the figures that two peaks occur for both pollutants, the smaller peak prior to the larger. This is due to the rainfall intensities of the storm. The rainfall intensity decreased at about 1450 and then increased around 1500. Figures 36 and 37 are the simulation run results of May 10, not the calibration run results. The pollutant flows end at approximately the same time as the runoff flow. This would be expected, since the dry weather flow was simulated as containing no pollutants. The total simulated pounds of 5-day BOD output was 77.99 and the total simulated pounds of suspended solids output was 992.85. These values are the simulated outputs from the test manhole 233.

The SWMM Model only routes BOD_5, suspended solids, and coliform bacteria through the storm sewer system. Calibration depends upon these parameters. However, for comparison purposes with the dry

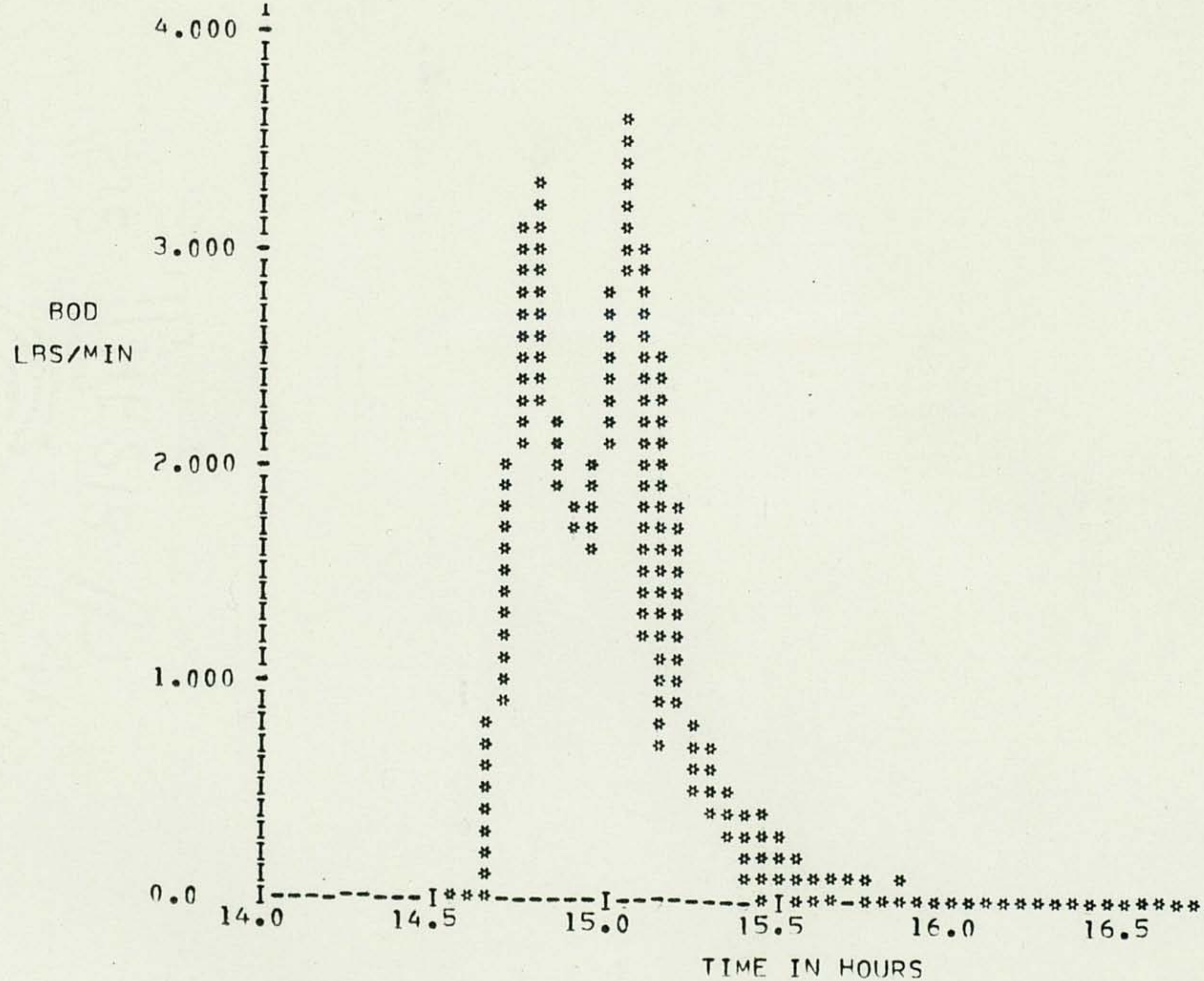


Fig. 36. BOD Pollutograph for Storm of May 10, 1975

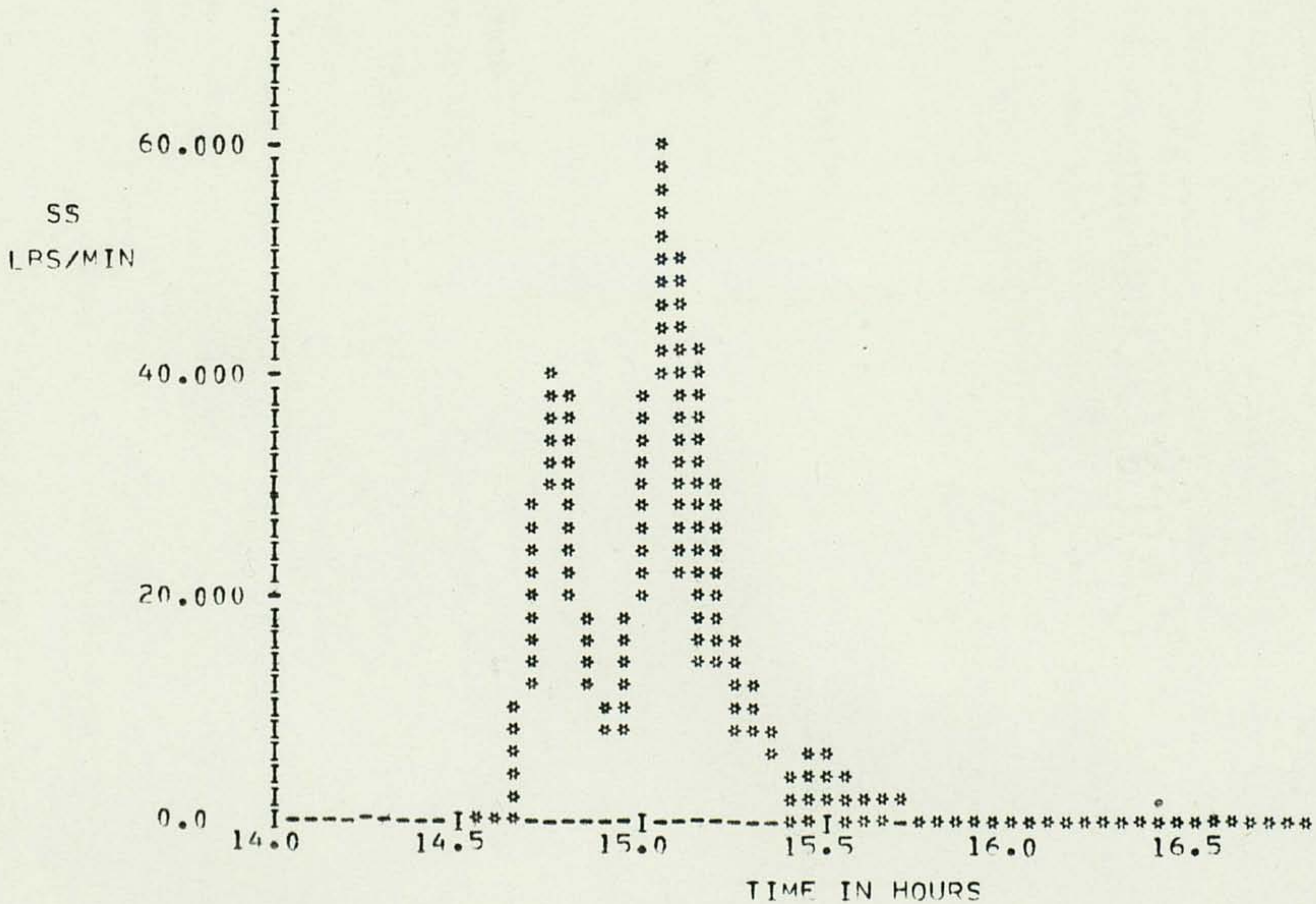


Fig. 37. Suspended Solids Pollutograph for Storm of May 10, 1975

weather flow quality parameters, the stormwater runoff was tested for additional quality parameters. These included phosphorus, total Kjedahl nitrogen and total solids. Table 30 lists these results for the storm of May 10. The grab samples in Table 30 were taken during the storm when peak runoff flows were occurring. The ½-hour composites were taken during the storm and the 3-hour composite was taken of the dry weather flow before the storm. The 1-hour composite was taken after the storm when flow had returned to dry weather flow.

Total Kjedahl nitrogen concentration was approximately ten times as much in the stormwater runoff as the dry weather flow. Total nitrogen of the stormwater runoff was about four times as much as that of the dry weather flow, 1.26 mg/l to 0.30 mg/l. Total phosphorus of the runoff was also slightly higher than the dry weather flow concentrations. Alkalinity and hardness of the runoff, expressed as mg/l as $CaCO_3$, was less than that in the dry weather flow. The dry weather flow had an alkalinity and hardness value of 149.9 and 150.4 mg/l respectively. The runoff had minimum values of 38.9 and 32.6 mg/l of alkalinity and hardness respectively for the second half hour composite. The runoff grab sample of 1445 had a peak value of 5-day BOD of 37.2 mg/l, the dry weather flow had a measured 5-day BOD of 6.6 mg/l. The pH of the runoff was about one less than that of the dry weather flow. From these comparisons, stormwater runoff was considered to be detrimental to Lake Eola.

First-flush effects can be noted in the stormwater runoff. Quality parameters peaked early in the runoff period and then declined slowly throughout the remainder of the period. When runoff ceases,

TABLE 30

STORMWATER QUALITY, MAY 10, 1975

Type of Sample	Time Filled	pH	TS (mg/l)	SS (mg/l)	BOD	COD	Nitrogen			Phosphorus (mg/l)		Alkalinity (mg/l as $CaCO_3$)	Hardness (mg/l as $CaCO_3$)
							TKN	NO_3-N	Total	OP	TP		
Grab	1445	----	528	372	37.2	111.0	----	0.59	----	0.35	0.76	-----	-----
Grab	1500	----	146	112	8.8	47.1	----	0.58	----	0.12	0.20	-----	-----
Grab	1530	----	146	26	----	38.0	----	0.83	----	0.17	0.24	-----	-----
Composite (3 hrs)*	1400	8.14	---	---	6.6	-----	0.07	0.23	0.30	0.47	0.54	149.9	150.4
Composite (½ hr)	1507	7.54	---	---	----	-----	0.74	0.52	1.26	----	----	74.9	61.0
Composite (½ hr)	1537	7.38	174	---	----	-----	----	0.84	----	----	----	38.9	32.6
Composite (½ hr)	1607	7.43	180	---	----	-----	0.19	0.91	1.10	0.19	0.30	44.4	52.6
Composite (½ hr)	1637	7.46	---	---	----	-----	----	0.91	----	----	----	-----	61.0
Composite (1 hr)**	0037	8.23	266	---	----	-----	----	----	----	----	----	149.9	162.6

* Before storm start
** Filled after runoff ended

quality returns to that of the dry weather flow. Further quality data was also obtained for the storms of May 5 and May 12.

Storm of May 5, 1975, Quality Results

The pollutographs resulting from the storm of May 5 are shown in Figures 38 and 39. Figure 38 is the 5-day BOD flow rate in lbs/min and Figure 39 is the suspended solids flow rate in lbs/min. Both peaks occur at approximately the same time as peak runoff flow. Again, the pollutographs closely resemble the runoff hydrograph for the storm contained in Figure 33. Both the starts and endings of the pollutants corresponds to the runoff. Total simulated 5-day BOD output from the test manhole 233 was 1.92 pounds. Total simulated suspended solids output was 10.28 pounds.

The measured water quality constituents of the actual runoff are given in Table 31. The three grab samples were taken during the peak stormwater runoff flow period. For these samples inorganic and total organic carbon concentrations were also obtained. Total organic carbon is at its maximum at about the same time as peak runoff flow occurs. It then decreases and inorganic carbon increases until dry weather flow conditions are reached. Again, as with the storm of May 10, pH values of the runoff are lower than the dry weather flow. Alkalinity and hardness also decrease from dry weather flow values as before. Nitrogen concentrations increase during the runoff period and subsequently decrease afterwards. Peak total Kjedahl nitrogen was 3.87 mg/l at 2245 and had decreased to 1.86 mg/l at 2305. The total solids content of the runoff was about three times that of the dry weather

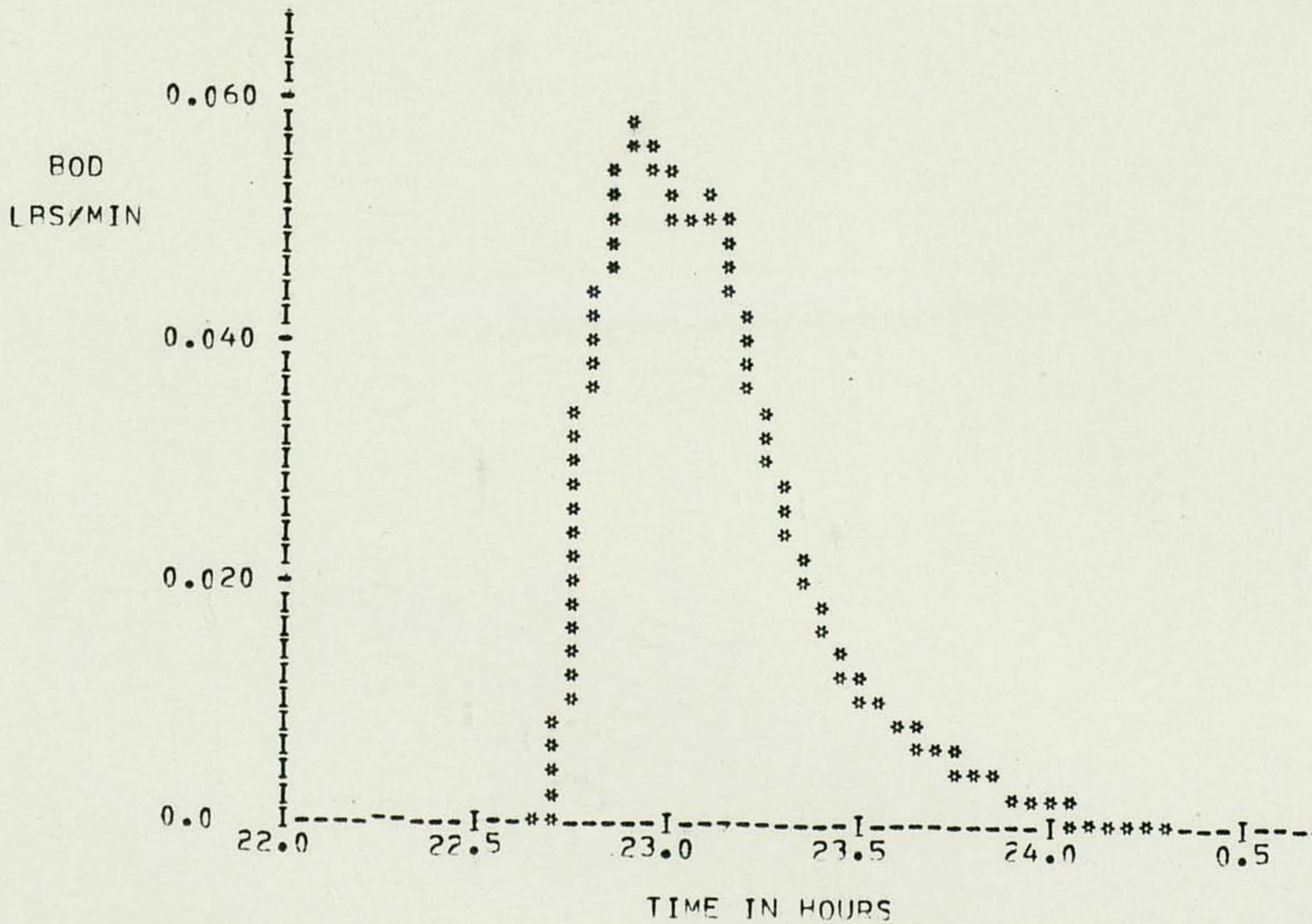


Fig. 38. BOD Pollutograph for Storm of May 5, 1975

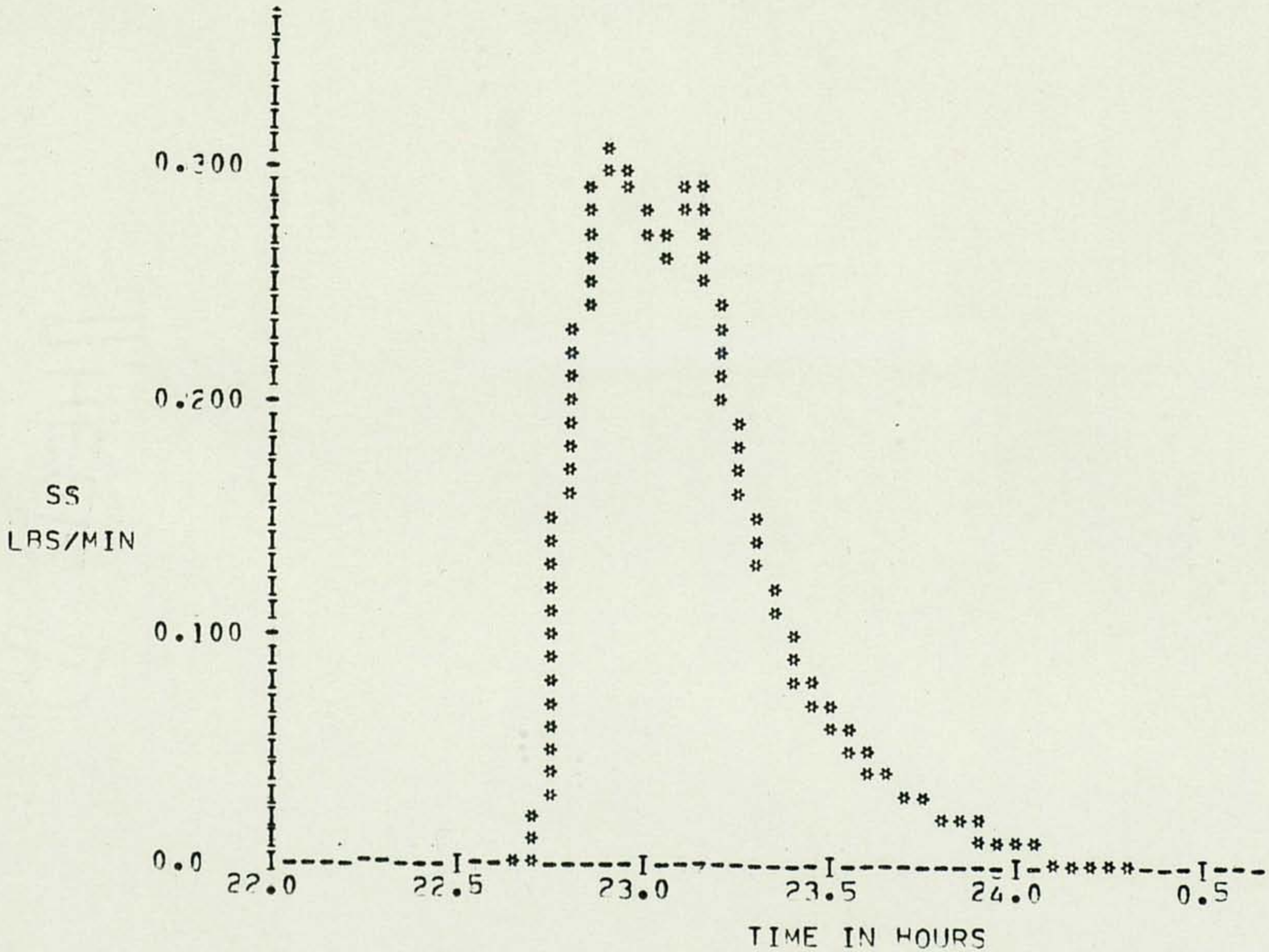


Fig. 39. Suspended Solids Pollutograph for Storm of May 5, 1975

TABLE 31

STORMWATER QUALITY, MAY 5, 1975

Type of Sample	Time Filled	pH	TS (mg/l)	SS (mg/l)	BOD	COD	Nitrogen (mg/l)			Phosphorus (mg/l)		Alkalinity (mg/l as $CaCO_3$)	Hardness (mg/l as $CaCO_3$)	Carbon (mg/l)	
							TKN	NO_3-N	Total	OP	TP			IC	TOC
Grab	2245*	7.02	910	---	---	----	3.87	1.31	5.18	0.59	0.76	52.7	130.1	14.3	146.0
Grab	2255*	7.02	650	---	---	----	3.11	1.24	4.35	0.70	0.92	52.7	101.8	9.9	133.0
Grab	2305*	7.25	372	---	---	----	1.86	1.11	2.97	0.38	0.50	55.5	93.5	9.6	98.0
Composite (1 hr)	2328*	7.71	308	---	---	----	----	0.88	----	0.36	0.60	97.1	128.0	21.1	30.5
Composite (1 hr)	0028**	8.11	294	1.3	7.9	----	----	0.60	----	0.38	0.64	130.4	160.5	28.2	10.8
Composite (1 hr)	0228**	8.12	290	1.0	4.0	----	----	0.33	----	0.39	0.60	147.1	160.5	29.5	9.4
Composite (1 hr)	0628**	8.36	---	---	3.2	14.0	----	0.15	----	0.39	0.60	149.9	158.2	31.5	1.0

*Filled during stormwater runoff occurence
**Filled after stormwater runoff occurence

flow, 910 mg/l compared to 290 mg/l respectively.

Comparing the various runoff concentrations for the storms of May 10 and May 5, correlations can be noted. Those in particular are alkalinity, hardness and pH values. Variations are due to the differences in the amount of rainfall, rainfall intensity and duration. The effects of these parameters on runoff quality have been discussed previously. The third storm, for which quality data was collected, occurred on the 12th of May.

Storm of May 12, 1975, Quality Results

The storm of May 12 was the largest sampled, having a total rainfall of 0.7 inches and a duration of one hour and 25 minutes. The pollotographs resulting from the Model simulation are contained in Figures 40 and 41. Figure 40 is the BOD pollutograph and Figure 41 is the suspended solids pollutograph. Maximum 5-day BOD flow rate was 5.744 lbs/min occurring at 1920. Maximum suspended solids flow was 99.498 lbs/min occurring at the same time. Total simulated suspended solids output from the test site manhole was 2309 pounds. The total simulated 5-day BOD output for the same manhole was 139 pounds.

The measured water quality parameters of the actual runoff are given in Table 32. All the samples taken were composites of ½-hour each, except for the final composite of 3-hours. Again, variations in water quality followed the same pattern as before. Slightly lower values for organic and total phosphorus were noted as compared to the values for the storm of May 10. A possible explanation would be that the storm of May 10 "washed" the study area of some of the phosphorus loadings contained on its surface. The storm of May 12 would then ex-

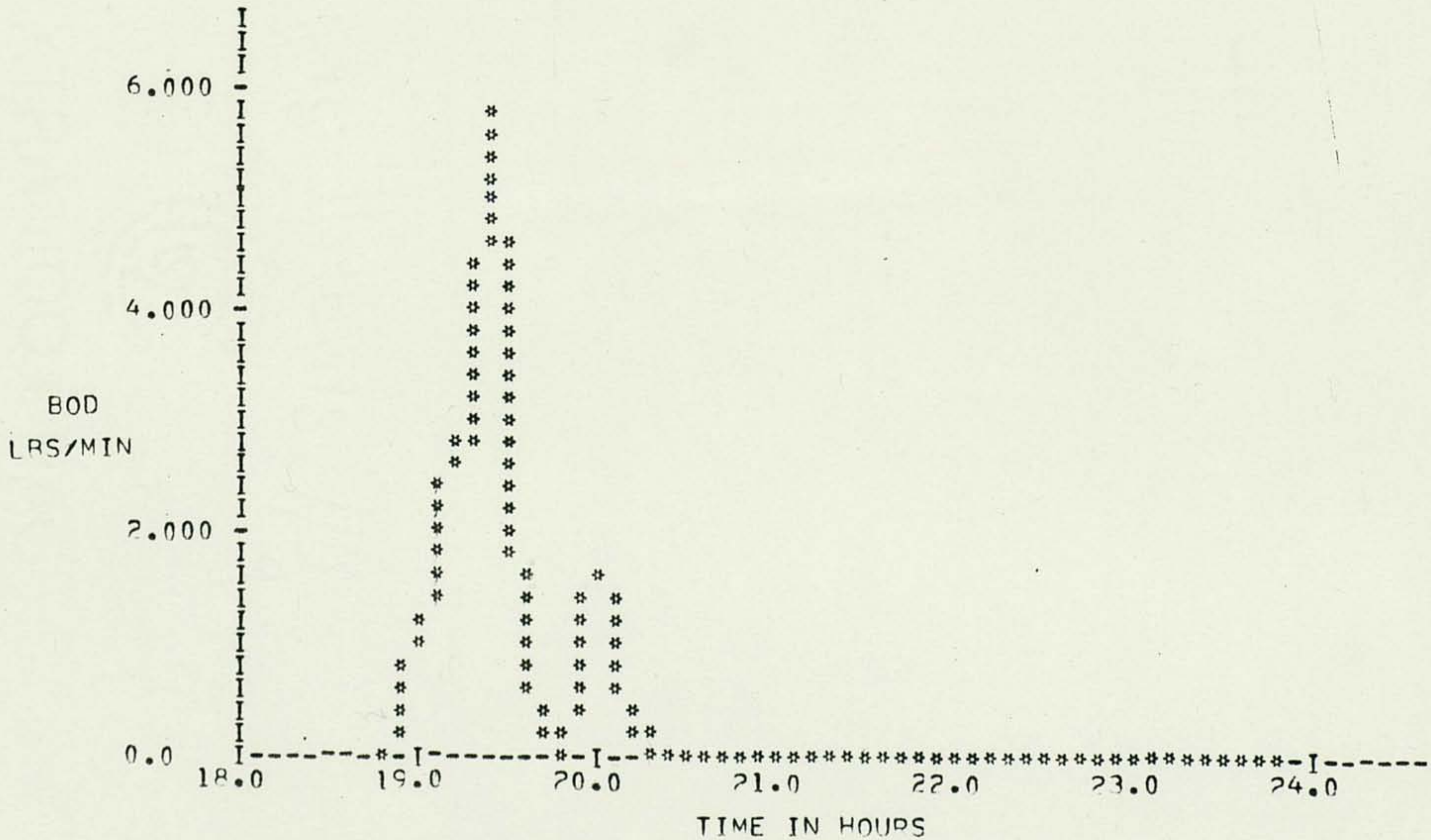


Fig. 40. BOD Pollutograph for Storm of May 12, 1975

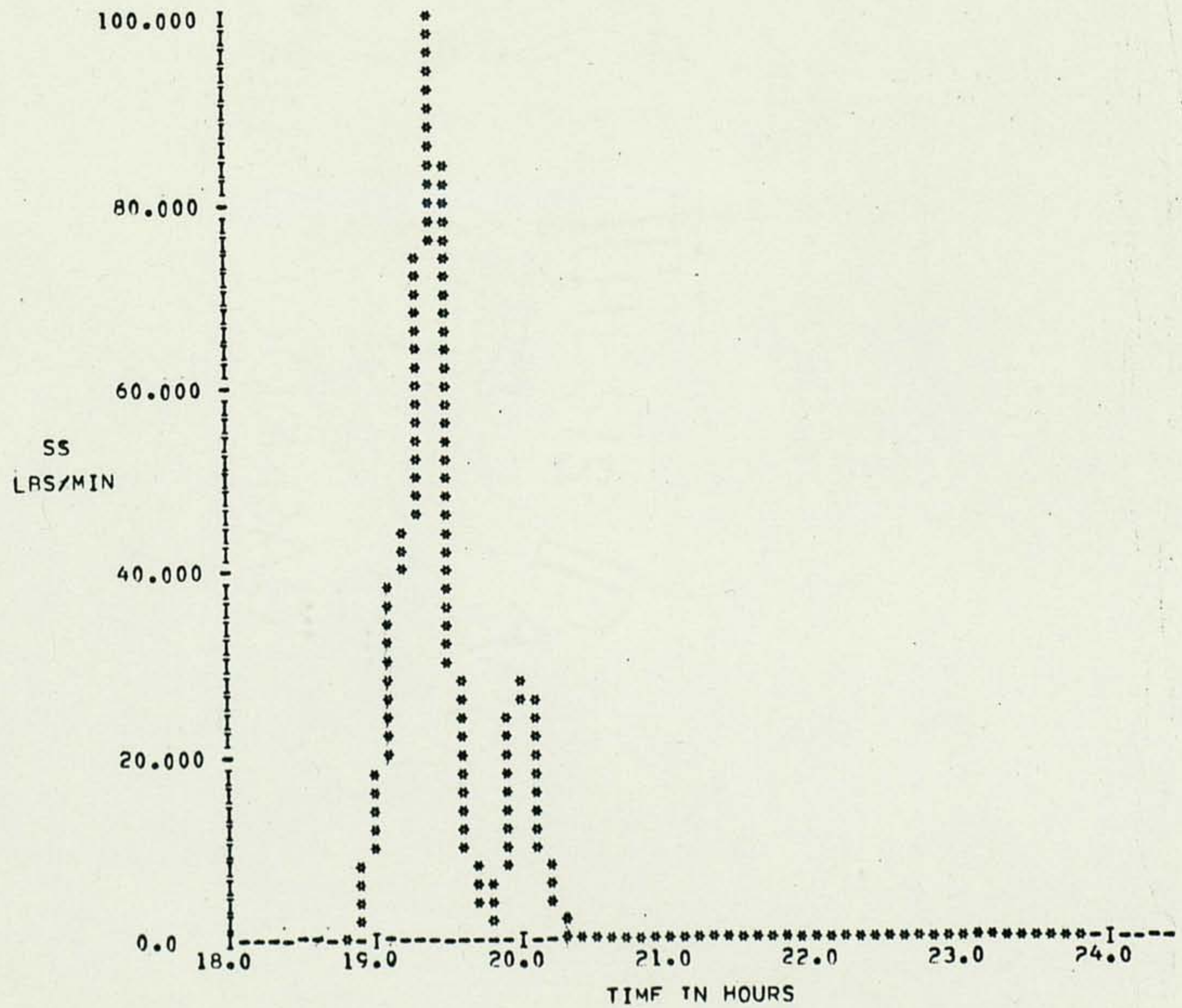


Fig. 41. Suspended Solids Flow (lb/min) Versus Time, May 12, 1975

TABLE 32

STORMWATER QUALITY, MAY 12, 1975

Type of Sample	Time Filled	pH	Alkalinity (mg/1 as $CaCO_3$)	Hardness (mg/1 as $CaCO_3$)	Nitrogen			Phosphorus	
					TKN	NO_3-N	Total	OP	TP
Composite (½ hr)	1705*	8.20	122.1	158.5	----	----	----	----	----
Composite (½ hr)	1735*	8.27	155.4	160.5	0.13	0.30	0.43	0.32	0.45
Composite (½ hr)	1835*	8.35	155.4	166.6	0.22	0.18	0.40	0.33	0.52
Composite (½ hr)	1905**	8.01	102.7	103.7	----	0.39	----	0.26	0.55
Composite (½ hr)	1935**	7.39	27.8	39.0	----	0.52	----	0.15	0.32
Composite (½ hr)	2035**	7.42	25.0	34.6	----	0.77	----	0.16	0.20
Composite (½ hr)	2135**	7.61	50.0	69.2	----	0.88	----	0.24	0.31
Composite (½ hr)	2235***	7.97	69.4	95.6	----	0.73	----	0.35	0.42
Composite (3 hr)	0135***	8.24	124.9	144.3	----	0.61	----	0.38	0.54

*Taken prior to stormwater runoff
**Taken during stormwater runoff
***Taken after stormwater runoff

hibit lower phosphorus concentrations.

The dry weather flow quality remained fairly constant throughout the study period. The various quality parameters had slight variations, but nothing major. The pH was around 8.20 and alkalinity and hardness varied by only about 6.0 and 8.0 mg/l, respectively, from the dry weather flow of May 10. The peak 5-day BOD and suspended solids flow rates of May 12 were higher than those of May 10. This, of course, was due to the rainfall intensities, durations, and total amounts. From this study, several conclusions and recommendations can be made.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

A literature review was done in order to determine the optimum stormwater simulation model to apply to the Lake Eola commercial study area. Upon completion of this research, the EPA Storm Water Management Model was selected. The reasons for choosing the EPA SWMM were as follows:

1. The Model is applicable to urban drainage basins,
2. The Model is very comprehensive and "fine" detail can be utilized,
3. The computer program and usage were well-documented and available, and
4. Testing and verification of the Model had been done.

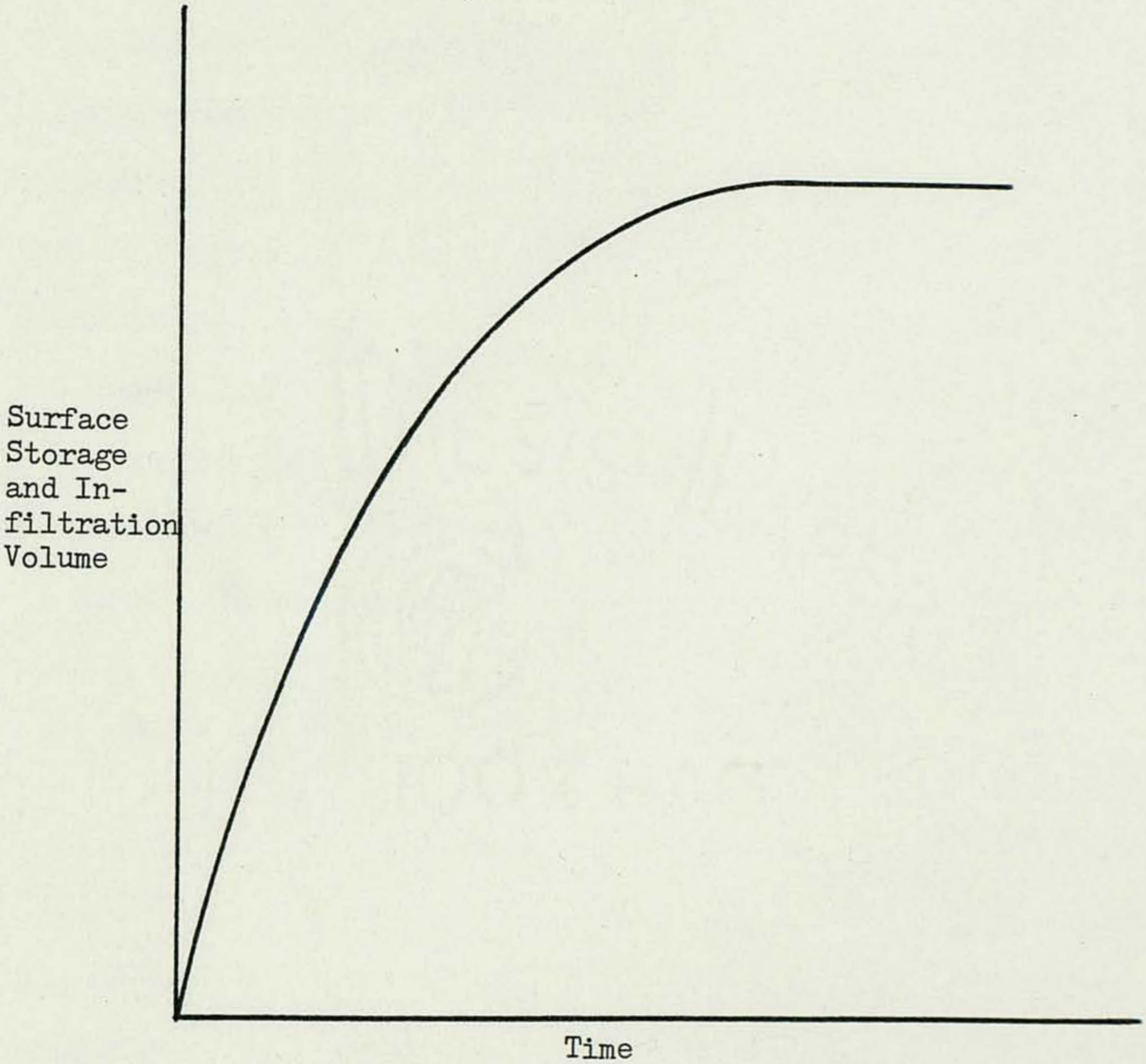
The SWMM was applied to the commercial study area after considerable data had been gathered. The data required ranged from surface roughness coefficients to storm sewer dimensions. Application of the SWMM was undertaken for actual rainfall events.

Three storms were simulated, with one storm being used for calibration purposes. Quantity and quality data were collected at the test site for comparison and calibration of the Model. Initial calibration runs were made to obtain reasonably accurate quality and quantity results. Quantity calibration of the Model was very successful with difference of only 66 cubic feet occurring between actual and simulated runoff volume. Upon application to the remaining two

storms, successful quantity simulations were also achieved. It is felt that the quantity calibration and simulations are refined to where accurate quantity simulation occurs. An interesting fact was also noted concerning surface storage and infiltration.

Comparing the simulated surface storage and infiltration volume of May 10 with that occurring on May 12 an increase of only 1301 cubic feet occurred. The total rainfall increase was 0.22 inches. In other words, an increase in total rainfall of 45.83% resulted in an increase of only 14.75% in surface storage and infiltration volume. It would appear that surface storage and infiltration is a function of the total rainfall taking into account the rainfall intensity and duration. At some point in time, surface storage and infiltration would reach a peak and remain constant. This would occur if a uniform rainfall occurred at a constant intensity for a continuous length of time. This rainfall intensity being greater than the maximum infiltration rate. This is illustrated in Figure 42. If surface storage and infiltration are examined separately, it could be assumed that given a storm of sufficient duration and constant rainfall intensity, surface storage would reach its peak level first. Infiltration would continue until a peak rate would be reached after maximum surface storage is achieved. This infiltration rate would remain relatively constant until the storm subsided.

For comparison purposes the runoff coefficient c, in the Rational Formula is calculated from the actual data for the May 10 storm. Actual runoff volume of 40,050 cubic feet is divided by the rainfall volume of 48,800 cubic feet to obtain a c of 0.82. Comparing this value with the



Assuming: 1. Uniform rainfall distribution
2. Constant rainfall intensity greater than infiltration rate

Fig. 42. Surface Storage and Infiltration Versus Time

coefficients for the same type of area contained in Table 9, page 30, the calculated coefficient appears reasonable. Runoff coefficients for downtown business areas range between 0.70 and 0.95.

Quality simulation of the stormwater runoff was also done with the Model. Laboratory analyses were done on runoff samples obtained for the various storms. Several water quality parameters were determined for the runoff. The Model routes only three quality parameters through the storm sewer system. These are 5-day BOD, suspended solids, and coliform bacteria. Quality calibration was done with the 5-day BOD and suspended solids parameters. Calibration was achieved to the point where initial concentrations of actual and simulated BOD and suspended solids were comparable. Pollutographs were produced from the simulated concentrations.

Recommendations

Additional work should be performed with the SWMM on other types of land uses. The proposed residential study area would be ideal. Work could also be done on simulating the entire Lake Eola drainage basin. There is a large amount of quality and quantity data now available for this basin that could be utilized. Additional quality and quantity calibration would be required to obtain satisfactory results. The calibration performed with the Commercial Study Area could serve as guidelines for future attempts.

Quality calibration of the Commercial study area simulation to obtain more accurate results should be undertaken. Since the quantity calibration is very successful, no flow data would be required but only rainfall data and water quality samples. Extensive sampling, primarily

with grab samples would be the optimal approach. In this manner, actual quality measurements taken at specific instants in time or at specific time steps could be compared with simulated quality data produced at the same time steps or instants in time. Grab samples obtained only a few time steps apart would be required for a "fine" quality calibration.

For reasonable water quality simulation results, actual pollutant loading rates for the drainage area should be determined. These would be loadings for the street surface contaminants and can then be input to the SWMM Model. Better correlation of actual and simulation quality data should occur with actual loadings for the drainage area being used. To obtain these loading rates would require a major study. Other quality parameters such as total gutter lengths, subcatchment widths, stored catchbasin BOD concentration, and street cleaning practices do affect the quality simulation results. These could possibly be varied to obtain good correlations, but this would require a "trial and error" approach. It is felt that the various pollutant loading rates for the land uses would have the most influence on the quality simulation.

It has been demonstrated that the storm sewer from the commercial study area contributes to the pollutant levels of Lake Eola. The quantities of pollutants for various storms were determined and compared with the dry weather flow pollutant quantities. A complete study of the Lake Eola drainage basin should be undertaken to determine what possible solutions to the problem, if any, exist. This study would serve as a guide to the City of Orlando, upon which decisions and possible solutions could be based. The EPA Storm Water Management Model would serve as an excellent tool for this study.

APPENDIX A

DATA CARDS FOR SIMULATION OF STORM OF MAY 10, 1975

```
//SWMM JOB (1173,0001,SWMM,FTU,350,400,50,,2400,3),'CHANCE',CLASS=G
/*SETUP          DISK=CCR017
//               EXEC    IRSWMM
//SYSIN          DD *
  0    9    9   10   10    9
  1    2    3    4    8
RUNOFF
      LAKE EOLA DRAINAGE BASIN COMMERCIAL STUDY AREA SIMULATION
         STORM OF MAY10,1975   DURATION OF 51 MINUTES,RUN ONE, STORM 5
   18  100 1432   2.    1   48
   60   1.
  0.0  0.0  0.0  0.0  0.0  1.2  1.2  1.2  1.2  1.2
  1.2  1.2  1.2 1.89 1.89 1.89 1.89 1.89 1.89 1.89
   .3   .3   .3  .12  .12  .12  .12  .12  .54  .54
  .54  .54  .54  .03  .03  .03  .03  .03  .03  .03
  .03  .03  .03  .03  .03  .03  .03  .03  .03  .03
  .03  .03  .03  .03  .03  .03  0.0  0.0  0.0  0.0

       217  217 480.2.336    95.008 .012        .13
       218  218 480.2.336    95.008 .012        .13
       219  219 350.1.579    95.005 .012        .13
       220  220 350.1.351    95.004 .012        .13
       221  221 170..4252    95.003 .012        .13
       222  222 110..2066    95.003 .012        .13
       223  223 160..2754    95.003 .012        .13
       224  224 170..3156    95.003 .012        .13
       225  224 180..6711    95.003 .012        .13
       226  226 280.1.2      95.004 .012        .13
       227  227 280.1.25     95.004 .012        .13
       228  228 340.1.57     95.007 .012        .13
       229  229 385.2.772    95.01  .012        .13
       230  230 650.2.62     95.0466.012        .13
       231  2311440.4.01     95.022 .012        .13
       232  232 890.2.984    95.029 .012        .13
       233  2331200.1.45     95.009 .012        .13
       234  234 200..65      95.0035.012        .13

         1         25          4         1  16.044      25.0      1
       217    3    1.          14.89
       218    3    1.          34.97
       219    3    1.          19.59
       220    3    1.           3.5
       221    3    1.           1.7
       222    3    1.           1.2
       223    3    1.           1.6
       224    3    1.           1.7
       225    3    1.           3.57
       226    3    1.           7.3
       227    3    1.           4.19
       228    3    1.           5.5
       229    3    1.          15.4
       230    3    1.          24.42
       231    3    1.          19.1
```

```
        232    3    1.          19.1
        233    3    1.0     20.5
        234    3    1.            .5
     3    2
   233  234  231
TRANSPORT
     0    0
              LAKE EOLA DRAINAGE SYSTEM 84
    57  100   18    1   2   2   0   3   4
       120.      .0001      25.
     0    1    0    1   0   0
   217   0   0   0  16
   218   0   0   0  16
   219   0   0   0  16
   220   0   0   0  16
   221 328   0   0  16
   222 329   0   0  16
   223 333   0   0  16
   224 335   0   0  16
   225   0   0   0  16
   226   0   0   0  16
   227   0   0   0  16
   228   0   0  00  16
   229   0   0   0  16
   230   0   0   0  16
   231   0   0   0  16
   232   0   0   0  16
   233 350   0   0  16
   234   0   0   0  16
    10 324 325 326  16
    11 327 330 351  16
    12 331 340   0  16
    13 338 339   0  16
    14 336 337   0  16
    15 334   0   0  16
    16 341 342 343  16
    17 344   0   0  16
    18 345 346   0  16
    19 347 348 349  16
    84 332   0   0  16
   324 217   0   0   1   35.      1.     2.3
   325 218   0   0   1   50.      1.     3.08
   326 219   0   0   1   25.      1.     9.6
   327  10   0   0   1   40.      2.      .5
   328 220   0   0   1   80.      1.      .5
   329 221   0   0   1  250.      1.      .45
   330 222   0   0   1   30.      1.    10.0
   331  11   0   0   1  200.      2.     2.
   332 233   0   0   1  135.      2.0    1.0                 3.0
   333  15   0   0   1   25.      1.     2.
   334 234   0   0   1   40.      1.     2.
```

```
335 223   0   0   1    5.       1.5       .5
336 224   0   0   1   20.       1.       2.
337 226   0   0   1   30.       1.       2.
338  14   0   0   1  190.       1.        .51
339 225   0   0   1   35.       1.       8.2
340  13   0   0   1   82.       1.25      .5
341  12   0   0   1  260.       2.       2.0
342 227   0   0   1   45.       1.       2.0
343 229   0   0   1   50.       1.       2.0
344  16   0   0   1  187.       2.       5.2
345  17   0   0   1   37.       2.       5.
346 230   0   0   1   38.       1.       5.
347  18   0   0   1  225.       2.       1.
348 232   0   0   1   30.       1.       2.
349 231   0   0   1   20.       1.       2.
350  19   0   0   1   22.       2.       1.1
351 228   0   0   1   20.       1.0       .11
   84  233
  233
   84  233
   120.
  282      0.0      6.0
    0    0    0    0   60  163  201  148  102    0    0    0
GRAPH
   10    1    3    2
   84  233
        LAKE EOLA DRAINAGE BASIN COMMERCIAL STUDY AREA
               TIME IN HOURS
  FLOW    IN CFS
   BOD    LBS/MIN
   SS     LBS/MIN
COLIFORM MPN/MIN
ENDPROGRAM
//
```

FOOTNOTES

[1] Association for Testing and Materials, Committee D-19, <u>Manual on Water</u>, Special Technical Publication No. 442 (Easton, 1969), p. 3.

[2] <u>Ibid</u>., p. 4.

[3] M. P. Wanielista, et al., <u>Interim Report - Non Point Source Effects</u>, delivered to Department of Pollution Control, Florida Technological University, Orlando, Florida, February 28, 1975.

[4] L. L. Ciaccio, ed., <u>Water and Water Pollution Handbook</u>, Volume 1 (New York: Marcel Dekker, 1971), p. 2.

[5] D. A. Wallace and R. R. Daque, "Modeling of Land Runoff Effects on Dissolved Oxygen," <u>Journal of Water Pollution Control Federation</u> 45:8 (August 1973), p. 1795.

[6] Norman H. Crawford, <u>Pesticide Transport and Runoff Model for Agricultural Lands</u>, Environmental Protection Agency (Washington, DC: U.S. Government Printing Office (December 1973)), p. 5.

[7] <u>Ibid</u>.

[8] <u>Ibid</u>.

[9] Environmental Protection Agency, Department of Biological and Agricultural Engineering, North Carolina State University at Raleigh, <u>Role of Animal Wastes in Agricultural Land Runoff</u>, (Washington, DC: U.S. Government Printing Office (August 1971)).

[10] <u>Ibid</u>., p. 1.

[11] <u>Ibid</u>., p. 2.

[12] <u>Ibid</u>., p. 94.

[13] J. D. Sartor, G. B. Boyd, and F. J. Acardy, "Water Pollution Aspects of Street Surface Contaminants," <u>Journal of the Water Pollution Control Federation</u> 46:3 (March 1974).

[14] <u>Ibid</u>., p. 461.

[15] <u>Ibid</u>.

[16] R. E. Pitt and G. Amy, <u>Toxic Materials Analysis of Street Surface Contaminants</u>, Environmental Protection Agency (Washington, DC: U.S. Government Printing Office, (August 1973)).

[17]Sartor, Boyd, and Acardy, p. 463.

[18]Pitt and Amy, p. 16.

[19]Ibid., p. 17.

[20]Ibid., p. 1.

[21]Ibid., p. 72.

[22]Ibid.

[23]Sartor, Boyd, and Acardy.

[24]Ibid.

[25]Pitt and Amy.

[26]Ibid.

[27]Ibid., p. 98.

[28]Ibid.

[29]J. W. Kluesener and G. F. Lee, "Nutrient Loading from a Separate Storm Sewer in Madison, Wisconsin," Journal of Water Pollution Control Federation 46:5 (May 1974), p. 932.

[30]J. D. Sartor and G. B. Boyd, Water Pollution Aspects of Street Surface Contaminants, Environmental Protection Agency (Washington, DC: U.S. Government Printing Office, (November 1972)), p. 77.

[31]Ibid.

[32]Sartor, Boyd, and Acardy.

[33]Sartor and Boyd.

[34]Kluesener and Lee.

[35]Ibid., p. 932.

[36]Sartor, Boyd, and Acardy, p. 464.

[37]Ibid.

[38]Sartor and Boyd.

[39]Sartor, Boyd, and Acardy, p. 463.

[40]Wanielista, et al.

[41]Wanielista, et al., p. II-9.

[42]A. G. Lamonds, Chemical and Biological Quantity of Lake Dicie at Eustis, Florida, with Emphasis on the Effects of Storm Runoff, U.S. Geological Survey, Water Resources Investigations 36-74, U.S. Department of Interior (December 1974).

[43]Ibid., p. 24.

[44]Ibid., pp. 16, 23.

[45]C. N. Papadakis and N. C. Prew, "Analysis of Synthetic Unit - Graph Methods," Journal of the Hydraulics Division 99:HV9 (September 1973), p. 1319.

[46]W. L. Moore and C. W. Morgan, eds., Effects of Watershed Changes on Streamflow (Austin and London: University of Texas Press, 1969), p. 235.

[47]Report of the Joint Committee of ASCE and WPCF, Design and Construction of Sanitary and Storm Sewers, by J. C. Lawler, Chairman (Washington, DC: Water Pollution Control Federation, 1970), p. 51.

[48]Ibid., p. 42.

[49]W. H. Espey and D. E. Winslow, "Quantity Aspects of Urban Storm Water Runoff," a short course application of Storm Water Management Models, University of Massachusetts, August 1974, p. 39.

[50]L. K. Sherman, "Streamflow from Rainfall by Unit-Graph Method," Engineering News Record, October 1932, p. 56.

[51]P. E. Morgan and S. M. Johnson, "Analysis of Synthetic Unit - Graph Methods," Journal, Trans American Society of Civil Engineers 129 (1964).

[52]W. W. Horner and F. L. Flynt, "Relation Between Rainfall and Runoff from Small Urban Areas," Journal, Trans American Society of Civil Engineers 60 (1942).

[53]Joint Committee of ASCE and WPCF.

[54]A. Brandstetter, Comparative Analysis of Urban Stormwater Models, study contracted by Environmental Protection Agency (Washington, DC: U.S. Government Printing Office, (1974)).

[55]L. H. Watkins, The Design of Urban Sewer Systems, Road Research Technical Paper 55 (London: Department of Scientific and Industrial Research, 1962).

[56]Ibid.

[57]M. L. Terstreip and J. B. Stall, Storm Sewer Design - An Evaluation of the RRL Method, EPA-R2-72-068, Office of Research and Monitoring, Environmental Protection Agency (Washington, DC: U.S. Government Printing Office, (1972)).

[58]A. L. Tholin and C. J. Keifer, "The Hydrology of Urban Runoff," Journal, Sanitary Engineering Division, American Society of Civil Engineers 85 (1959).

[59]S. W. Jens and M. B. McPherson, "Hydrology of Urban Areas," Handbook of Applied Hydrology, Sec. 20 (New York: McGraw-Hill Book Company, 1965).

[60]A. B. Kaltenbach, "Storm Sewer Design by the Inlet Method," Public Works 94 (January 1963).

[61]Papadakis and Prew, p. 1321.

[62]W. I. Hicks, "A Method of Computing Urban Runoff," Journal, Trans American Society of Civil Engineers, Vol. 109 (1944), p. 1217.

[63]C. F. Izzard, "Hydraulics of Runoff from Developed Surfaces," Proceedings, Highway Research Bulletin, No. 26 (1946).

[64]U. T. Chow, "Hydrologic Design of Culverts," Journal, Hydraulics Division, American Society of Civil Engineers H74 (1962).

[65]Watkins.

[66]Terstreip and Stall.

[67]N. H. Crawford and R. K. Linsley, Digital Simulation in Hydrology: Stanford Watershed Model IV, Technical Report No. 39 (Palo Alto: Stanford University, Department of Civil Engineering, (July 1966)).

[68]Ciaccio, p. 265.

[69]Brandstetter, p. 25.

[70]Papadakis and Prew.

[71]Brandstetter, p. 31.

[72]Papadakis and Prew, p. 1322.

[73]Brandstetter, p. 26.

[74]Ibid., p. 30.

[75]Ibid., p. 34.

[76]U.S. Environmental Protection Agency, Stormwater Management Model, Volume I, Final Report, Water Pollution Control Research Series (Washington, DC: U.S. Government Printing Office (July 1971)).

[77]U.S. Environmental Protection Agency, Stormwater Management Model, Volume II, Verification and Testing, Water Pollution Control Research Series (Washington, DC: U.S. Government Printing Office (August 1971)).

[78]U.S. Environmental Protection Agency, Stormwater Management Model, Volume III, User's Manual Version II, Environmental Protection Technology Series (Cincinnati: Office of Research And Development (Marcy 1975)).

[79]U.S. Environmental Protection Agency, Stormwater Management Model, Volume IV, Program Listing, Water Pollution Control Research Series (Washington, DC: U.S. Government Printing Office (October 1971)).

[80]Stormwater Management Model, Volume I, p. 3.

[81]Warren Andersen et al., Control of Lake Levels in Orange County, Florida, Information Circular No. 47, United States Geological Survey (Tallahassee, FL: Florida Geological Survey-22 (1965)), p. 13.

[82]Ibid., p. 2.

[83]Charles J. Boyter, "The Restoration of Lake Waters," (Master's Thesis, Florida Technological University, November 1972).

[84]Kluesener and Lee.

[85]Florida Technological University and Orange County Pollution Control Department, Lake Eola Restoration: Phase III, presented to Mr. Paul Matthes, Public Works Director, City of Orlando, FL (December 1972), p. 22.

[86]Ibid., p. 23.

[87]Stormwater Management Model, Volume III, p. 35.

[88]Ibid.

[89]Ibid.

[90]Ibid.

[91]Ibid., p. 108.

[92]Ibid.

[93]Ibid.

[94]*Stormwater Management Model*, Volume III, p. 269.

[95]*Ibid*.

[96]*Ibid*., p. 138.

[97]*Ibid*., p. 48.

[98]*Ibid*.

[99]*Stormwater Management Model*, Volume IV, p. 54.

[100]U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Data Service, Herndon Airport Station, *Local Climatological Data*, September 1973.

[101]Boyter.

BIBLIOGRAPHY

Anderson, Warren, et al. Control of Lake Levels in Orange County, Florida. United States Geological Survey, 1965. Information Circular #47. Printed by the Florida Geological Survey-22, Tallahassee, FL, 15 pages.

American Society for Testing and Materials, Committee D-19. Manual on Water. Special Technical Publication No. 442, Easton, MD, May 1969.

Boyter, Charles J. "The Restoration of Lake Waters." Masters Thesis, Florida Technological University, Orlando, Florida, November 1972.

Brandstetter, A. Comparative Analysis of Urban Stormwater Models. Environmental Protection Agency, Washington, DC: U.S. Government Printing Office, 1974.

Chow, U. T. "Hydrologic Design of Culverts." Journal, Hydrologic Division, Proc. American Society of Civil Engineers, H74 (1962), pages 88-97.

Ciaccio, L. L., ed. Water and Water Pollution Handbook, Volume 1 New York: Marcel Dekker, 1971.

Crawford, Norman H. Pesticide Transport and Runoff Model for Agricultural Lands. Environmental Protection Agency, Washington, DC: U.S. Government Printing Office, December 1973.

Crawford, N. H. and Linsley, R. K. Digital Simulation in Hydrology: Stanford Watershed Model IV. Stanford University, Palo Alto, California, Department of Civil Engineering, Technical Report No. 39, July 1966.

Deininger, R. A., ed. Models for Environmental Pollution Control. Ann Arbor, Michigan: Ann Arbor Science, 1973.

The Effects of Agricultural Pesticides in the Aquatic Environment, Irrigated Croplands, San Joaquin Valley. Pesticide Study Series, Volume 6, Office of Water Programs, Environmental Protection Agency, Washington, DC, June 1972, pages 210-268.

Ehlers, V. M. and Steel, E. W. Municipal and Rural Sanitation. New York: McGraw-Hill Book Company, 1965.

Espey, W. H. and Winslow, D. E. "Quantity Aspects of Urban Storm Water Runoff." A short course application of Storm Water Management Models, University of Massachusetts, August 1974.

Fair, G. M., Geyer, J. C. and Okun, P. A. Water and Water Engineering, Volume 1, Water Supply and Wastewater Removal. New York: John Wiley and Sons, Inc., 1966.

Hicks, W. I. "A Method of Computing Urban Runoff." Trans American Sec. Civil Engineer, Volume 109 (1944), pages 1217-1229.

Horner, W. W. and Flynt, F. L. "Relation Between Rainfall and Runoff from Small Urban Areas." Trans American Society of Civil Engineers, Volume 60 (1942), pages 1135-1178.

Izzard, C. F. "Hydraulics of Runoff from Developed Surfaces," Proceedings, Highway Research Bulletin, No. 26, 1946, pages 129-138.

Jens, S. W. and McPherson, M. B. "Hydrology of Urban Areas." Handbook of Applied Hydrology, Sec. 20. New York: McGraw-Hill Book Co., 1965.

Joint Committee of American Society of Civil Engineers and Water Pollution Control Federation, J. C. Lawler (Chairman). Design and Construction of Sanitary and Storm Sewers. Water Pollution Control Federation, Washington, DC, 1970.

Kaltenbach, A. B. "Storm Sewer Design by the Inlet Method." Public Works, Volume 94 (January 1963).

Kluesener, J. W. and Lee, G. F. "Nutrient Loading from a Separate Storm Sewer in Madison, Wisconsin." Journal of Water Pollution Control Federation, Volume 46, No. 5 (May 1974), pages 920-936.

Lake Eola Restoration: Phase III. Florida Technological University and Orange County Pollution Control Department. Orlando, Florida, December 1972.

Lamonds, A. G. Chemical and Biological Quantity of Lake Dicie at Eustis, Florida, with Emphasis on the Effects of Storm Runoff. U.S. Geological Survey, Water Resources Investigations 36-74, United States Department of Interior, Washington, DC: U.S. Government Printing Office, December 1974.

Mallory, C. W. The Beneficial Use of Storm Water. Washington, DC: U.S. Government Printing Office, January 1973.

Moore, W. L. and Morgan, C. W., eds. Effects of Watershed Changes on Streamflow. Austin: University of Texas Press, 1969.

Morgan, P. E. and Johnson, S. M. "Analysis of Synthetic Unit - Graph Methods." Trans American Society of Civil Engineers, Volume 129 (1964), pages 727-738.

Papadakis, C. N. and Prew, N. C. "Testing of Urban Runoff." Journal of the Hydraulics Division, Volume 99, n HV9 (September 1973), pages 1319-1335.

Peterson, F. L. and Nargis, D. R. "Subsurface Disposal of Storm Runoff." Journal of Water Pollution Control Federation, Volume 45, No. 8 August 1973), pages 1663-1670.

Pitt, R. E. and Amy, G. Toxic Materials Analysis of Street Surface Contaminants. Environmental Protection Technology Series, Office of Research and Development, U.S. Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, August 1973.

Role of Animal Wastes in Agricultural Land Runoff. Department of Biological and Agricultural Engineering, North Carolina State University at Raleigh, Environmental Protection Agency, Washington, DC: U.S. Government Printing Office, August 1971.

Sartor, J. D. and Boyd, G. B. Water Pollution Aspects of Street Surface Contaminants, Environmental Protection Technology Series, Office of Research and Monitoring, U.S. Environmental Protection Agency, Washington, DC: U.S. Government Printing Office, November 1972.

Sartor, J. D., Boyd, G. B., and Acardy, F. J. "Water Pollution Aspects of Street Surface Contaminants." Journal of the Water Pollution Control Federation, Volume 46, No. 3 (March 1974), pages 458-467.

Sheffield, C. W., et al. Lake Restoration Methods. Orlando, Florida: Orange County Pollution Control Department, 1972.

Sherman, L. K. "Streamflow from Rainfall by Unit-Graph Method," Engineering News Record, October 1932, pages 54-62.

Stormwater Management Model, Volume I, Final Report. Water Pollution Control Research Series, Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, July 1971.

Storm Water Management Model, Volume II, Verification and Testing. Water Pollution Control Research Series, Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, August 1971.

Storm Water Management Model, Volume III, User's Manual Version II. Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, March 1975.

Stormwater Management Model, Volume IV, Program Listing. Water Pollution Control Research Series, Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, October 1971.

Terstreip, M. L. and Stall, J. B. Storm Sewer Design - An Evaluation of the RRL Method. EPA-R2-72-068, Environmental Protection Agency, Office of Research and Monitoring. Washington, DC: U.S. Government Printing Office, 1972.

Tholin, A. L. and Keifer, C. J. "The Hydrology of Urban Runoff." Journal, Sanitary Engineering Division, American Society of Civil Engineers, Volume 85 (1959), pages 47-106.

University of Cincinnati, Department of Civil Engineering. Urban Runoff Characteristics. Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, October 1970.

Urban Storm Runoff and Combined Sewer Overflow Pollution. Envirogenics Company for the Environmental Protection Agency. Washington, DC: U.S. Government Printing Office, December 1971.

U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Data Service, Herndon Airport Station, Local Climatological Data. Washington, DC: U.S. Government Printing Office, September 1973.

Vilaret, M. R., et al. Storm and Combined Sewer Pollution Sources and Abatement, Atlanta. Environmental Protection Agency, Water Quality Office. Washington, DC: U.S. Government Printing Office, 1971.

Wallace, D. A. and Daque, R. R. "Modeling of Land Runoff Effects on Dissolved Oxygen." Journal of Water Pollution Control Federation, Volume 45, No. 8 (August 1973), pages 1795-1809.

Wanielista, M. P., et al., Interim Report - Non Point Source Effects, delivered to Department of Pollution Control, Florida Technological University, Orlando, Florida, February 28, 1975.

Watkins, L. H. The Design of Urban Sewer Systems. Department of Scientific and Industrial Research, London, England, Road Research Technical Paper, 55 (1962).