TREATMENT OF OVERFLOWS FROM

COMBINED SEWERS

by

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INTRODUCTION

Automation and the danger of over productivity have created unprecedented amounts of leisure time in the United States. We are now in the process of finding that major potential recreational areas have been seriously polluted by the by-products of the same automation and productivity which have created our leisure time. Increasing urbanization has been accompanied by progressively more stringent demands on water quality and quantity while steadily impairing the same parameters. One of the major sources of water quality impairment due to concentrated urban growth is combined sewer overflow.

Of the sewered communities in the United States, approximately thirty percent of the smaller communities and fifty percent of the major cities have separate storm and sanitary lines (99). More than sixty million people in the United States presently reside in communities which allow overflows from combined sewers. The annual average extent of such overflow has been estimated by the U.S. Department of Health, Education and Welfare (8) to be three to five percent of the total untreated sewage produced. During times of combined sewer overflow, it was estimated that 95 percent of the total untreated domestic sewage in the system escapes to the receiving watercourse. These percentages do not include the significant amounts of biochemical oxygen demand and suspended solids contributed by the actual stormwater runoff. The pollution caused by overflows from combined sewers has recently received increased attention because of increased use of waterways for recreation, particularly swimming, boating, and fishing, which require, because of human contact, a high degree of water cleanliness. Frequent overflow of substantial amounts of objectional material during the recreational months often prevents the consumptive and recreational uses of our water courses and beaches for weeks at a time.

The complete separation of sanitary and storm sewers is considered to be the ultimate solution to the problem. Eventually, even the complete treatment of the stormwater will probably be required in the larger metropolitan areas in order to obtain water quality suitable for bathing during the recreational season. The complete separation of all existing combined sewers was estimated to cost 20 to 30 billion dollars in 1964 (8). All new construction supposedly incorporates separate systems. Cities with combined sewer systems are changing their building codes to specify that all storm drainage from gutters and yards must be carried to the street in separate sewers to await the coming separate sewer system.

A significant portion of the pollution discharged in the overflow from combined sewers is due to the large amounts of sludge scoured from the sewer by the excess stormwater. The city of Buffalo, New York, reported that 1/3 of its annual production of sewage solids was lost through overflows, while only two or three percent of the annual sewage volume was lost (8). Most authors agree that the volume of sewage lost through overflows is around 3 to 6 percent, but disagree on the percentage of suspended solids and organic material lost. The biochemical oxygen demand concentration contained in an overflow from a combined sewer is approximately that of the sanitary sewage alone. The suspended solids concentration is usually six or seven times that of sanitary sewage.

Methods of treatment other than complete separation are being considered and studied. Some of these possibilities include holding tanks for the excess stormwater, chlorination of overflows, enlargement of the existing sewer systems to provide more storage capacity with eventual treatment at the existing sewage treatment plant, and new zoning codes. A new treatment unit for overflows from combined sewers is proposed in this study. Treatment of the overflows from combined sewers has been estimated to cost less than 10 percent of the cost for complete separation (8).

Recognition of a problem is the first step toward a solution. The need to separate sanitary waste water lines from stormwater lines is presently recognized in the design and construction of new sewer systems which incorporate a complete separation. However, the question of the most economical solution of the existing pollutional problem of overflows from combined sewers remains unanswered. The work described in this thesis is aimed at achieving guide lines leading to the solution of this problem.

LITERATURE REVIEW

To fully understand the effect, significance, and alternatives connected with stormwater overflow of combined sewers, each parameter related to the problem must be studied. It is for this purpose that the literature review is divided into sections dealing with the characteristics of the overflow and with different treatment alternatives. The literature review does not discuss all of the theoretically possible treatment devices, but only those considered to be practical at the present time.

Frequency And Amount Of Overflow

Stream pollution during wet weather is primarily caused by overflows from combined sewers, stormwater discharged directly to the stream, and wastewater bypasses around treatment plants and pumping stations. The frequency and amount of such overflows are very significant. Economical design of intercepting sewers and sewage treatment facilities is dependent upon accurate knowledge of the peak dry weather flow and of the magnitude, frequency, and effect of overflows from combined sewers. There are many existing methods to estimate the average dry weather flow (DWF), but stormwater runoff is much more difficult to analyze since many combinations of intensity and duration of precipitation produce flows far in excess of the capacity of interceptors.

McKee (70) in his classical study of overflows from combined sewers, found in the Boston, Massachusetts area, that a 0.01 inch per hour of runoff, after the impervious surfaces were wetted, produced a storm drainage runoff equal to the DWF. This study was undertaken to determine to what extent stormwater runoff was intercepted, what amount of sewage escaped as overflow from combined sewers, and how often overflows from combined sewers can be expected.

McKee made a probability study of all hourly precipitation during the dry weather months of June through November for the years 1934 to 1945 inclusive. He found that rainfall in Boston during the dry months equaled or exceeded 0.01 inches per hour approximately six percent of the total time. If traces of precipitation were included, McKee found that during the dry months, precipitation should be expected 14.9 percent of the time or approximately one-sixth of the time.

Horner (56) noted that surface runoff was retarded and decreased by:

a. retention on vegetation

b. infiltration into the soil

c. wetting of impervious surfaces

d. depression storage

e. surface detention required to build up a sheet of water sufficiently thick to cause runoff.

These factors are usually incorporated into the coefficient of runoff, which is largely dependent upon the relative imperviousness of the area, and the intensity and duration of prècipitation.

The coefficient of runoff will increase with the duration of precipitation as Horner's assumptions are fulfilled. Even though evaluations of the change in the coefficient of runoff for high intensity rainfalls have been made, McKee doubted their usefullness for the lower intensity rainfalls. McKee (70), in the absence of precise information concerning the coefficient of runoff for low intensity rainfalls, made the following assumptions:

a. that only the low intensity rain that fell upon impervious surfaces during the six dry months would be considered as surface runoff

b. that after the initial wetting of impervious surfaces, including depressions storage, all rain falling was considered to run off.

These assumptions were justified by noting that some of the runoff from impervious areas runs onto pervious areas and is absorbed there. Hicks (55) estimated that ten percent of the runoff from impervious areas was lost. Hicks and McKee agree that the amount of rainfall from a low intensity storm required to start runoff from an impervious surface is about 0.02 to 0.03 inches.

Since the low intensity rainfalls occur most frequently and are thus significant in determining the amount of overflow from combined sewers, the effect of the wetting of impervious surfaces has to be considered. McKee, allowing 0.03 inches to effect runoff from an impervious area, re-examined the rainfall probability for Boston. By eliminating the first 0.03 inches of precipitation for each period of rainfall, he found that he could more accurately predict the runoff attributed to each storm.

The quantity of stormwater runoff that a combined sewer may accommodate depends upon the size of the interceptor in terms of the DWF and the quantity of sanitary sewage being discharged at the time of the storm. If a storm occurs at night or early in the morning, the available capacity for storm water will be greater than if the storm occurred during the peak sanitary flow. McKee found that the rainfall showed no tendency to favor any certain hours of the day. This observation means that the probability of storm runoff coinciding with the peak sanitary flow is balanced by the increased capacity of the system for stormwater during periods of low sanitary flow. Consequently, on the average, if the system is designed for 2 X DWF, the quantity of stormwater intercepted will be equal to the average sanitary flow (70).

The proportion of sewage escaping (E) as overflow from a combined sewage system for a particular rainfall intensity i is given by the following formula:

$$E = \frac{i - r}{i + .01} \quad (70)$$

where the interceptor sewer has a capacity of (r+1) times dry weather flow. The term (i-r) is related to the quantity of overflow from the combined sewer system, and the term (i+.01) is related to the total flow in the sewer. For example, for a rainfall intensity of 0.15 inches per hour and combined sewer with a capacity of 3 X DWF, the amount of sewage escaping to the receiving watercourse without treatment will be 0.814 or 81.4 percent of the total amount.

In the long run, the proportion of all sanitary sewage that enters the receiving stream untreated will be a function of the probability of occurrence of each rainfall intensity and the proportion of sewage escaping at that intensity. The sum of the products of these two functions over the range of storm intensities will give the total percentage of raw sewage escaping to the watercourse.

In order to determine the frequency of overflows for various interceptor capacities, McKee examined the rainfall records for five years for the months of June through September. No storms were counted unless the total rainfall exceeded 0.04 inches, of which 0.03 inches were allowed for initiating flow, and the first 0.01 inch per hour was assumed to be intercepted. Each storm had to be followed at least by twenty-four hours during which no overflow occurred.

McKee concluded that by these methods and techniques, engineers may determine the optimum size of interceptor with regard to pollution abatement and economy of design. He also concluded that the added cost of larger interceptors does not appear to be justified by the reduction of sewage overflow quantities (70).

Johnson (59) reported the results of a study of the overflow frequency and duration of the combined sewers of Washington, D. C.. Johnson defined "dilution ratio" as the ratio of the capacity of the interceptor to the average DWF which is being intercepted. Given in Table I, is a summary of overflows to the streams during the period when Johnson considered the number of overflows excessive. Included in these overflows are the dry weather overflows of sanitary sewage which occurred only during the summer months for stations 3, 5, and 7. It was noted that at every gage, the number of overflows during the summer months exceeded those during the winter months.

A study was made of the rainfall and runoff at Kansas City, Missouri, for March to November from 1950 to 1959 by Benjes (28) utilizing United States Weather Bureau data.

TABLE I. Summary of Overflows at Five Locations in Washington, D. C. as Reported by Johnson (59).

Station No.	Dilution Factor	Overflows in Six Summer Months	
		Average Number Per Month	Average Duration Per Month(hr)
3	2.2	8.7	63
4	8.3	5.0	30
5	1.3	10.2	57
6	2.0	5.3	24
7	1.9	16.8	110

It was found that a measurable amount of rainfall occurs only five percent of the time. In computing the runoff, several simplifying assumptions were made:

a. that the first 0.04 inches of rain were required to initiate runoff

b. that the runoff from impervious areas was 100 percent after the initial wet down

c. that the runoff from pervious areas was equal to the rate of precipitation minus 0.10 inches per hour, the assumed rate of infiltration into the ground, and

d. fifty percent of the area was considered impervious and fifty percent pervious.

The results of deducting the first 0.04 inches from each storm for surface wetting reduced the frequency of storms producing runoff to 3.7 percent of the time. The average DWF was estimated to be 0.0035 CFS per acre or 0.0035 inches per hour. Using this figure, it was found that a runoff equal to the DWF or greater would occur 3.55 percent of the time and that a runoff of two times DWF or greater would occur 3.22 percent of the time. With the peak DWF from Kansas City equal to 1.5 X DWF, various sized interceptors were studied to determine the percent of time an overflow would occur. Benjes (28) concluded that since the quantity of combined sewage is great, and the cost of increasing the interceptor size is high for a small reduction in the percent of time of occurrence of overflow, that the maximum effective capacity of a combined interceptor is not appreciably greater than the peak DWF.

The results of a one year study of combined sewer overflow from a 24,500 acre town system in Michigan was reported by Hubbell (57). The resident population was estimated to be 302,000. A high weir structure was built at the outlet of the system and provided 100 acre-feet of storage in the sewers before overflow. The maximum interceptor flow was 120 CFS or 2 X DWF. The dry weather sewage was reported to be of medium strength and characteristics. During the year, nineteen separate overflows occurred and an automatic sampler took a composite sample of each overflow. During the 12 months, overflows of combined sewage occurred for 187.9 hours or 2.15 percent of the time. The total rainfall from May of 1965 through April of 1966 was 26.37 inches. The rainfall occurred on 133 days and the system did not overflow on 86 percent of the days.

Hubbell (59) concluded that there was no constituent strength change throughout the overflow period and that treatment of combined sewer overflows by conventional methods would be costly and not uniformly effectual. A storage capacity of 1375 acre-feet or about fourteen times that provided would have retained all of the overflows.

Greeley and Langdon (53) described the New York City

Jamaica Bay problem and solution in detail. The beaches in the Jamaica Bay area were not continually suitable for recreation during the summer months due to bacterial pollution caused by overflows from combined sewers. Studies of the location of overflow outlets, areas served, and the sources of sewage and stormwater runoff revealed a number of abuses in the system. As no records of the frequency, duration, and quantities of overflows were available, an approach relating rainfall to sewage quantities and tributary areas was utilized. Hourly records of rainfall for the five summer months, May through September, for the years 1950 to 1957 were utilized to establish the relationships between the number of storms, the total rainfall in the storm, and the duration of the storm. The expected number of storms during the five month period was found to be forty with an average interval between storms of 3.8 days. The relationship between the total rainfall in a storm and the percentage of the storms in which this amount of rainfall was exceeded, and the relationship between the total rainfall in a storm and its probable duration were found. These were used to estimate the probable number of storms, the total amount of rainfall of each, and the probable duration of each rainstorm expected during the summer months.

Several important parameters were then estimated in order to utilize gathered data for a rational analysis of the problem. The average DWF was found. All flows less than 2 X DWF were assumed intercepted and given full treatment. The first 0.03 inches of a rainstorm were assumed to initiate runoff. After the initial 0.03 inches, forty percent of the rainfall was considered to enter the combined system as runoff.

Using these factors in conjunction with the rainfall data, computations were made to estimate the overflow conditions for a typical 1000 acre urban basin. Forty rainstorms were considered to occur during the summer months, with 25 of these causing overflows. The average interval between overflows was found to be six days with a total duration time during the summer months of 229 hours or 6.2 percent of the time. Under these conditions, it was discovered that 2.6 percent of the sewage was lost through overflows, and that 52.5 percent of the total runoff was intercepted and received treatment at the sewage treatment plant.

Each period of runoff of combined sewage defined as that time after a rainfall when the flow was 0.1 MGD greater than the expected DWF, was carefully examined at Northampton. The average percentage runoff for the whole drainage area was found to be 38.9 percent, or 77 percent when based on the impervious area alone. For storms with a high total rainfall but not a significantly high intensity, the total runoff was approximately equal to the rainfall on the impervious area (50).

The duration of flows in excess of particular values of the DWF (0.323 MGD) for the period of study were investigated. The authors found that over the range of one to 98 percent of the occasions of overflow, there was an almost linear relationship between the duration and the percent of time of occurrence. This result indicated that the distribution of durations of overflows from a combined sewer system was very close to being log-normal (50). The volume of combined sewage discharged from a hypothetical overflow, if one had existed at the measuring flume, for each overflow setting (expressed in terms of DWF) was constructed for the Northampton study. It was noted that for each increment of 1 X DWF in the overflow setting, the number of overflow occurrencies per year was reduced by approximately four percent (50).

Characteristics Of Overflows From Combined Sewers

Various exploratory studies have been conducted on the characteristics of urban stormwater runoff. Since the studies vary in pattern and background, the results have not been consolidated to one set of data which may be considered as representative of the United States.

Palmer (77) sampled stormwater runoff from land surfaces at various street catch basins in Detroit in 1949 and 1960 (76). He found that the BOD ranged from 96 to 234 mg/1; total solids from 310 to 914 mg/1; and coliform MPN's from 25,000 to 930,000 per 100 ml. In 1960, he found the suspended solids averages from two storms to be 213 and 102 mg/1. The concentrations varied considerably between catch basins and at the same point at different times. He noted that in some cases the quality of runoff worsened with time and in other instances improved.

Wilkinson (100) in 1954, published the results of a study of surface runoff from a 611 acre estate with separate sewers at Oxney, England. The surface water runoff contained BOD up to 100 mg/1 and suspended solids concentrations of up to 2,045 mg/1. The BOD increased with an increase in the length of time of antecedent rainfall up to a period of eight to ten days, after which no increase was noted. A study was made to assess the actual discharge of BOD and suspended solids to the stream as opposed to a hypothetical treatment plant which would treat the combined wastewater if a combined sewer were present. It was concluded that the separate system reduced the BOD reaching the stream, but the suspended solids loading was increased by six or seven times. Wilkinson also noted that the first flushings from the storm sewers did not contain more objectionable material than did the subsequent flows, except after long antecedent dry periods.

Schigorin (84) published the results of a 1936 sampling survey of stormwater runoff in Moscow, which indicated a BOD of 186 to 285 mg/l and a suspended solids content of 1000 to 3,500 mg/l. In 1948-50, in studies conducted to Leningrad's cobblestone streets (84), the stormwater runoff was found to contain a BOD of 36 mg/l and suspended solids of 14,541 mg/l.

In 1959 and 1960, Sylvester (92) made a study of the characteristics of stormwater from the Seattle street gutters. He found that the constituent concentrations were highest when the antecedent rainfall had been low.

Akerlinch (2) in 1945 through 1948 found the median concentrations of the summer stormwater runoff from streets and parks in Stockholm, Sweden. The median value of coliforms was 4000 per 100 ml; COD, 188 mg/l; total solids, 210 mg/l; and BOD, 17 mg/l.

Riis-Carstensen (78) found that the combined sewage of Buffalo, N.Y. in one hour during a storm contained 28.4 times the amount of suspended solids normally found in the DWF. He also noted that any evaluation of the pollutional effect should not be made on volume alone due to the wide variance of the constituent strengths of the overflow.

In 1964, a rather extensive study was initiated by the United States Public Health Service (98) of the stormwater runoff of a 27 acre residential and light industrial drainage basin with separate sewers in Cincinnati, Ohio. Approximately 37 percent of the land was considered impervious with a ground slope of two to three percent. The stormwater runoff was sampled from July 1962 through September 1963. The range and mean constituent concentrations of the stormwater runoff are presented in Table II, exclusive of January and February when the runoff was largely due to snow melt which was high in chlorides due to street salting. The BOD and COD averaged 19 and 99 mg/l, respectively, which is about the quality of a secondary sewage treatment plant effluent. The suspended solids averaged 210 mg/l which is approximately that of raw sewage. The percent volatile suspended solids content was lower than that for sanitary sewage alone. Samples of stormwater runoff were also subjected to a 20 minute sedimentation period after which the supernatant was analyzed for suspended solids, volatile suspended solids, BOD, and COD. The range of suspended solids reduction was 27 to 53 percent, depending on the concentrations in the initial runoff. Similarly, reductions of volatile suspended solids ranged from 17 to 50 percent; BOD, 3 to 17 percent; and COD, 5 to 34 percent.

The authors of the Cincinnati study concluded that they could find no correlation between antecedent rainfall and runoff loads. The authors also compared the stormwater runoff constituents, computed for the years runoff

TABLE II. Reported Constituent Concentrations of Stormwater Runoff From the Cincinnati Study (98) From July 1962 through September 1963.*

Parameter		Range	Mean
		(units)	
Turbidity		30-1,000	170
Color		10-380	81
pH		5.3-8.7	7.5
		(mg/1)	
Alkalinity		10-210	59
Hardness (as $CaCO_3$)	Ca	24-200	63
	Mg	2-46	15
	Total	29-240	78
Cl^-		3-35	12
SS		5-1,200	210
VSS		1-290	53
COD		20-610	99
BOD		2-84	19
Nitrogen (as N)	NO_2	0.02-0.2	0.05
	NO_3	0.1-1.5	0.4
	NH_3	0.1-1.9	0.6
	Org.	0.2-4.8	1.7
PO_4(total soluble as PO_4)		0.07-4.3	0.8

* January and February of 1963 are not included.

from the 27 acre site, with the estimated amount of the same constituents that might be contributed by the raw sewage in the area. A density of nine persons per acre existed. This comparison of the stormwater runoff load and the sanitary sewage load in pounds per acre per year indicated that the suspended solids in stormwater runoff was 1.4 times that in normal sewage and that the BOD load contributed by the stormwater was negible. The results of this study indicated as the authors point out, that urban stormwater runoff can not be overlooked in considering the wastes discharged from an urban community.

A limited sampling survey of the storm runoff at eleven catch basins was run in Washington, D. C. (8). Several samples were taken from a catch basin during the storm. The BOD was found to average 126 ppm while the average concentration of the suspended solids was 2,100 ppm. These concentrations indicate a substantial pollutional load from the stormwater runoff.

Dunbar and Henry (42) presented a compilation of five day BOD, suspended solids and coliform concentrations for combined stormwater and raw sewage discharges. This data, gathered from a number of investigators, is presented in Table III. The authors suggested that field observations have not borne out the idea that, after a first flushing of the streets and roof areas, a marked decrease in the

TABLE III. Characteristics of Combined Sewer Overflow As Reported by Dunbar And Henry (42).

City	Coliform (MPN/100 ml)	Total Solids (mg/l)	Suspended Solids (mg/l)	BOD (mg/l)
Buffalo, N. Y.	—	—	172 to 1,220	—
Buffalo, N. Y.	—	498 to 754	158 to 544	100 to 162
Buffalo, N. Y.	—	461 to 785	126 to 436	121 to 127
Detroit	4,300,000	—	250	50
Toronto, Ont. (Eglinton Ave.)	23,000 to 2,400,000	—	130 to 930	40 to 260
Toronto, Ont. (North York Storm Trunk Sewer)	70,000 to 3,500,000	Includes 100 mg/l zinc (toxic)	17 to 580 (2 days)	0 to 100
Welland, Ont. (Burgar St.)	210,000	850 to 960	168 to 426	220 to 614

constituent concentrations of the combined sewage occurs.

A very comprehensive two-year stormwater study was described in 1963 by Gameson and Davidson (50). This study was concerned with the composition and flow from a combined sewerage system in Northampton, England serving an area of 229 acres and a population of 9,600. Fifty percent of the area was considered impermeable. The total length of sewers was nine miles with ninety percent of them being egg-shaped brick sewers of sizes ranging from 2 by 1.5 feet to 3 by 2.5 feet. Eighty percent of the sewer system had a gradient between one in 30 and one in 160 with the median being one in 77. A flume was specially constructed for the study at the lowest point or outlet of the drainage area. Two rainfall recorders were installed in the area along with an automatic sampling apparatus for the storm flows. The rainfall during the two year study was only five percent more than expected during average conditions, which meant that the study should be very representative of average conditions in the area. The average DWF for weekdays, Saturdays, and Sundays was found for each month from February 1960 to January 1962. The average DWF was found to be 0.323 million gallons per day.

The dry weather sewage was sampled on 51 days when 737 samples were taken and analyzed. The average dry weather concentrations of BOD and suspended solids were found to be 333 ppm and 368 ppm, respectively. Over 3,200 samples were taken by the automatic sampler during some 300 storms. The variation of the quality of combined sewage with the time of day was studied and the average concentration of BOD and suspended solids in the combined sewage was compared with that of the DWF. The most significant result was that the concentration of suspended solids was greater in the combined sewage than in the average DWF. The BOD of the combined sewage was generally less than that of the DWF, except at night. A study was also made of the variation of BOD with flow and time since the start of the storm. A first flushing effect was found. The average BOD for the first five minutes of runoff was 351 ppm, while the average for the complete runoff of all the storms was 123 ppm. This would indicate that either large quantities of solid matter are carried into the sewers by the surface water, are flushed from house connections, or are scoured from the sewers.

An examination was made into the reasons for the high concentrations of the solid matter found in the combined sewage. The Northampton investigators obtained samples of surface water runoff from a storm sewer in an adjacent drainage area on four occasions in 1961, and the analysis of these samples indicated that it was very unlikely that the high concentrations of suspended matter found in the Northampton combined sewage could be caused by the entry of solids into the surface water. On thirteen occasions there was a substantial accidental discharge of clean water into the Northampton system in dry weather. An examination of the composition of the sewage during these occasions tended to indicate a scouring of the deposition in the sewers, which indicated that the major source of the solid matter in the combined sewage must be within the sewer system itself. In order to further assess the effect of a discharge of clean water into the Northampton sewer system, the investigators made arrangements to have 0.1 million gallons of clean water released from the city reservoir. The release occurred early on a Sunday morning when the flow and composition of the sewage was the lowest. In three minutes the solids rose from 100 to 7,500 ppm, and in another five minutes had fallen to 1000 ppm. The total weight of suspended solids passing the sampling station during this experiment was estimated to be 330 pounds - compared with less than 5 pounds estimated from the normal DWF during the same time interval. The corresponding figures for BOD were 88 and 3 pounds. The authors noted that the values for suspended solids in this experiment will normally not include any contribution by grit or sand as most of this material was expected to pass under the sampling shoe. At the beginning of this controlled experiment the stilling chamber at the flume was cleaned, but at the end there was an estimated 100 pounds of grit in the chamber. It was estimated that the total solid matter carried through the flume must have exceeded 430 pounds as considerable grit was transported through the flume. The total solid weight was estimated to be 0.25 to 0.50 tons. If one-half a ton of material had been evenly distributed over the one and one-half miles of sewers carrying the discharge of clean water, this would equal three ounces per foot (50).

A study of the average velocities in the Northampton sewer system showed that during the minimum night flows, 38 percent of the sections had a velocity of less than 0.5 feet per second. The corresponding figure for peak daily flow was 15 percent. It was concluded that the deposition of solids in the Northampton sewage system during DWF was very significant (50).

The conclusions drawn from the Northampton study were: (50)

a. That raising the overflow setting by one DWF had approximately the same proportional effect in the range from three to thirty times the DWF, and that no small change in overflow setting would bring about a substantial reduction in the pollutional load discharged to the stream.

b. The BOD of the combined sewage was least for storms occurring in the latter part of the night and decreased with increases in flow. The BOD decreased with time since the storm started and increased with time since antecedent rainfall. The suspended solids content was not substantially affected by the time of day and the flow, and was generally greater in combined sewage than in raw sewage. The first flush of combined sewage contained the greatest concentration of objectionable material.

c. The deposition of considerable quantities of grit and organic solids in the sewer system during DWF was largely responsible for the first flush during storms.

d. A storage capacity equivalent to two hours of DWF would reduce the BOD discharged in the overflow of combined sewage by forty percent; the corresponding figure for a detention capacity of six hours of DWF was sixty percent.

e. The average yearly BOD load discharged from an overflow setting of six times DWF would be approximately equal to the total BOD discharged as effluent from a sewage treatment plant.

As a result of three separate stormwater investigations in England, the Ministry of Technology published some interesting results and observations (14). The areas studied were parts of Bradford and Brighouse, as well as the Northampton, which has already been discussed. When allowance was made for the differences in rainfall, impermeable area, and dry weather flow, it was found that there was a remarkable agreement in the data from the three areas. An equation describing the average yearly duration of flows in excess of the DWF was developed for the three areas. The equation was:

$$D = 20R[1+16.2(Q-q)/A]^{-4}$$

where:,

D = duration in hours during which the combined sewage exceeds a particular value (Q)

Q = the particular value of combined sewage flow exceeded in MGD

q = the DWF in MGD

A = the sum of the roofed and paved areas or impervious area in acres

R = the total rainfall in inches during the period considered which sould be at least a year.

The three study areas had populations ranging from five to ten thousand and times of concentrations of from 12 to 23 minutes. The chief difference in the three sites was the degree of impervious area which ranged from 11 to 50 percent. This equation was applied to a fourth area in England and compared with the known values of duration. As a result it was suggested that for areas with times of concentration greater than 30 minutes, the equation should be modified (14). The total volume combined sewage discharged as a result of any particular setting can be found by the integration of the observed distribution curve. The author stated that the equation cannot be considered valid for the highest flows (14).

A study of a 177.2 acre combined sewer drainage area in Baltimore was made in 1941 by Stegmaier (91). The area was mainly composed of private dwellings with approximately thirty percent of the drainage area being impervious. It was noted that the peak amounts of total, fixed, and volatile solids occurred with the peak discharge. During peak flow, the concentration of total solids was 8.7 times the average concentration in the DWF; the fixed solids, 13.8 times greater; and the volatile solids, 6.5 times greater. The data gathered indicated that the amount of solids carried by the flow was directly proportional to the discharge and that the first flush through the sewer did not scour out or reduce the solids during the peak flow.

The results of a study of the characteristics of discharges from separate storm sewers and combined sewers at two locations in Michigan provided information on the bacteriological quality of the discharges during periods of rainfall (29). The combined sewer system served an area of 21,000 acres in Detroit, while a separate storm sewer in

Ann Arbor was chosen which served 3,800 acres. Automatic sampler were installed at both locations to gather composite samples of the discharge. The discharges from the separate sewer system contained quantities of total coliforms, fecal coliforms and fecal streptococci with the mean density of total coliforms for the six summer months being 1,200,000 organisms per 100 ml. The mean density of total coliforms for the combined system during the six summer months was 9,400,000 per 100 ml. This result indicated that a considerably larger quantity of total coliforms are discharged from combined sewers than from separate storm sewers. During this study the mean coliform concentration of the combined discharge was eight times that of the separate storm sewer.

The suspended solids in the overflow from a town system in Michigan varied from 42 to 1300 mg/l with an average of 198 mg/l, which was 112 percent of that found in the dry weather flow. The volatile suspended solids concentration ranged from 15 to 82 mg/l with an average of 50 mg/l, which was 42 percent of that of DWF. The BOD ranged from 10 to 229 mg/l with an average of 52 mg/l, which was 37 percent of that of DWF. The MPN values varied from 93,000 to 1,100,000 averaging 500,000 which was 86 percent of that of DWF (57).

Holding Tanks As A Means Of Reducing Overflows From Combined Sewers

The design capacity of a holding tank is governed by the desired frequency of combined sewer overflow to the receiving stream, and is therefore dependent upon local rainfall conditions. An elimination of pollution may be realized by the construction of holding tanks which function in four ways:

1. To serve as a storage reservoir for the complete retention of excess storm flows during periods of light to moderate runoff
2. To serve as a detention device for the combined sewage overflow and to effect the removal of suspended and floating solids
3. To equalize flow to the existing treatment plant. This equalization enables some of the potential overflow to receive treatment.
4. To capture the first flush of an overflow in situations involving high intensity storms.

The city of Columbus, Ohio constructed stormwater holding tanks in 1934 for the purpose of providing storage and partial sedimentation of the combined sewage overflow. Prior to the construction of these tanks, the dissolved oxygen level in the receiving streams was zero in certain locations and the flow contained large quantities of solids.

The rainfall intensity-duration-frequency curves selected for design purposes allowed combined sewer overflow not more than four times per year for some sections and not more than eight times per year for the remaining sections. One group of these tanks was still operating effectively as of January, 1966. This group consisted of three tanks, each 180 x 105 x 10 feet. Automatic controls have been added for the regulators, emergency bypass channel, drain gates, and inlet gates (54).

The city of Toronto has two storm holding tank systems. An open tank handles flows in excess of nine MGD and is reported to be quite effective in retaining solids, even though it must be dredged periodically. The BOD removal efficiency of a 90,000 cubic foot storage system for a 2,196 acre drainage area was estimated to be fifty percent, but this efficienty was believed due to the small drainage area (16).

Dunbar and Henry (42), suggested that banks of large storm water holding tanks at points of intersection between the interceptor and laterals may not prove to be economically feasible and possible even physically impossible. A system of smaller local tanks in the drainage basin of interest may be more economical as well as practical. An advantage of this type of system would be that tanks could be added as the area developed. Dunbar and Henry concluded that by providing sufficient volumetric stormwater storage for a storm with an intensity of 1.2 inches per six hours, the number of occurrences of overflow per year may be reduced by greater than ninety percent.

A combined sewage holding basin was recently designed for Macomb County, Michigan to alleviate the problem of combined sewage overflow which normally occurred about 35 times per year. The reinforced concrete box has a total capacity of twelve million gallons and cost $800,000. The structure has a reinforced concrete top and a forced air blower to disperse odors (13).

The effect of storage at a hypothetical overflow was studied to determine the reduction in pollutional load to the stream for various overflow settings at Northampton, England (39). The theoretical tank was artificially operated to receive the first flushings of the stormwater as soon as the flow in the sewer exceeded the overflow setting. This material was to be returned to the sewer as soon as the flow fell below the overflow setting. The overflow setting selected for this limited study was 6 times DWF for two values of tank storage capacity, which gave two and six hours detention of the average DWF. It was pointed out that in practice the storage capacity could be provided by a sufficient enlargement of the sewer, the flow to the treatment plant being restricted to the desired value by means of a throttle pipe. The overflow would not operate until the entire storage provided had been utilized. Calculations of the reduction of BOD load discharged to the stream by utilization of the theoretical detention tanks were made for 68 storms, when the flow rose above 2 MGD (6 X DWF). It was found that the reduction realized by the smaller tank for suspended solids and BOD would have been equivalent to raising the overflow setting from 6 to 8.6 and 8.4, respectively. The corresponding values for the larger tank would be equivalent to raising the overflow setting from 6 to 12.0 and 11.0, respectively. This amounts to a percentage saving in total BOD load discharged during the year of 13 percent for the smaller tank and 20 percent for the larger tank (50).

A sewage treatment plant near Skegness, England was provided with holding tanks to enable partial treatment of flows between three and six times DWF. When the rate of inflow to the plant exceeds three times DWF, a stormwater pump comes into operation and the excess flow is pumped to one of four horizontal flow holding tanks with a total capacity of 86,000 gallons. At the end of the storm the tanks are flushed back into the incoming sanitary flow (22).

Two primary settling tanks with the equipment removed act as holding tanks at Falkirk, Scotland. The raw sewage arriving at the sewage treatment plant first passes through the holding tanks. The two tanks with a combined storage of 450,000 gallons provide a 36 minute detention time before discharge to the receiving water course (21). Many other English localities have made use of holding tanks (41,17,18).

Chicago is presently studying a totally new concept for controlling combined sewer overflow. The plan envisions temporary storage underground of excess sewage flow from combined sewers. The storage would be provided by a system of tunnels and chambers, 800 feet below the city in solid rock. After the storm, the stormwater would be pumped to the surface for treatment. This proposal is still under study (19).

The disposal of surface water from a 5,900 acre concentrated development area in England caused a problem because the receiving watercourse was not large enough to handle the storm flows. A study indicated that a man-made lake would suffice to limit the discharge of stormwater to the receiving watercourse. The lake would provide a storage capacity of 3.5 million cubic feet, which was adequate for the ten year storm (36).

A retention basin of 400,000 cubic feet per 1000 acres for Jamaica Bay, New York was studied which would prevent the discharge of floating sewage solids and provide sufficient contact time for chlorination. The size of the retention basin was such as to guarantee a 15 minute detention time of the maximum flow capacity. This procedure was, in general, adopted for the abatement of the Jamaica Bay pollution problem. A typical arrangement and operating policy was described (53). The total cost of the elimination of marginal bacterial pollution from Jamaica Bay was estimated to be $40,000,000, of which $23,000,000 was for changes and alterations of the existing sewer system. The remaining $17,000,000 was for the construction of seven overflow treatment devices. The cost of the treatment facilities was $1,100 per acre. It was noted that this was less than seven percent of the total cost of complete separation (53).

Chlorination And Sedimentation Of Overflows From Combined Sewers

The use of chlorine for treatment of sewage and municipal water supplies was patented more than 100 years ago and was first used in the United States in 1908 for the continuous disinfection of the Jersey City, New Jersey, water supply (46). The amount of chlorine required to produce the desired kill of bacteria and other organisms in sewage depends on the quantities of soluble and insoluble materials and the nature of the substances present. In 1936 it was pointed out (81) that settleable solids had a considerable chlorine demand and that their rate of absorption was slower than that of the suspended solids. Even though settleable solids do not absorb much chlorine per unit weight, they are very important due to the fact that they represent a high percentage of the total solids. Rudolfs and Gehm (81) investigated the chlorine demand of the different fractions of sewage. Their results are reported in Table IV, which shows that while the suspended and non-settleable fraction represents a small percentage of the total solids, their chlorine demand per gram of solids is such that they are responsible for a significant portion of the overall chlorine demand.

Numerous investigators (34, 3, 4, 49) have studied the effect of contact time and concentration of chlorine on the effectiveness of chlorination of sewage and stormwater. Camp, (34) investigating the effectiveness of penetration of chlorine in comminuted raw sewage, found that the chlorine demand after two minutes ranges from 4 to 23 ppm. He also noted that with chlorine doses of 20 to 38 ppm, coliform kills in excess of 99.9 percent were achieved in less than ten minutes in all but one of his experiments. After dechlorination, the samples were blended in a Waring blender and showed a five to ten fold increase in MPN.

The effect of chlorination on enteric viruses has been studied by Kelly, Sally, and Sanderson (61,62). In one study five strains of polio and two strains of Coxsackie

TABLE IV. The Relative Chlorine Demand of Various Constituents of Sewage As Presented By Rudolfs And Gehm(81).

Fraction of Total Solids	Cl_2 Consumed ppm	Cl_2 Demand of Total %	Total Solids ppm	Cl_2 per gram solids mgm
Settleable	5.3	30	592	9
Non-settleable	3.1	17	26	119
Suspended	4.2	23	16	262
Dissolved	5.3	30	634	8

virus were examined. At pH values between six and eight a free chlorine residual of at least 0.3 ppm for a contact time of at least thirty minutes was required for a 99.9 percent kill or inactivation of enteric viruses. A contact period of one hour was required for a combined residual chlorine content of 10 ppm to insure a 99.7 percentage kill or inactivation of enteric viruses. In another experiment (71), it was concluded on the basis of laboratory experiments that 0.3 to 0.5 mg/l of free chlorine is sufficient to inactivate three types of polio virus and six other types of enterovirus suspended in buffered saline water at pH seven at 22 degrees Centigrade for ten minutes. Camp (34) suggested that if a ten minute chlorine contact period was provided for the capacity of the combined sewers and that if a chlorine dose of about ten ppm greater than the one hour demand was applied for flows up to five to ten times the DWF, that there would be a sufficient contact time and combined chlorine residual to inactivate the enteric viruses.

Frost, Balch, and LaCava (49) made a thorough study of the expected bactercidal effects under varying conditions of chlorine dose, demand, residual and detention time. The samples were allowed to settle for two hours in Imhoff cones, after which 99 ml of settled sewage were removed by siphoning. The settled sewage samples were then innoculated with 5, 10, 15, 20, and 40 ppm of chlorine with detention times of 1, 5, 10, 15, 30, 60, 120 minutes and two days. The two day detention was used to determine the aftergrowth. This study was conducted in order to formulate a "safe trend curve", for the kills which "could be anticipated" 100 percent of the time. There were approximately seventy test results for each specific chlorine addition. Some of the results of these tests are presented in Figure 1.

Studies were undertaken by McCarthy (34,69) in 1960 to investigate the required chlorine doses for sewage mixed with stormwater. Tap water was substituted for the stormwater. The dilution ratios of tap water to sewage studied were 10 to 1, 25 to 1, 50 to 1, and 100 to 1. Sufficient chlorine was added to produce a free chlorine residual of one ppm after twenty minutes. Camp (34) suggested that tap water was a satisfactory substitute for stormwater because of the combined chlorine demand of the high free ammonia content of the tap water. The diluted sewage was gently stirred during the contact period. The membrane filter technique (MF) was used for coliform counts with an occassional check by the conventional dilution tube method. The coliforms were counted after varying contact periods and in some of the runs were also counted after two minutes in a Waring blender. The results of the 10 to 1 dilution indicated that the required chlorine dose to produce a one ppm free chlorine was 8.5 ppm. These tests indicated that a substantial coliform reduction can be achieved with

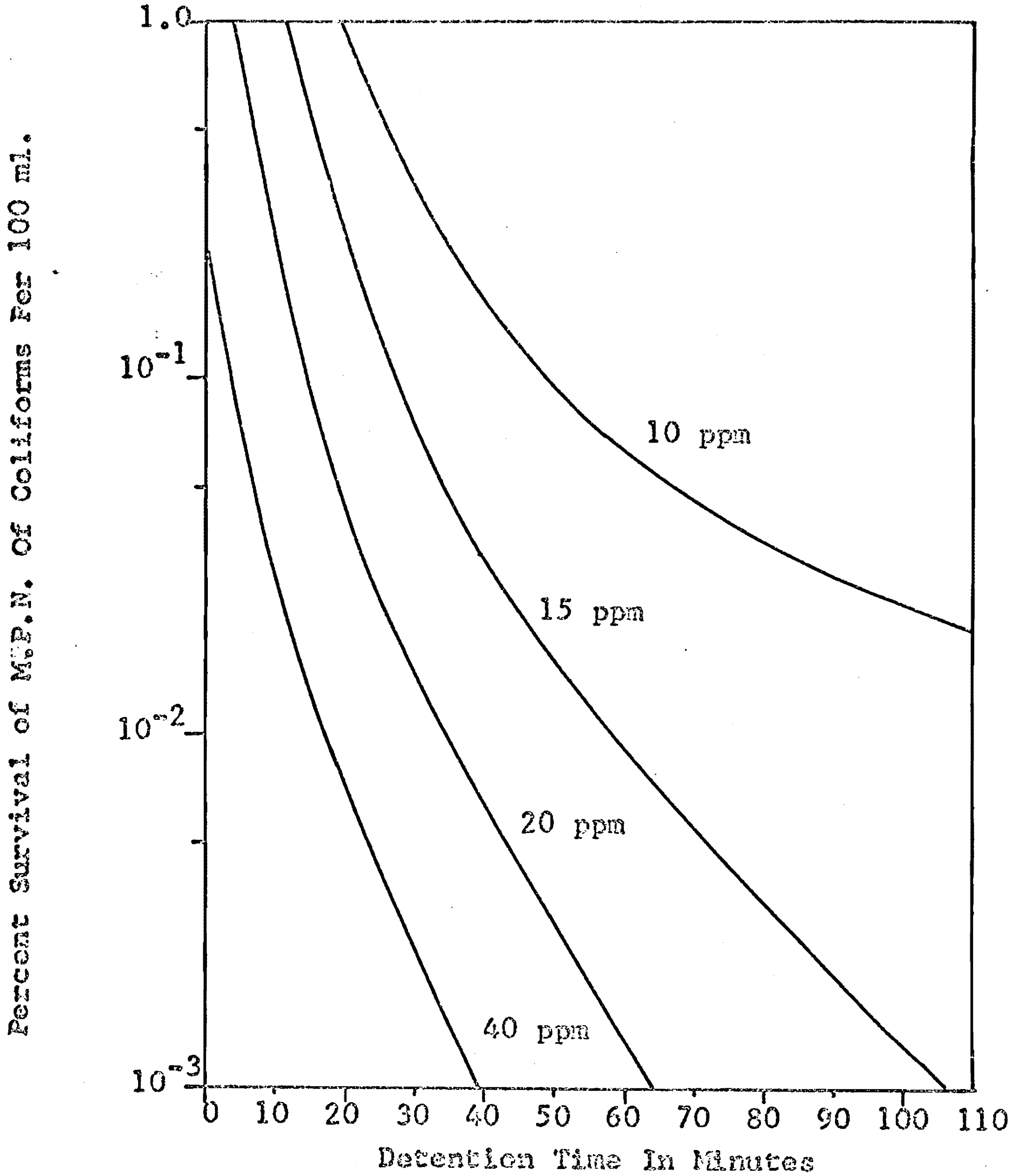

FIGURE 1. The Bactericidal Effects of Varying Chlorine Dose And Detention Time on settled Sewage As Reported by Frost, Balch, and LaCava(49).

chlorine doses very much less than are required with normal municipal sewage. Camp (34) suggested that these results should be verified with data on actual stormwater.

Romer (15) disagreed with McCarthy's assumption that the diluted raw sewage was representative of combined sewage. He stated that the BOD and suspended solids concentration in combined sewagegreatly exceeds that of DWF. Romer stated that it is not unusual to have a suspended solids content of as high as 1000 mg/l or 2000 mg/l and that 5000 mg/l was not impossible.

The Cincinnati Water Research Laboratory of the United States Department of the Interior published in January of 1967, the results of a two year study on stormwater (45). Fifty storms were included, and the results of laboratory settling and chlorination studies of stormwater runoff from nine storms in the Cincinnati area were presented. The runoff was treated by settling for periods of ten minutes to twenty-four hours, and by chlorination of two to nineteen mg/l with a twenty minute contact time, followed immediately by dechlorination for purposes of determining the bactercidal effect of the chlorine.

The concurrent settling and chlorination studies were made in 2,000 ml graduates. A standardized chlorine solute was added to each graduate and thoroughly mixed. Samples were withdrawn after twenty minutes, dechlorinated, and the free residual and total residual chlorine were found along with the bacterial reductions of the total coliform group, fecal coliforms and fecal streptococci. Examinations were also made to determine the aftergrowth of each indicator. The membrane filter technique was used to determine the densities of the total coliforms, fecal coliforms, and fecal streptococci. The authors attributed the wide variation in the amount of chlorine to effect a 99.99 percent kill to the components creating the chlorine demand rather than the number of coliforms present. The densities of the total coliform organisms after twenty minute concurrent settling and chlorination ranged from 2 to 4,500 per ml. In Figure 2, Evans, et. al. (45), present the bactercidal results on the total coliforms, fecal streptococci and fecal coliforms with concurrent twenty minute settling and increasing chlorine dosage. Evans, et. al., noted that twenty minutes of settling alone effected practically no removal of coliforms, but that twenty minutes of chlorination produced approximately the same rate of kill of all three groups. It was also concluded that the leveling off of the curve after six mg/l of chlorination indicated that some well protected organisms survived the twenty minute detention. The investigators also concluded that the presence of free chlorine after twenty minutes would not guarantee that virtually all the coliforms had been destroyed.

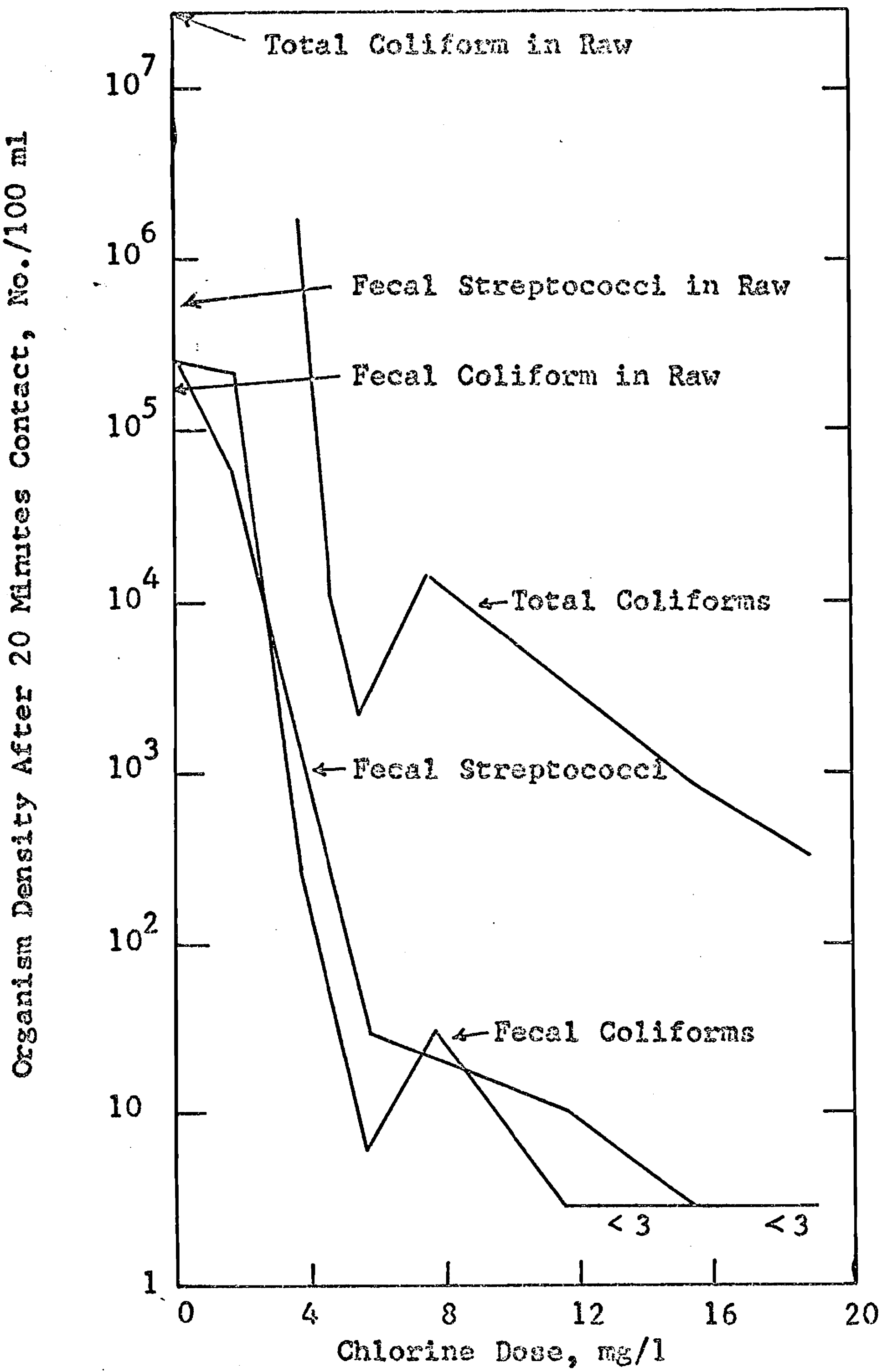

FIGURE 2. Bactercidal Results of 20 Minute Concurrent Settling And Varied Chlorination on Stormwater Runoff as Reported by the Cincinnati Water Research Laboratory(45).

Aftergrowth was experienced for total coliforms even at the highest chlorine dosages (45). The aftergrowth was followed by a natural die-off. Fecal streptococci and fecal coliforms did not show the aftergrowth exhibited by the total coliform group. Therefore, the aftergrowth of the total coliforms was not attributed to the fecal coliform segment, but to Aerobacter aerogenes, as it is the most responsive organism to very minimal amounts of nutrients. Evans, et. al., (45) recommended that fecal coliforms be used as an indicator of the effects of the chlorinated stormwater discharged to the stream.

Camp (34), in order to determine whether aftergrowths of coliform bacteria in the receiving stream should be expected after discharges of chlorinated sewage, diluted the chlorinated effluent with tap water. After a ninety minute chlorine contact time, the samples were diluted in ratios of 16 to 1, 80 to 1, and 200 to 1. Coliform counts were then made immediately after dilution, and at 24, 48, and 72 hours. The results indicated that there was no appreciable aftergrowth as all counts were less than 240 per 100 ml. Camp, based on his other experiments, then concluded that the post-chlorination of a well settled sewage provides no better bactericidal results in the receiving stream than the chlorination of comminuted raw sewage.

In March of 1960, the staff of the New Hampshire

Water Pollution Commission published their report on the bactericidal efficiency of concurrent chlorination and fifteen minute settling. This study was divided into three separate, but closely related chlorination studies (3).

The purpose of the first study was to determine whether or not a practical level of chlorination of raw sewage existed such that 99 percent of the time the effluent would not contain more than 240 coliforms per 100 ml (3). The sewage was not settled and the effluent was subjected to mixing in a Waring blender to break up the particulate matter before the coliform count was made. The samples during chlorination were slowly stirred to prevent settling. Chlorine residuals were measured at the end of the fifteen minutes and the samples were dechlorinated with a dilute sodium thiosulfate solution. Chlorine dosages of 10 to 170 ppm were used. This study showed that no practical chlorine dose of 40 ppm or less existed such that the New Hampshire B-1 bathing water quality standard of 240 coliforms per 100 ml or less could be met with this type of treatment.

The second study (3) was initiated by the staff of the New Hampshire Water Pollution Commission to determine if a practical level of chlorination, contact time, with concurrent high rate agitation of raw sewage existed which would guarantee that 99 percent of the time the coliform count in the effluent would not exceed 240 per 100 ml, when the effluent was subjected to particulate break up. The procedures followed here were essentially those of test one, except that the raw sewage was agitated during the chlorine contact time, and the contact time was varied. As a result of this study, the authors concluded that 30 ppm of chlorine, with a 15 minute contact time in a Waring blender was the lowest value of each parameter which would guarantee that the coliform concentration of the effluent would not exceed 240 per 100 ml. The authors also pointed out that consideration should be given to the energy costs of the agitation.

In another study by Camp of the effect of violent agitation and chlorination (34), it was concluded that increased violence of mix in the chlorine contact chamber was not sufficient to significantly increase the rate and percentage of kill above what might be expected with no mixing. In the chlorination of raw sewage, McCarthy (69) found that increased agitation in a model chlorine contact chamber did not significantly raise the percentage kill of coliform organisms with respect to the percentage kill in gently stirred sewage. Camp (34) concluded that no agitation is needed except to thoroughly mix the chlorine with the sewage and that agitation does not significantly increase the efficiency of chlorination.

The third study was initiated by the staff of the

New Hampshire Water Pollution Commission to determine if a practical level of 15 minute concurrent chlorine contact and settling time existed which would guarantee that 99 percent of the time the effluent would have a total coliform count of less than 240 per 100 ml (3). As in studies one and two, the effluent was subjected to violent breakup of particulate matter after chlorination. The procedures followed were essentially those of studies one and two. The staff found that 100 percent of the time, the supernatant of the raw sewage, subjected to 15 minute concurrent settling and contact time at a chlorine dose of 20 ppm, contained an estimated coliform density of less than 3.2 per 100 ml. In the effluent subjected to violent agitation estimated coliform densities of less than 6.7 and MPN's of equal to or less than 23 per 100 ml were found 100 percent of the time. It was concluded that of the coliforms remaining in suspension after the concurrent 15 minute contact and settling time, only an insignificant amount were not killed by the chlorine due to their protection in the dissolved solids.

It has been concluded that the chlorination of mixtures of stormwater and raw sewage effectively reduce the bacterial concentrations in the effluent. The total quantity of storm runoff is approximately forty percent of the total sanitary sewage flow in the Eastern parts of the United

States and Canada (15,34,42). The total yearly quantity of chlorine required to treat the overflows from combined sewers has been estimated to be only about twenty percent greater than that required to treat the sanitary sewage alone.

Cost data for chlorine contact tanks for stormwater overflows has been gathered for three cities in Massachusetts (8). The average unit cost was about $4,200 per acre. For these cities the total cost of chlorine per person per year was about fifty cents. This data is presented in Table V.

Alleviation of Overflows From Combined Sewers by Complete Separation of Storm and Sanitary Flows

The present practice in the United States tends to favor separate sanitary and stormwater systems for newly developed urban areas. One of the reasons for this trend is the need for elimination of overflows containing raw sewage solids. A possible drawback is that often stormwater from densely populated areas contains highly objectionable material. Eventually, even the complete treatment of the stormwater will probably be required in the larger metropolitan areas in order to guarantee bathing quality standards for the receiving watercourse during the recreational season. The complete separation of all existing combined sewers was estimated to cost 27.4 billion dollars in 1964 (8). All new construction supposedly incorporates separate systems.

TABLE V. Cost data for Chlorine Contact Tanks for Partial Disinfection of Combined Overflows (8).

City	Total Project Cost	Cost Per Acre	Cost Per Capita	Annual Chlorine Cost
1	$11,500,000	$4,050	$250	$30,000
2	9,800,000	4,400	140	24,000
3	23,700,000	4,060	264	56,000

A separate sewer system is desirable when the sewage must be collected at a single point, or it must be pumped. Sanitary sewers must be at sufficient depth to guarantee basement drainage, while storm sewers may be located at a depth which would permit gravity flow. It is also difficult to guarantee a cleansing velocity in combined sewers during the DWF.

The construction of a separate system in the older parts of cities has proven to be very expensive. The estimated cost for separation of the District of Columbia's combined sewers was $214,000,000 or about $18,000 per acre. In Chicago, the complete separation of sewers was estimated to cost $2,250,000,000, or about $17,000 per acre (35,42,53,59,75). In Detroit, the estimated cost of complete separation of the existing combined system was $235,000,000 or approximately $130 per capita (42,77). The estimated cost of providing the whole city of Toronto with a separate system of sewers was $21,500 per acre. The cost of providing a new sanitary sewer system for a study area in Toronto was estimated to be $17,000 per acre (16,42). A combined sewer in Minneapolis was separated into sanitary and storm lines by the installation of oval corrugated steel pipe in the bottom of the existing combined sewer. Concrete was placed around the sides of the corrugated pipe to anchor it and also to provide a smooth surface for the stormwater(11).

This system was also utilized recently in a section of Cleveland, Ohio (12).

The District of Columbia amended its plumbing code in 1959 requiring every new building to install separate plumbing systems to a point beyond their property line. This was also required of all buildings undergoing remodeling. By the year 2000, the city hopes to have a completely separate system (23). A 552 acre section in southwest Washington was completely separated at a cost of 81,000 dollars per acre. In 1965 it was reported that separate systems had been installed in 1,150 buildings with an average cost of 1,500 dollars per house (23).

The U. S. Department of Health, Education and Welfare estimated that the average cost per acre for separation was $12,500, and the average cost per capita was $465, based on fifteen observations of different metropoliton areas. The costs varied from $1,800 to $30,000 per acre. The higher costs were for the more densely populated cities. The monetory loss of individuals, businesses and communities which would occur as a results of the physical inconvenience during the construction of separate systems was not considered (8).

The high cost of complete separation has led the U. S. Department of Health, Education and Welfare, and the Federal Water Pollution Control Administration to give grants to organizations to study other and hopefully cheaper ways of combating pollution from overflows of combined sewers (8).

Decreasing The Quantity of Combined Sewer Overflow By Increasing The Interceptor Size

The problem of sizing interceptors for combined sewers in terms of the average DWF has received considerable attention in England and the United States. The requirement of the Ministry of Housing and Local Government in England for combined sewers is that the sewage treatment plant must be able to provide full treatment for flows up to three times the DWF and sedimentation for flows between three and six times DWF. The interceptors are normally designed for six times DWF with the overflow devices also set for six times DWF (63). The overflow setting and treatment standards in England are much higher than those usually practiced in the United States, due to the low consumption of water in England and the high yearly rainfall (42).

An interceptor capacity of 1.5 to 5 times the DWF is a normal design in the United States. If the peak DWF occurs simultaneously with the stormwater runoff, then the capacity for the stormwater in the system is seriously reduced. McKee (70) presented an analysis of the relationship between interceptor capacity and the amount of sanitary sewage lost through stormwater overflows and the frequency of overflows for Boston. His basic conclusions are presented in Table VI. In order to obtain a fifty percent reduction in the amount of sewage lost in overflows, it would be necessary to provide five times the DWF capacity in the interceptor. Even this would only reduce the average number of overflows per month from 5.5 to 4.

The Mersey River Board in England has recently suggested that combined sewage overflow settings be raised from six to eight times the DWF (67). The raising of the overflow setting would mean that the frequency of overflows would be reduced, the quantity discharged to the stream reduced, and the initial flush of the sewers would have a better chance to be intercepted before the actual overflow began. The disadvantage of increasing the overflow setting would be the additional cost of increasing the interceptor size. The work at Northampton (50) indicated that an increase in the overflow setting from six to eight times the DWF would reduce the quantity of combined sewage overflow by twenty-five percent.

A study by Banjes, et. al., was made to determine the relative sewer costs when varying quantities are intercepted (28). A base of comparison was the minimum design capacity of the peak DWF, which was assumed equal to 1.5 times the DWF. A 36 inch pipe, n=0.013, flowing full, with a velocity of two feet per second was assumed. Based on

TABLE VI. Relationship Between Interceptor Capacity And The Amount of Combined Sewage Lost Through Stormwater Overflows For Boston As Reported By McKee (70).

Ratio of Int. Capacity to Dry Weather Flow (1)	Percent of Sewage Lost (2)	Number of Overflows Per Month (3)	Average Interval Days (4)
2	2.6	5.5	5.5
3	1.9	5	6
4	1.5	4.5	6.7
5	1.2	4.0	7.5
10	0.5	3.1	9.7
20	0.2	2.1	14.3
50	0.1	0.8	37.0

these assumptions, the relative costs of various interceptor sizes were determined. To obtain a reduction from two percent of the combined sewage loss for an interceptor capacity of two times the DWF, to one percent with an interceptor capacity of five times the DWF, the interceptor costs would be nearly doubled. Benjes concluded that the small reduction obtained by increasing the interceptor capacity by 2.5 times did not appear to justify the expenditure.

Economic Analysis

The success of an economic analysis depends on the accuracy of the unit-process data used in the model. Economic considerations should always be a very fundamental part of any engineering analysis. In fact, from a practical point of view, an engineering design without economic considerations is meaningless. In order for economic analysis to be meaningful, cost, efficiency, and technical considerations should each be an inherent part of the engineering design.

Utilization of the relationship between cost and efficiency of individual units or processes make it theoretically possible to design and develop the optimal economic plant for a given set of conditions. From the economic considerations, there is only one optimum or most economical solution for a given set of constraints. There are, however, many practical considerations other than cost which must play a significant part in any decision making process. While these enviromental and technical considerations may outweigh the purely economic factors, there is nevertheless an important need to be able to economically discriminate between the alternatives.

Schroepfer (86) in 1939 made one of the first attempts to directly correlate the related costs and efficiencies in the field of sanitary engineering. Schroepfer analyzed the relationship between construction and maintenance cost of treatment with the suspended solids and BOD removal efficiencies.

The development of the digital computer with its time-saving features has encouraged more intensive study of the relationship between costs and efficiencies in sanitary engineering. Basic unit costs, aside from the original Schroepfer study, are not readily available as most recent cost analyses in sanitary engineering have only dealt with the overall process or system costs. Overall cost studies, such as those of Rowan (79,80), Thoman (82), Velz (93), and Logan (65), do satisfy the need for economic guidelines in the choice between different waste treatment methods. The United States Department of Health, Education and Welfare have issued three publications (7,9,10) dealing with the costs of sewage treatment and proposed two new cost indices to be used in connection with the cost estimation of sewage treatment plants and sewer construction.

The Department of Health, Education, and Welfare, noting that existing economic indices were not significantly applicable to sewer construction, developed a Sewage Treatment Plant Construction Cost Index in 1963 (9). This index does not reflect the cost of land, engineering, legal, and fiscal services, but does include the following items which are common to sewage treatment works:

1. cost of materials
2. cost of process equipment
3. construction labor cost
4. contractors equipment cost
5. overhead and profit.

The base period selected for this index was the thirty-six month period from January 1, 1957 through December 31, 1959 (9).

A comparison of the Engineering News Record Construction Cost Index (ENR) and the Public Health Service Sewage Treatment Plant Construction Cost Index (PHS-STP), disclosed that there were substantial differences in the two indices. The ENR Index had increased at a rate almost double that of the PHS-STP Index between 1956 and 1962. The 1963 ENR Index was 430, while the PHS-STP index was only

310. It is believed that since the PHS-STP index is based on information peculiar to the sewage treatment plant construction, its value is more significant for this specialized field.

Aware that there were vast differences in quantities of material, men, and equipment required for sewer construction, as compared to sewage treatment plant construction, the Public Health Service prepared the Sewer Construction Cost Index (PHS-S) in 1964 (10). The same base period was selected for the PHS-S index as was for the PHS-STP index. The PHS-S index was designed to reflect the same parameters as was the PHS-STP index. A total of 733 sewer projects constructed between 1956 and 1962 were studied. The pipe size varied from 8 to 96 inches in diameter with a combined total length of 3,456,000 linear feet. The Public Health Service suggested that for the purpose of estimating the land, legal, and engineering fees, an allowance of 20 percent of contract cost could be used (9).

An analysis of the cost per diameter inch per foot of reinforced concrete pipe over a number of years revealed that there was a constant price differential between pipes of varying diameters. The method of least squares was used to determine the best fitting linear curve of the form $Y = A + BX$; where Y is the cost per diameter inch per linear foot and X is the diameter of the pipe in inches. The resultant equation for reinforced concrete sewer pipe for Kansas City, Missouri, in August of 1962 was:

$$Y = 0.032066 + 0.021368\ X \qquad (10)$$

The present price of reinforced concrete sewer pipe may be found by comparing the PHS-S index of the area and time of interest with the August 1962 PHS-S index of Kansas City. Both PHS cost indices are published monthly for 20 areas of influence in the United States.

General Summary

The problem of pollution caused by stormwater overflow of combined sewers is one which has only recently begun to receive significant attention as a source of water quality impairment. Polluting discharges from combined storm and sanitary sewers occur during wet-weather periods when the interceptor capacity is not sufficient to carry the abnormal discharge. The average cost of separation of stormwater and sanitary sewers is around $13,000 per acre and the stormwater itself contains a significant amount of pollutants.

The degree of contamination and pollution caused by stormwater overflow of combined sewers is often very significant, especially when the discharge during the recreational season is once every few days. Containing the overflow by increasing the interceptor size does not appear economical as a large increase in interceptor size does not produce a substantial reduction in overflow. The use of local stormwater holding basins appears to offer a feasible solution to the problem. Chlorination of the overflow at 30 ppm was reported to attain a significant reduction in the coliform density.

The complex, long range nature of the overflow problem should be recognized as an essential consideration in urban planning. A present concern now evident is that the problem of stormwater overflow will increase at a rate greater than appropriate corrective measures can be found and applied. The work undertaken in this study is aimed at defining reasonable alternatives in an effort to close this gap.

DEVELOPMENT OF THE GRAVITY SEDIMENT SEPARATOR TREATMENT DEVICE

Many new schemes and ideas for the elimination or control of overflow from combined sewers have been recently proposed. Some of the proposals aimed at overflow elimination utilize underwater storage in inflatable rubber tanks, while others advocate pumped underground storage. The proposed alternatives for the solution of the problem have advocated either physical separation of the sanitary sewage and storm runoff, or alternatively have systems incorporating storage plus increased treatment facilities. Physical separation has not been demonstrated to be practical due to the cost involved. Systems involving storage plus treatment have either been limited by cost or effectiveness.

One new approach to the control of the combined sewer overflow problem is the gravity sediment separator proposed by W. A. Parsons. This unit would be designed to enable chlorination and removal of settleable solids from the overflow and would assist in the maintenance of water quality standards throughout the recreational months.

The gravity sediment separator is a unit conceived to separate, principally by gravity, a concentrated stream containing all the readily settleable solids, the solids carried as a bed load, a significant fraction of the slow settling suspended load, and tank skimmings for transfer to the sewage treatment plant in the interceptor. The proposed unit would also feature chlorination of the overflow from the combined sewer at a dose of about 30 ppm. The object of the treatment unit would be the diversion of a concentrated underflow of solids to the sewage treatment plant in the existing interceptor, and the release of an essentially sediment-free bacteriologically safe discharge to the receiving watercourse at a reasonable overall cost. The sediment separator is expected to maintain the esthetic as well as the sanitary quality of the receiving stream.

The proposed unit would employ submerged longitudinal baffles to effect a progressively decreased turbulence from the upper to lower strata in the unit. Physically, the unit may be expected to have more particle migration, as a result of more turbulence, in the upper than in the lower strata. Such a condition would provide a net transfer of sediment from the upper strata to the lower strata of the sediment separator irrespective of gravity. This mechanism is expected to be particularly effective at high flow rates.

The submerged longitudinal baffles should promote a well distributed shear gradient that will promote collisions between suspended particles and captalize on any tendency for flocculation. Aggregated suspended matter is more readily captured by the mechanism of gravity settling. The mechanism of flocculation is expected also to be particularly effective at high flow rates.

The gravity sediment separator is envisioned to have efficient inlet and outlet structure and efficient geometry to enable it to provide outstanding performance relative to gravity sedimentation. The gravity sediment separator would be a device designed to perform efficiently as a conventional gravity settling tank that in addition will incorporate mechanisms of positive flocculation and controlled turbulence migration. The unit differs from a conventional primary settling tank in that the objective is not only to capture debris deposited on the bottom of the tank, but to separate and transmit to the sewage treatment plant a solids-laden underflow consisting of deposited solids, bed load, and suspended matter flowing adjacent to the tank bottom. The underflow or sludge volume withdrawn from a conventional primary sedimentation tank is about 0.5 percent of the inflow. The gravity sediment separator is envisioned to have an underflow of approximately 20 times that of the primary sedimentation basin or 10 percent of the dry weather flow. A scum weir would also be provided to capture the floating solids.

The gravity sediment separator would be applicable for installation at combined sewer overflow regulators and at overflow bypasses of sewage treatment plants. The solids-laden underflow from the separator would be transmitted in the existing sewer to the sewage treatment plant. The separator would require a washdown after use and would be kept empty during dry weather so the storage capacity would be available for overflows caused by moderate storms. The envisioned gravity sediment separator is presented in Figure 3. The concept of the proposed operation of the unit is similar to other schemes for storage treatment and chlorination of overflows such as the one proposed by Greeley and Langdon (47), but differs in that the envisioned gravity sediment separator would feature improved sedimentation of solids and fuller utilization of the existing sewage treatment plant.

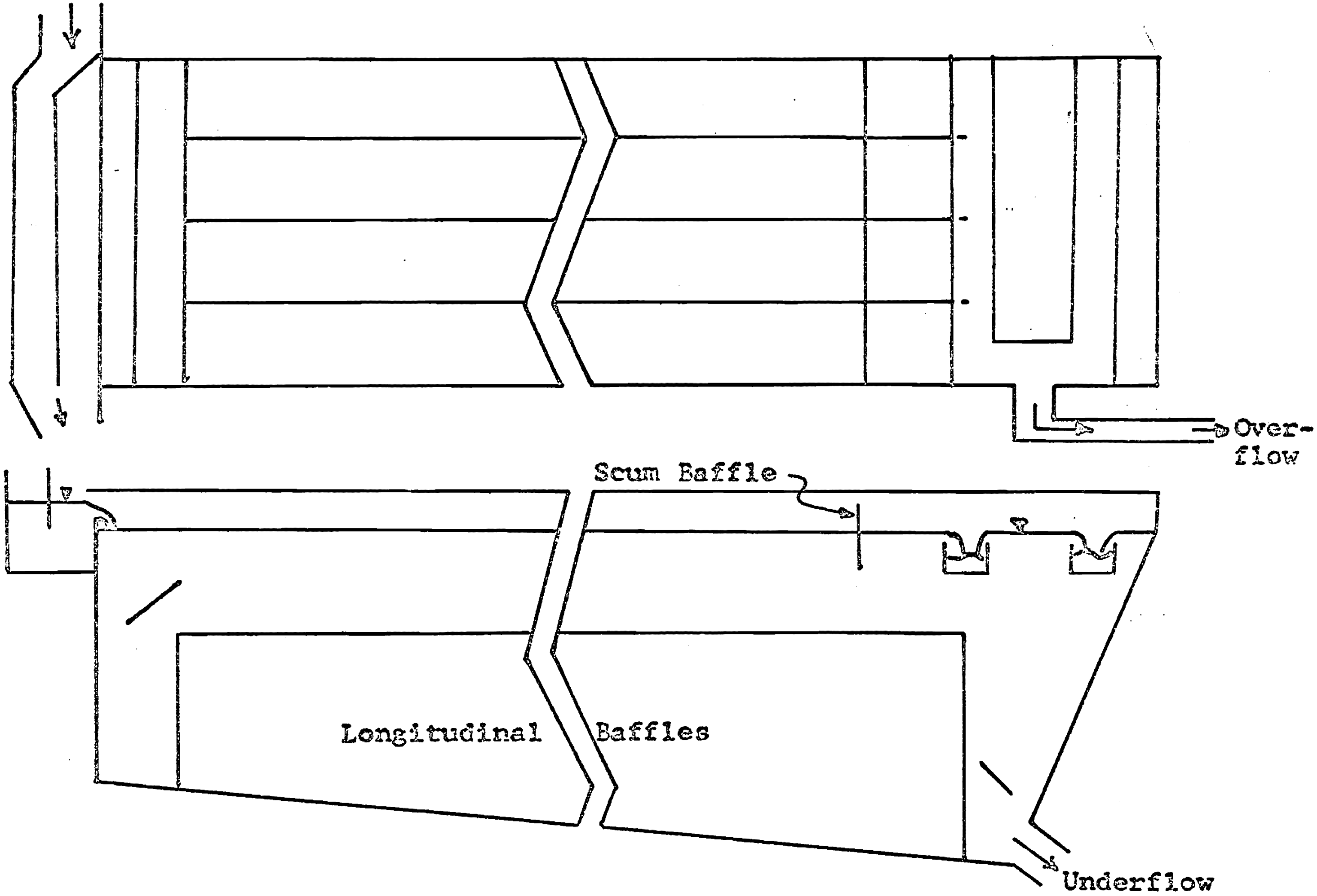

FIGURE 3. Skematic Representation of Proposed Gravity Sediment Separator.

METHODS OF ANALYSIS

The evaluation of the performance, characteristics and cost of a stormwater holding basin and of the proposed gravity sediment separator is the primary object of this study. The complete evaluation of any physical system including a description of the existing situation is prerequisite to satisfactory evaluation of any proposed modifications to the system. An existing combined sewer system may first be mathematically modeled and described in terms of the appropriate governing criteria. The stormwater holding basin and the gravity sediment separator may then be described by mathematical models in order to evaluate the performance of each as opposed to the original system. Both preventive systems may then be evaluated in terms of the reduction of combined sewer overflow of such parameters as BOD and suspended solids. In addition the annual cost of treatment per acre including maintenance may be calculated.

The recreational season of May through September was chosen for this study as it represented the period of the most critical pollutional problem as more overflows occur during this season as opposed to the remaining portion of the year. The increased demand for recreational use by the public during this season justifies confining the principal study to this period.

The existing combined sewer system was described in terms of its capacity, and the average dry weather flow (DWF) was expressed as a function of tributary area. The interceptor capacity was expressed in multiples of the DWF. The capacities studied were 2, 3, 4, 5, and 6 times the DWF. The DWF was assumed equal to a runoff of 0.01 inches per hour per acre of drainage basin. The size of the hypothetical basin may be negated by describing all the appropriate parameters in units per acre.

The hydrologic influences and descriptions are very important in their effect upon the frequency, magnitude and duration of the combined sewer overflow. In order to mathematically model the hydrologic influences on the system, the precipitation characteristics during the recreational season were first described as to frequency, intensity and average duration. A precipitation duration curve, similar to the concept of flow duration curve and representative of the middle East coast, was constructed for the recreational season. A relationship between the amount of precipitation in a storm during the recreational season and the average duration of the storm in hours, and titled the Most Probable Duration of a Storm, was developed as representative of the middle East coast. From historical records of precipitation, the average number of storms expected during the five month period may be ascertained. The number of storms assumed for the purpose of this study was 30, 40, and 50. By the relationships which exist between the number of storms, the precipitation duration curve, and the storm duration curve, the average intensity, in inches per hour, and duration, in hours, of each storm may be found.

A modified form of the rational formula was utilized to describe the amount of runoff contributable to each storm. The equation was

$$Q = \text{Coef}[\text{Rain} - 0.03]\ \text{Area}$$

where:

Coef = the runoff coefficient of the basin of interest

Area = the area of the drainage basin in acres

Rain = the total amount of precipitation in the storm in inches

0.03 = the amount of rain required to initiate runoff

Q = the total amount of runoff contributable to the storm in cubic feet.

The amount of runoff per unit time may then be found by dividing Q by the duration of the storm. The rational formula provides a rectangular hydrograph which is defended by Sarginson (88), who states that the shape of the inflow hydrograph has very little affect upon the discharge of a combined sewer. The selected runoff coefficients, considered to be representative of urban areas, were 0.3, 0.4, 0.5, 0.6, and 0.7. The total amount of precipitation in inches for each storm was found from the precipitation duration relationship. For the purpose of this study, the number of storms expected during the recreational season was assumed to have a rectangular distribution with respect to the percent of storms with precipitation equal to or greater than any stated amount as expressed in the precipitation duration curve.

The BOD and suspended solids concentration in the DWF were both assumed to be 180 ppm, while the BOD and suspended solids concentration in the combined sewer overflow were assumed to be 180 and 800 ppm, respectively. All flows less than the interceptor capacity were assumed to be intercepted and given full treatment at the existing sewage treatment plant.

Utilizing the forementioned parameters, each storm was analyzed to find the total runoff, the total flow in the interceptor, the total amount of overflow, and the pounds of suspended solids and BOD overflowing. For each set of conditions, the percent of the suspended solids and BOD produced during the entire five months in the sanitary and stormwater runoff and overflowing to the receiving watercourse during the recreational season was found.

The effect of a holding basin for the area on the parameters of interest was then studied. The size of the holding basin was varied from 50 to 2,700 cubic feet per acre of drainage basin, in increments of 50 cubic feet. The performance of the holding basin was analyzed with respect to the percent reduction in overflow as compared to the existing combined sewer system. The holding basin was optimized with respect to the minimum cost of concrete in place for wall and the floor slab. A maximum depth of 21.5 feet was chosen, including 1.5 feet of freeboard.

The cost analysis was largely based on the 1962 edition of Means, Building Construction Cost Data, (72) and updated by considering the "Engineering News Record" cost index of 850 for 1962, and 1050 for spring of 1967. The thickness of the walls and slabs were assumed to be 0.75 feet. The 1962 cost per cubic yard of reinforced concrete in place for walls and slabs was \$55.00 and \$27.00, respectively. The 1962 cost of excavation per cubic yard was \$1.00 (72). An allowance factor of 1.2 was utilized for the concrete and excavation to account for additional work for miscellaneous items such as footings for unit and pump slabs. The cost of the pump and return lines was \$1500.00 (37). Engineering, land and legal fees were assumed to be 20 percent of the project costs (9,10,37). Profit and overhead were assumed to represent 18 percent of the total project cost (37,48). Annual operation and maintenance costs were estimated to be 5 percent of the original project cost (37). The annual cost per acre of each holding basin of a given size of interest was found based on a 30 year design period and an interest rate of 5.0 percent.

The efficiency and cost of the gravity sediment separator was studied to determine as many of its characteristics as possible. The separator was designed in a manner physically akin to a primary sedimentation basin with a length to width ratio of 6.0 to insure a system approximating plug flow. The depth of the separator was assumed equal to the width, subject to a maximum depth of 20 feet, not including 1.5 feet of freeboard. The volume per acre of the separator was varied from 13 to 380 cubic feet. The removal of suspended solids by the gravity sediment separator was based on conservative equations for detention time of a primary sedimentation basin (24,43). The highest percent removal attainable under this constraint was 72 percent. The additional removal by the controlled turbulence, shear gradient, and the concentrated underflow carried as a bed load could not be taken into account as these effects have never been defined as a result of studies under field conditions. The removal of BOD was determined from known equations based on the overflow rate of a primary sedimentation basin (24,43).

Chlorination at 30 ppm was assumed adequate for bactercidal purposes, based on the work cited in the literature review. The total percent removal of suspended solids and BOD of the potential overflow for the five month recreational season was found for each gravity sediment separator size of interest.

A cost analysis of the gravity sediment was made. The concrete and excavation costs were determined in the same manner as that for the holding basin. The total chlorinator cost, including chlorinator, platform scale, chlorine solution lines, chlorine fittings, electric control, electric primary meter, chlorinator fittings and valves, chlorine pump, and installation came to $4,587.00 (94). A duplex pump system with electric controls designed to pump 10 percent of the DWF, was ascertained to be equal to:

$$\text{Cost} = 40X + 500.00 \quad (37)$$

where X designates the quantity pumped in gallons per minute. A 15 percent installation cost was added (37). The cost of the sludge return lines in place was estimated to be 25 percent of the cost of the duplex pump system (37). The cost of the weir was estimated at $8.50 per linear foot in place (37). The cost of an electric flow monitor and the associated electric controls in place was $1,000.00 (94). The cost of a building to house the chlorinator and pumps was $1,000.00 (48). Electrically controlled inlet and outlet gates were $1,000.00 (37).

The fees for land, legal, engineering, profit and overhead were the same as those cited for the holding basin. A chlorine cost of 13 cents per pound was added to the 5 percent annual overhead and maintenance. The annual cost per acre was based on a 30 year design and an effective interest rate of 5 percent.

A computer program in Fortran IV was written to properly analyze the efficiencies, operation, economics, and characteristics of the systems, and is presented in Appendix A. A flow model of the combined sewer analysis is presented in Figure 4 and the flow model for the proposed treatment methods is presented in Figure 5. A non-linear regression program developed by Professor Robert C. Heterick of the Civil Engineering Department of V. P. I. was utilized in presenting the data. The efficiency of the holding basin and the gravity sediment separator in reducing the objectionable combined sewer overflow for different values of the significant parameters were then compared on an annual cost per acre basis.

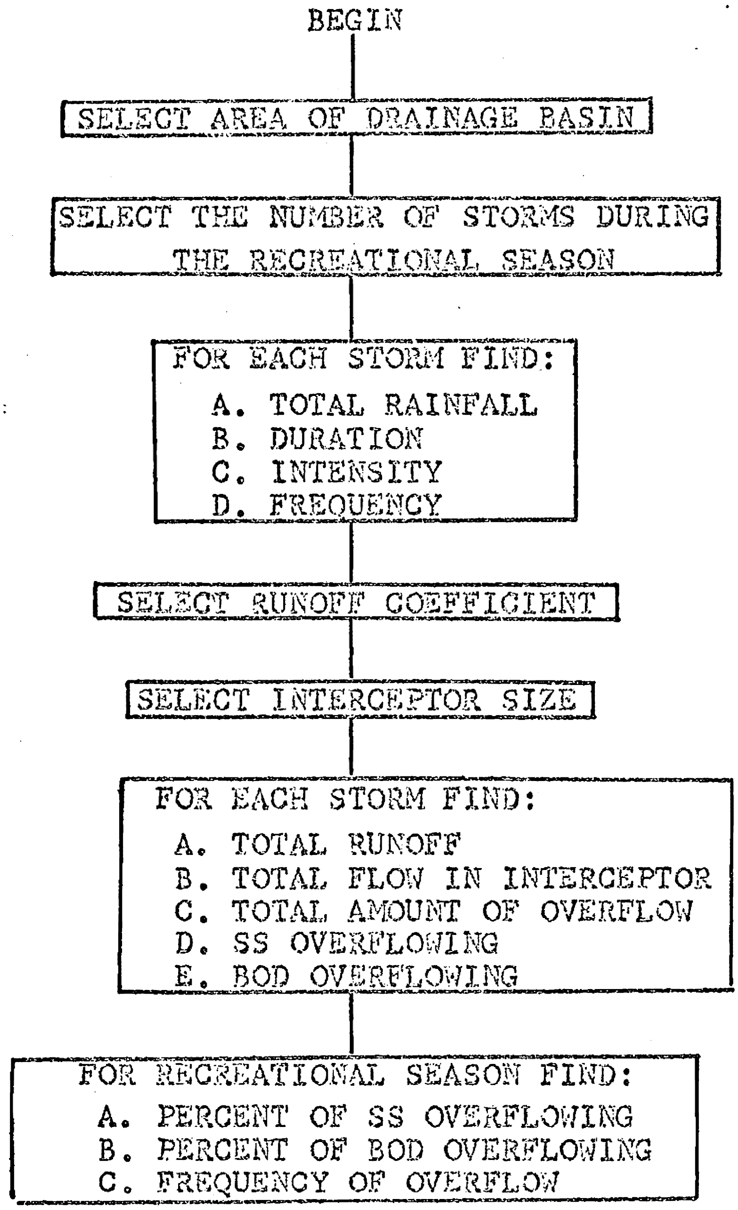

FIGURE 4. Computer Flow Diagram For Combined Sewer Overflow Analysis.

HOLDING BASIN

SELECT LIMIT OF DEPTH

OPTIMUM BASIN GEOMETRY

FOR SELECTED CAPACITY PER ACRE FIND:
A. PERCENT REDUCTION OF OVERFLOW
B. FREQUENCY OF OVERFLOWS
C. NUMBER OF OVERFLOWS
D. TOTAL COST OF HOLDING BASIN BASED ON ENR COST INDEX OF 1050
E. ANNUAL COST PER ACRE INCLUDING MAINTENANCE

GRAVITY SEDIMENT SEPARATOR

SELECT RANGE OF WIDTHS

DEPTH = WIDTH

MAXIMUM DEPTH = 20.0 FT. NOT INCLUDING FREEBOARD

FIND CAPACITY PER ACRE

FOR EACH STORM FIND:
A. DETENTION PERIOD
B. PERCENT REMOVAL OF SS
C. OVERFLOW RATE
D. PERCENT REMOVAL OF BOD

FOR RECREATIONAL SEASON FIND:
A. TOTAL PERCENT REMOVAL OF SS
B. TOTAL PERCENT REMOVAL OF BOD
C. INTERVAL BETWEEN OVERFLOWS
D. TOTAL COST OF GRAVITY SEDIMENT SEPARATOR BASED ON ENR COST INDEX OF 1050
E. ANNUAL COST PER ACRE INCLUDING MAINTENANCE AND CHLORINE

FIGURE 5. Computer Flow Diagram For Proposed Treatment Methods.

RESULTS

The presentation of the results of this study is divided into five sections. These subdivisions are concerned with the extent and characteristics of the synthesized rainfall, the characteristics of the combined sewer overflow, the effects of the proposed treatment devices, including the holding basin and the gravity sediment separator, and the economics of the proposed treatment methods.

Storm Characteristics of Synthetic Rainfall

A rainfall duration curve constructed for the five month recreational season and representative of the East coat was initially developed. From Figure 6, which indicates the characteristic amounts of rainfall during the five month recreational season, the expected amount of rainfall in each storm may be found. For the purpose of this study, 30, 40, and 50 storms were assumed to occur during the period in question. This represents an average interval of less than four days between storms. Figure 7, entitled The Most Probable Duration of a Storm for a Given Amount of Rainfall, was constructed to give the relationship between the total amount of rainfall in a storm and the most probable storm duration. From these two figures, the total amount of rainfall, the duration, and the average intensity of each hypothetical storm may be ascertained.

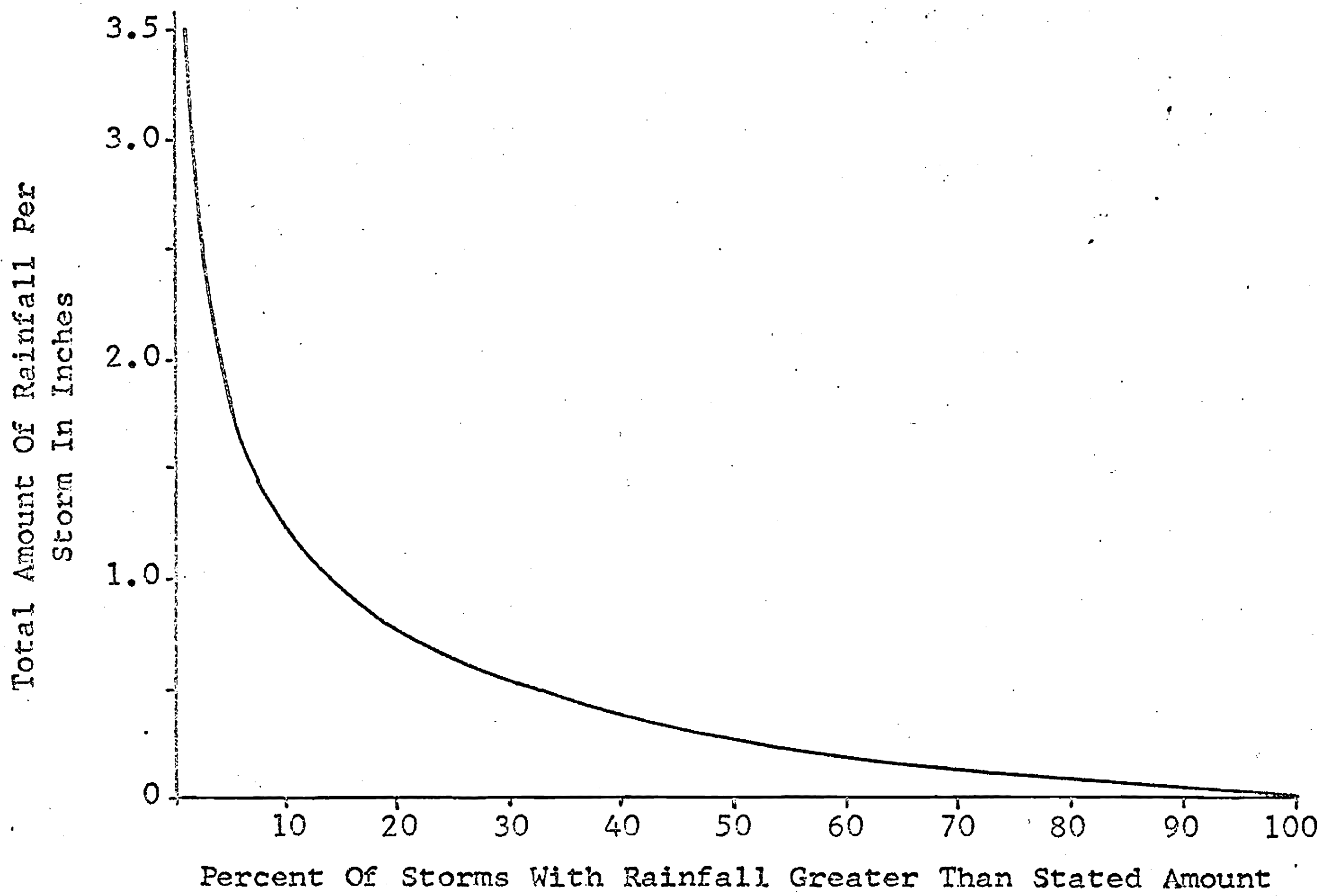

FIGURE 6. Characteristic Amounts Of Rainfall During The Recreational Season.

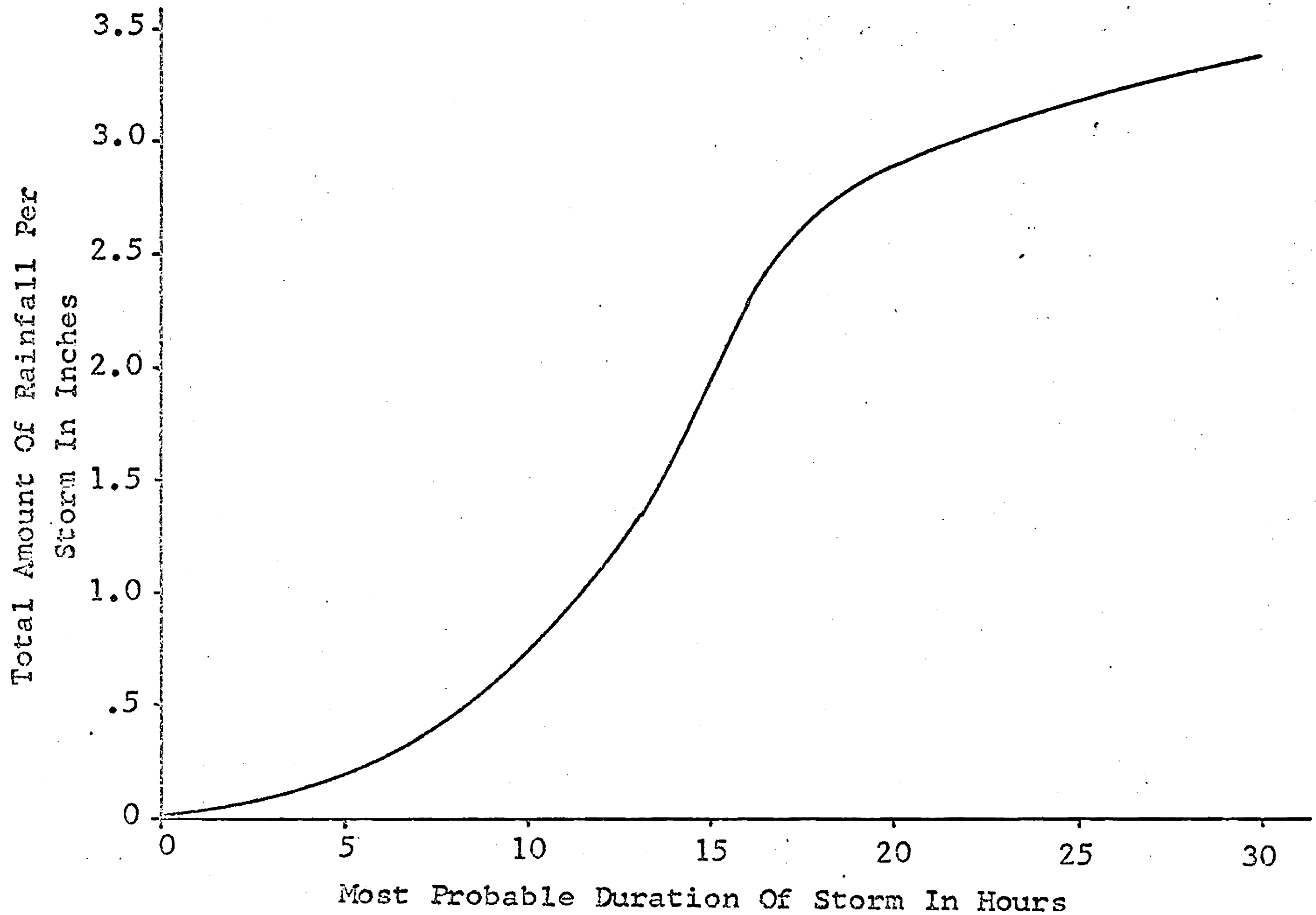

FIGURE 7. The Most Probable Duration Of A Storm For A Given Amount Of Rainfall.

Given in Appendix B are the individual characteristics of each synthetically derived storm occuring during the recreational season.

<u>Characteristics and Extent of Untreated Overflow</u>

The percent of the total five month production of BOD escaping untreated from a combined sewer due to overflow during the recreational season as a function of the coefficient of runoff and the interceptor capacity is presented in Figures 8, 9, and 10 for 30, 40, and 50 storms, respectively. From Figure 9 for 40 storms, for a runoff coefficient of 0.6 and an interceptor capacity of 3 X DWF, it can be seen that 13 percent of the total five month production of BOD escapes untreated to the receiving watercourse. The portion of BOD escaping to the receiving watercourse increases with an increase in the number of storms during the recreational season, with an increase in runoff coefficient, or with a decrease in the interceptor size.

The percent of the total five month production of suspended solids escaping from a combined sewer due to overflow during the recreational season is presented in Figures 11, 12, and 13 as a function of the number of storms, the interceptor capacity, and the runoff coefficient, for 30, 40, and 50 storms, respectively. From Figure 12 for 40 storms, for a runoff coefficient of 0.6 and an interceptor capacity

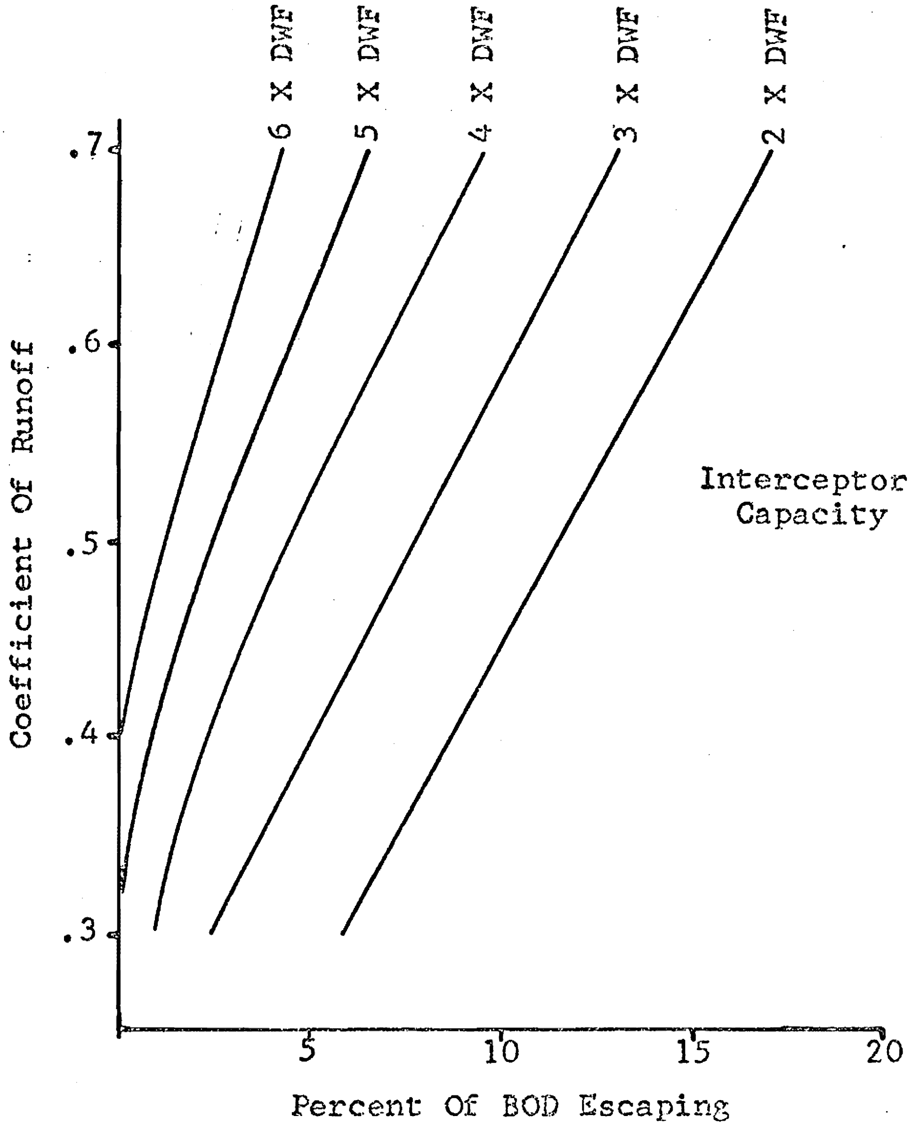

FIGURE 8. Percent Of Total BOD Escaping During The Recreational Season In The Combined Sewer Overflow As A Result Of 30 Storm.

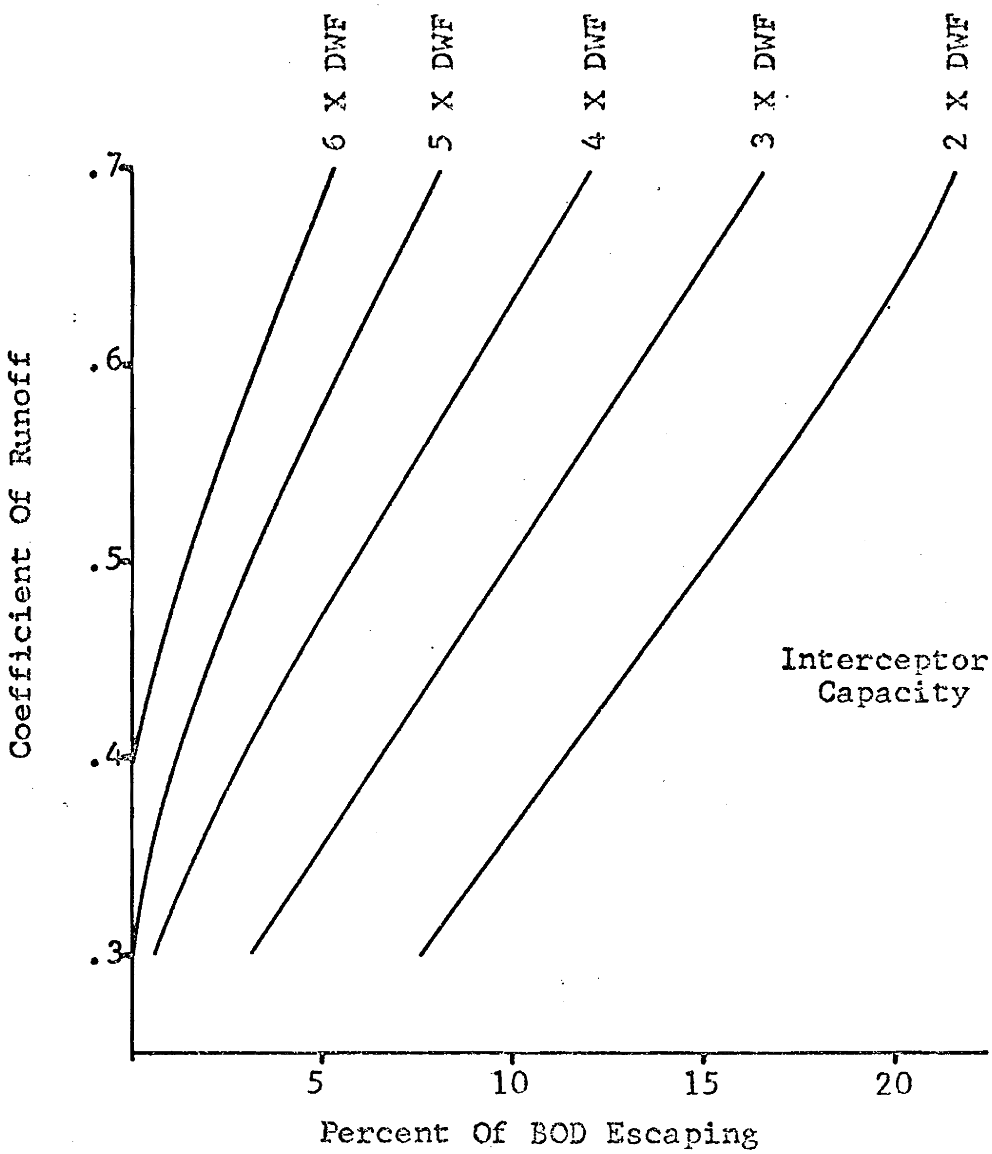

FIGURE 9. Percent Of Total BOD Escaping During The Recreational Season In The Combined Sewer Overflow As A Result Of 40 Storms.

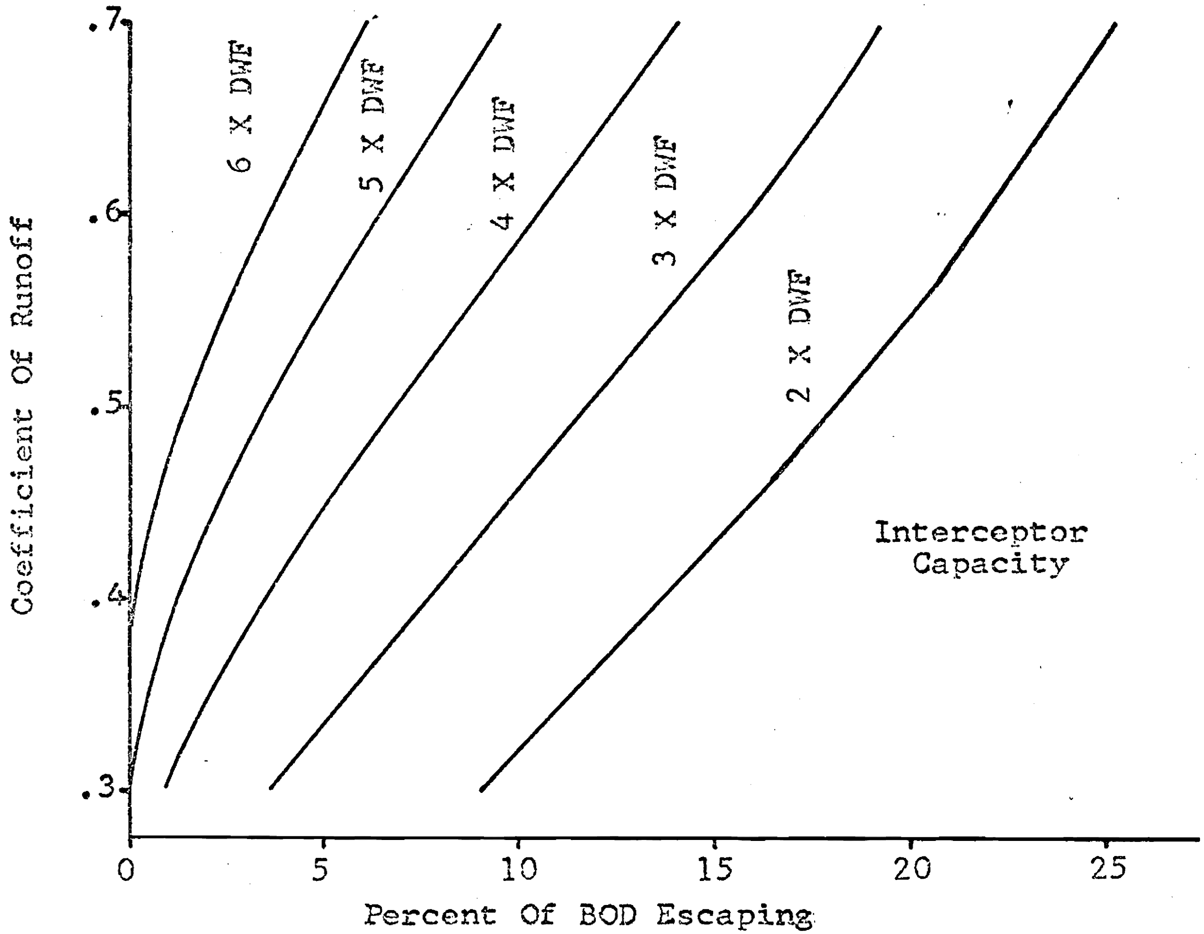

FIGURE 10. Percent Of Total BOD Escaping During The Recreational Season In The Combined Sewer Overflow As A Result of 50 Storms.

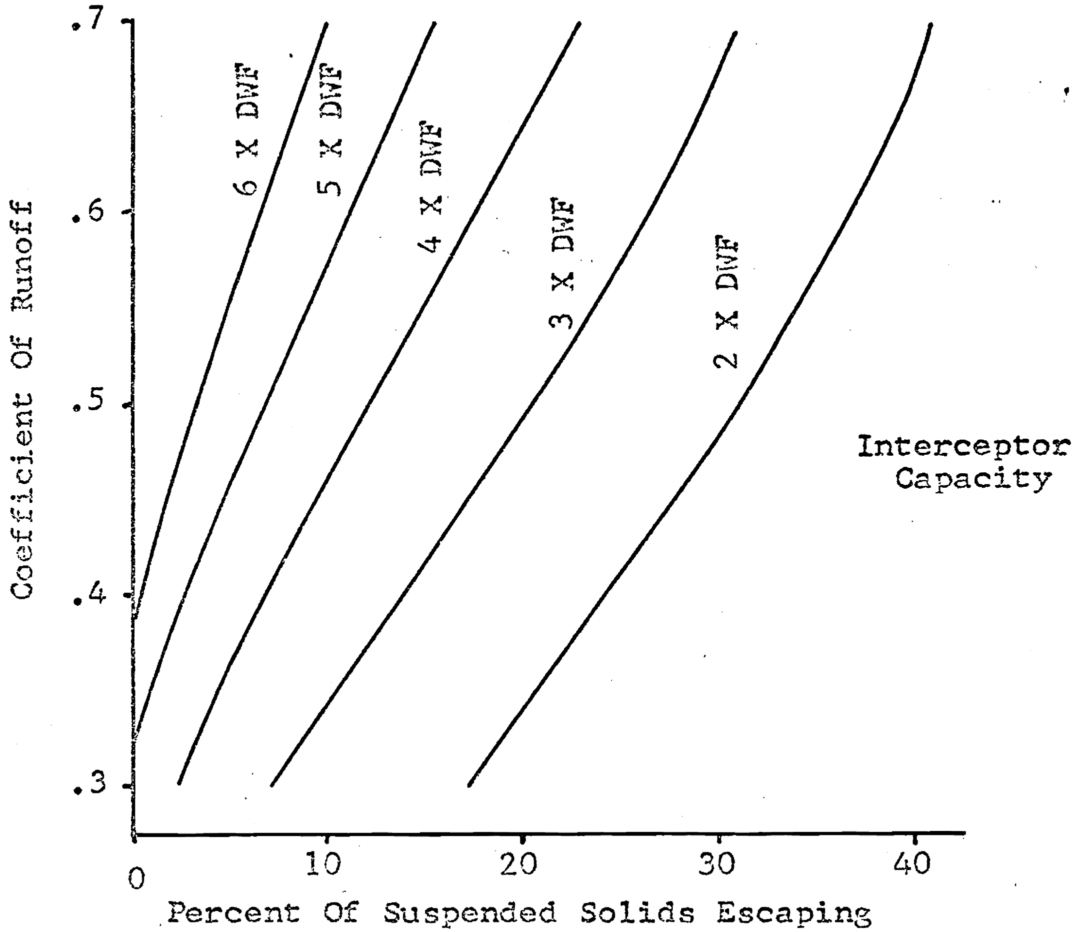

FIGURE 11. Percent Of Total Suspended Solids Escaping From A Combined Sewer During The Recreational Season As A Result Of 30 Storms.

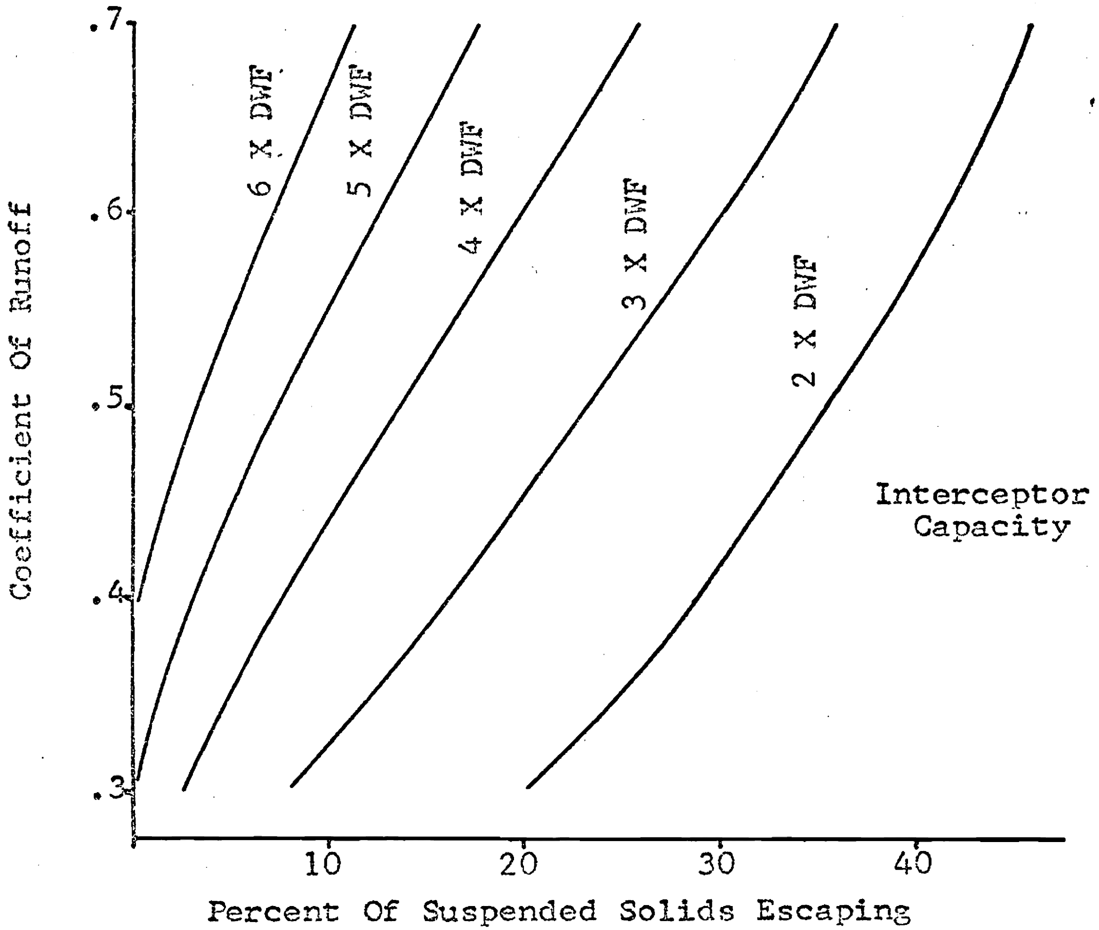

FIGURE 12. Percent Of Total Suspended Solids Escaping From A Combined Sewer During The Recreational Season As A Result Of 40 Storms.

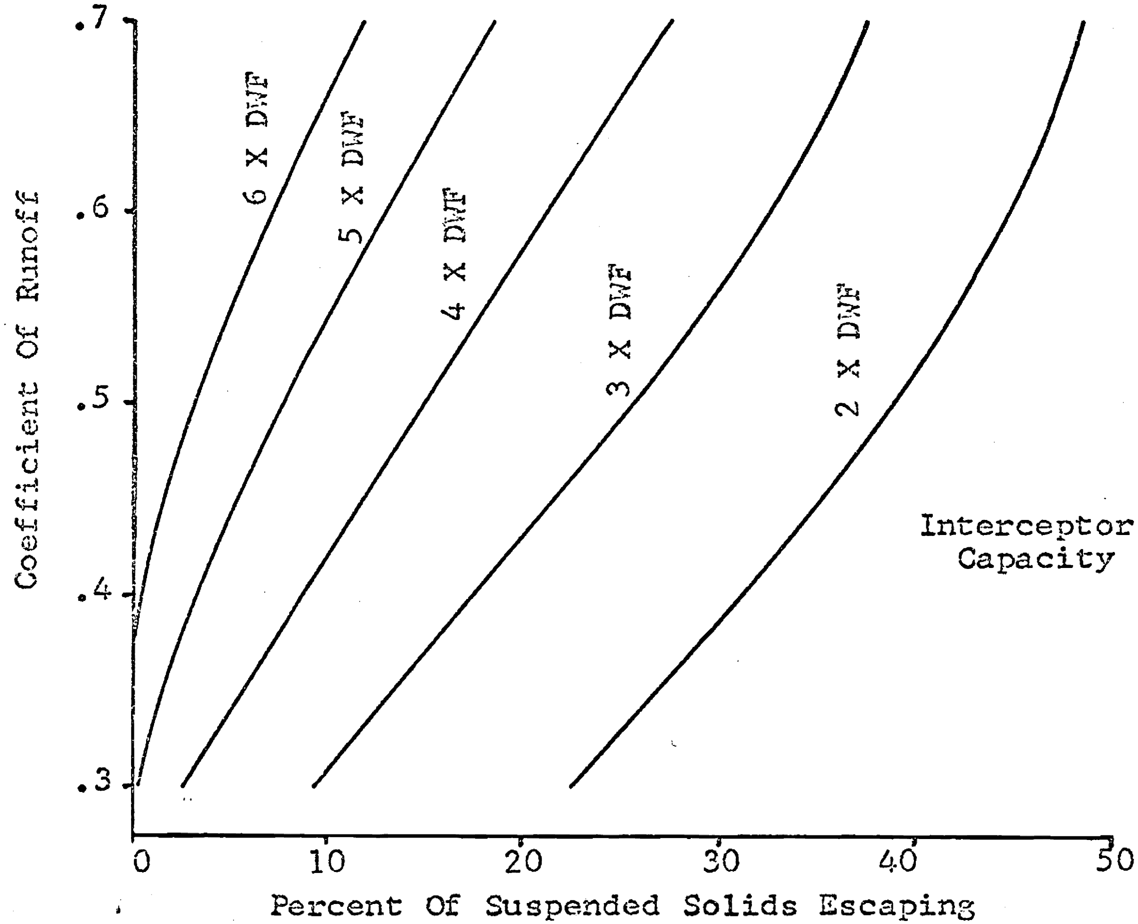

FIGURE 13. Percent Of Total Suspended Solids Escaping From A Combined Sewer During The Recreational Season As A Result Of 50 Storms.

of 3 X DWF, it may be seen that 30 percent of the total five month production of suspended solids escapes untreated to the receiving watercourse. The fraction of the five month production of suspended solids overflowing also increases with an increase in the number of storms during the recreational season, with an increase in the runoff coefficient, or with a decrease in the interceptor capacity. A non-linear regression model was fitted to this date to give the percent of suspended solids escaping during the recreational season as a function of the runoff coefficient, the interceptor capacity, and the number of storms. The empirical equation developed in this manner is:

$$PCSS = -60.51 + 108.59 \times CINT^{-0.36} \times N^{0.10} \times COEF^{0.35}$$

$$R = .9981$$

where:

PCSS = the percent of the total five month production of suspended solids escaping due to overflow of the combined sewer

CINT = the interceptor capacity expressed in multiples of the average DWF

N = the number of storms expected during the recreational season.

This model has a correlation coefficient (R) of 0.9981. The correlation coefficient is a measure of the "goodness of fit" of the model and is applicable within the examined ranges of the variables.

Effect of Storage on the Extent and Characteristics of Combined Sewer Overflow

The capacity of a holding basin in cubic feet per acres required to obtain a stated reduction in the overflow of a combined sewer is a function of the interceptor capacity, runoff coefficient, and the number of storms expected during the recreational season. Representative values of the storage capacity required are given in Figures 14 through 20. Figures 14, 15, and 16 show the influence of the runoff coefficient on the storage capacity, while Figures 17, 18, and 19 show the influence of the interceptor capacity. From Figure 15, for 40 storms and a fixed interceptor capacity of 4 X DWF, the storage capacity of a holding basin in cubic feet per acre required to effect a specified reduction in combined sewer overflow may be found for varying runoff coefficients. For example 1400 cubic feet per acre of storage will reduce the combined sewer overflow by 80 percent for an area with a runoff coefficient of 0.5. The storage capacity required to obtain a stated reduction in overflow increases with an increase in the coefficient of runoff, or with a decrease in the interceptor capacity.

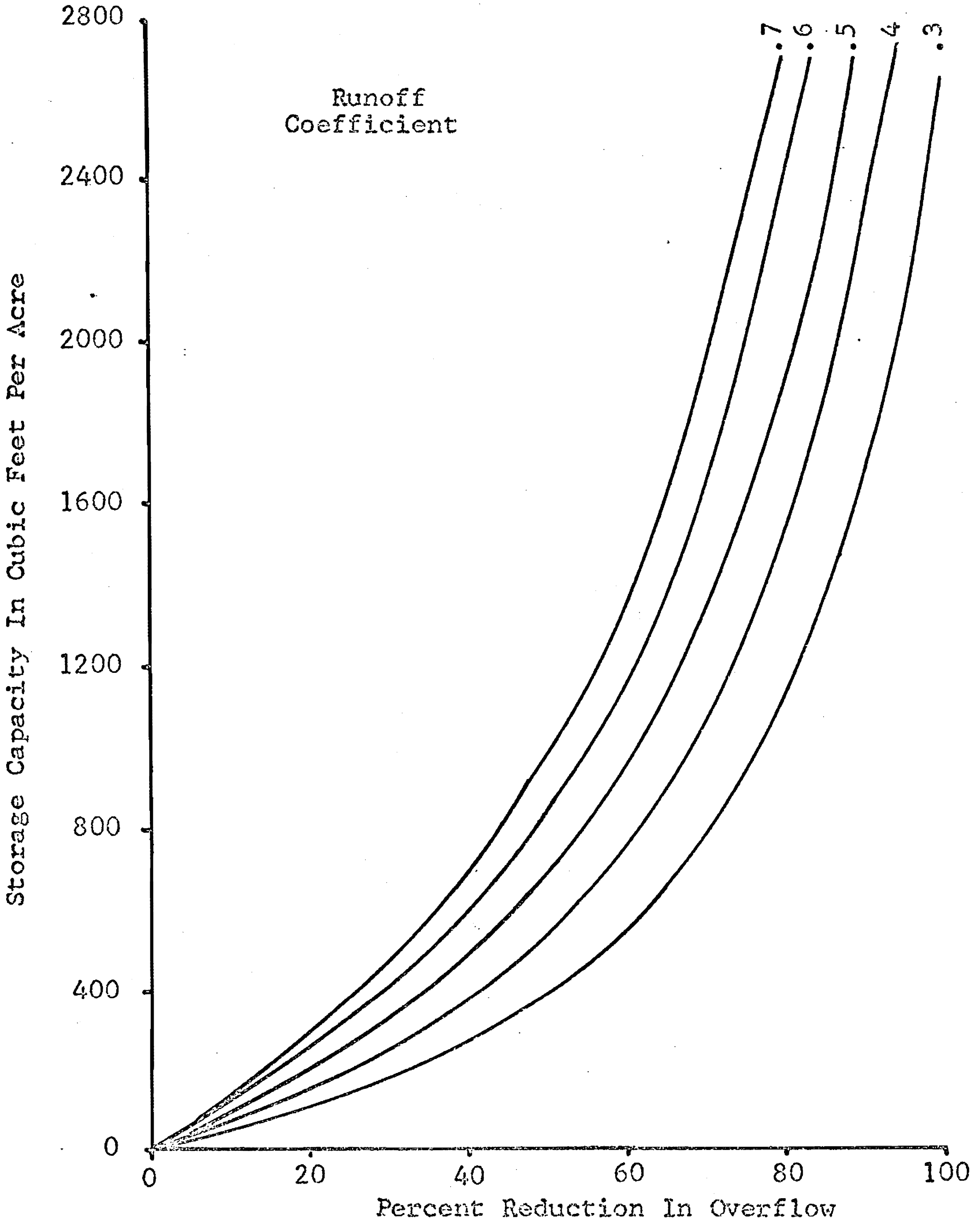

FIGURE 14. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With An Interceptor Capacity Of 2 Times DWF And 40 Storms During The Recreational Season.

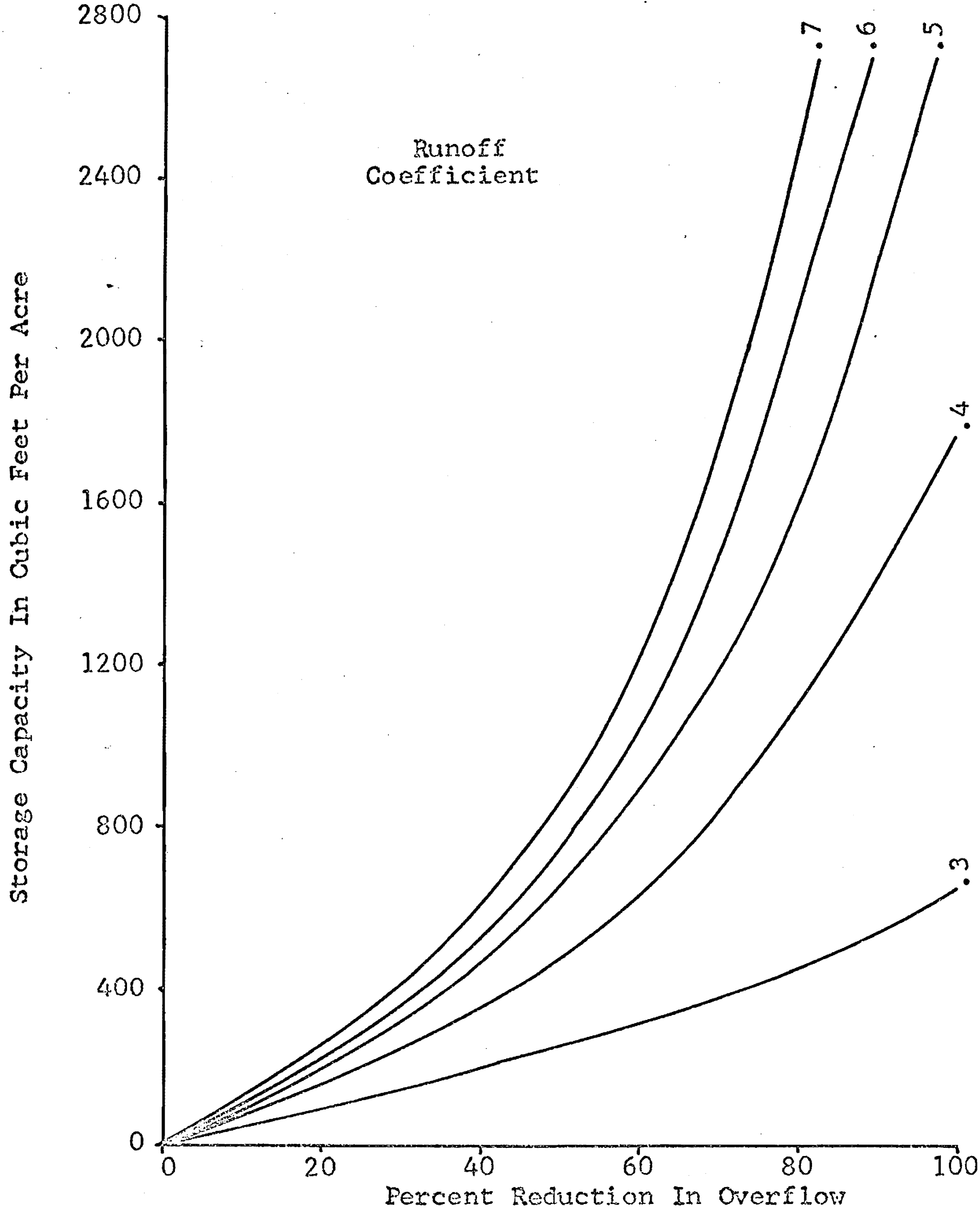

FIGURE 15. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With An Interceptor Capacity Of 4 Times DWF And 40 Storms During The Recreational Season.

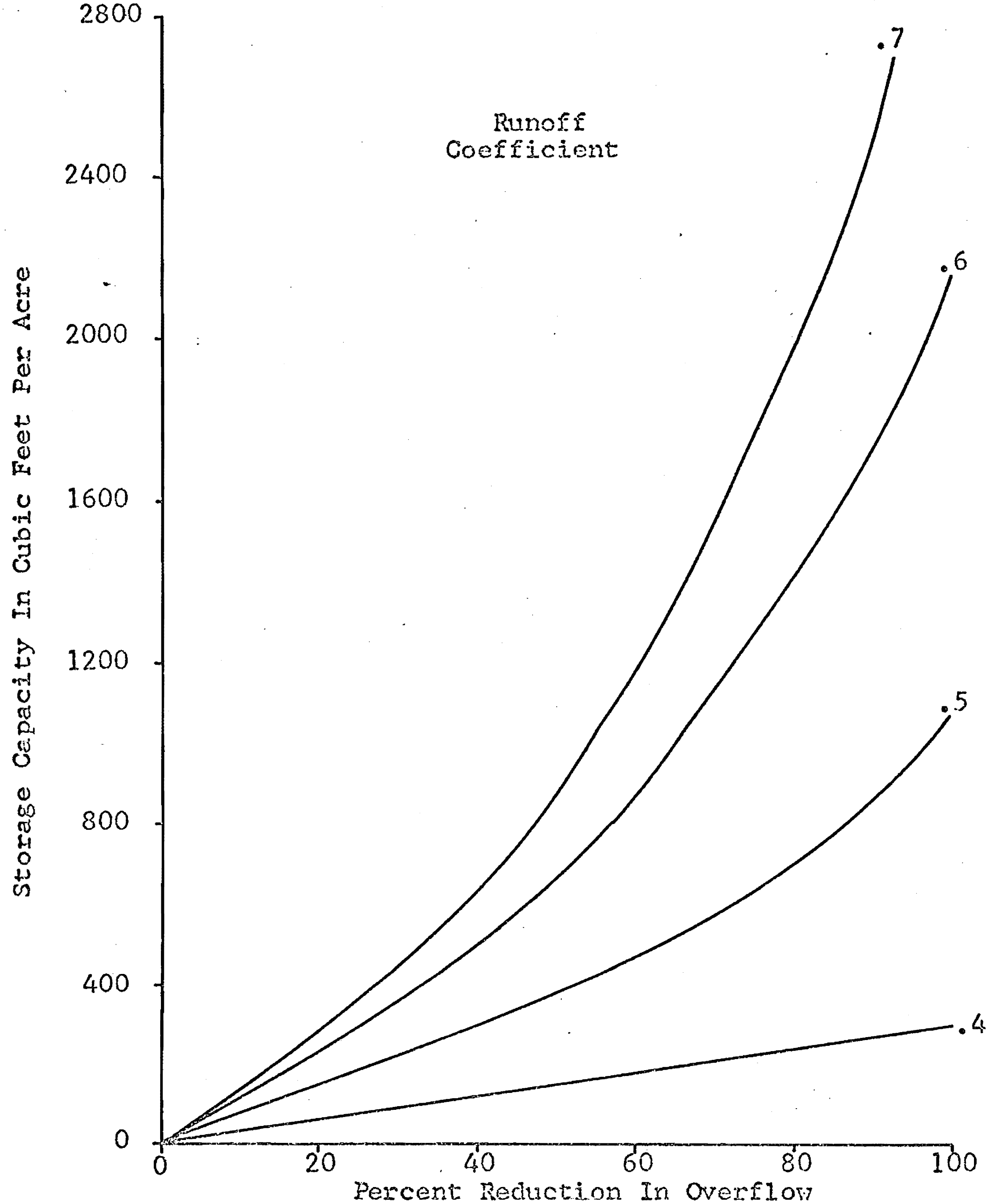

FIGURE 16. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With An Interceptor Capacity Of 6 Times DWF And 40 Storms During The Recreational Season.

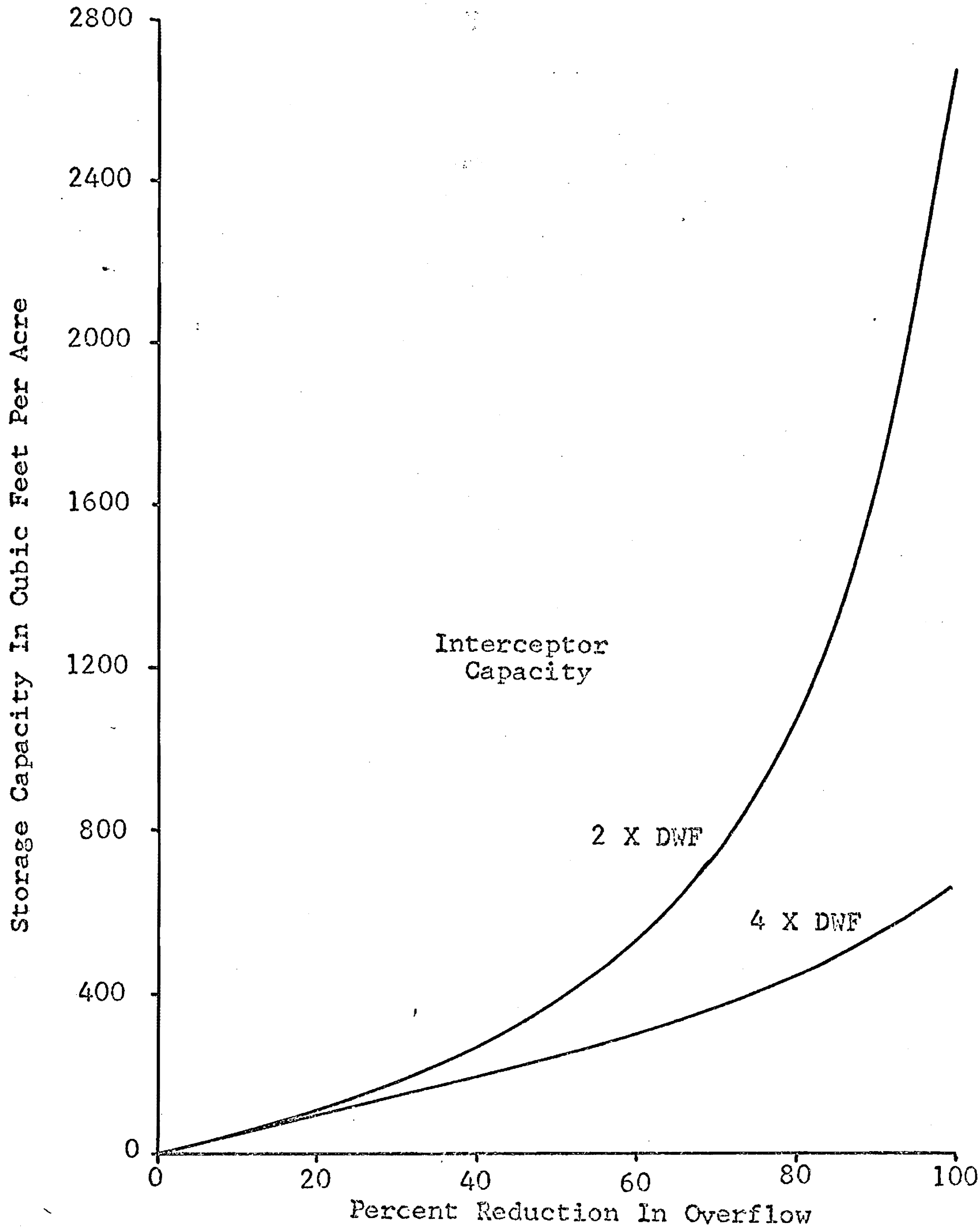

FIGURE 17. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With A Runoff Coefficient of 0.3 And 40 Storms During The Recreational Season.

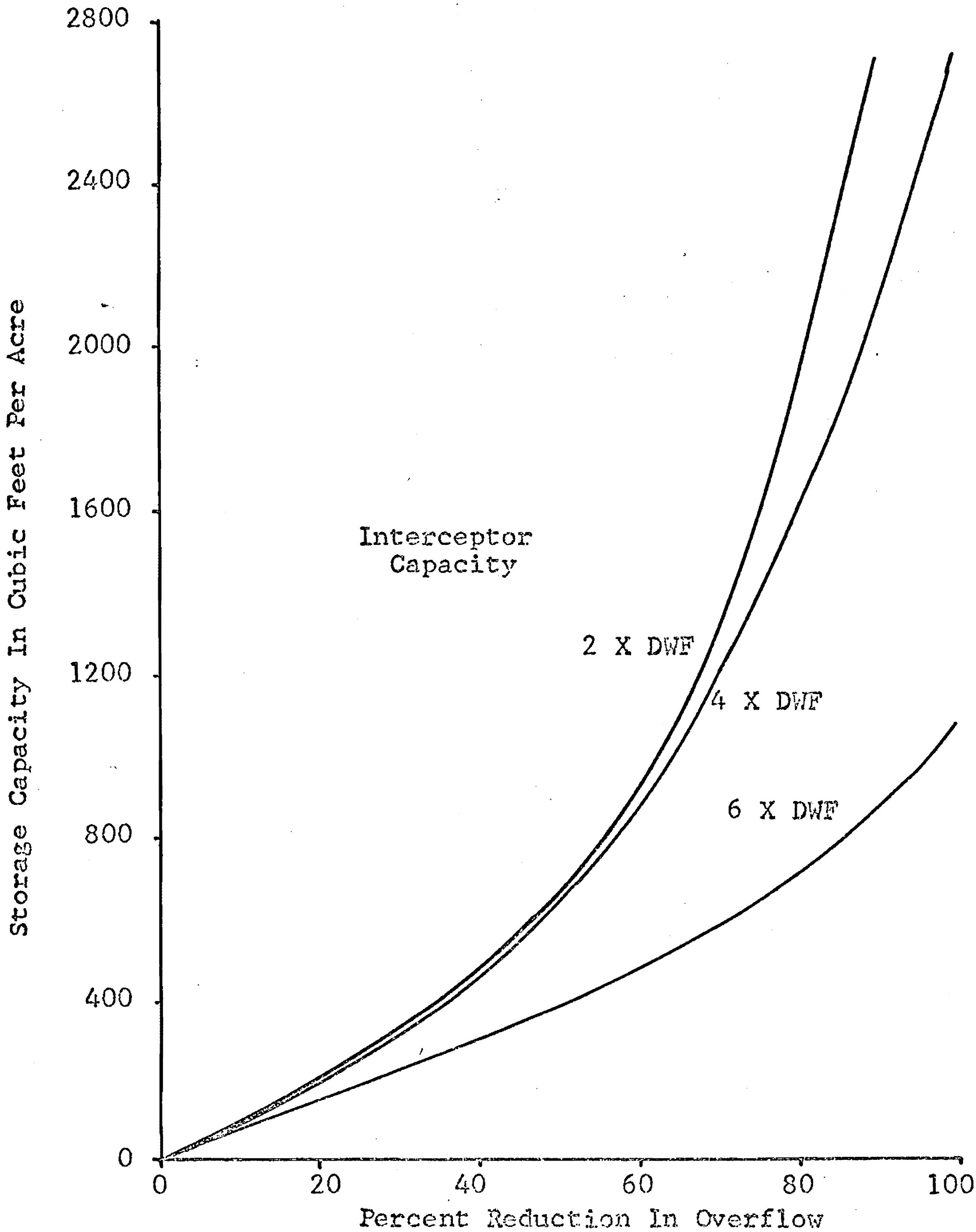

FIGURE 18. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With A Runoff Coefficient of 0.5 And 40 Storms During The Recreational Season.

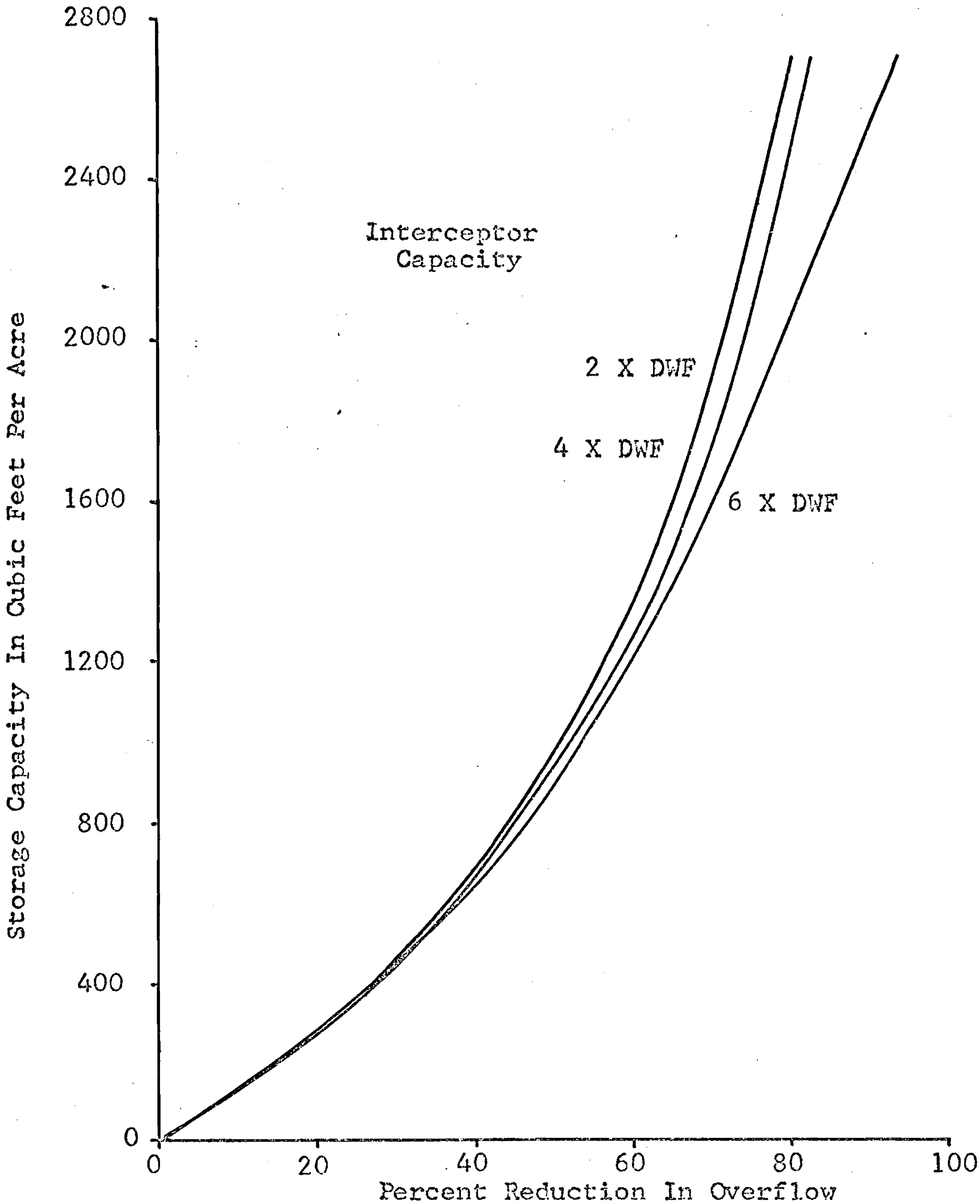

FIGURE 19. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With A Runoff Coefficient Of 0.7 And 40 Storms During The Recreational Season.

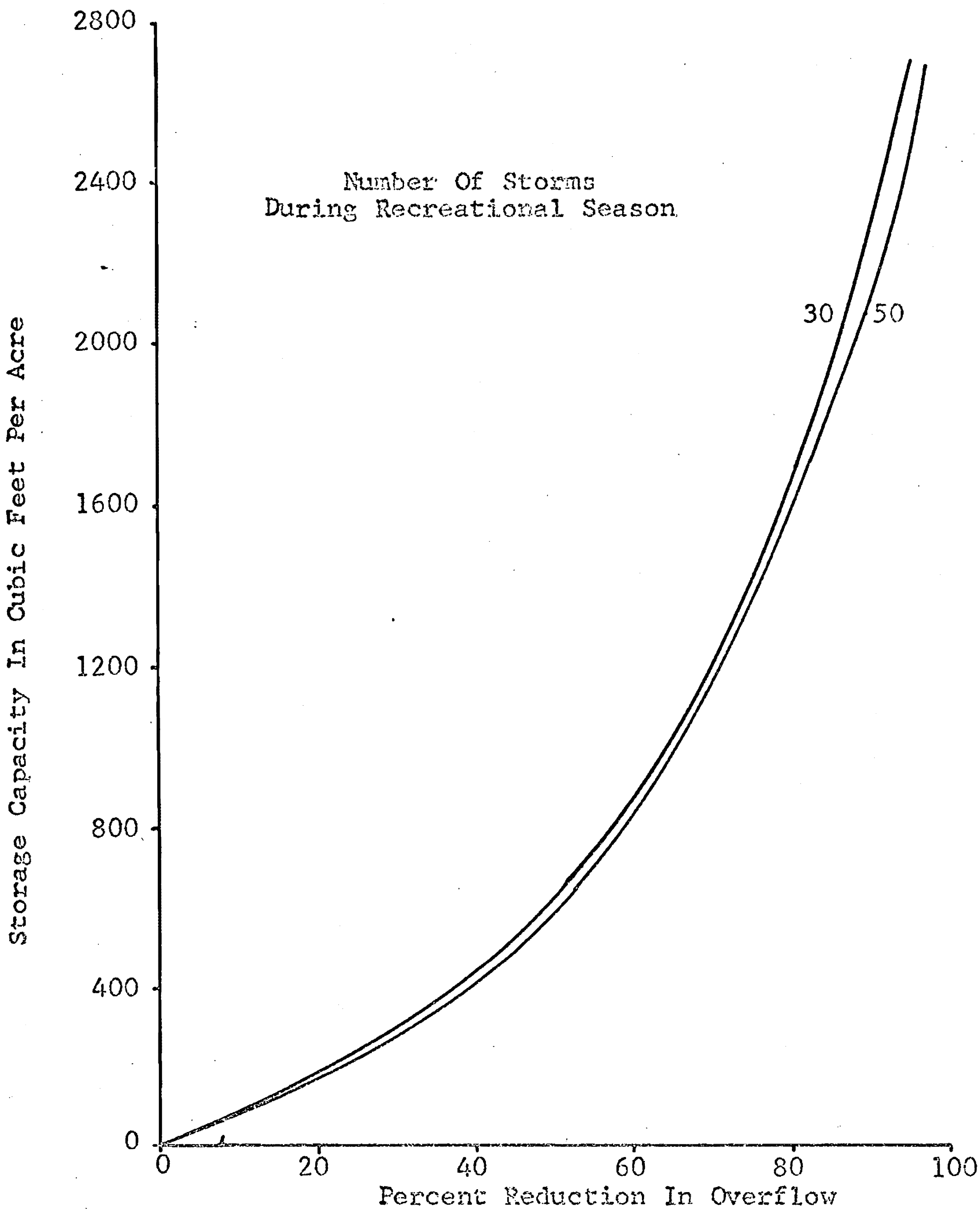

FIGURE 20. The Capacity Per Acre Of A Holding Basin Required To Obtain Stated Reduction In Quantity Of Overflow From A Combined Sewer With An Interceptor Capacity of 4 Times DWF and A Runoff Coefficient Of 0.5.

From Figure 20, it can be seen that the storage capacity of a holding basin required to obtain a stated reduction in overflow is not significantly affected by the number of storms expected during the recreational season. The non-linear prediction equation for the required holding basin to achieve a given reduction in the quantity of overflow was found to be:

$$Y = 5.99 + CPA^{0.46} \times COEF^{-0.57} \times N^{0.05} \times CINT^{0.21}$$

$$R = .9969$$

where:

Y = the percent reduction in the quantity of overflow being discharged to the receiving watercourse without treatment

CPA = the capacity per acre of the holding basin in cubic feet

COEF = the runoff coefficient

CINT = the interceptor capacity expressed in multiples of the average DWF

N = the number of storms during the recreational season.

This model has a correlation coefficient of 0.9969 within the investigated ranges.

Effect of Gravity Sediment Separator on the Extent and Characteristics of Combined Sewer Overflow

The capacity per acre of a sediment separator required to achieve a specified removal of suspended solids from the overflow of a combined sewer is illustrated in Figures 21 through 25. Figures 21, 22, and 23 illustrate the influence of the runoff coefficient on the required capacity per acre. Figure 24 illustrates the influence of the interceptor capacity on the size of the sediment separator required, while Figure 25 illustrates the influence of the number of storms. From Figure 22, for 40 storms, and a fixed interceptor capacity of 4 X DWF, it can be seen that the capacity per acre of a gravity sediment separator required to obtain a 60 percent reduction of suspended solids overflowing from a drainage basin with a runoff coefficient of 0.5 is 50 cubic feet. The capacity per acre of the gravity sediment separator required to obtain a stated reduction in the overflow of suspended solids from a combined sewer increases with an increase in the runoff coefficient or with a decrease in the interceptor size. From Figure 25, it can be seen that the number of storms during the recreational season does not have a significant effect on the capacity of the separator required to obtain a stated reduction on the overflow of suspended solids. The non-linear regression equation for the percent

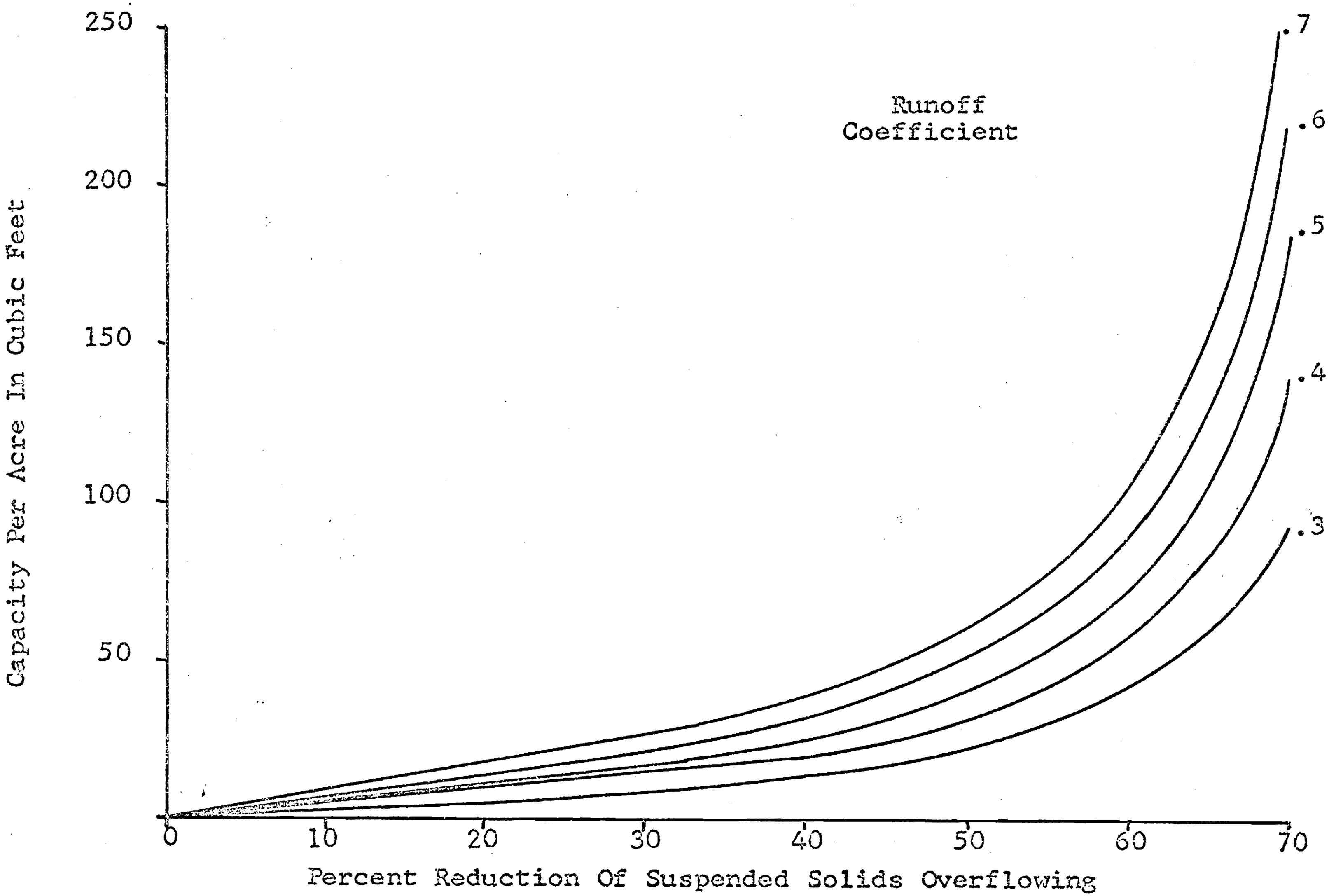

FIGURE 21. The Capacity Per Acre Of A Gravity Sediment Separator Required To Obtain Stated Reduction In Quantity Of Suspended Solids Overflowing From A Combined Sewer With An Interceptor Capacity Of 2 Times DWF And 40 Storms During The Recreational Season.

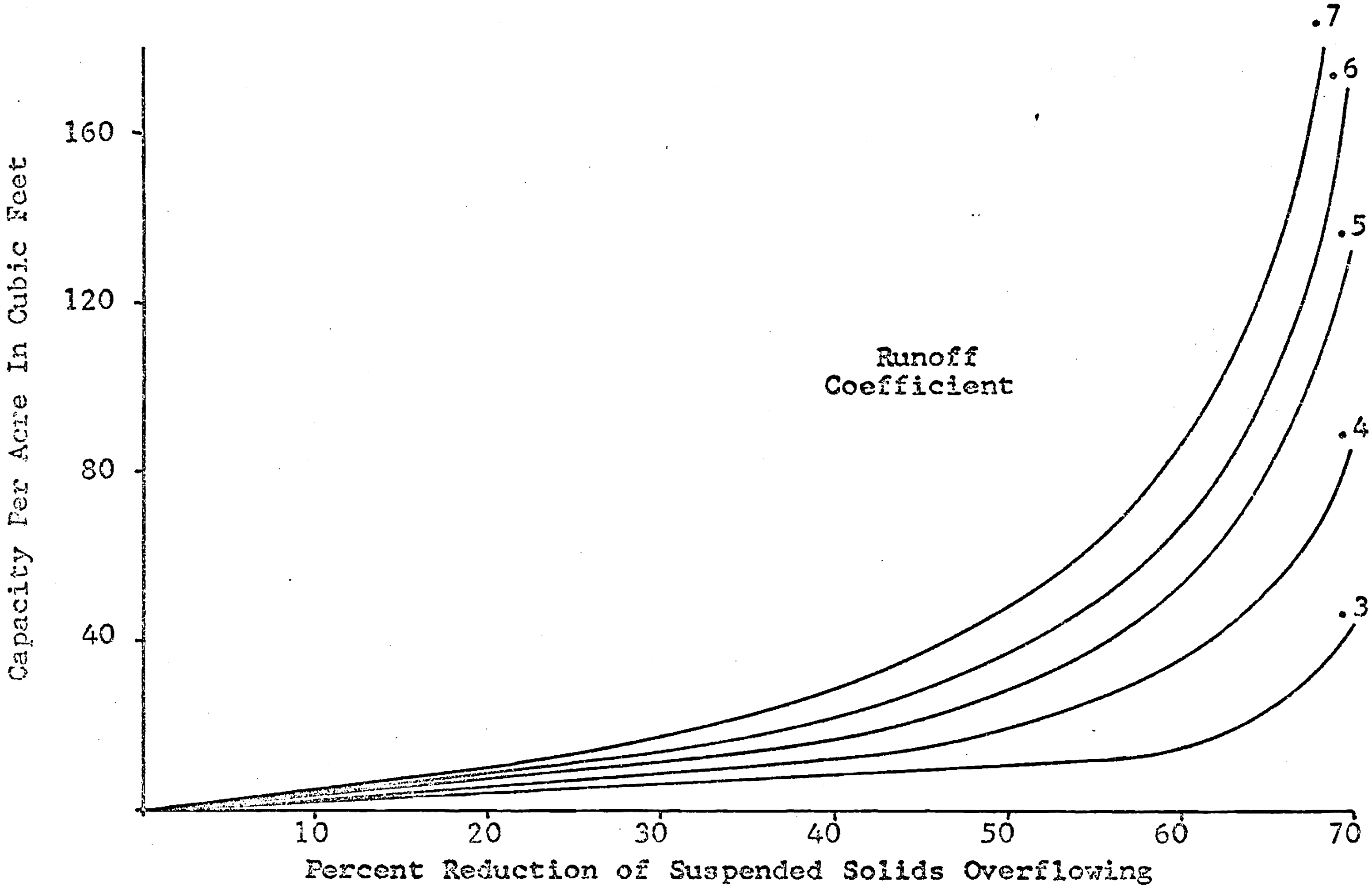

FIGURE 22. The Capacity Per Acre of a Gravity Sediment Separator Required to Obtain Stated Reduction in Quantity of Suspended Solids Overflowing From A Combined Sewer With an Interceptor Capacity of 4 Times DWF and 40 Storms During The Recreational Season.

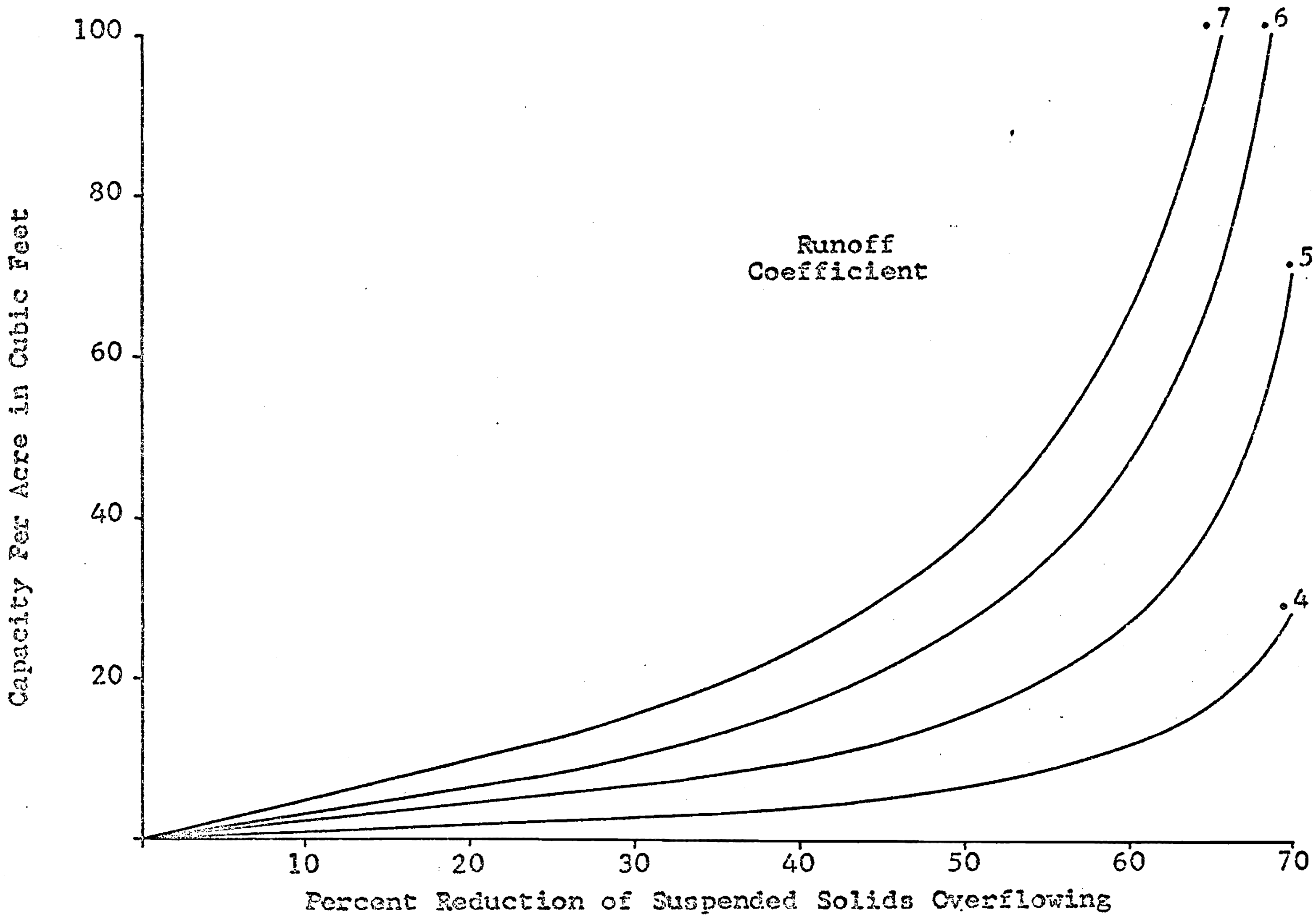

FIGURE 23. The Capacity Per Acre of a Gravity Sediment Separator Required to Obtain Stated Reduction in Quantity of Suspended Solids Overflowing From a Combined Sewer With An Interceptor Capacity of 6 Times DWF and 40 Storms During the Recreational Season.

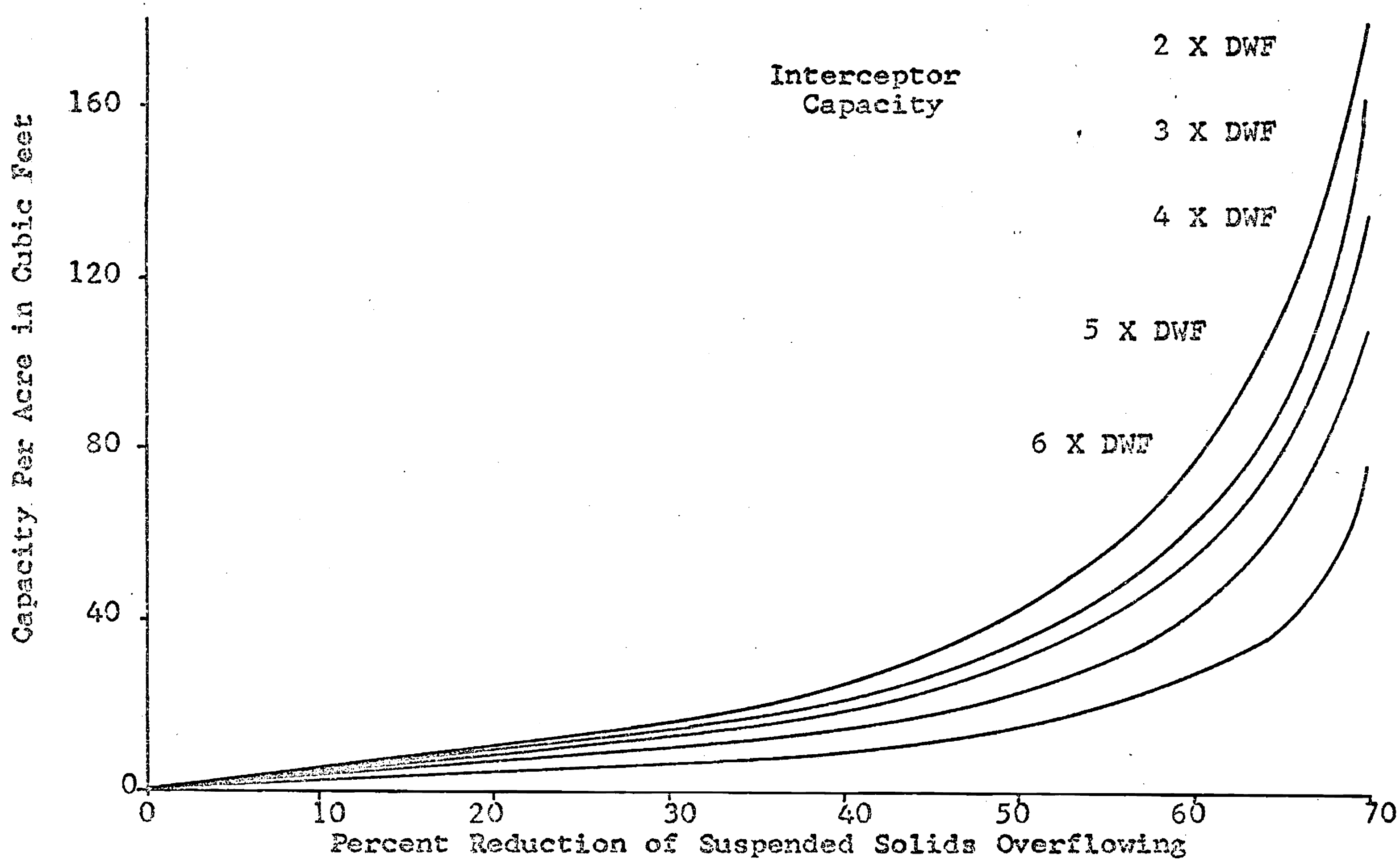

FIGURE 24. The Capacity Per Acre of a Gravity Sediment Separator Required to Obtain Stated Reduction in Quantity of Suspended Solids Overflowing From a Combined Sewer With a Runoff Coefficient of 0.5 and 40 Storms During the Recreational Season.

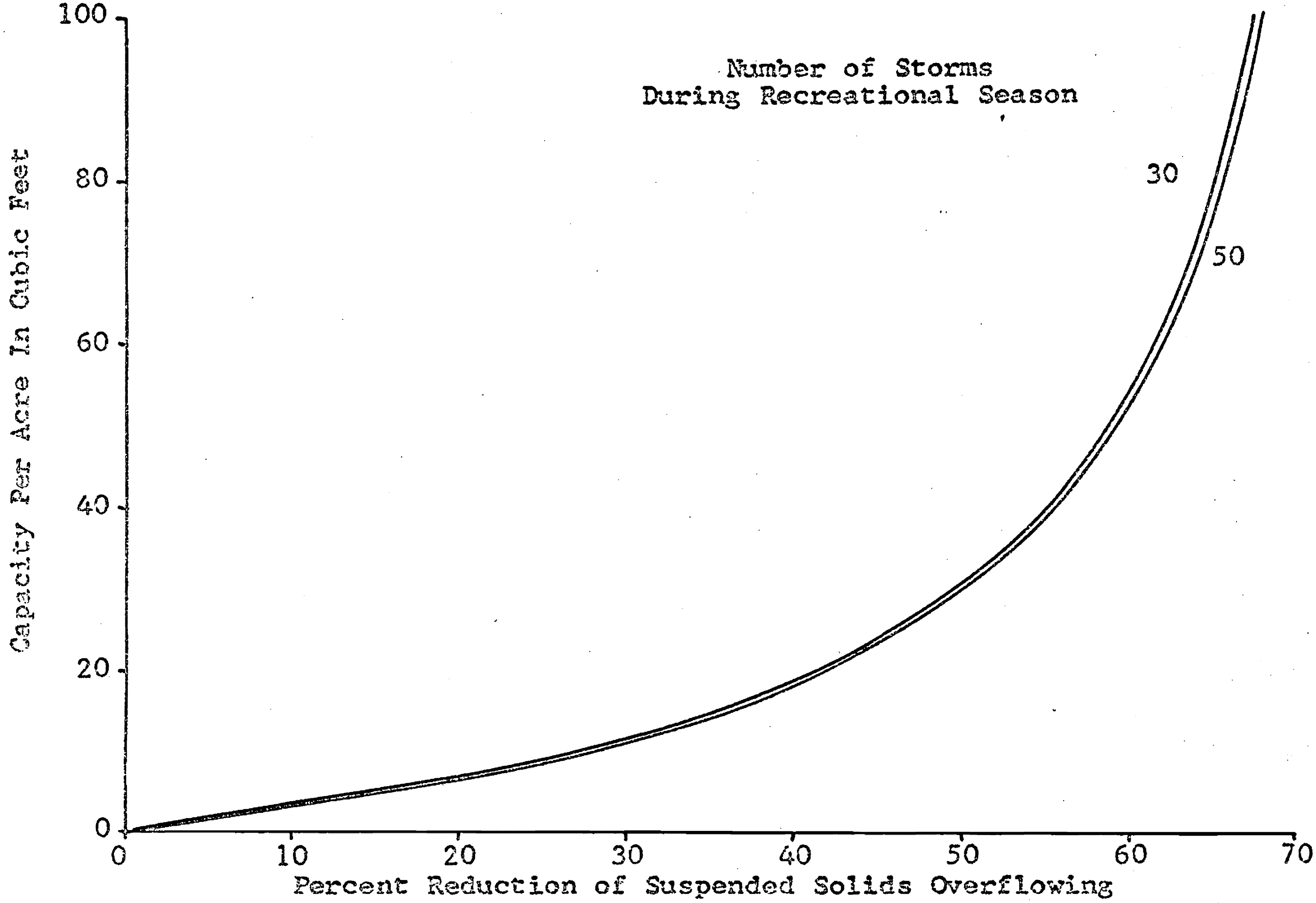

FIGURE 25. The Capacity Per Acre of a Gravity Sediment Separator Required to Obtain Stated Reduction in Quantity of Suspended Solids Overflowing From a Combined Sewer With a Runoff Coefficient of 0.5 and an Interceptor Capacity of 4 Times DWF.

reduction of suspended solids achieved by the sediment separator is

$$Y = -83.53 + 71.62 \times CPA^{0.11} \times COEF^{-0.18} \times CINT^{0.07} \times N^{0.01}$$

$$R = .9979$$

where:

Y = the percent reduction in the overflow of suspended solids achieved by the sediment separator. This equation is based on data spanning a removal range of between 30 and 70 percent.

CPA = the capacity per acre of the sediment separator in cubic feet.

COEF = the runoff coefficient.

CINT = the interceptor capacity in multiples of the average DWF

N = the number of storms expected during the recreational season.

This model has a correlation coefficinet of 0.9979.

Economics of Proposed Treatment Methods

The annual cost per acre at an interest rate of five percent for a period of 30 years of the sediment separator and the holding basin are compared in Figures 26 through 33. From Figure 30, for 40 storms, an interceptor capacity of 4 X DWF, and a runoff coefficient of 0.5, it can be seen

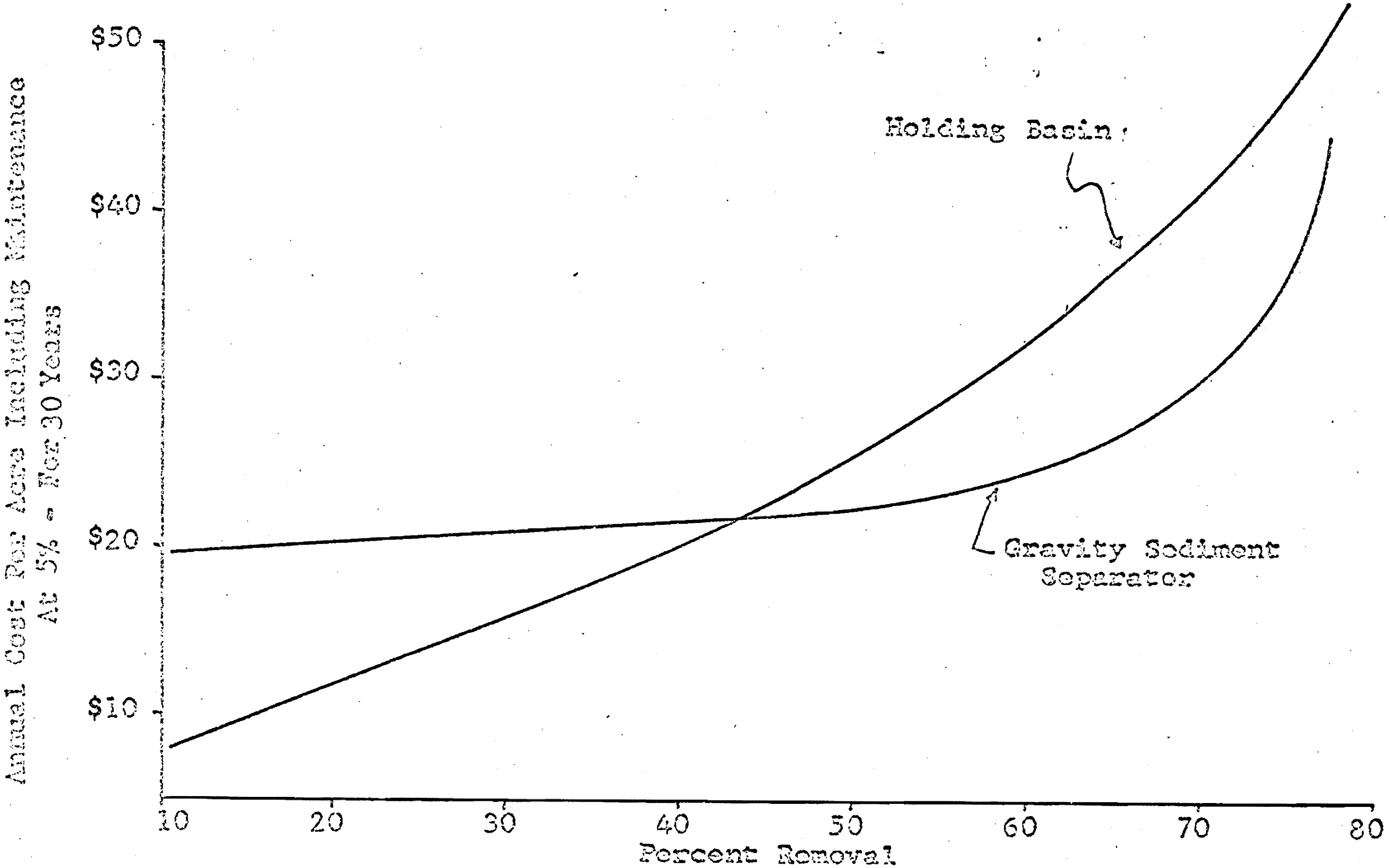

FIGURE 26. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With an Interceptor Capacity of 2 Times DWF, a Runoff Coefficient of 0.3, and 40 Storms During the Recreational Season.

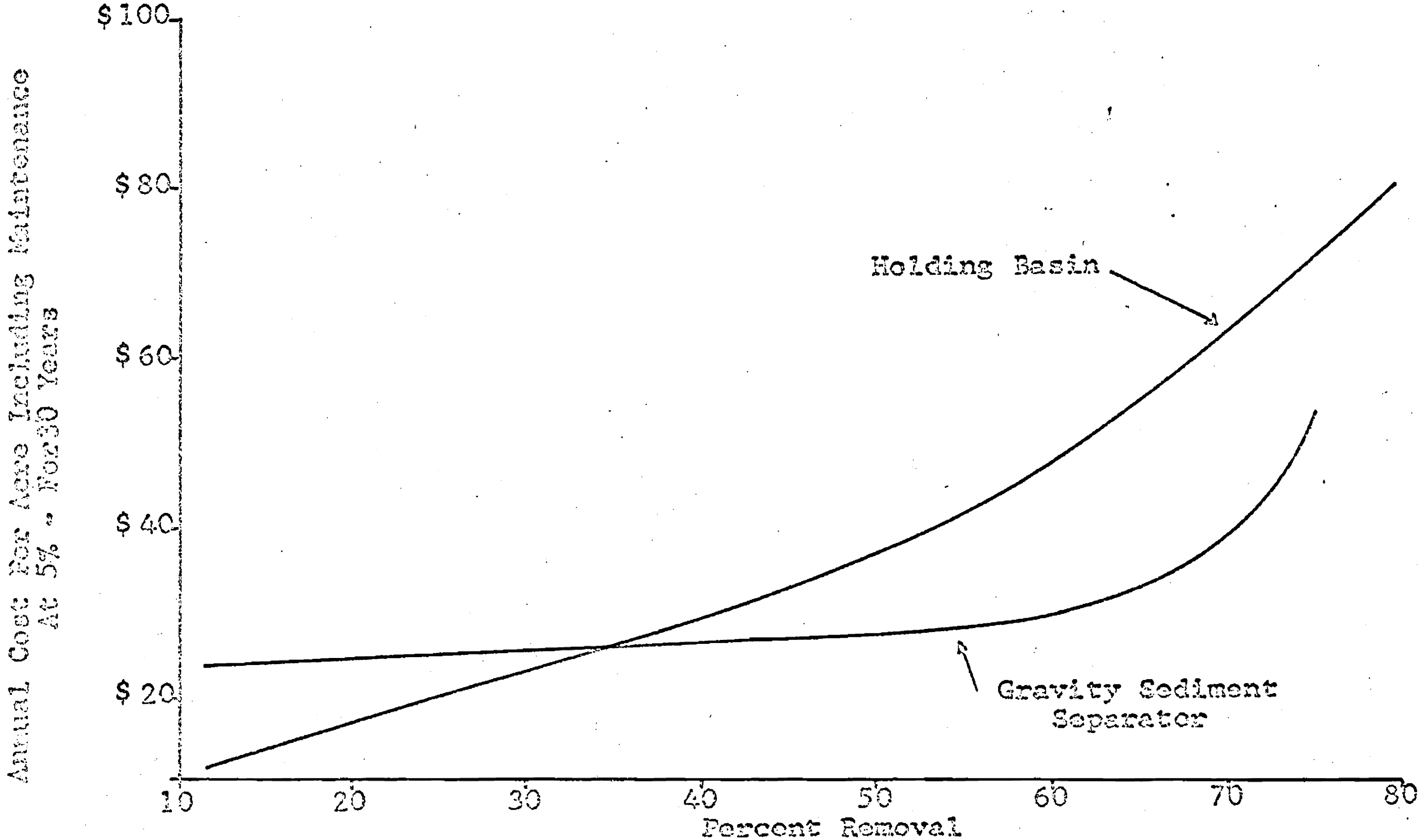

FIGURE 27. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With an Interceptor Capacity of 2 Times DWF, a Runoff Coefficient of 0.5, and 40 Storms During the Recreational Season.

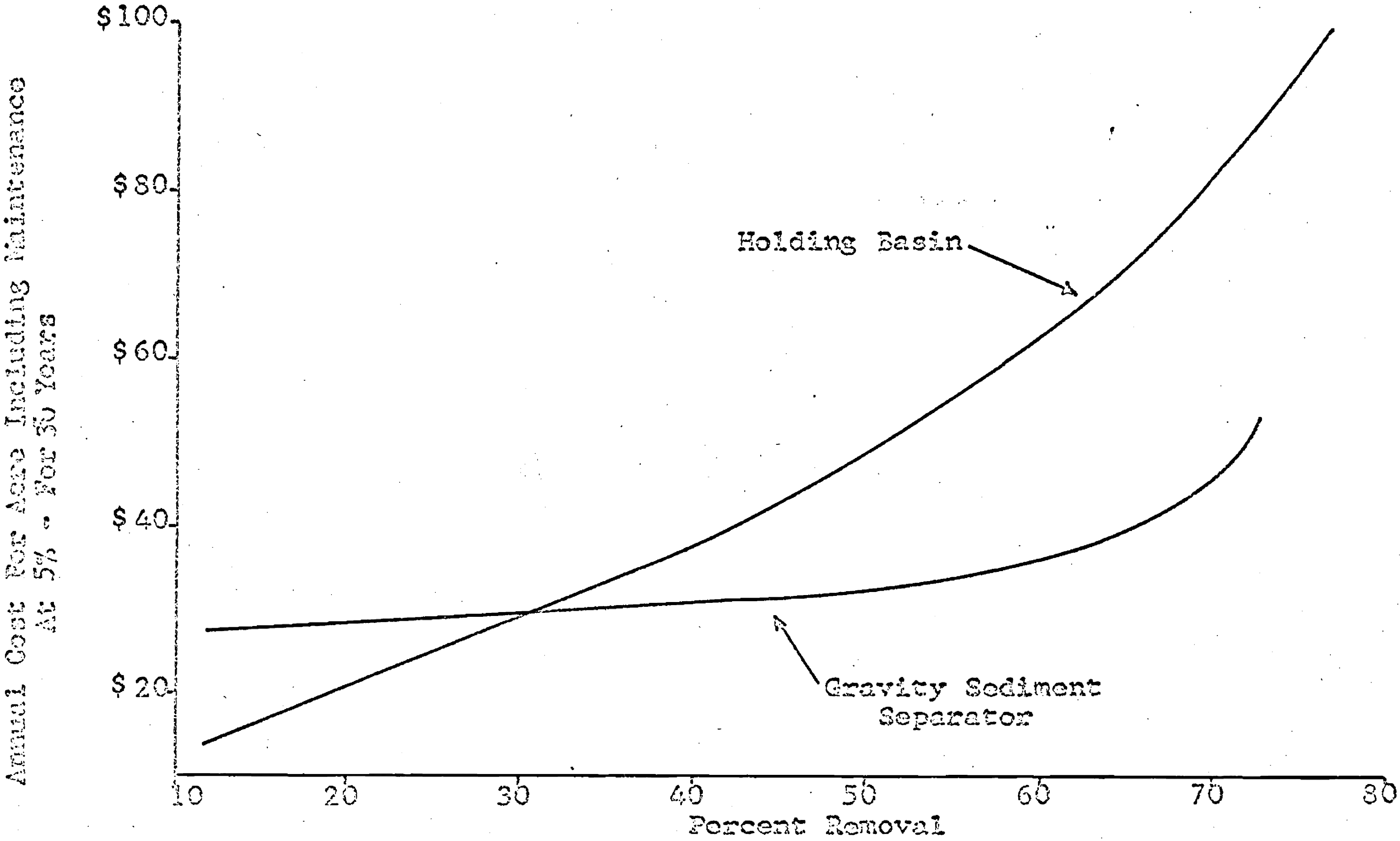

FIGURE 28. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With An Interceptor Capacity of 2 Times DWF, a Runoff Coefficient of 0.7, And 40 Storms During the Recreational Season.

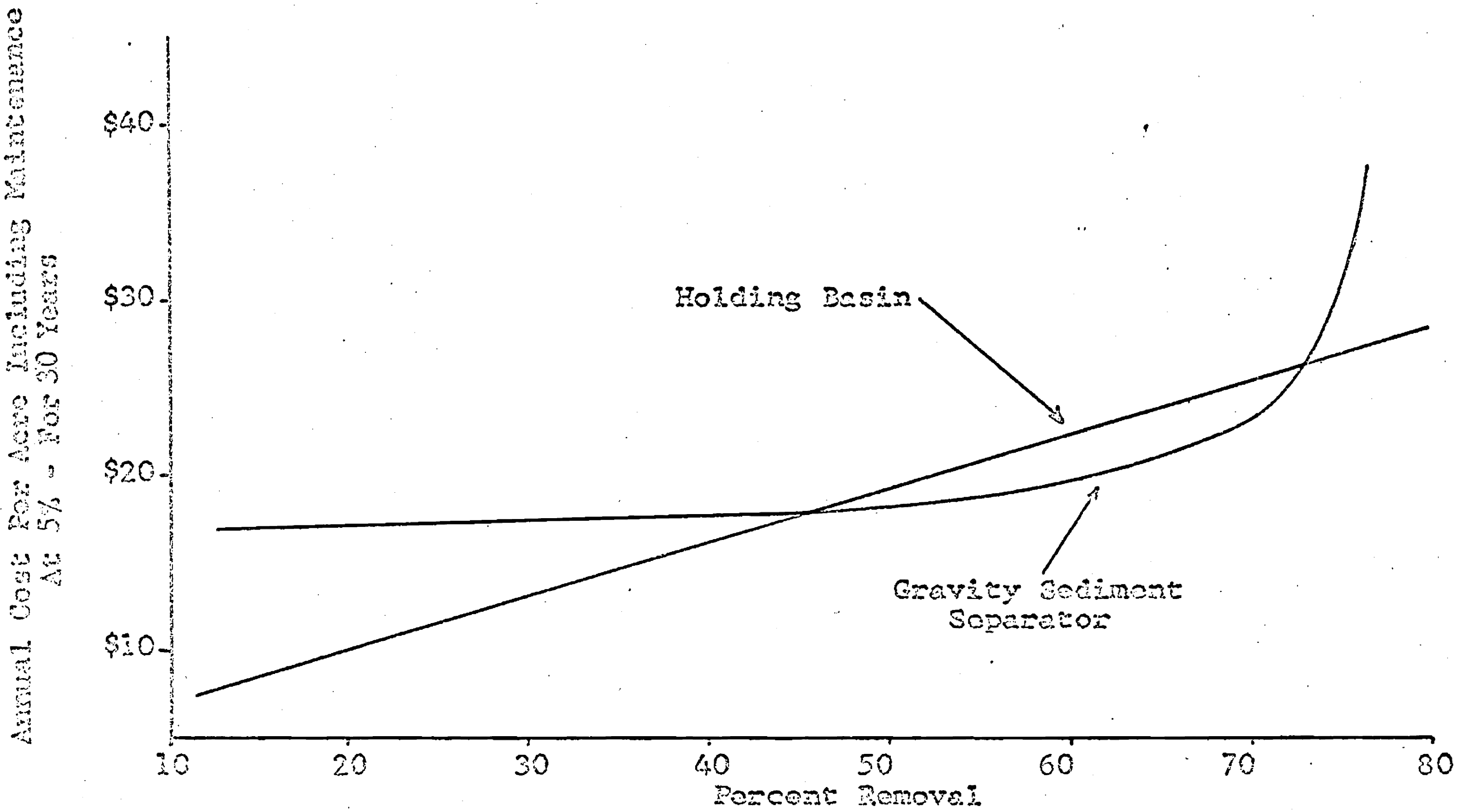

FIGURE 29. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With An Interceptor Capacity of 4 Times DWF, a Runoff Coefficient of 0.3, And 40 Storms During The Recreational Season.

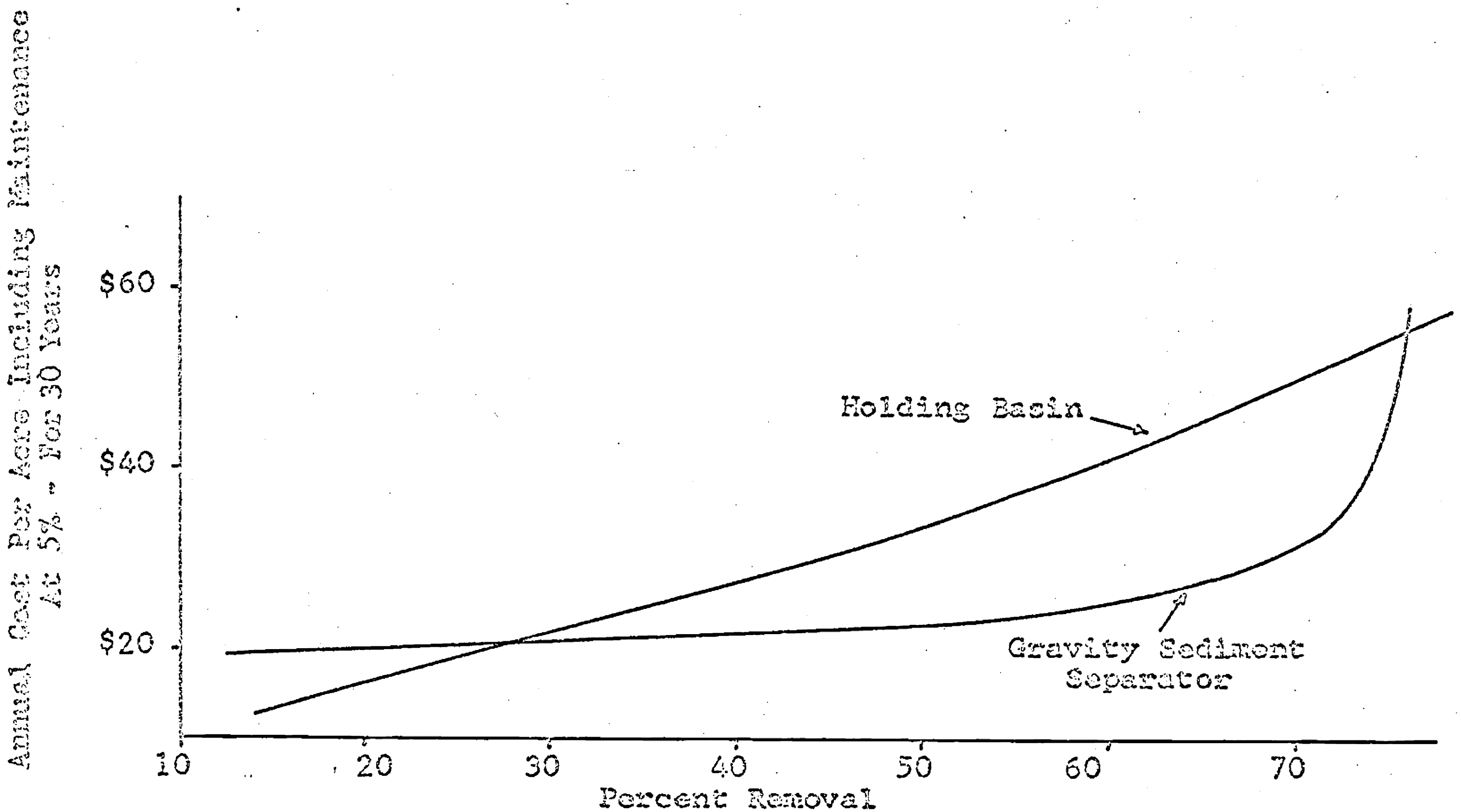

FIGURE 30. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With An Interceptor Capacity of 4 Times DWF, a Runoff Coefficient of 0.5, And 40 Storms During the Recreational Season.

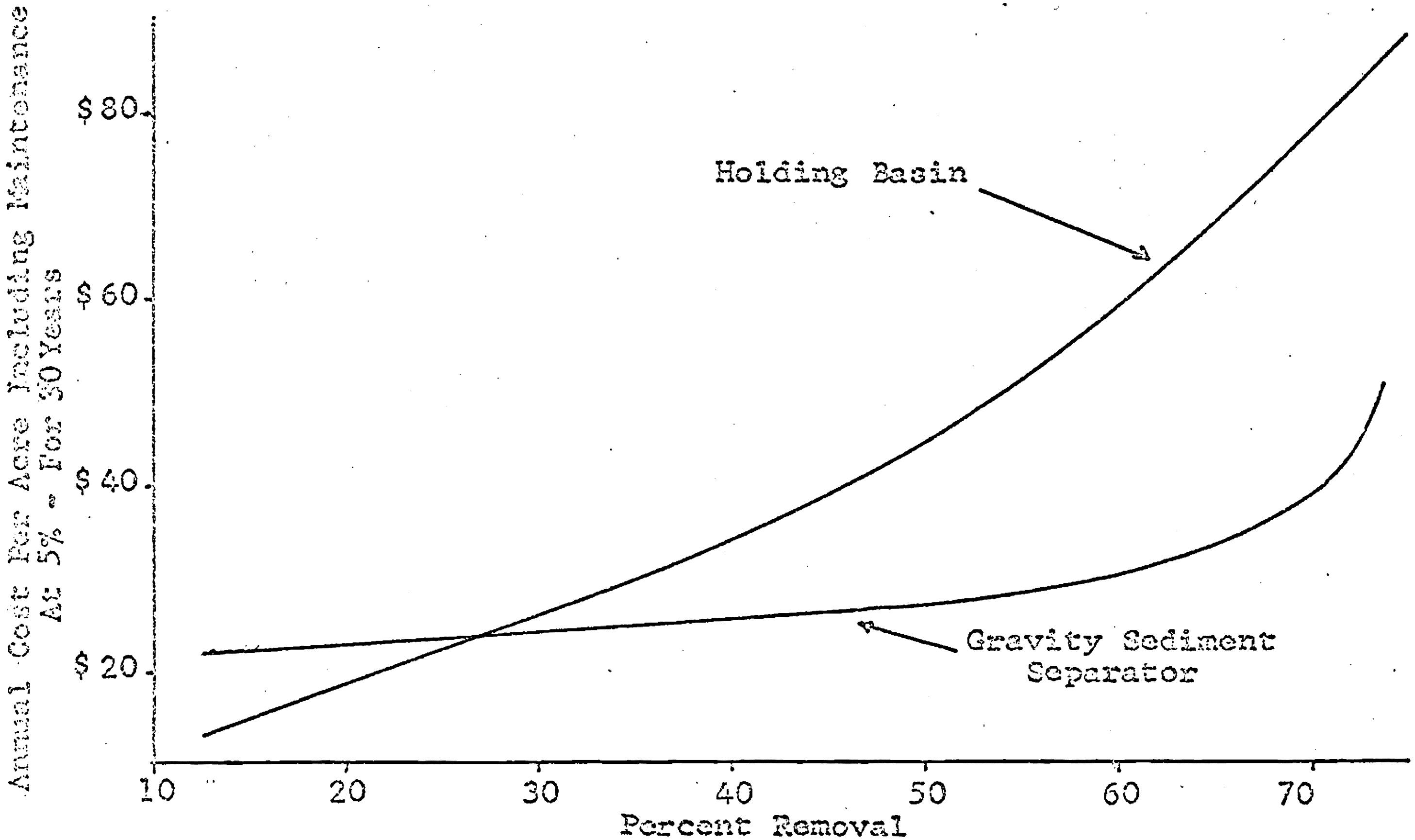

FIGURE 31. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With An Interceptor Capacity of 4 Times DWF, a Runoff Coefficient of 0.7, and 40 Storms During the Recreational Season.

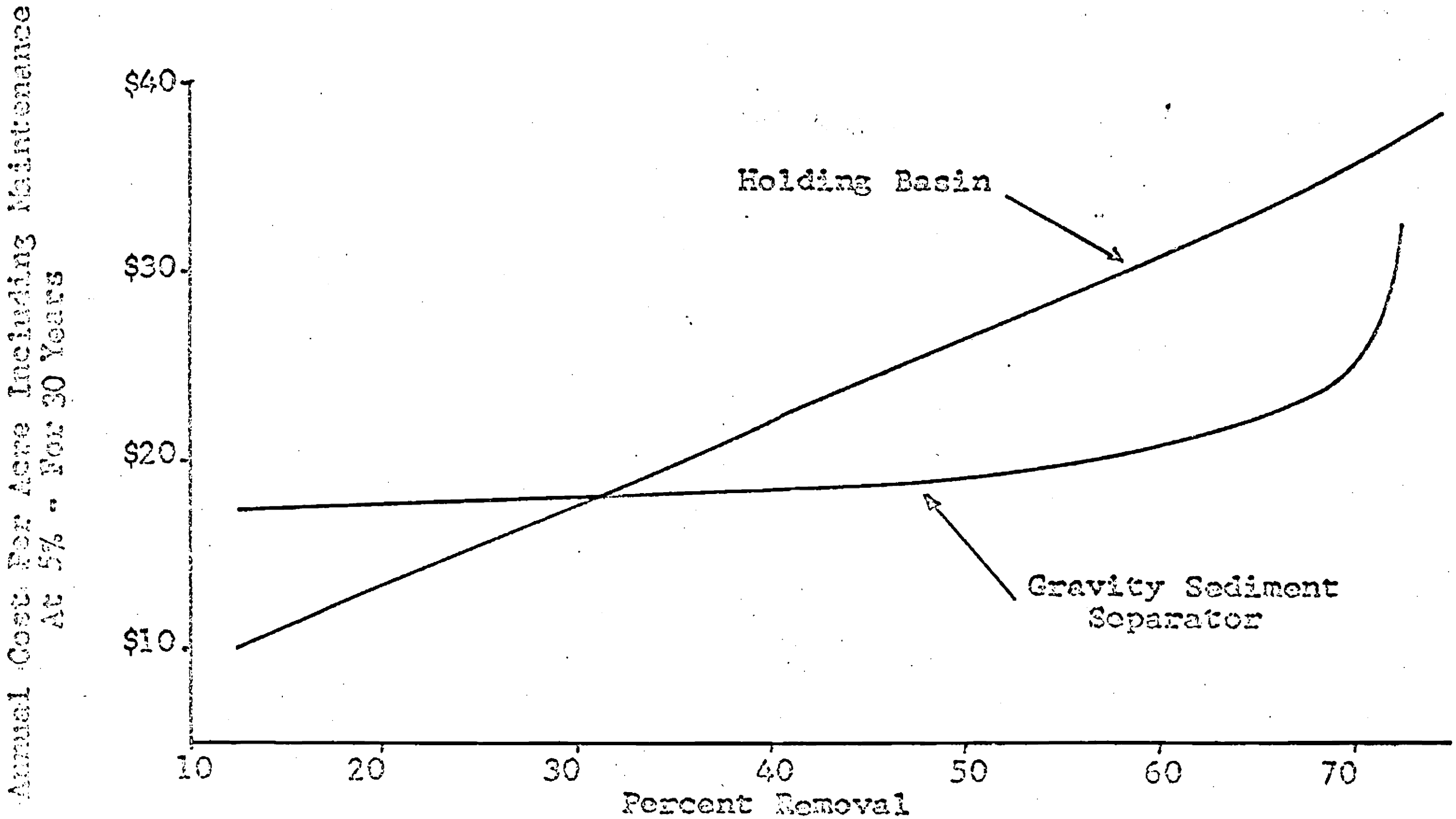

FIGURE 32. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With An Interceptor Capacity of 6 Times DWF, a Runoff Coefficient of 0.5, And 40 Storms During the Recreational Season.

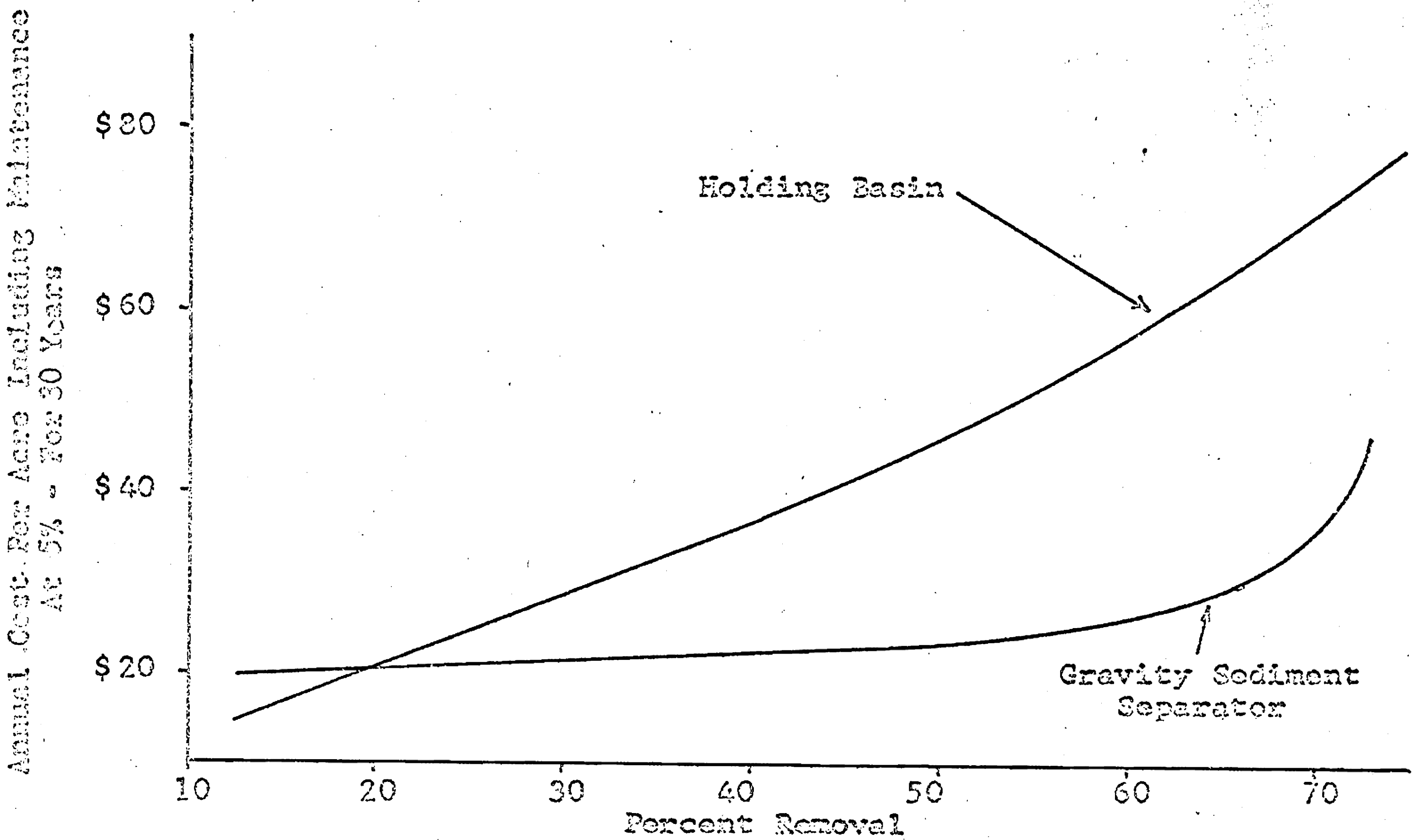

FIGURE 33. Comparison of Cost of Alternative Regulation Devices To Obtain Stated Reduction of Overflow of Suspended Solids From a Combined Sewer With an Interceptor Capacity of 6 Times DWF, a Runoff Coefficient of 0.7, and 40 Storms During The Recreational Season.

that the annual cost per acre including maintenance of the holding basin required to affect a 60 percent reduction in the overflow is \$41.00 while the annual cost including maintenance of the gravity sediment separator required to achieve the same reduction in suspended solids is \$25.00. The holding basin is cheaper than the separator in the lower and upper ranges of reduction while the separator normally appears to have a cost advantage in the range of 30 to 75 percent reduction. The gravity sediment separator loses its cost advantage if a suspended solids removal of 75 percent or greater is required. In this regard it should be noted that the sediment separator efficiency is based on conservative equations for the suspended solids removal as developed for a primary sedimentation tank. As previously indicated better predicted removal efficiencies may be warranted following field testing of the proposed treatment device. The general non linear cost equation for the holding basin was found to be:

$$COST = 4.59 + 0.15 \text{ X } CPA^{0.83}$$

$$R = 1.000$$

where:

COST = the annual cost per acre at five percent for a period of 30 years. This cost includes maintenance.

CPA = the capacity per acre in cubic feet of the holding basin.

This equation has a correlation coefficient of 1.000.

The non-linear cost equation for the sediment separator was found to be:

$$COST = 19.22 + 0.04 \times PCSS^{1.45} \times N^{0.38} \times COEF^{1.65} \times CINT^{-0.79}$$

$$R = .9965$$

where:

COST = the annual cost per acre based on five percent interest for a period of thirty years. This cost includes operation and maintenance costs.

PCSS = the percent of suspended solids removed by the sediment separator. The removal should be between 30 and 70 percent.

N = the number of storms expected during the recreational season.

COEF = the runoff coefficient.

CINT = the interceptor capacity expressed in multiples of the average DWF.

This model has a correlation coefficient of 0.9965.

The effect of the 5 percent interest rate and the 30 year design period on the annual cost per acre for both the gravity sediment separator and the holding basin were investigated. A change in the interest rate will not significantly alter the annual cost of the separator or the holding basin as the annual cost per acre including maintenance is in the range of $50. The design period has a much greater effect upon the annual cost per acre. The annual cost per acre including maintenance will be about 10 percent less if the design period is extended to 40 years and approximately 20 percent more if the design period is shortened to 20 years.

DISCUSSION OF RESULTS

This investigation was conducted in order to determine the characteristics of the overflow from combined sewers and the effect, efficiency, and cost of proposed treatment methods or regulation devices. These parameters are of interest to the design engineer in planning the sewerage systems of a new urban community or in analyzing the existing conditions of an older system. In adequate and outdated combined sewerage systems in the older downtown sections pose the question of the most economical solution to the problems of overflow resulting from storm runoff and even, perhaps, infiltration. The design engineer should be fully aware of the characteristics of the existing system and the effect of any proposed alternative solutions on the parameters of interest.

The rainfall duration and frequency graphs were constructed to be representative of the Middle Atlantic region of the East coast. Similar graphs could be synthesized for regions which have different meteorological characteristics. The characteristics of the actual precipation encountered in any particular place in the region during any particular recreational season will obviously not be identical with those forecast by this procedure. However, in the long run, the synthesized data should be representative of typical meteorological patterns in the middle Atlantic region.

The runoff coefficient of a developed area may be estimated in two ways. The historical approach assigns each area a representative value by assuming that a fixed percent of all precipitation is converted to storm runoff after the initial wetdown. The second approach, which may be more applicable to heavily developed areas, involves the assumption that only the precipitation which falls on impervious surfaces, such as roads, sidewalks, and roofs contributes directly to runoff, and thus this portion of the total area has a runoff coefficient of 1.00 after the initial wetdown. The overall runoff coefficient is then estimated by the percent of impervious area as estimated from an aerial photograph. If there is an infiltration problem, then the runoff coefficient should be increased to compensate for this added source.

If the average DWF is not equal to a direct runoff of 0.01 inches per hour, as assumed in this investigation, the coefficient of runoff may be adjusted to negate this assumption in order to utilize the results presented herein. For example, if the DWF is equal to a direct runoff of 0.02 inches per hour, then the runoff coefficient should be reduced by one-half.

The precipitation in this investigation was assumed to have a characteristically uniform intensity over the drainage basin and in addition was considered to be constant over the course of any particular storm. It is felt this assumption has a insignificant effect on the validity of the results obtained and is realistic. Urban drainage basins are usually less than 100 acres in size and consequently are smaller than those usually studies by hydrologists. Smaller basins of this type characteristically exhibit less variation in intensity and duration than a larger basin. The assumption of a rectangular hydrograph can be defended by the same argument. Since the drainage basin is relatively small, it is feasible to assume that the peak runoff will occur more quickly and will remain constant as long as the whole basin is contributing. This characteristic will tend to make the hydrograph rectangular in shape. The effect of the interceptor will be to flatten the larger peak flows. This interceptor effect and the rectangular tending hydrograph associated with it one most typical of the larger urban runoffs. These heavier storms are the most significant in terms of the quantity of overflow of the combined sewer.

The characteristics of the unregulated combined sewer overflow are very important in analyzing the existing system and feasible alternatives. The percent of the summer's production of BOD escaping in the overflow as presented is identical with the percent of the total quantity of flow escaping, since the concentration of BOD in the overflow and during DWF is assumed to be the same. Figures 8, 9, and 10 can therefore be used to estimate the total quantity of overflow as well as the percent of BOD escaping. The percent of the summer's production of suspended solids escaping during periods of combined sewer overflow is very significant in terms of the quantity overflowing. The actual amount is greatly dependent on the runoff coefficient and the interceptor size. The quantities of suspended solids as presented herein are also dependent upon the quantity of overflow and the ratio of the suspended solids concentration in the overflow to the concentration during periods of DWF. The ratio under consideration, as developed in this work, is assumed to be 800/180 or 4.45. The results presented are thought to be significant for ratios between 4.0 and 5.0. If the ratio increases, then the percent of suspended solids escaping will increase commensurately. This assumed ratio is believed to be representative of the average situation as cited in the literature review. The average suspended solids concentration in any typical overflow should be used in finding this ratio and not the suspended solids concentration of the first flush which typically would be significantly higher.

The holding basin approach appears to offer a feasible solution under some circumstances to the problem of combined sewer overflow. A typical holding basin will require a large amount of land to provide the required amount of storage. This characteristic might be a detering factor in the more heavily developed urban areas. The holding basins could perhaps be located beneath public parks, play grounds, or highways. A system of smaller local holding basins might offer a better alternative, since it would often be easier to locate several small tracts of land than one large tract. The percent reduction in combined sewer overflow obtained by the holding basin is dependent upon the coefficient of runoff and the interceptor capacity. As shown in the results of this research, the capacity required to obtain a stated reduction in overflow is almost completely independent of the number of storms expected during the recreational season.

The predicted efficiency of the sediment separator in the removal of the suspended solids from the overflow as used in this study is believed to be very conservative, as its efficiency is based on the Ten State Standard removal rates for a primary sedimentation tank and consequently does not take into account the additional sedimentation features of the gravity sediment separator. The increasing slope of Figures 21 through 25, are reflections of the variable removal rates of the primary sedimentation unit. It is relatively easy to remove a small fraction of the suspended solids. Removal of increasing quantities is progressively more difficult. As the removal rate approaches seventy percent, the curves increase rapidly, illustrating that it requires a proportionately larger settling time to remove more than seventy percent. As the capacity of the gravity sediment separator increases, it begins to act more like a holding basin on the smaller overflows. The efficiency of the separator appears to be fairly independent of the number of storms expected. The characteristics of the gravity sediment separator demand that field tests should be made to more properly evaluate the efficiencies, operating characteristics, and full potential of the unit.

The cost comparison on an annual basis as illustrated in Figures 26 through 33 is very interesting. Both the holding basin and the gravity sediment separator have characteristics which make them desireable in certain ranges of removal. The range in which the separator is cheapest increases as the coefficient of runoff and interceptor capacity increase. Since the performance of both units was relatively independent of the number of storms, the costs associated with each treatment scheme will be largely independent of this factor. The separator always tends to lose its economic advantage at a removal rate of about 75 percent.

The non-linear regression equations are believed to be very descriptive of the data they represent. They should only be applied within the ranges and limitations described previously. These equations are presented in lieu of a cumbersome number of graphs.

The interest rate does not appear to have an appreciable effect upon the annual cost per acre of the gravity sediment separator or the holding basin as the annual cost is relatively small. The design period of either the separator or the holding basin has an effect on the annual cost per acre as might be expected.

The results of this study of the nature and characteristics of combined sewer overflow yield results that have practical implications. It is felt that either the specific results of this study or results utilizing modifications of such a technique would make the method of analysis an acceptable design procedure for large urban communities which possess combined sewer overflow problems.

CONCLUSIONS

The following conclusions may be drawn from this investigation.

1. The percent of suspended solids escaping to the receiving watercourse is more dependent on the coefficient of runoff and the interceptor capacity than on the number of storms during the recreational season.
2. The storage capacity of a holding basin or gravity sediment separator required to obtain a stated reduction of overflow is more dependent on the runoff coefficient and interceptor capacity than on the number of storms.
3. The gravity sediment separator and the holding basin each possess qualities which allow them to be the least expensive for certain removal efficiencies, but within the range of most interest, 30 to 75 percent reduction, the separator is the most feasible method of treatment.
4. The characteristics of the gravity sediment separator indicate that developmental investigations should be made. These studies would perhaps offer a rapid realization of the potential of the process.
5. The cost per acre of the gravity sediment separator approaches five percent of the cost of complete separation which is estimated at approximately $13,000 per acre.

SUMMARY

Stormwater overflows from combined sewers constitute one of the major sources of quality impairment of natural watercourses in urban areas. This pollution can render the receiving watercourse unfit for human recreation or shell fish propagation for weeks at a time.

The suspended solids concentration is mainly responsible for the pollution of the stream due to its high concentration during periods of overflow. The suspended solids concentration is four to six times greater in the overflow of a combined sewer than in the average dry weather flow. This increase is due to the scouring of sludge and grit in the interceptor as well as the suspended solids carried into the interceptor by the stormwater runoff. The effect of this discharge is particularly significant due to the shock or intermittant loading it places on the stream. Periodic flushing of combined sewers during dry weather conditions would do much to alleviate the high concentration of solids in overflows.

The problem of stormwater overflows from combined sewers is growing with the increasing urbanization and water demands in certain metropolitan areas of the United States. The complete separation of storm and sanitary lines is considered to be the ultimate solution, even though the stormwater will eventually have to be given treatment in some

localities. The average cost of physical separation of storm and sanitary lines appears to be approximately $13,000 per acre.

An engineering economic analysis of remedial methods of treatment has been conducted. Attention has been given to a system of either local holding basins or newly-conceived treatment units known as gravity sediment separators. Each of these alternatives possess qualities which are desireable. The analysis of the gravity sediment separator indicates that a developmental investigation should be made. This type of research might well demonstrate the feasibility of the process and lead to a rapid realization of its potential. The cost of the gravity sediment separator including maintenance and operating costs appears to approach five percent of the cost of complete separation.

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APPENDIX A

APPENDIX. IBM 7040 COMPUTER PROGRAM IN FORTRAN IV

```
      DIMENSION X(16),Y(16),X1(8),Y1(8),RAIN(50),DUR(50),
     1XINT(50),XDWF(50),TRNOF(50),DWF(50),TFLO(50),OFLO(50),
     2OSS(50),BOD(50),Z(9),R(9),FR(5),RBOD(5),PCRSS(50),
     3DUROF(50),OUTFL(50),OM(60),SCOST(60),ACPA(55,2),
     4R1(50),R2(50),R3(50),R4(50),R5(50),R6(50),RED(55,5)
     5PCRBOD(50),DU1(60)
C
C     COEF = RUNOFF COEFFICIENT
C     CINT = INTERCEPTOR CAPACITY IN MULTIPLES OF THE DWF
C     C1 THROUGH C14 = COST PARAMETERS
C     RAIN = TOTAL PRECIPITATION IN INCHES
C     DUR = DURATION OF THE STORM IN HOURS
C     N = NUMBER OF STORMS DURING RECREATIONAL SEASON
C     XNO = INTERVAL IN PERCENT BETWEEN STORMS
      READ1,N,COEF,AREA,ENR62,ENR67,CINT
      READ 2,(X(J),J = 1,16)
      READ 2,(Y(J),J=1,16)
      READ 3, (X1(J),J=1,8)
      READ 3, (Y1(J),J=1,8)
    1 FORMAT(5X,I3,5F7.2)
    2 FORMAT(5X,16F4.0)
    3 FORMAT(5X,8F4.0)
      READ 65 , C1,C2,C3,C4,C5,C6,C7,C8,C9
```

```
   65 FORMAT  (2X,9F7.2)
      READ 66,C10,C11,C12,C13,C14
   66 FORMAT (2X,5F7.2)
C
C     READ IN REMOVAL OF SUSPENDED SOLIDS
      READ 42,(Z(I),I=1,7)
      READ 42,(R(I),I=1,7)
   42 FORMAT (5X,7F5.2)
   81 B=N
      NOP = 0
      XNO=100./(2.*B)
      J=0
C     FIND TOTAL RAIN IN STORM(I)
      DO 14 I=1,N
    5 J=J+1
    8 IF(X(J)-XNO) 5,6,7
    6 RAIN(I)= Y(J)
      GO TO 4
    7 RAIN(I)=Y(J)+(Y(J-1)-Y(J))*(X(J)-XNO)/(X(J)-X(J-1))
    4 XNO=XNO+100./B
   14 J=0
C     FIND DURATION
      J=0
      DO 9 I=1,N
   12 J=J+1
```

```
   13 IF(Y1(J)-RAIN(I))12,11,10
   11 DUR(I)=X1(J)
      GO TO 9
   10 DUR(I)=(X1(J)-X1(J-1))*(RAIN(I)-Y1(J-1))
      DUR(I)=DUR(I)/(Y1(J)-Y1(J-1))+X1(J-1)
    9 J=0
C
      KK=0
      DO 41 JJJ=2,6
      M2=0
      DO 21 JJJ1=3,7
      KK=KK+1
      M2=M2+1
      NOP = NOP + 1
      XXX=JJJ1
      COEF= XXX/10.
      CINT=JJJ
      RATIO=CINT/COEF
      DO 15 I=1,N
C     FIND AVERAGE INTENSITY OF STORM
      XINT(I)= RAIN(I)/DUR(I)
      DU1(I)=(RAIN(I)-.03)*DUR(I)/RAIN(I)
      IF(DU1(I)) 17,17,23
   17 DU1(I)=0.0
   23 XDWF(I)=(COEF*XINT(I)+.01)/.01
```

```
C     TRNOF = TOTAL RUNOFF DUE TO RAIN (I)
      TRNOF(I)=COEF*(RAIN(I)-.03)*AREA*43560./12.
      IF(TRNOF(I)) 69,69,70
C
   69 TRNOF(I)=0.0
C     DWF= AVERAGE DRY WEATHER FLOW
   70 DWF(I)=.01*AREA*DU1(I)*43560./12.
C     TFLO= TOTAL FLOW IN INTERCEPTOR = TRNOF + DWF
      TFLO(I)=TRNOF(I)+ DWF(I)
C     OFLO = AMOUNT OF COMBINED SEWAGE OVERFLOW
      OFLO(I)=TFLO(I)-CINT*DWF(I)
      IF(OFLO(I))67,67,68
   67 OFLO(I)=0.0
C     OSS = AMOUNT OF OVERFLOW OS SUSPENDED SOLIDS
   68 OSS(I)=800.*OFLO(I)*7.48*8.34/10.**6
C     BOD = AMOUNT OF BOD OVERFLOWING
   15 BOD(I)=180.*OFLO(I)*7.48*8.34/10.**6
      SS1=0.0
      SS2=0.0
      DO 83 I=1,N
      IF (TRNOF(I)) 83,83,86
   86 SS2 = TFLO(I) - OFLO(I) + SS2
      SS1=DU1(I)+SS1
   83 CONTINUE
      CSS = SS2*800.*7.48*8.34/10.**6
```

```
      CBOD = 180.*CSS/800.
      IF (JJJ*JJJ1-6) 106,106,105
C
  106 PRINT 100
  100 FORMAT(1H1)
      PRINT 85,N
   85 FORMAT (10X,4HN = ,I3)
      PRINT 99,COEF,CINT,RATIO
   99 FORMAT(10X,7HCOEF = ,F7.1,5X,7HCINT = ,F7.1,5X,
     18HRATIO = ,F7.1)
      PRINT 16
   16 FORMAT(5X,9HINTENSITY,5X,8HDURATION,6X,10HTOTAL RAIN,
     14X,11HMULT OF DWF,3X,12HTOTAL RUNOFF,2X,10HTOTAL FLOW,
     24X,11HAMT OF OVER,4X,10HLBS OF BOD,4X,9HLBS OF SS,/,
     37X,5HIN/HR,9X,5HHOURS,9X,6HINCHES,23X,5HIN CF,9X,
     45HIN CF,6X,10HFLOW IN CF,5X,11HOVERFLOWING,3X,
     511HOVERFLOWING,/)
C
      DO 18 I=1,N
   18 PRINT 19,XINT(I),DUR(I),RAIN(I),XDWF(I),TRNOF(I),
     1TFLO(I),OFLO(I),BOD(I),OSS(I)
   19 FORMAT(F14.3,8F14.2)
C
  105 CONTINUE
      CORR = 153.-SS1/24.
```

```
      TBOD=CORR*.01*AREA*180.*.04356*7.48*8.34*24./12.
      TSS=TBOD
      TBOD = TBOD+CBOD
      TSS = TSS+CSS
      ZZ=0.0
      XX=0.
      YY=0.
      DO20 I=1,N
      ZZ=ZZ+OFLO(I)
      XX=XX+BOD(I)
   20 YY=YY+OSS(I)
      XBOD=100.*XX/(TBOD+XX)
      XSS=100.*YY/(TSS+YY)
C
      R1(NOP)= XSS
      R2(NOP) = XBOD
      SS3=0.0
      DO 24 I=1,N
      IF(OFLO(I)) 24,24,25
   25 SS3= DU1(I) + SS3
   24 CONTINUE
      R3(NOP)=SS3/24.
      R4(NOP)=SS3
      R5(NOP)=SS3/5.
      R6(NOP)=100.*R3(NOP)/153.
```

```
C
C     STORM WATER HOLDING BASIN
C     CAPACITY OF TANK IN   CU FT PER ACRE
      M1=0
      DO 30 I=50,2700,50
      M1=M1+1
      XI=I
      CAP = XI*AREA
      W=ABS((2.*C2*CAP/C1)**0.3333)
      D=CAP/W**2
C
C     1.5 FEET OF FREEBOARD
      IF(I) 84,84,48
   48 IF(D-20.) 88,88,87
   87 D1 = 20.
      W=(CAP/D1)**0.5
      D=21.5
      GO TO 84
   88 D=D+1.5
   84 WIDTH = W
      IF(JJJ*JJJ1-6) 63,63,55
C
C     DETERMINE COST OF HOLDING BASIN
C     20 PC P+O + 18 PC ENGR AND LAND FEES
   63 COST=((4.*C2*W*D+C1*W*W)*1.2*.75/27.+CAP*1.2*C3/27.)
```

```
      COST = (COST+C4)*1.38*ENR67/ENR62
      XMAIN = .05*COST/AREA
      SCOST(M1)=COST/AREA
   61 XI=.05
      L2=30
C     FIND ANNUAL COST PER ACRE INCLUDING MAINTENANCE
      ACPA(M1,1)=(COST*XI/(1.-(1./(1.+XI))**L2))/AREA
      ACPA(M1,2)=ACPA(M1,1)+XMAIN
C
   55 DO 32 J=1,N
      IF (CAP-OFLO(J))33,34,34
   33 OUTFL(J)= OFLO(J)-CAP
      GO TO 32
   34 OUTFL(J)=0.0
   32 CONTINUE
C
      W=0.
      DO 36  J=1,N
   36 W=W+ OUTFL(J)
C     RED = PERCENT REDUCTION IN OVERFLOW OF COMBINED SEWER
      RED(M1,M2)=100.-100.*W/ZZ
   30 CONTINUE
C
C     SEDIMENT SEPARATOR
C
```

```
C     L = LENGTH TO WIDTH RATIO
   52 L=6
      K = 0
      DO 21 I=3,23
      K=K+1
      WIDTH = I
      IF (I-12) 75,75,76
   75 DEPTH = I
      XLENTH=L*I
      GO TO 77
   76 DEPTH = 12.
      XLENTH=L*I
   77 CAP= WIDTH*DEPTH*XLENTH
C     CPA = CAPACITY PER ACRE OF SEDIMENT SEPARATPR IN FT**3
      CPA=CAP/AREA
      DWF1=.01*AREA*43560.*7.48/(60.*12.)
C
      DO 60 M=1,N
      GPM=7.48*OFLO(M)/(DUR(M)*60.)
C     DET = DETENTION TIME
      DET = CAP*60.*DUR(M)/(OFLO(M))
      ORATE = OFLO(M)       *7.48*24./(DUR(M)*WIDTH*XLENTH)
C
C     FIND REMOVAL OF SS BASED ON DETENTION TIME
      IF(OFLO(M)-CAP) 72,72,73
```

```
C      PCRSS = PERCENT REMOVAL OF SUSPENDED SOLIDS
   72 PCRSS(M)=1.0
      PCRBOD(M) = 1.0
      GO TO 53
   73 J=0
   47 J=J+1
      IF(120.-DET) 45,45,46
   45 PCRSS(M)=0.72
      GO TO 43
   46 IF(DET-Z(J+1)) 44,44,47
   44 PCRSS(M)=R(J)+(R(J+1)-R(J))*(DET-Z(J))/(Z(J+1)-Z(J))
   43 IF(4600.-ORATE) 49,49,50
   49 PCRBOD(M)=0.0
      GO TO 53
   50 PCRBOD(M)=-ORATE*(.405/4600.)+0.405
C
   53 V=M
      RINT = 153./(7.*V)
      CFS=OFLO(M)/(DUR(M)*60.*60.)
      RPCSS=100.*PCRSS(M)
      RPCBOD=100.*PCRBOD(M)
   60 CONTINUE
C
C
C      COST OF TREATMENT WITH SEDIMENT SEPARATOR
```

```
C         SCRAPER COSTS 78 CENTS PER CUBIC FOOT OF CAPACITY
C        WEIR COSTS $8.50 PER LIN FOOT + 25 PERCENT INSTALLATION
C        DUPLEX PUMP SYSTEM   ADD 20 PERCENT FOR INSTALLATION
C        COST OF SLUDGE LINES = 20 PERCENT COST OF PUMP
C        PUMP COST = 40*X + $500     WHERE X = GPM
C        20 PERCENT FOR P+O + 18 PERCENT ENGINEERING + LAND FEES
C        10 PERCENT EXTRA ON CONCRETE FOR ADDITIONAL STRUCTURES
C
C
         CWAL=2.*C2*(DEPTH+1.5)*(WIDTH+XLENTH)* .75*1.1
         CFLOR=C1*XLENTH*WIDTH*.75*1.1
         CEX=C3*CAP*1.2
         COS=(CWAL+CFLOR+CEX)*ENR67/(ENR62*27.)
         CPUMP=40.*C.1*DWF1+500.
         CWEIR=(WIDTH+5.)*2.*C12
         CO=COS+1.4*CPUMP+1.2*CWEIR+C5+C9+C13+.78*CAP
         COST=CO*1.38
         OM(K) = .05*COST/AREA
C
      22 SUM=0.0
         SUMBOD=0.0
         OFLO1=0.0
         SUMSS=0.0
         SUM1=0.0
C
```

```
      DO 62 M=1,N
      SUMBOD = SUMBOD + BOD(M)
      OFLO1=OFLO1+OFLO(M)
      SUM=SUM + PCRBOD(M)*BOD(M)
      SUMSS =SUMSS + OSS(M)
   62 SUM1 =SUM1 +PCRSS(M)*OSS(M)
      CCL = 30.*8.34*7.48*OFLO1*C7/10.**6
      FBOD = 100.*SUM/SUMBOD
      FSS=100.*SUM1/SUMSS
      XI=.05
      L2=30
        B3        =(COST*XI/(1.-(1./(1.+XI))**L2)+CCL)/AREA
      B4=B3+OM(K)
      X17=COST-CAP*.78*1.38
       B1 =(X17*        XI/(1.-(1./(1.+XI))**L2)+CCL)/AREA
      B2=B1+.05*X17/AREA
      B5=COST/AREA
      B6=X17/AREA
      IF(I-3) 27,27,28
   27 PRINT 100
      PRINT 54, N,JJJ,JJJ1
   54 FORMAT(20X,18HSEDIMENT SEPARATOR,/,20X,4HN = ,I3,
     15X,7HCINT = ,I3,5X,7HCOEF = ,I3,/,10X,8HCAP/ACRE,6X,
     25HPCRSS,7X,6HPCRBOD,4X,9HCOST/ACRE,6X,4HACPA,5X,
     39HACPA+MAIN,4X,10HTC-SCRAPER,5X,4HACPA,5X,9HACPA+MAIN,/)
```

```
 28 PRINT 29,CPA,FSS,FBOD,B6,B1,B2,B5,B3,B4
 29 FORMAT(4X,9F12.2)
    PUNCH 139,CPA,B,CINT,COEF,FSS
139 FORMAT(5X,5F12.3)
    PUNCH 140,B2,B,CINT,COEF,FSS,CPA
140 FORMAT(4X,6F12.3)
 21 CONTINUE
    Z1=.2
    DO 141 I=1,5
    Z1=Z1+.1
    II=0
    DO 141 IX=50,2700,50
    Z2=IX
    II=II+1
141 PUNCH 142 ,Z2,B,CINT,Z1,RED(II,I)
142 FORMAT(3X,5F12.3)
    IF(N-30)145,145,41
145 II=0
    DO 144 I=50,2700,50
    XI=I
    II=II+1
144 PUNCH 143,ACPA(II,2),XI
143 FORMAT(5X,2F12.3)
 41 CONTINUE
    PRINT 100
```

```
      II=0
      DO 147 I=2,6
      CINT =I
      DO 147 J=3,7
      COEF = J
      COEF=COEF/10.
      II=II+1
  147 PUNCH 146,R1(II),CINT,COEF,B
  146 FORMAT(1X,4F12.3)
      N = N + 10
      IF(N-50) 81,81,82
   82 STOP
      END
$ENTRY
C
C     ****PLACE DATA HERE *******
C
$IBSYS
```

GND TOTAL

APPENDIX B

Characteristics of 30 Separate Storms During the Recreational Season.

Storm No.	Intensity In/Hr	Duration Hours	Total Rain Inches
1	0.133	23.03	3.07
2	0.119	14.25	1.70
3	0.103	13.28	1.37
4	0.091	12.11	1.10
5	0.081	11.05	0.90
6	0.075	10.46	0.79
7	0.070	9.97	0.70
8	0.067	9.34	0.63
9	0.065	8.77	0.57
10	0.062	8.27	0.52
11	0.060	7.83	0.47
12	0.057	7.39	0.42
13	0.054	6.92	0.37
14	0.050	6.42	0.32
15	0.047	6.10	0.29
16	0.043	5.75	0.25
17	0.039	5.38	0.21
18	0.037	5.19	0.19
19	0.034	4.84	0.16
20	0.032	4.06	0.13
21	0.031	3.77	0.12
22	0.030	3.48	0.11
23	0.029	3.18	0.09
24	0.028	2.89	0.08
25	0.026	2.60	0.07
26	0.024	2.31	0.06
27	0.021	2.02	0.04
28	0.018	1.73	0.03
29	0.013	1.44	0.02
30	0.005	1.15	0.01

Characteristics of 40 Separate Storms During the Recreational Season.

Storm No.	Intensity In/Hr	Duration Hours	Total Rain Inches
1	0.117	29.09	3.40
2	0.138	15.59	2.16
3	0.113	13.89	1.58
4	0.101	13.16	1.33
5	0.092	12.24	1.13
6	0.085	11.45	0.97
7	0.079	10.83	0.86
8	0.074	10.38	0.77
9	0.070	10.03	0.70
10	0.068	9.58	0.66
11	0.067	9.13	0.61
12	0.065	8.70	0.56
13	0.063	8.33	0.52
14	0.061	8.00	0.49
15	0.059	7.67	0.45
16	0.057	7.33	0.42
17	0.054	6.98	0.38
18	0.051	6.60	0.34
19	0.049	6.30	0.31
20	0.047	6.06	0.28
21	0.044	5.80	0.25
22	0.041	5.52	0.22
23	0.038	5.31	0.20
24	0.036	5.17	0.19
25	0.034	4.94	0.17
26	0.033	4.35	0.14
27	0.032	3.95	0.13
28	0.031	3.73	0.12
29	0.030	3.51	0.11
30	0.030	3.29	0.10
31	0.029	3.08	0.09
32	0.028	2.86	0.08
33	0.026	2.64	0.07
34	0.025	2.42	0.06
35	0.023	2.20	0.05
36	0.021	1.98	0.04
37	0.018	1.76	0.03
38	0.015	1.55	0.02
39	0.010	1.33	0.01
40	0.004	1.11	0.00

Characteristics of 50 Separate Storms During the Recreational Season.

Storm No.	Intensity In/Hr	Duration Hours	Total Rain Inches	Storm No.	Intensity In/Hr	Duration Hours	Total Rain Inches
1	0.110	32.73	3.60	26	0.044	5.83	0.26
2	0.146	16.67	2.43	27	0.042	5.60	0.23
3	0.119	14.25	1.70	28	0.039	5.38	0.21
4	0.110	13.67	1.50	29	0.038	5.26	0.20
5	0.099	13.09	1.30	30	0.036	5.15	0.19
6	0.093	12.32	1.14	31	0.034	5.00	0.17
7	0.087	11.68	1.02	32	0.033	4.53	0.15
8	0.081	11.05	0.90	33	0.032	4.06	0.13
9	0.078	10.69	0.83	34	0.032	3.88	0.12
10	0.074	10.34	0.76	35	0.031	3.71	0.12
11	0.071	10.05	0.71	36	0.030	3.53	0.11
12	0.069	9.72	0.67	37	0.030	3.36	0.10
13	0.067	9.34	0.63	38	0.029	3.18	0.09
14	0.066	9.00	0.59	39	0.028	3.01	0.09
15	0.064	8.66	0.56	40	0.028	2.84	0.08
16	0.063	8.36	0.53	41	0.027	2.66	0.07
17	0.062	8.09	0.50	42	0.025	2.49	0.06
18	0.060	7.83	0.47	43	0.024	2.31	0.06
19	0.058	7.57	0.44	44	0.023	2.14	0.05
20	0.057	7.30	0.41	45	0.021	1.96	0.04
21	0.055	7.02	0.38	46	0.019	1.79	0.03
22	0.052	6.72	0.35	47	0.016	1.61	0.03
23	0.050	6.42	0.32	48	0.013	1.44	0.02
24	0.048	6.23	0.30	49	0.009	1.26	0.01
25	0.046	6.04	0.28	50	0.003	1.09	0.00

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TREATMENT OF OVERFLOWS FROM COMBINED SEWERS

by

Newton Vaughan Colston, Jr.

ABSTRACT

Stormwater overflows from combined sewers constitute one of the major sources of quality impairment of natural watercourses in urban areas. This pollution can render the receiving watercourse unfit for human recreation or shell fish propagation for weeks at a time.

The suspended solids concentration is mainly responsible for the pollution of the stream due to its high concentration during periods of overflow. The suspended solids concentration is four to six times greater in the overflow of a combined sewer than in the average dry weather flow. This increase is due to the scouring of sludge and grit in the interceptor as well as the suspended solids carried into the interceptor by the stormwater runoff. The effect of this discharge is particularly significant due to the shock or intermittant loading it places on the stream. Periodic flushing of combined sewers during dry weather conditions would do much to alleviate the high concentration of solids in overflows.

The problem of stormwater overflows from combined

sewers is growing with the increasing urbanization and water demands in certain metropolitan areas of the United States. The complete separation of storm and sanitary lines is considered to be the ultimate solution, even though the stormwater will eventually have to be given treatment in some localities. The average cost of physical separation of storm and sanitary lines appears to be approximately $13,000 per acre.

An engineering economic analysis of remedial methods of treatment has been conducted. Attention has been given to a system of either local holding basins or newly-conceived treatment units known as gravity sediment separators. Each of these alternatives possess qualities which are desireable. The analysis of the gravity sediment separator indicates that a developmental investigation should be made. This type of research might well demonstrate the feasibility of the process and lead to a rapid realization of its potential. The cost of the gravity sediment separator including maintenance and operating costs appears to approach five percent of the cost of complete separation.