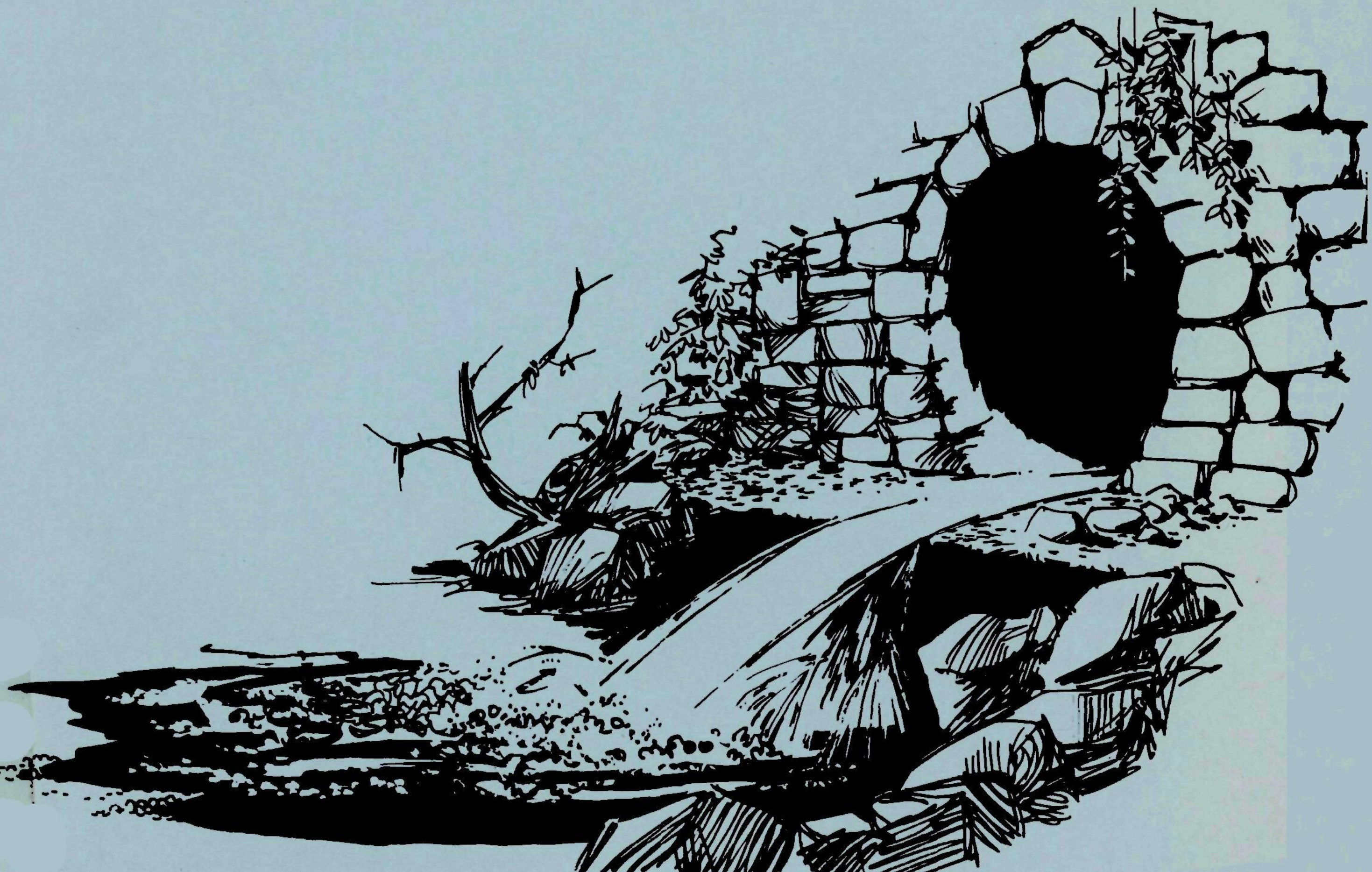
WATER POLLUTION CONTROL RESEARCH SERIES • 11024 EXF 08/70

Combined Sewer Overflow Abatement Alternatives

Washington, D.C.



ENVIRONMENTAL PROTECTION AGENCY • WATER QUALITY OFFICE

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To be continued on inside back cover...

Combined Sewer Overflow Abatement Alternatives Washington, D.C.

by

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for the
WATER QUALITY OFFICE
ENVIRONMENTAL PROTECTION AGENCY

Program No. 11024 EXF
Contract No. 14-12-403
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ABSTRACT

Objectives of the project were: 1) define the characteristics of urban runoff; 2) investigate the feasibility of high-rate filtration for treatment of combined sewer overflow; and 3) develop and evaluate alternative methods of solution.

Investigative activities included: review of pertinent reports and technical literature; field monitoring of combined sewer overflows and separated storm water discharges at three sites; laboratory studies of ultra-high-rate filtration of combined sewer overflow; hydrological analysis; and evaluation of feasible alternatives (based on conceptual designs, preliminary cost estimates, and other factors).

Reservoir Storage, Treatment at Overflow Points, Conveyance Tunnels and Mined Storage, and Sewer Separation were the approaches considered sufficiently promising for detailed evaluation. Tunnels and Mined Storage with treatment at the Blue Plains plant and at Kingman Lake after subsidence of the storm is recommended. Estimated capital costs (based on the 15-year, 24-hour storm) are $318,000,000 (ENR=1800) with annual operation and maintenance costs of $3,500,000. This approach also was preferable to the others on the basis of systematic evaluation of reliability, flexibility, public convenience and other non-quantifiable factors.

This report was submitted in fulfillment of Contract 14-12-403 (11024 EXF) between the Environmental Protection Agency-Water Quality Office and Roy F. Weston, Inc.

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SECTION I

CONCLUSIONS

1. The indicated appropriate solution to the problem of combined sewer overflows in Washington is one that provides a network of large tunnels and mined areas to convey and store overflow, with treatment of stored overflow at both the Blue Plains plant and at a facility near Kingman Lake.

2. The information developed in this study further points to a design capacity equal to the overflow from the 15-year, 24-hour storm. This design would provide effective treatment of more than 99 percent of the long-term-averaged annual volume of overflow that now contributes significantly to the pollution of the Potomac and its tributaries. Even with this design, the 25-year, 24-hour storm would result in a total 24-hour BOD loading of 70,000 pounds, well over four times the recommended maximum allowable daily loading from all BOD sources in the entire metropolitan Washington area.

3. A network of tunnels and mined storage, plus an additional facility near Kingman Lake, represents the least-cost alternative designed for the 15-year, 24-hour storm. It would cost $318,000,000 (ENR=1800) to construct and $3,500,000 per year to operate and maintain this system; in contrast, a program of complete sewer separation would cost about $610,000,000 (ENR=1800).

4. An evaluation of non-quantifiable factors such as reliability, flexibility, land requirements, public convenience, implementation, and solids removal suggested that an approach incorporating tunnels and mined storage offers more advantages than an approach based on storage reservoirs, treatment at overflow points, or sewer separation.

5. Based on the results of a field monitoring program and a detailed rainfall-runoff analysis, the overflow from combined sewers within Washington, D.C. discharges each year the following pollutant loads to the Potomac and its tributaries:

Biochemical Oxygen Demand	3,200,000 pounds
Total Phosphorus	500,000 pounds
Total Nitrogen	500,000 pounds
Suspended Solids	59,000,000 pounds

6. Overflows from combined sewers do not occur as continuous steady discharges, but rather as slug loadings. This characteristic, in combination with the long effective residence times of estuarine waters, explains the particularly serious impact combined sewer overflows have on water quality. For example, the 24-hour BOD load expected in the 616 million gallon overflow from the 2-year, 24-hour storm (for the existing sewer system) is 160,000 pounds, nearly ten times the recommended maximum allowable daily loading from all BOD sources in the entire metropolitan Washington area.

7. Any abatement alternative utilizing storage or in-line treatment will remove practically all of the 59,000,000 pounds of suspended solids now discharged yearly as overflow to the Potomac. The resulting quantity of sludge (30,000 tons per year on a dry

basis) is greater than the quantity of sludge generated at the Blue Plains plant. Besides this, the generation of this sludge is concentrated in the 50 to 60 overflows per year, rather than coming on a uniform, daily basis as at Blue Plains. In light of the general problems of solid waste disposal in the Washington area, a high level of sophistication and a high degree of planning will be required to handle the combined sewer sludge as is employed to handle the sewage treatment plant sludge. The handling and disposal of this sludge will probably cost $750,000 per year.

8. A limited six-month field monitoring program of combined sewer overflow from two combined sewer drainage basins and separated storm sewer discharge from one separated sewer drainage basin during storm conditions suggested the following mean values for certain significant waste constituents:

	Combined Sewer Flow	Separated Storm Sewer Flow
Biochemical Oxygen Demand	71 mg/L	19 mg/L
Settleable Solids	229 mg/L	687 mg/L
Fecal Coliform	2,400,000/100 ml	310,000/100 ml
Total Phosphate	3.0 mg/L	1.3 mg/L

Values of BOD as high as 470 mg/L in combined sewer flow and as high as 90 mg/L in separated storm sewer flow were detected during the monitoring program.

9. With reference to the laboratory investigations of ultra-high-rate filtration, the following conclusions are drawn:

   a. Tri-media filtration at rates less than 10 gpm/sq.ft. provides a satisfactory effluent.

   b. Ultra-high-rate filtration (15 gallons or more per minute per square foot) of combined sewer overflows is not technically feasible for upflow filtration through a garnet bed, because of poor effluent quality.

   c. The fiberglass filter operated successfully at filtration rates of 15-30 gpm/sq.ft., with removals of 90 percent suspended solids and 70 percent non-soluble BOD.

   d. The addition of flocculant aids did not improve the removal characteristics of the fiberglass filter.

   e. Soluble BOD_5 was not significantly reduced by the addition of low dosages of activated sludge to the filter influent.

   f. The economic feasibility of the fiberglass medium will depend upon the extension of the useful life of the medium and the improvement of backwash techniques.

10. The selection of the design storm event is properly made only in the context of basin-wide quality water management. Many factors beyond the scope of this study, such as the impact of wastewater discharges outside the District of Columbia, low-flow augmentation, etc., have too great an influence to be ignored.

SECTION II

RECOMMENDATIONS

Based on the findings and conclusions developed in the course of this combined sewer overflow study, the following actions are recommended.

1. Discontinue the current sewer separation program and develop pollution abatement programs for both the combined and separated sewer areas if the pollutional characteristics of storm water determined in the current study are confirmed in other areas of the District.

2. Proceed with engineering and construction of conveyance tunnels, mined storage, and a treatment facility near Kingman Lake, somewhat like the facilities described in reference (6), in conjunction with the following:

   a. Conduct the sub-surface investigations necessary to confirm the assumed bedrock characteristics.

   b. Initiate a long-term monitoring program using monitoring equipment developed for the current project to obtain long term comparative and detailed information to define long-term impact of pollution loads on required treatment facilities and receiving waters.

   c. Confirm the selection of the design storm frequency in the context of basin-wide water quality management.

3. Implement additional EPA-WQO sponsored laboratory and full-scale development studies on the use of fiberglass as a filter medium with the following objectives:

   a. To develop an improved filter bed design with respect to density gradation, depth, and combination with granular media.

   b. To refine the backwashing techniques, including underdrain design, stagewise removal of backwash effluent, and regeneration of the fiberglass filter medium.

   c. To optimize design parameters.

4. Re-evaluate, under EPA-WQO sponsorship, the validity and accuracy of the present conventional approach to the determination of sewer flow rates (combined-sewer areas) during the surge period associated with each storm.

SECTION III

INTRODUCTION

General

Recent efforts stimulated by the Federal Water Pollution Control Act of 1956 (amended in 1961, 1965, 1966, and 1970) have brought to light the significance of combined sewer overflows as a source of pollution.

Most United States cities today are served both by combined sewers and by separate sanitary and storm sewer systems. As determined from a 1967 survey sponsored by the Water Quality Office of the Environmental Protection Agency, approximately 29 percent of the total sewered population of the United States is served by combined sewer systems. Approximately three percent of the total annual flow of sewage and as much as 95 percent of the sewage produced during periods of rainfall is carried with combined sewer overflows to the surface waters.

The District of Columbia follows this pattern, with an area of approximately 20 square miles (one third of the total area of the District) being served by combined sewers. The hydraulic capacity of the system is often exceeded during periods of precipitation, and raw sewage mixed with surface runoff is discharged to the watercourses of the District.

The Potomac Estuary is polluted and continues to experience problems with low dissolved oxygen, excessive algal growths, sediments, high concentrations of fecal bacteria and repulsive floating matter. All of these problems, except sediments, are complicated by combined sewers. Although combined sewer overflow adds to the sediment load, the primary source of sediment is the heavy silt load included in the runoff from areas with significant agricultural and construction activity. In addition to the effects of combined overflow on the Potomac River, overflows into Rock Creek detract from the natural and recreational features of this small, scenic stream flowing through the District.

In 1957, the District prepared a separation schedule for conversion of all combined sewers into separate storm and sanitary systems. This conversion, however, would not be completed until after the year 2000. The cost of this program is extremely high, estimated at $610 million (ENR=1800), and budgeting and other problems have delayed progress. In the meantime, the overflows from unseparated sewers will continue to contribute to pollution problems in the Potomac Estuary.

Project Objectives and Scope

The primary objectives stipulated for this study were to: define the problem of combined sewer overflows within the District of Columbia; investigate the feasibility of high-rate filtration (greater than 15 gallons/square foot/minute) for treatment of combined sewer overflows; and study alternative methods of solution to the problem

The general scope of the project is outlined below as taken from the Statement of Work of Contract No. 14-12-403 (between the Federal Government and Roy F. Weston, Inc.) and as stated in the latter's proposal "Study of Pollution Abatement from Combined Sewer Systems" dated 21 November 1967:

1. Develop a quantitative definition of the combined sewer pollution problem by review of reports, studies, and data concerning water quality and rainfall intensities, surface runoff, frequency of occurrence, and hydrologic data pertinent to assessing methods for controlling combined sewer system pollution.

2. Collect data and subsequent water quality analyses of continuous rainfall and sewer flow measurements made at three selected sites.

3. Evaluate by laboratory research the possibility of removing flocculated solids and associated BOD by high-rate filtration of combined sewer overflows.

4. Define the technical and economic feasibility of flocculation and high-rate filtration as a means of treating combined sewer overflows.

5. Investigate other selected practical solutions to provide a meaningful evaluation of the relative merits of high-rate filtration and to provide a comprehensive study of the combined sewer problem of Washington, D.C.

6. Present generalized capital cost and annual cost estimates for alternatives which appear to be technically feasible for the Washington, D.C. system.

7. Develop technical, economic, and operational comparisons of practical alternatives.

8. Further develop the general formulation and application of a methodology of analysis for determining feasible solutions for eliminating combined sewer pollution from municipal systems.

SECTION IV

PROBLEM DEFINITION

Description of Study Area

General Conditions

The District of Columbia lies largely along the "Y" formed by the junction of the Anacostia and Potomac Rivers, extending eleven miles along the northeasterly side of the Potomac and straddling the Anacostia for six miles. Figure 1 illustrates the general area. The average flow of the Potomac River is 13,300 cfs, whereas the Anacostia averages 150 cfs. The total land area of the District is approximately 61 square miles, and the water area within the District is some seven square miles. The District is the center of a metropolitan area which includes parts of Maryland and extends across the Potomac River into Virginia.

The central and older portion, comprising approximately one-third the total land area of the District, is the principal area of interest for the present investigation. This area rises gradually from the confluence of the Potomac and Anacostia Rivers to the encircling hills, and was originally drained by several sizable streams discharging to these two rivers.

The metropolitan Washington area has experienced a phenomenal growth in population since 1930. Although the population of the area surrounding the District continues to increase significantly each year, the District itself may have reached its peak population, for some time, in 1950. Out-migration since then has outweighed population increases from in-migration and the birth-less-death increases. It is uncertain whether or not population will continue to decrease or increase; nevertheless, existing and planned land use will probably impede any drastic increases in population. The population of the District from 1900 to 1970 is given in Table 1. The possible future populations are indicated in a February, 1957 Board of Engineers report to the District on improvements to the sewerage system (1); the report projected a population of 1,125,000 in the year 2000. However, the same report projected 1,010,000 population in 1970.

Table 1

Population of the District of Columbia

Year	Population	Year	Population
1900	278,718	1940	663,091
1910	331,069	1950	802,178
1920	437,571	1960	763,956
1930	486,869	1970	746,169[1]

[1]Preliminary value from 1970 Census.

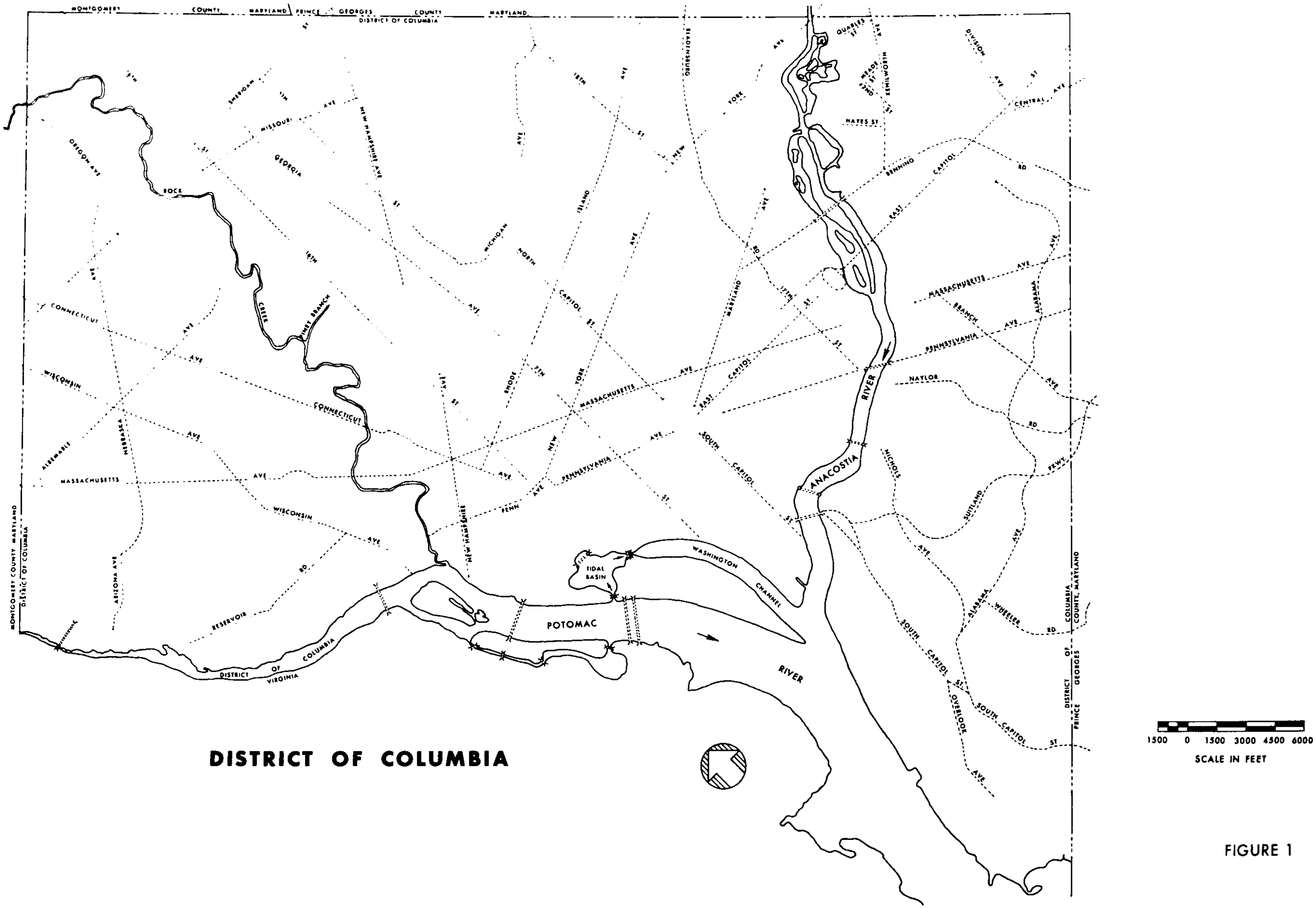

DISTRICT OF COLUMBIA
MONTGOMERY COUNTY MARYLAND
PRINCE GEORGES COUNTY MARYLAND
DISTRICT OF COLUMBIA
POTOMAC
RIVER
ANACOSTIA
RIVER
WASHINGTON CHANNEL
TIDAL BASIN
ROCK CREEK
CONNECTICUT AVE
MASSACHUSETTS AVE
WISCONSIN AVE
PENNSYLVANIA AVE
NEW HAMPSHIRE AVE
GEORGIA AVE
NEW YORK AVE
SOUTH CAPITOL ST
EAST CAPITOL ST
NORTH CAPITOL ST
RHODE ISLAND AVE
BENNING RD
NAYLOR RD
ALABAMA AVE
RESERVOIR RD
DISTRICT OF COLUMBIA
VIRGINIA
1500 0 1500 3000 4500 6000
SCALE IN FEET

FIGURE 1

District activities largely concern government offices, public buildings, and related activites; there are no large industries within its borders. The many large government buildings in the District cover extensive areas and generate heavy concentrations of employees. Considerable areas of the District are occupied by parks, recreational areas, institutional grounds, and cemeteries; for example, in 1966, approximately 7,830 acres (20 percent of the total land area of the District) were occupied by parks and playgrounds.

The District had a very low ratio of runoff to rainfall when the original combined sewer system was installed, primarily because of the numerous parks and other unpaved areas. However, as the population density increased, as more and more buildings were erected, as street pavings grew wider and wider, and as parking areas were paved, the rainfall-runoff coefficient has increased tremendously. This has contributed to the present inadequacy of all storm water facilities in the central portion of the District.

The land surface in the District varies in elevation from the low areas (elevations of about 4 feet, USGS Datum) adjacent to the Potomac River and in East and West Potomac Park, along the Washington Channel, and along the Anacostia River, to elevations as high as 410 feet near the intersection of Wisconsin and Nebraska Avenues NW. The area west of 16th Street NW, in Rock Creek Park, is quite rugged, with a number of places where the land rises to elevations of 300 feet and more.

Southeast of the Anacostia River and east of the Potomac River in the southern park of the District, the land surface is low and flat along the rivers, as in the area of Bolling Air Force Base and the Anacostia Naval Air Station. The remainder of this southeast part of the District is hilly, reaching elevations of 200-300 feet in several areas.

Except for the low-lying lands along the rivers, the terrain of the District favors the design of gravity sewers. The District generally drains well because of the good slopes that can be obtained. There are a few bowl-like depressions surrounded by higher territory; however, except for the area in the vicinity of 5th and Ingraham Streets NW, none is subject to severe flooding in times of heavy rains. Some basement flooding has been experienced from backup of storm water in surcharged sewers. This problem is a function of the sewer system itself and will be discussed in greater detail in a subsequent section. Surface water draining from the land areas of the District is carried off by the Potomac River, the Anacostia River, Rock Creek, Foundry Branch, Oxon Run, and their minor tributaries.

Geological Features

Information on the geologic and other natural conditions is essential to a complete evaluation of the various possible approaches to the abatement of pollution caused by the inadequacies of combined sewers, including the feasibility of tunneling beneath the City of Washington to provide for storage of combined sewer overflows.

Data were obtained from published geologic reports, from engineering reports prepared under the auspices of the Metropolitan Area Rapid Transit Authority, and from verbal communication with personnel of the Authority. The published geologic data are fairly broad in coverage, yet sufficiently detailed to provide information helpful in forming conclusions in regard to the feasibility of tunneling. The engineering reports are extremely detailed and provide an abundance of data concerning the soils, bedrock, and rock mechanics in certain restricted areas developed for the purpose of underground tunneling for rapid transit. Although somewhat restricted as to area-wide application, the data included in these reports provide enough coverage to be representative of the general Washington area.

The District of Columbia lies within portions of two physiographic provinces; the southeastern portion is located within the Coastal Plain province, which consists of relatively flat-lying sediments overlying deep bedrock, and the northwestern portion is in the Piedmont province, which in general is characterized by a thin layer of overburden covering crystalline bedrock. The Fall Line separating the two provinces extends roughly southwest from Blair Park in the northeast through Farragut Square and on toward the Pentagon. Figure 2 presents the generalized geology of the District of Columbia.

Previous sub-surface investigations in the District have resulted in grouping the materials into five major categories: bedrock, Cretaceous sediments, Pleistocene terrace deposits, recent river alluvium, and drainage channels and man-made fills. These major categories of materials in various parts of the District are found in the following five vertical profiles:

1. Recent alluvium over bedrock or Pleistocene terrace deposits.

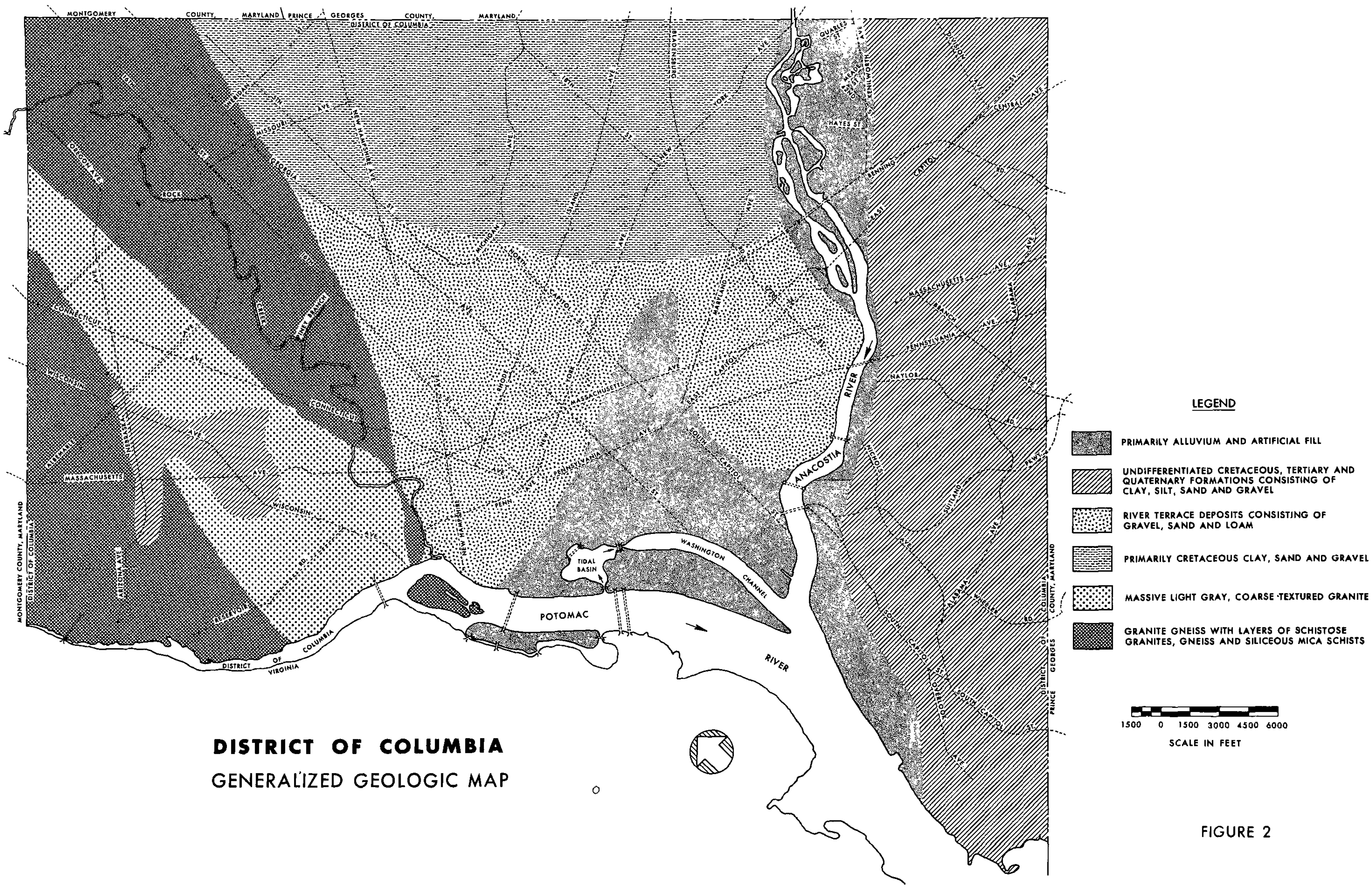
2. Overburden of Pleistocene terrace and Cretaceous coastal plain soils above deep bedrock.

3. Comparatively thick cover of Pleistocene terrace and Cretaceous coastal plain soils above deep bedrock.

4. Thin to moderately thick cover of Cretaceous coastal plain materials above decomposed rock and bedrock.

5. Relatively thin cover of man-made fill and decomposed rock over bedrock at shallow to moderate depths.

Geological and related natural conditions in the Washington, D.C. area and the implications for tunneling are described in more detail in Appendix A.



DISTRICT OF COLUMBIA

GENERALIZED GEOLOGIC MAP

FIGURE 2

Description of Present D.C. Sewer System

Historical Development

Several studies have been made of the District of Columbia sewerage system and sewage treatment facilities. A list of pertinent reports and related technical articles, which were reviewed to obtain detailed information relevant to the present pollution problem, is presented as Appendix B.

The installation of culverts and drains in the District began as early as 1810. In 1840, the first piped water supply to a few homes began; it was followed by a few sewers discharging to the nearby culverts. This was the beginning of the combined sewer system in Washington, D.C. With the introduction of the Potomac River water supply in 1859, the extent of the combined sewer system discharging into nearby watercourses increased very rapidly, and by 1874 there were approximately 80 miles of sewers. The sewers drained into the principal watercourses and thence to the Potomac River. The discharges to the watercourse from the combined sewers resulted in pollution of the streams and created unsanitary conditions along the waterfront.

Upon the recommendations of the Hering-Grey-Sterns Report, a comprehensive system of sewer construction began in 1890. Interceptor sewers were then installed to collect all of the dry-weather flows and some surface-water runoff from the combined sewers for conveyance to an outfall sewer for ultimate discharge into the deep water of the Potomac River.

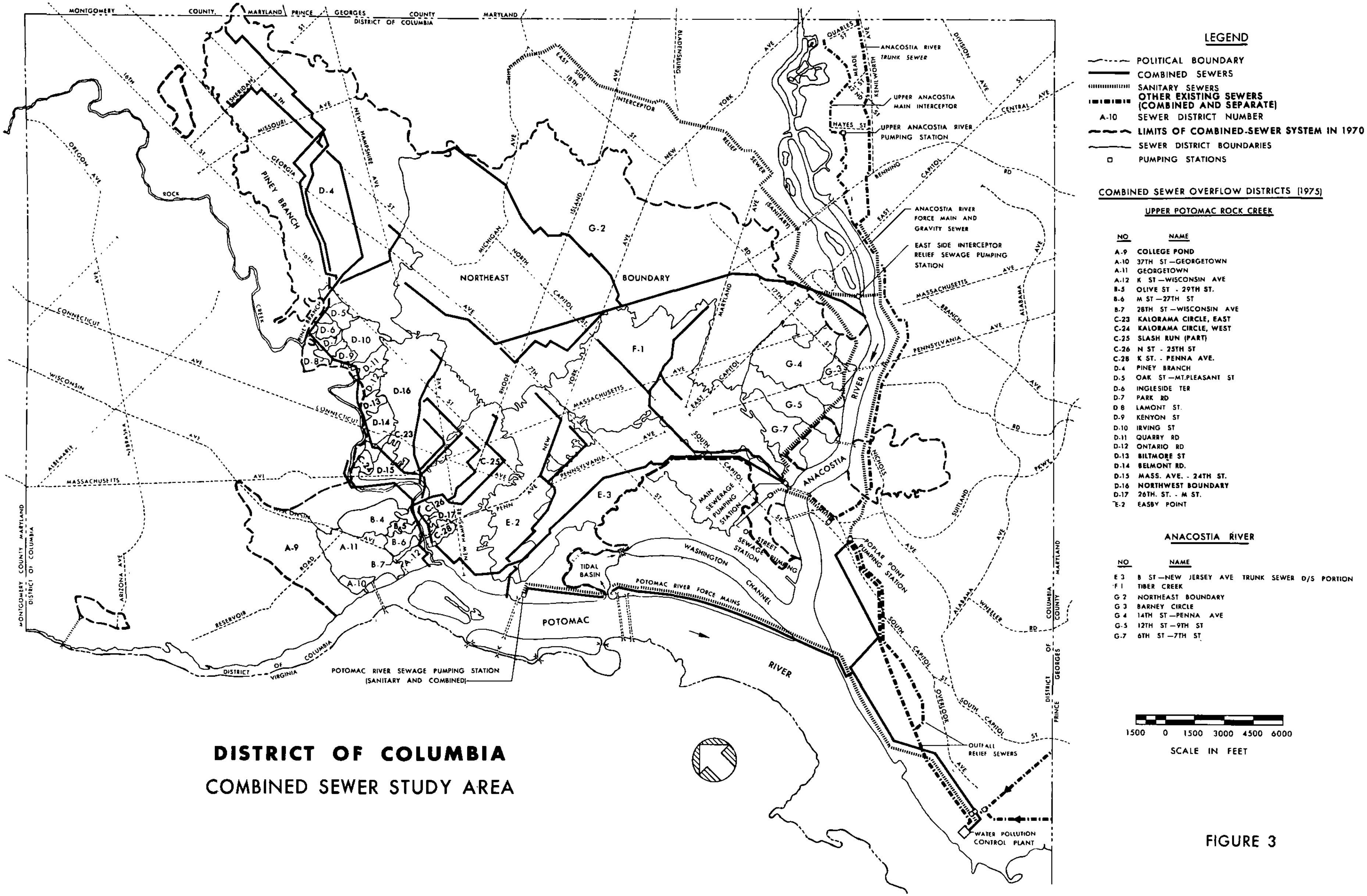
Through 1890, the sewers constructed in the District were combined sewers. Subsequently, however, the approach generally adopted was that combined sewers were tolerable in the areas already sewered but that any additions (new areas of the District) must be separated storm and sanitary sewers. By 1929, the construction of separated sewer systems in the new, outlying areas was common practice.

Existing Sewer System

The present District sewer system is designed to serve an area of approximately 725 square miles, comprising the entire District of Columbia and adjoining areas in Maryland and Virginia. The U. S. Congress authorized the District to serve the upstream adjacent areas of the two states so that the streams flowing through the District could be protected from pollution.

Approximately 12,000 acres of the sewage-producing areas of the District are served by the existing combined sewer system Substantially all of this acreage is in the central part of the District. The remainder of the District and substantially all of the adjoining areas of Maryland and Virginia are served by separated sewers. The existing sewer system is shown in Figure 3. Table 2 lists the acreage of the combined sewer districts scheduled for separation after 1975.

The District of Columbia has been divided into some 93 sewer drainage districts, arranged in 11 groups under the names of the principal sewers to which the districts are tributary. A tabulation of the statistics pertinent to these districts is presented in Chapter 6 of the investigative report of June 1955 (2). The existing and required capacities of the intercepting sewers are described in Chapter 16 of the same report.



DISTRICT OF COLUMBIA

COMBINED SEWER STUDY AREA

FIGURE 3

Table 2

Combined Sewer Districts
Scheduled for Separation after 1975

No.	Name	Net Area (acres)	Drainage Basin
A-10	37th. St.-Georgetown	19	Upper Potomac
A-11	Georgetown	183	Upper Potomac
A-12	K St. - Wisconsin Ave.	32	Upper Potomac
B-3	Q Street	8	Rock Creek
B-4	Q Street - 31st. St.	105	Rock Creek
B-5	Olive Street 29th St.	14	Rock Creek
B-6	M St. 27th. St.	35	Rock Creek
B-7	28th. St. - Wisconsin Ave.	13	Rock Creek
C-23	Kalorama Circle, East	8	Rock Creek
C-24	Kalorama Circle, West	14	Rock Creek
C-25	Slash Run	417	Rock Creek
C-26	N St. 25th. St.	12	Rock Creek
C-28	K St. - Penna. Ave.	20	Rock Creek
C-29	I St. - 22nd. St.	95	Rock Creek
D-4	Piney Branch	2,175	Rock Creek
D-5	Oak St. - Mt. Pleasant	26	Rock Creek
D-6	Ingleside Ter.	18	Rock Creek
D-7	Park Road	17	Rock Creek
D-8	Lamont Street	17	Rock Creek
D-9	Kenyon Street	17	Rock Creek
D-10	Irving Street	76	Rock Creek
D-11	Quarry Road	36	Rock Creek
D-12	Ontario Road	22	Rock Creek
D-13	Biltmore St.	21	Rock Creek
D-14	Belmont Road	44	Rock Creek
D-15	Mass. Ave. 24th Street	70	Rock Creek
D-16	Northwest Boundary	534	Rock Creek
D-17	26th. St. - M St.	6	Rock Creek
E-2	Easby Point	518	Upper Potomac
E-3	Trunk Sewer	375	Anacostia
F-1	Tiber Creek	1,000	Anacostia
G-2	Northeast Boundary	3,728	Anacostia
G-3	Barney Circle	41	Anacostia
G-4	14th. St. - Penna. Ave.	252	Anacostia
G-5	12th. St. - 9th. St.	151	Anacostia
G-7	6th. St. - 7th. St.	121	Anacostia

Source: Board of Engineers (Greeley, S.A. et al), "Report to District of Columbia Department of Sanitary Engineering on Improvements to Sewerage Systems," February 1957.

There are three principal interceptors which serve the major portion of the combined sewer area: Upper Potomac Interceptor, Rock Creek Main Interceptor, and East Side Interceptor. All three serve large areas of separated systems along the upstream reaches and essentially are separate trunk sewers in these areas. They become interceptors in the true sense in the downstream reaches in the combined sewer areas, and convey this flow through an outfall sewer to the sewage treatment plant at Blue Plains.

The District sewer system is generally in good physical condition; however, many of the combined and separated sewers and certain intercepting sewers do not have adequate capacity. As development in the combined-sewer area of the District (39 percent of the land area in 1957) increased, it was noted that the hydraulic capacity of the system was exceeded during periods of precipitation. In order to prevent local flooding and the spilling of sanitary sewage onto the surface of the ground, overflow structures and interceptor chambers were built to relieve the excessive sewer flow by discharging it directly into the natural watercourses.

In the past, the District has conducted detailed studies to evaluate the contribution to the problem of pollution at these overflow structures. Nine automatic, continuous-recording, depth-of-flow gauges were installed at various critical storm water points to obtain data on the frequency and duration of overflows. It was found that some of these overflow structures discharge as many as 40 to 50 times a year, while others discharge only 2 to 3 times a year, depending on the drainage area characteristics and the available interceptor capacities.

One of the most significant and comprehensive studies was the 1957 investigation of the Board of Engineers. The Board of Engineers study indicated that many of the combined and separate sewers were inadequate by generally accepted design standards. Considerable surcharging occurred in the combined sewers, and there were excessive overflows of mixed sewage and storm water to the streams, even including some overflows during dry weather. It was evident that certain interceptors needed relief.

The situation relative to the inadequate capacities of various trunk sewers is summarized in Table 3. This gives a picture of the magnitude of the problem. The required capacities were based on conditions of development estimated for the year 2000.

Table 3

Deficiency in Sewer Capacities

Description of Sewer	Percent of Total Length Deficient in Capacity	Present Capacity as Percent of Required Capacity
Upper Potomac Interceptor	98	40
Rock Creek Main Interceptor	77	54
East Side Interceptor	69	71
Piney Branch Trunk Sewer	100	56
Northwest Boundary Trunk Sewer	92	57
Slash Run Trunk Sewer	85	53
B St.-New Jersey Avenue Trunk Sewer	98	49
Easby Point Trunk Sewer	93	47
Tiber Creek Trunk Sewer	70	50
Northeast Boundary Sewer	86	54

To evaluate the problem of pollution of streams attributable to the overflows, the Board of Engineers collected data on the frequency and duration of overflows at critical points for a period of one year. It was found that the average number of overflows per month at the various locations ranged from 5 to 16.8 during the summer months, and from 3.8 to 4.7 during the winter months. The average duration of overflows per month ranged from 24 to 110 hours during the summer months and from 26 to 38 hours during the winter months.

Sewer Separation Programs

Studies by Eddy, Gregory and Greeley in 1933 and by Sherman and Horner in 1935 recommended separation of the combined sewer system in several smaller areas of the city. However, it was not until the 1957 Board of Engineers study that a unified program for the elimination of combined sewers was presented.

The Board of Engineers outlined several alternative plans for abating or eliminating the pollution that results from combined sewer overflows and from inadequate capacities of combined sewers and interceptors. Two of the alternatives deemed to have the greatest merit were those designated as Project A and Project C. Both of these alternatives require the conversion of some combined sewers into separate sewers.

Project A was defined to convert all combined sewer areas to the separated system and to provide interceptors to carry all sewage to the water pollution control plant. At the completion of Project A, there would no longer be any overflows of untreated sewage into the river. However, no appreciable reduction of pollution in the river would result until substantially all of the work outlined in Project A was completed. Minimum construction time required to complete Project A was optimistically estimated at twenty years, and the estimated cost for this project was 238 million dollars at 1957 prices.

Under Project C, only 10 percent (approximately) of the combined sewer area was to be separated initially, but new interceptors would be provided to carry additional quantities of mixed sewage and storm water flows to the pollution control plant. Wet-weather discharges to the river would occur even with these enlarged interceptors, but only during heavier rainfall and on a scale substantially below present volumes. It was estimated that Project C could be completed in ten years or less at an estimated cost of 72 million dollars at 1957 prices. Between July 1957 and July 1970 the Engineering News Record Construction Cost Index has increased from 725 to 1414, an increase of 94 percent; thus, in terms of 1970 dollars, Project C costs would be approximately 140 million dollars.

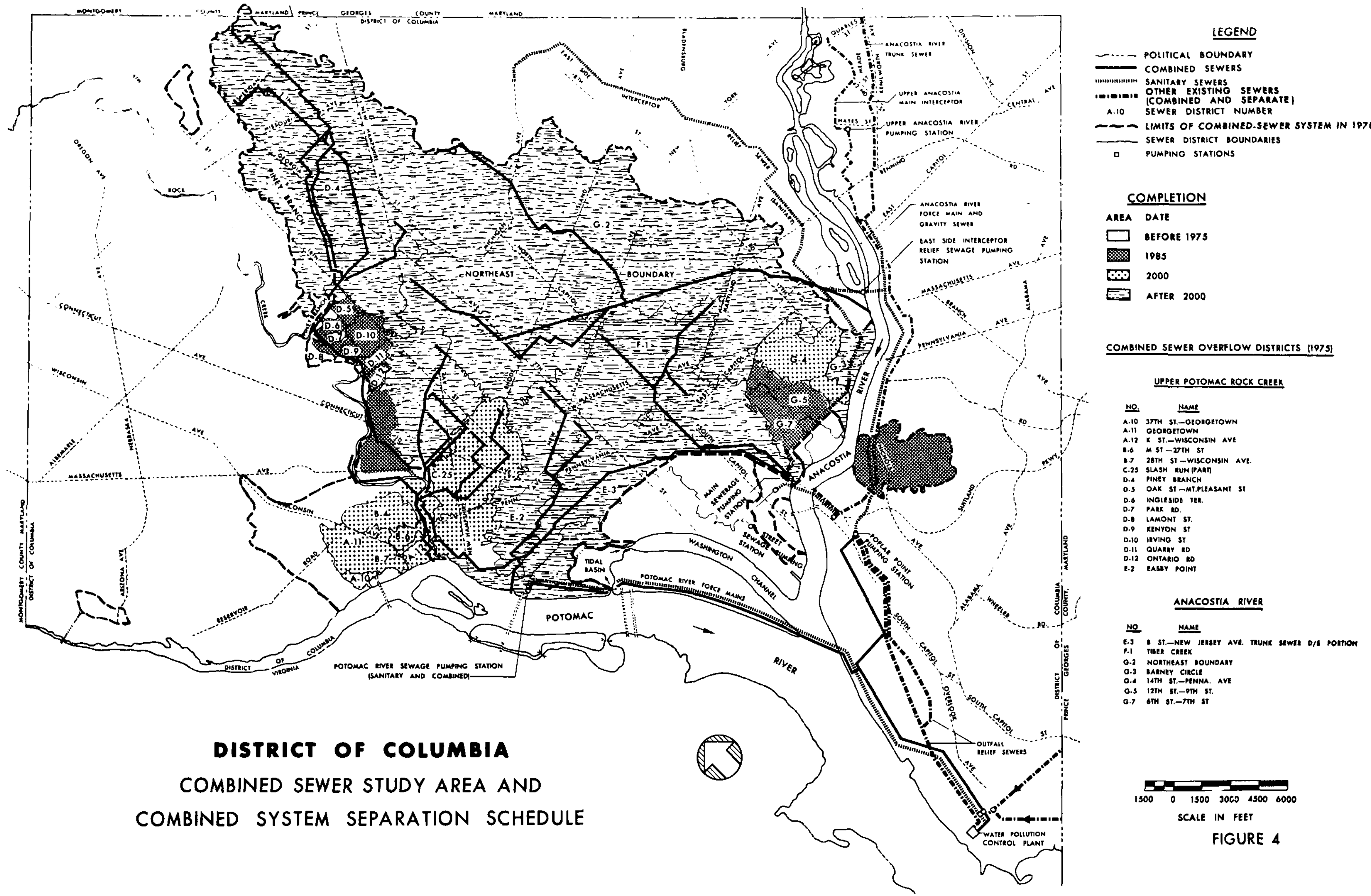
The Board had recommended Project C over other alternatives because it was the least expensive and could be completed in about ten years, and because the benefits (in terms of reduction of pollution) could be realized as the work progressed.

The District of Columbia approved the Board's recommendations and later modified Project C, the principal change being to design a two-level conduit for the Upper Potomac Interceptor Relief Sewer and two force mains from the Potomac River Sewage Pumping Station to carry the flow to the sewage treatment plant. One reason for conveying the flow in two conduits was to prevent a complete breakdown of the system in case of any problems. Another factor was the advantage, from the long-range standpoint, of being able to separate the flow of sewage from the combined flow of sewage and storm water. By this modification, all sewage originating upstream of Georgetown (in Maryland, Virginia, and the District) would be handled by an interceptor serving only separated system areas. No overflow structures would be provided in this interceptor, and all the facilities could be so designed that most of the objectionable overflows would be eliminated at an early date.

As the sewers in the older section of the city are gradually converted to a separated system under the Storm Sewer Separation Program the two-line interceptor would have the built-in capacity to carry the entire flow of sanitary sewage to the treatment plant. Eventually, Project C would achieve the same result as Project A but at reduced cost. When the work outlined under modified Project C is completed, there will be no overflows to the streams until the average sewage flow is diluted at least five times by storm water.

Work outlined under the separation program has been initiated, and conversion, until recently, has kept pace with the schedule. Budgeting problems have delayed the progress of this program. Figure 4 shows the District's present combined sewer separation schedule. A large segment of the central section is not scheduled for separation until after the year 2000, but it is possible that areas listed to be separated after 2000 could be acted on sooner, depending upon other improvement programs and the availability of funds.

Nevertheless, the problem of combined sewer overflows still exists. In 1961, there were approximately 86 permanent built-in storm water overflow structures (some combined sewer districts have more than one overflow structure) that discharge to the watercourses. A few of these overflow structures have been plugged since then, but about 60 are still operative, and a mixture of combined sewer overflows and storm water discharges through them with each rainfall.



DISTRICT OF COLUMBIA

COMBINED SEWER STUDY AREA AND COMBINED SYSTEM SEPARATION SCHEDULE

FIGURE 4

D.C. Sewage Treatment Facilities

The first primary treatment plant to serve the District of Columbia was put into operation in 1938, and secondary treatment facilities were added by 1959. The present treatment facilities consist of: a pumping station; screening and grit removal units; primary sedimentation, aeration, and secondary sedimentation tanks; chlorination facilities; effluent outfall into the river; and sludge heating, thickening, digesting, elutriating, and vacuum filtration facilities. The secondary (biological) treatment process is modified aeration.

The most recent improvements and additions to the plant include: secondary treatment by modified aeration and sedimentation; new thickeners and heat exchangers; a doubling of the elutriation tanks; mixing of the contents of the sludge digestion tanks; and a series of improvements to the sludge-gas piping.

The treatment plant has the capacity to handle peak flows up to 300 mgd and in the present setup, flows in excess of 300 mgd would bypass the treatment plant. In reviewing the performance of the plant (3), operating data of the past three years (fiscal 1966-1968) were utilized. It was observed that the plant was providing 70 to 80 percent reduction of BOD and suspended solids, with removals lowest for the year 1968. For fiscal year 1969, the average flow to the treatment plant was 248 mgd. Table 4 lists the 1969 average daily pollutant loadings discharged in the effluent from the Blue Plains Plant. Also listed, for purposes of comparison, are the maximum allowable loadings recommended by the conferees of the May 8, 1969 conference on pollution of the Potomac River (4).

Table 4

Daily Pollution Loads from Blue Plains Plant

	BOD lbs/day	SS lbs/day	TP lbs/day	TN lbs/day
Average 1969 Effluent	124,000	139,000	87,000	75,000
Recommended Maximum Allowable Discharge	12,700	--	560	6,130

The District has made concerted efforts to maintain plant efficiency under increased flow conditions and has had to develop specialized operating procedures and techniques. However, major modifications and additions to the plant are required to meet the recommended waste loadings. The District has prepared a plan to complete these improvements by the year 1974. The requirements for advanced waste treatment and the limited land area available restrict the ultimate capacity of the plant to 309 million gallons per day. The plant will be constructed to handle this 309 mgd capacity by 1974. However, in addition to the 309 mgd flow, the plant will be constructed to provide complete treatment for an incremental increase of 289 mgd for a period not to exceed 400 hours per year. For short durations, the plant will also be able to provide partial treatment of flows 2 to 5 times the 309 mgd flow.

Although the sewage flow from the District is not expected to increase significantly in the future, the sewage flow from the suburbs served by the plant is increasing at such a rate that the 309 mgd capacity will be exceeded in 1977. Therefore, the responsible agencies outside of the District have scheduled the construction of additional regional plants.

Peak Rate and Volume of Overflow

General Basis of Data

The significance of the combined sewer problem is measured by the pollutant loadings discharged in the overflow to the District's watercourses. The selection of the capacities of facilities required in alternative solutions to the problem must rest, in part, on parameters such as volume of overflow and peak flow rate. Particular values of each of these parameters vary with each storm, and it is necessary to perform an analysis of rainfall-runoff relationships to define the frequency and range of values. This type of information would provide a basis for evaluating alternatives.

To perform this analysis, extensive data are required concerning rainfall intensities and frequencies, characteristics of each drainage basin and the existing sewer system, average dry-weather flow, etc. Most of the required data have been developed in previous, unrelated investigations. To avoid the duplication of previous efforts and to assure that the best available data were used, the analysis was coordinated with District officials. In this analysis, only the areas scheduled for separation after the year 1975 were considered in the determination of the magnitude and intensity of the combined sewer overflow problem.

For the purpose of this investigation, an attempt had been made to use the existing design storm basis for the Washington, D.C. sewer system to evaluate the peak flow rates and volume of runoff. This design storm, one which occurs once every fifteen years with a duration of one hour, provides a basis for calculating peak flow rates, but does not provide a sound basis for calculating the volume of runoff. For example, the amount of rainfall associated with the 15-year, one-hour storm is 2.66 inches, whereas the maximum daily rainfall occurring once in fifteen years for a 24-hour duration is 5.5 inches. It is obvious then that all further considerations of volume of overflow should be based on a 24-hour duration. The peak flow rates, however, should be based on storms of shorter duration, since the mean intensity is higher.

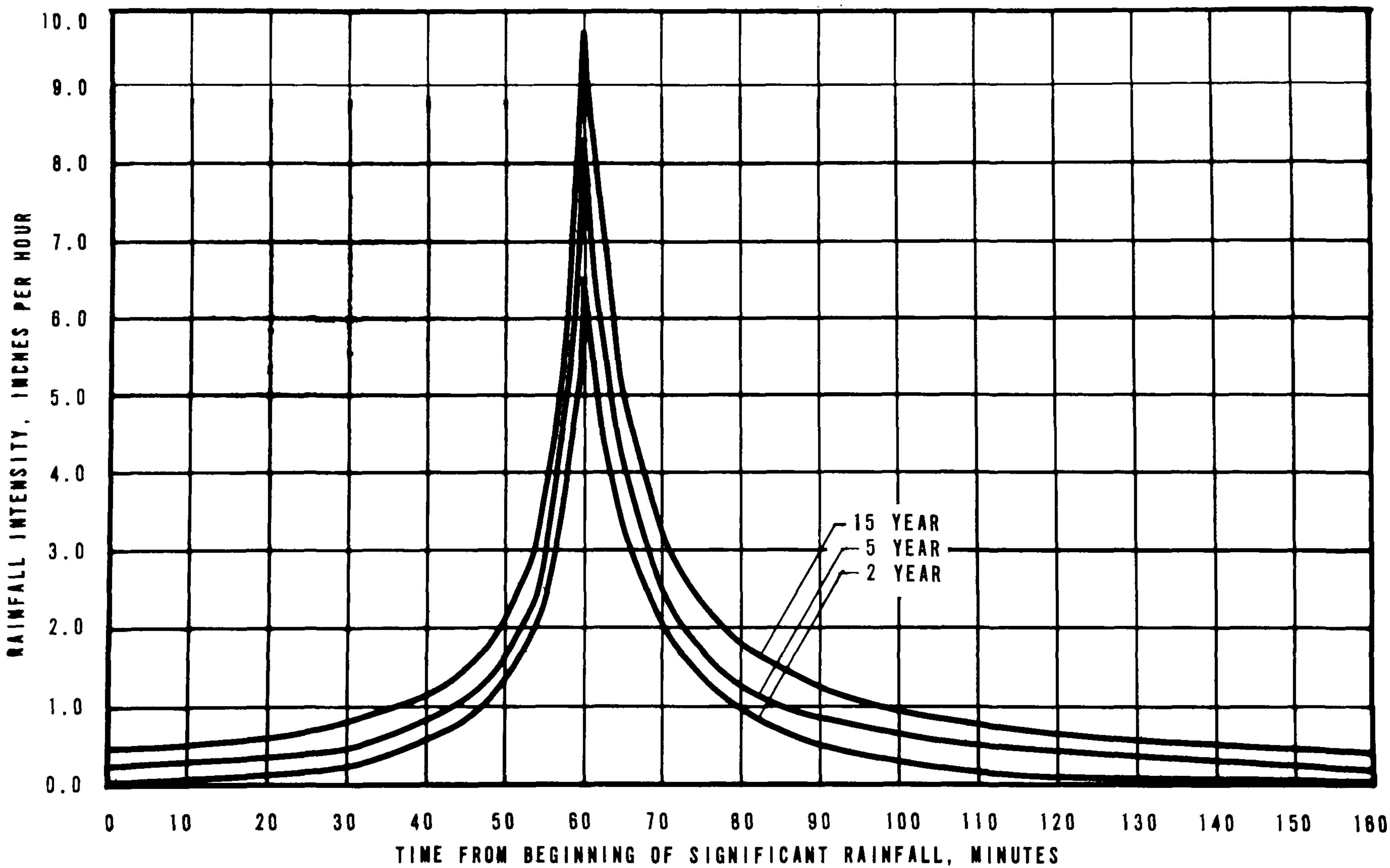
Rainfall-Runoff Analysis

The hydrologic analysis of the current study involved two procedures. The first procedure, the Rational Method, is limited to estimation of peak flow rates; it cannot provide the second necessary component of the analyses, i.e., volume of runoff. In the second procedure, for determination of the volume of overflow, several different approaches were examined. Some methods seemed to provide reliable results, but were too elaborate, requiring lengthy and involved data collection and analysis, to be completed within the time and budget limitations of this study. Some simpler approaches such as the unit hydrography method seemed to provide unreliable results. After a thorough examination, an approach was selected similar to the approach used in another combined sewer study (5) in Washington, D.C. This approach incorporates two methods. In the first method, the volume of surface runoff of rainfall into combined sewers was calculated by subtracting the losses due to infiltration into pervious soils and retention in surface depressions from the total amount of precipitation. An explanation of the methodology used for the calculations of runoff is given in detail in Appendix C as "The Hyetograph Method Volume of Overflow". In general, the methodology includes:

1. Construction of hyetographs for storms of various frequencies (Figure 5).

FIGURE 5

HYETOGRAPH FOR VARIOUS RAINFALL FREQUENCIES



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2. Determination of the relative percentages of pervious and impervious areas of each sewer district.

3. Determination of the relationship of infiltration capacity to time (Figure 6).

4. Construction of "accumulated mass rainfall curves" and "actual accumulated mass infiltration curves" and subsequent determination of rainfall in excess of infiltration (Figures 7 and 8).

5. Determination of the volume of rainfall retained in surface depressions.

6. Use of an area-depth correction factor to account for application of point rainfall data to the drainage basin.

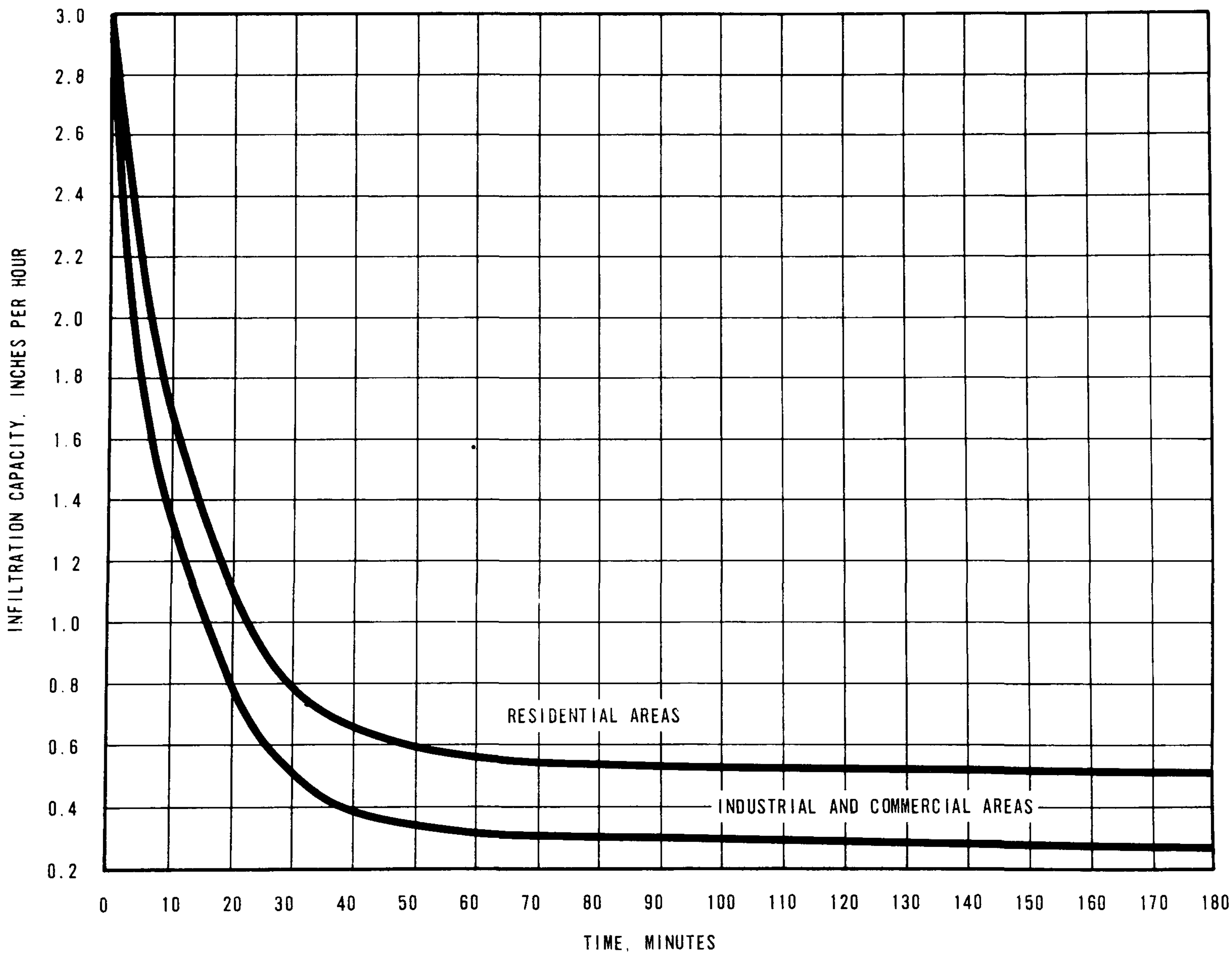
The calculated volume of runoff accounted for all runoff from pervious areas during the period of a storm when rainfall was in excess of infiltration capacity, and from impervious areas during the 24-hour period of extreme rainfall. The rainfall intensity values were read from the updated Washington, D.C. intensity-duration-frequency curves (recorded values 1896-1897, 1899-1950, 1951-1969).

This method provides reliable values for the volume of runoff from any sewer district; however, this value may differ from the resulting volume of overflow. To determine the volume of overflow, a method of hydrograph routing was employed. Appendix C likewise presents a discussion of this method. In general, this method used the Rational Method to predict the peak flow rate, which was plotted at the time of concentration for each drainage basin. Based on the peak runoff and the total volume of runoff previously calculated, a simple triangular hydrograph was assumed and plotted.

The results of the hydrograph routing were smoothed and extended to reflect the pattern of actual hydrographs, yet the volume of runoff determined by the hyetograph method was maintained. Following this, a cursory examination was made of each point of overflow. Proposed interceptor capacity, dry-weather flow, the particular operation of the diversion structure (when it closes, when it opens), etc. were accounted for to determine the peak rate of overflow and the volume of overflow. Table 5 lists the results for four storm frequencies.

Not all of the existing combined sewer districts are included in Table 5. Two districts, A-9 and part of B-2, were scheduled for separation prior to 1975. It should also be noted that overflow will not occur at 13 combined sewer districts along the lower reaches of Rock Creek, even during storms as intense as the 25-year storm. There will be no overflow at these districts due to the relief provided by the capture of combined sewer flow upstream (any District-wide solution must deal with the overflow along the upstream reaches of Rock Creek) and the large capacity of the relief interceptors in the lower reaches of Rock Creek. While the relief interceptors have the capacity to contain the peak rates of runoff along Rock Creek, the pumping station near the mouth of Rock Creek does not have the capacity to force the peak rates to the Blue Plains plant. For example, the Rock Creek interceptors have a capacity of over 1,200 mgd, while the Potomac Sewage Pumping Station has a capacity of less than 500 mgd available to pump the Rock Creek flow plus sanitary sewage from Maryland and Virginia and combined sewer flow from the Upper Potomac. It is obvious that although some districts will not experience overflows within the districts themselves, their runoffs will overflow at a subsequent point in the sewer system.

FIGURE 6
STANDARD INFILTRATION-CAPACITY CURVES FOR PERVIOUS SURFACE



SOURCE: DESIGN AND CONSTRUCTION OF SANITARY AND STORM SEWERS, ASCE, MOP.NO.37, NEW YORK, 1960.

FIGURE 7
DETERMINATION OF INFILTRATION OFFSET
5-YEAR RAINFALL FREQUENCY
PERVIOUS RESIDENTIAL AREA

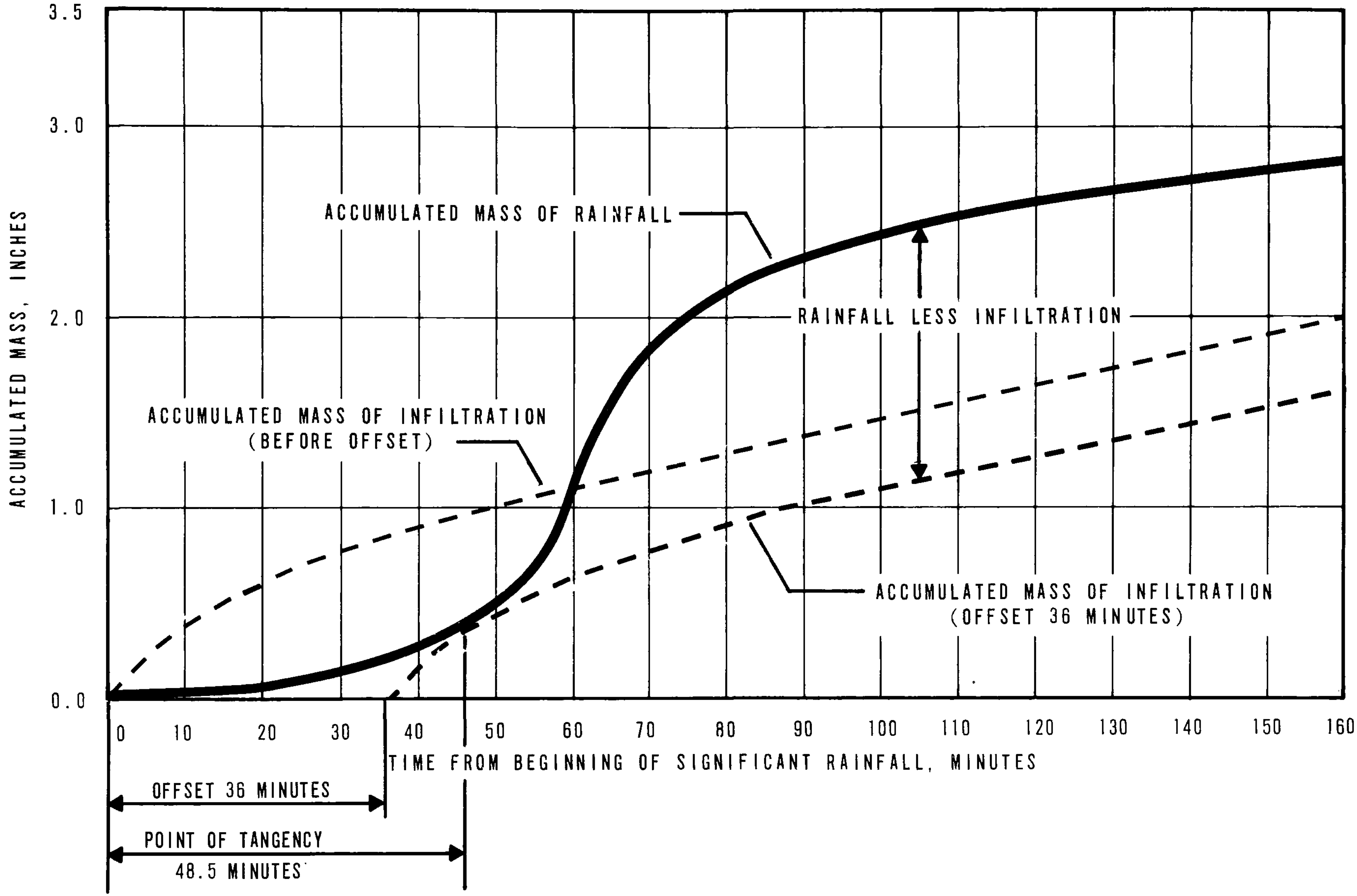


FIGURE 8
DETERMINATION OF POINT OF INTERSECTION OF INFILTRATION CAPACITY AND PRECIPITATION RATE 5-YEAR RAINFALL FREQUENCY PERVIOUS RESIDENTIAL AREA

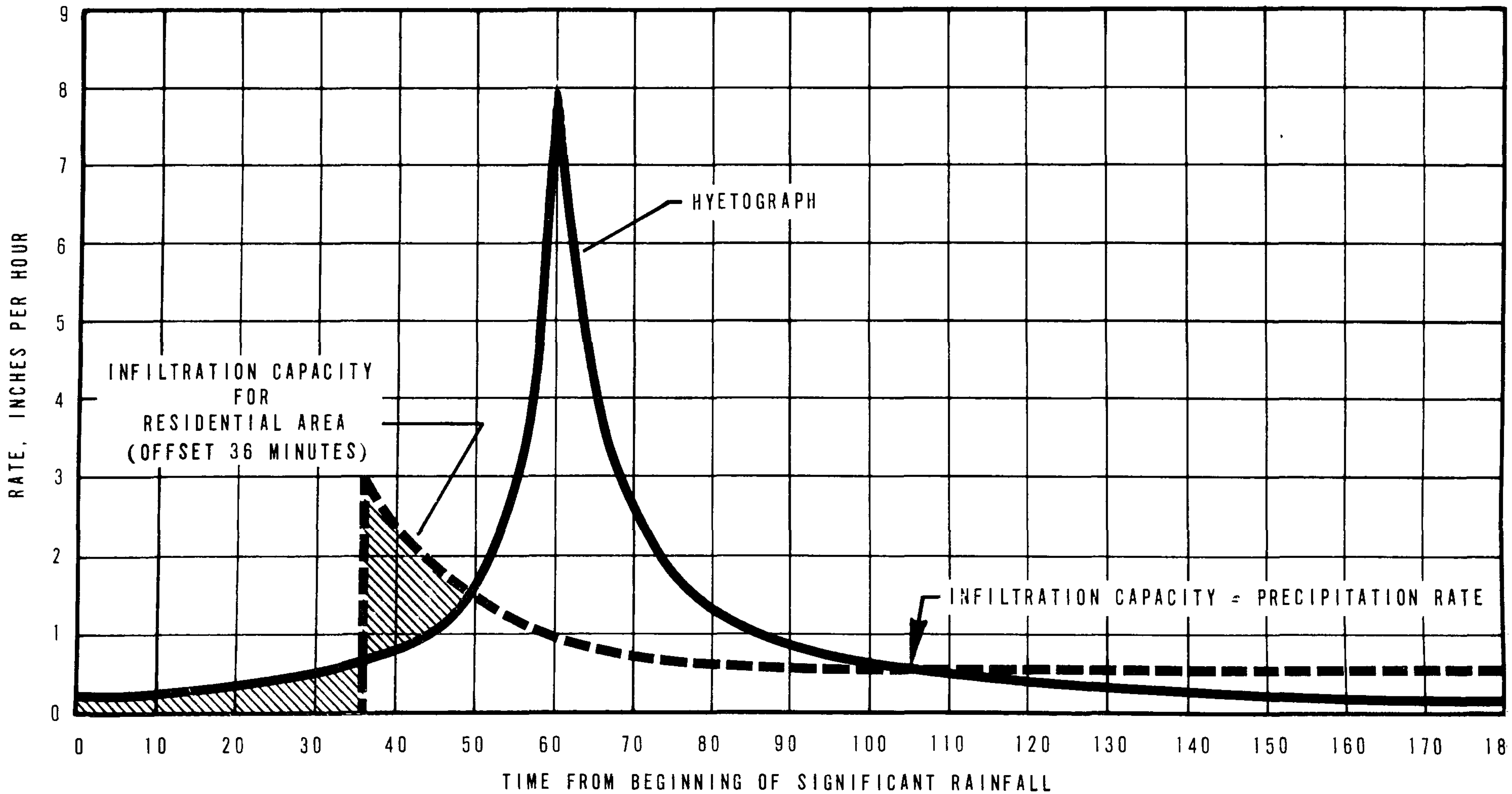


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Table 5

Combined Sewer Overflow Characteristics
(After 1975)

District No.	Location of Overflow: District Name	Storm: 2-Year, 24-Hour: Volume mil. gallons	2-Year, 24-Hour: Peak Rate mgd	5-Year, 24-Hour: Volume mil. gallons	5-Year, 24-Hour: Peak Rate mgd	15-Year, 24-Hour: Volume mil. gallons	15-Year, 24-Hour: Peak Rate mgd	25-Year, 24-Hour: Volume mil. gallons	25-Year, 24-Hour: Peak Rate mgd
A-10	37th St. - Georgetown	0.2	25	0.4	32	0.5	42	0.8	45
A-11	Georgetown	11	330	15	400	20	480	24	500
A-12	K Street - Wisconsin Ave.	2	81	3	100	4	110	4	120
B-6	M Street - 27th Street	0.0	0	0.3	50	2	90	2	110
B-7	28th Street - Wisconsin Ave.	0.0	0	0.3	30	1	40	2	50
C-25	Slash Run (Part) at 22nd and M Sts.	1	120	4	240	8	380	11	440
D-4	Piney Branch	64	1,500	100	2,100	160	2,700	190	2,900
D-5	Oak Street - Mt. Pleasant	1	45	2	65	3	71	4	81
D-6	Ingleside Terrace	0.9	29	1	40	2	47	2	50
D-7	Park Road	0.8	24	1	34	2	39	2	41
D-8	Lamont Street	0.5	15	0.7	26	1	29	1	32
D-9	Kenyon Street	0.4	13	0.6	19	1	19	2	26
D-10	Irving Street	2	51	4	96	6	120	6	126
D-11	Quarry Road	1	14	2	34	2	45	3	50
D-12	Ontario Road	0.5	8	1	20	1	25	2	27
E-2[1]	Pre-Potomac Sewage Pumping Station	97	450	158	620	214	810	260	890
	Sub Total	182		293		428		516	
E-3	O Street Pumping Station	58	1,500	79	1,800	100	2,200	120	2,300
F-1[2]	Main Pumping Station	70	1,600	96	2,000	130	2,400	150	2,500
G-2	Northeast Boundary	280	3,600	370	4,400	490	5,500	560	6,000
G-3	Barney Circle	0.2	31	0.4	38	0.8	54	1	62
G-4	14th Street - Penna. Ave.	15	420	20	550	28	660	32	630
G-5	12th Street - 9th Street	6	200	9	250	13	320	15	340
G-7	6th Street - 7th Street	5	190	7	230	10	290	12	310
	Sub Total	434		581		772		890	
	Districtwide Total	616		874		1,200		1,406	
	Inches of Rain	3.3		4.3		5.5		6.0	

[1] Overflow results from runoff in all sewer districts in Rock Creek Basin. Overflow occurs through overflow structures located between Rock Creek Pumping Station and Potomac Sewage Pumping Station. Sewer District E-2 is the nearest district and the major contributor to overflow.

[2] Overflow results from runoff in sewer districts along the Anacostia River. Overflow occurs at Main Pumping Station, and Sewer District F-1 is the major contributor to overflow.

In all, there would still remain many points of overflow beyond 1975. The largest volume of overflow was observed from the Northeast Boundary Trunk Sewer, which would have 490 million gallons of overflow with a peak rate of 5,500 mgd for a storm of 15-year frequency. For this storm the volume of overflows, in general, ranged from 0.1 to 35 million gallons for small drainage districts, up to 490 million gallons for the Northeast Boundary area, and the peak rates ranged from 19 mgd to 5,500 mgd. As can be noticed, the peak rates of overflow were generally high, even for the drainage districts where the volume of overflow was small.

Field Monitoring Program

General

A significant secondary objective of this study was to obtain data concerning actual overflows from some of the combined sewers in the District. Data concerning actual rates of overflow, pollutant concentrations versus time, etc. provide a basis to predict the characteristics of combined sewer overflows at other locations in the District.

The comprehensive field survey conducted as part of the definition of the pollution problem involved three automated monitoring systems installed in different sewer districts; two for combined sewer overflows and one for separated storm water discharges. The monitoring systems were operated during the period April 1, 1969 to September 23, 1969.

Selection of Sites

Three sampling sites were selected at different geographical areas in Washington, D.C., based on the following considerations:

1. Size of Drainage Area--The drainage area of the sewer district must be large enough to have a good combination of different surface development but also small enough to be monitored economically. The optimum size was determined to be in the range of 100 to 300 acres.

2. Population Density--The population density within the monitored district should be reasonably close to the overall population density of the entire combined sewered area, which was estimated to be about 33-35 people per acre.

3. Integrity and Simplicity of the Sewer System--Multiple diversion or intercepting points in the system to be monitored interfere with valid correlation between runoff and rainfall. All the sewage flow and the surface runoff of the sewer district must converge into one trunk line equipped with one overflow point.

4. Feasibility of Construction and Equipment Installation--The geographical configuration at the monitoring site must be flat, a power source must be available at or near the site, and there must be reasonable expectation of obtaining permission to use the site.

5. Traffic and Public Impact--The monitoring site must not involve any public annoyance or traffic congestion attributable to the installation of the monitoring equipment. Exposure to malicious vandalism must also be avoided.

6. Parity Between the Sites for the Combined and Separated Sewer Systems--The site for the separated sewer system and one of the sites for the combined sewer system should be as similar as possible. Their drainage areas and population densities should also be similar. This makes comparison of the hydrological, geological, geographical, and meteorological data for both kinds of monitored areas more meaningful.

7. Size of the Trunk Sewer--In view of the necessity of installing various items of equipment in the sewer, the trunk sewer must be spacious enough to work inside. Minimum usable size was set at three feet in diameter.

8. Underground Installation--The existence of any sizable underground structures or installations in the vicinity of the monitoring stations is undesirable because of possible interference with the underground conduits required for the monitoring operation. The information concerning different utility lines such as gas, water, and electricity, must be reviewed and their exact locations noted.

Based on the detailed study of various pertinent information and field observations, three sewer districts were selected as meeting these criteria:

1. Sewer District B-4
2. Sewer District G-4
3. Sewer District of Good Hope Run

Sewer Districts B-4 and G-4 (as designated by the D.C. Department of Sanitary Engineering) are in the combined sewer system whereas the Good Hope Run Sewer District is in the separated sewer system. Sewer District B-4 is in the Georgetown area, and G-4 and Good Hope Run are along the Anacostia River. The basic data for the physical descriptions of the sewer districts selected for monitoring are summarized in Table 6. Figures 9, 10, and 11 show the major sewers in each of these districts and the locations of monitoring equipment.

Operation of Monitoring Systems

The monitoring systems were operated by ROY F. WESTON personnel, with the assistance of personnel of the Sewer Operation Division of the Washington, D.C. Sanitary Engineering Department. Storms of varying intensities were monitored at each of the three sites; seven in Sewer District B-4, seventeen in Sewer District G-4, and ten in Good Hope Run Sewer District. Starting time, duration, total rainfall, maximum intensity, numbers of samples collected, and monitoring sites involved for each storm are listed in Table 7. Storms were missed at each of the monitoring sites from time to time, mostly because of storm caused damage to equipment located in the sewers; these storms are listed, along with the problems involved, in Table 8.

The stations were designed to operate automatically, because of the difficulties of predicting the beginning of significant precipitation, assembling personnel on short notice, and assembling personnel during the night hours. A schematic diagram of the monitoring equipment is shown in Figure 12.

Selection of a satisfactory technique for flow measurement presented a problem. A weir setup could not be used, because backwater elevations would have caused surcharging and flooding at the anticipated high flow rates. Depth-of-flow measurements with the use of one of the steady state empirical equations (Manning, Kutter, etc.) for calculating flow would not be applicable, since flow conditions were not steady-state during periods of precipitation. The approach finally selected for measuring flow rates was to use a tracer solution and a form of the continuity equation; this procedure is described mathematically in subsequent paragraphs of this section. Physically, it involves the release of a tracer solution (lithium chloride in this case) of known concentration and feed rate at an upstream manhole, sampling of combined sewer overflows at a downstream manhole, and analysis for tracer concentration. From the upstream feed rate and concentration and from the downstream concentrations, the flow can be calculated accurately. Previous experience with this method on steady-state flows has indicated that accuracy within ±4 percent

Table 6

Physical Descriptions of the Monitoring Sites

Sewer District	Location	Monitoring Operation	Sewer System	Drainage Area acres	Population Density 1970 persons/acre	Trunk Sewer Size
B-4	Rose Park Playground	Lithium Chloride Release	Combined	105	43.6	4′ Diam.
	Rock Creek Parkway	Sample Collection				
	26th - O St.	Rainfall Measurement				
G-4	14th - L St. S. E.	Lithium Chloride Release	Combined	252	52.6	5′ - 6″ Diam.
	14th - M St. S. E.	Sample Collection				
	14th - L St. S. E.	Rainfall Measurement				
Good Hope Run	17th - Minn. St. S. E.	Lithium Chloride Release	Separated Storm	265	37.6	6′ x 6′
	1630 16th S. E.	Sample Collection				
	16th - Minn. St. S. E.	Rainfall Measurement				

FIGURE 9

COMBINED SEWER DISTRICT B-4

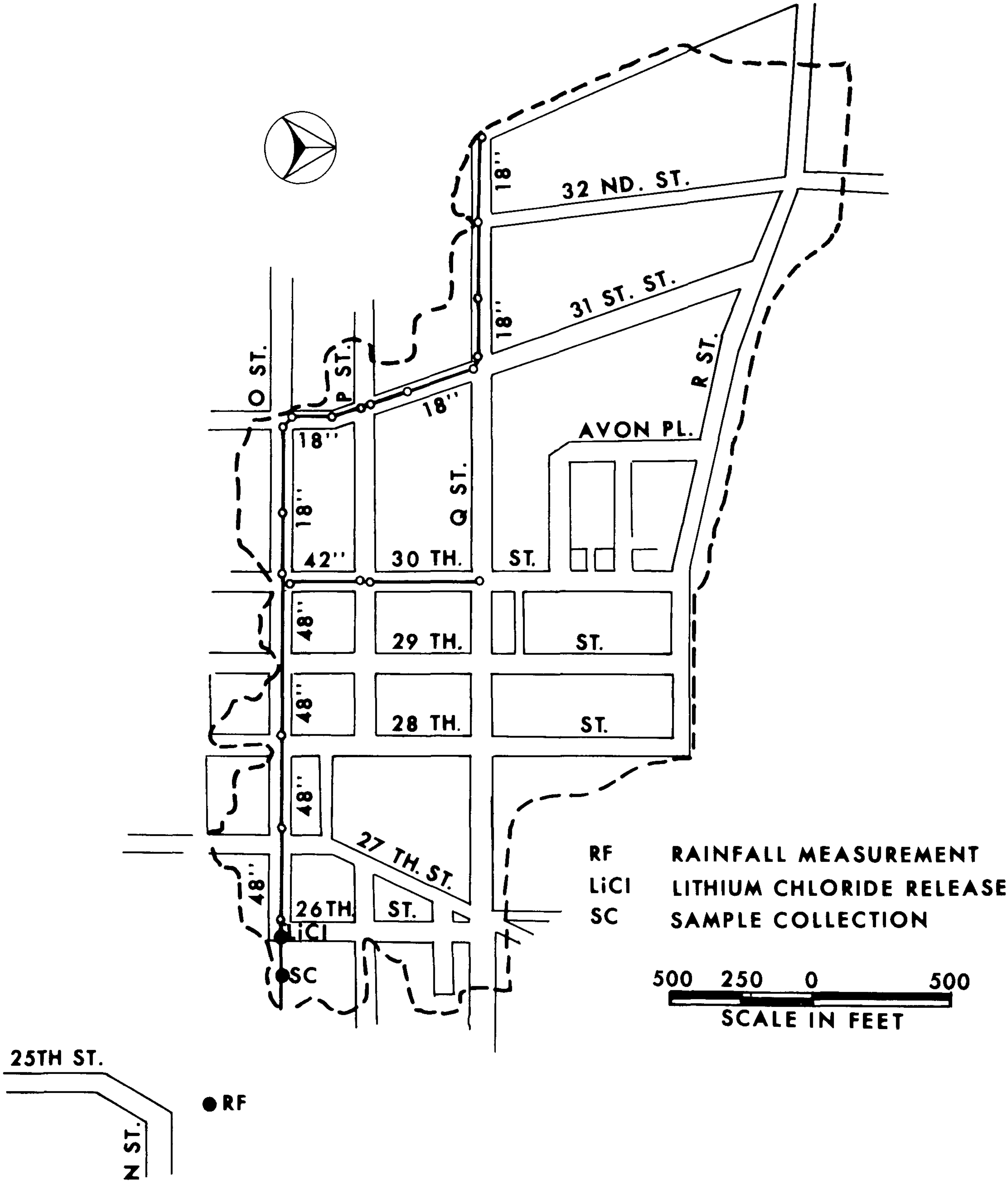

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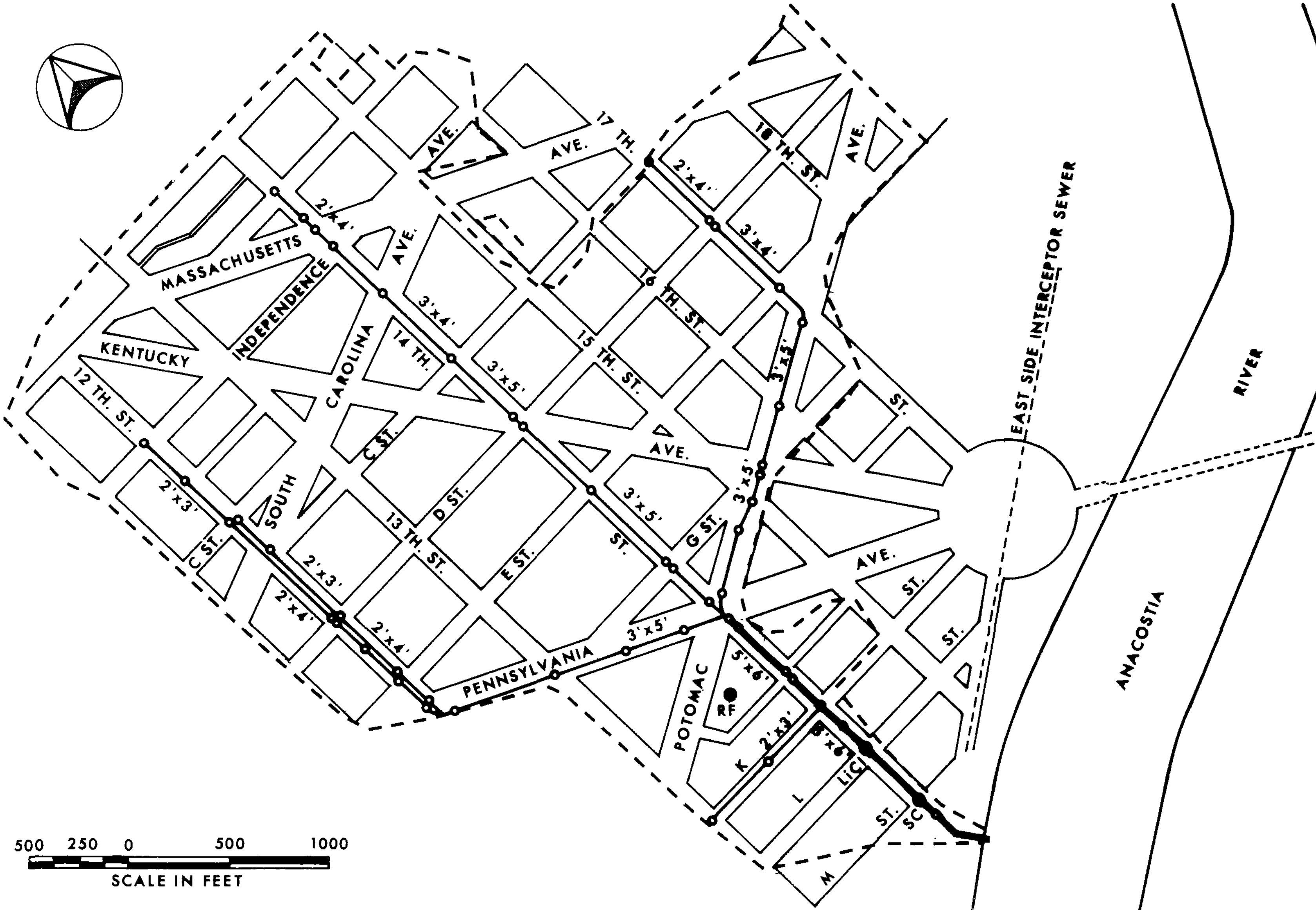
FIGURE 10
COMBINED SEWER DISTRICT G-4



RF	RAINFALL MEASUREMENT
LiCl	LITHIUM CHLORIDE RELEASE
SC	SAMPLE COLLECTION

FIG. 10

FIGURE 11
GOOD HOPE RUN SEWER DISTRICT

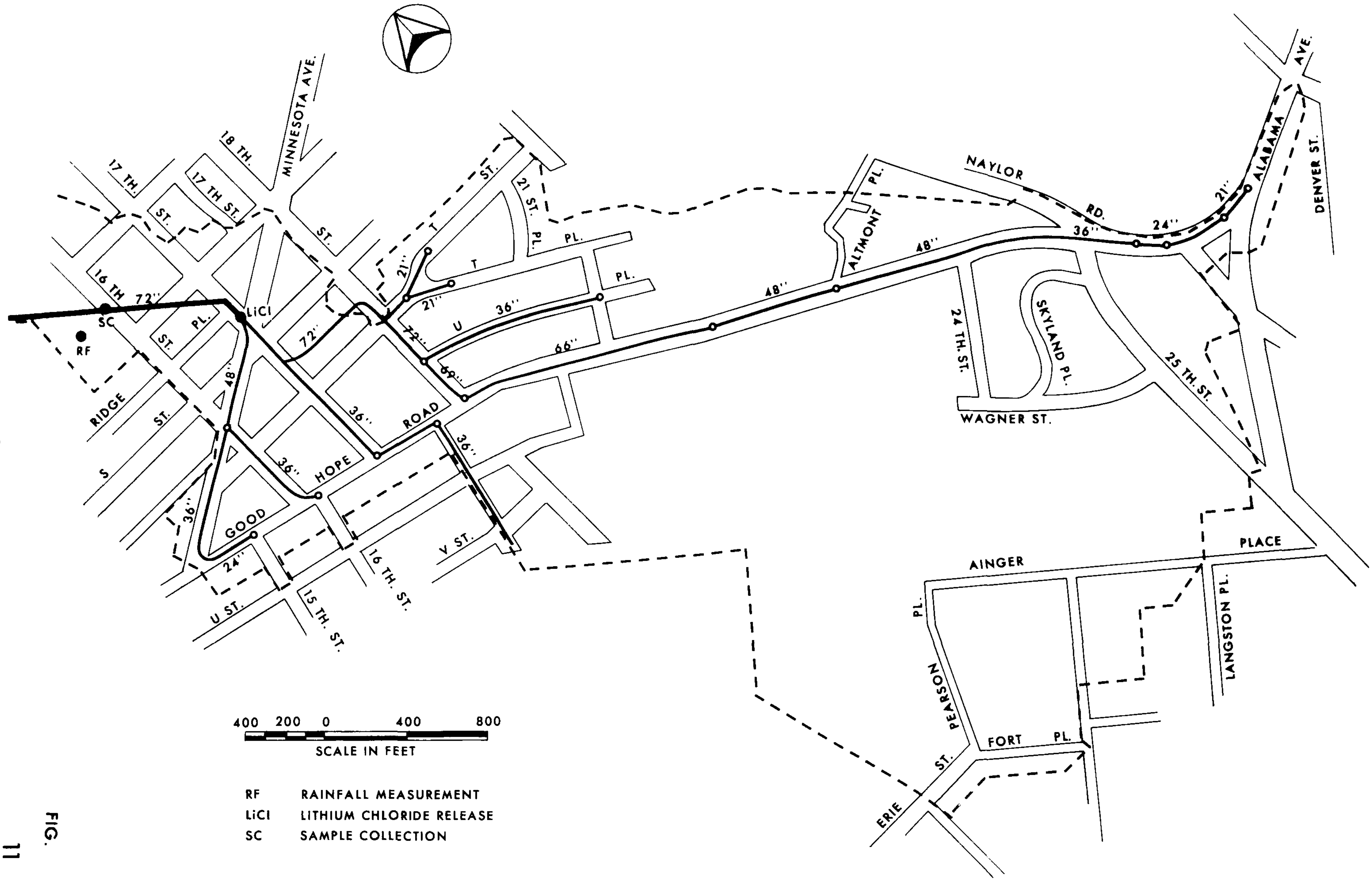


FIG. 11

Table 7

Storms Monitored

Date	Starting Time	Site	Total Rainfall (inches)	Duration (min.)	Maximum Intensity (in./hr.)	Number of Samples Collected
May 9	9:25 a.m.	G-4+	0.8	20	2.4	1
May 19	1:42 a.m.	G-4	0.4	3	3.0	4
May 20	11:42 p.m.	G-4	0.6	7	6.0	3
June 1	7:25 p.m.	B-4o	1.4	22	7.2	6
June 2	7:45 p.m.	B-4	0.9	20	6.0	3
June 3	12:25 a.m.	B-4	0.95	15	6.0	3
June 8	5:44 p.m.	G.H.R.*	0.7	6	6.0	1
June 8	5:50 p.m.	G-4	0.7	13	2.8	3
June 15	2:00 p.m.	G-4	0.7	40	6.0	4
July 6	7:40 p.m.	G-4	0.4	20	0.8	2
July 27	11:35 p.m.	G-4	2.1	39	8.0	12
July 27	11:35 p.m.	B-4	1.3	20	7.0	2
July 28	2:30 a.m.	G-4	0.6	22	4.6	6
July 28	11:30 a.m.	G-4	0.6	15	4.0	6
July 28	1:20 p.m.	G.H.R.	1.6	40	6.0	22
July 28	1:28 p.m.	G-4	1.3	35	5.6	5
July 28	5:00 p.m.	G.H.R.	0.2	30	0.4	8
Aug. 1	7:30 p.m.	G-4	0.6	210	1.0	1
Aug. 1	8:45 p.m.	G.H.R.	0.5	135	1.6	1
Aug. 2	8:05 p.m.	G-4	2.8	65	6.0	8
Aug. 2	8:10 p.m.	B-4	3.9	50	7.5	6
Aug. 2	8:17 p.m.	G.H.R.	2.9	73	5.6	12
Aug. 3	10:30 p.m.	B-4	0.4	30	4.0	3
Aug. 9	9:20 p.m.	G-4	1.1	17	8.0	5
Aug. 9	9:20 p.m.	G.H.R.	1.5	25	7.2	7
Aug. 9	11:00 p.m.	G.H.R.	0.4	30	2.0	5
Aug. 9	11:20 p.m.	G-4	1.6	10	7.2	2
Aug. 9	11:22 p.m.	B-4	1.6	15	8.4	3
Aug. 10	12:25 a.m.	G.H.R.	0.65	20	0.6	2
Aug. 19	6:40 p.m.	G-4	1.35	13	8.0	4
Sept. 4	3:45 p.m.	G-4	3.4	75	6.0	1
Sept. 17	8:15 p.m.	G-4	0.7	5	4.2	1
Sept. 17	8:20 p.m.	G.H.R.	0.6	100	0.6	4
Sept. 20	3:00 p.m.	G.H.R.	0.1	90	0.1	1

G-4+ Combined Sewage in Sewer District G-4
B-4o Combined Sewage in Sewer District B-4
G.H.R.* Storm Runoff in Sewer District Good Hope Run

Table 8

Major Storms Missed[1]

Date	Starting Time	Site	Loss Involved	Problems
June 1	10:40 p.m.	G-4[2]	LiCl and Sample	Submersible Pump Clogged
June 2	12:40 a.m.	G-4	LiCl and Sample	Submersible Pump Clogged
June 1	10:18 p.m.	G.H.R.[3]	Sample	Bubbler Line Broken
June 1	7:20 p.m.	B-4[4]	Instruments Flooded	Poor Drainage at Site
June 2	7:45 p.m.	B-4	Instruments Flooded	Poor Drainage at Site
June 15	1:53 p.m.	G.H.R.	Sample	Bubbler Line Broken
June 15	12:45 p.m.	B-4	Sample and LiCl	Electrical System Flooded
June 18	9:30 p.m.	All Sites	Sample and LiCl	In Process of Repairing Equipment in Sewers
July 20	5:38 p.m.	All Sites	Sample and LiCl	Submersible Pump Clogged
July 22	7:00 p.m.	All Sites	Sample and LiCl	In Process of Repairing Equipment
Aug. 19	6:50 p.m.	G.H.R.	Sample and LiCl	Sampling Head not in Barrel
Sept. 4	3:52 p.m.	G.H.R.	Sample and LiCl	Sampler Switch System Broken Down
Sept. 4	3:45 p.m.	G-4	Sample and LiCl Submersible Pump	Submersible pump Washed Away
Sept. 8	1:10 p.m.	G-4	None	Submersible Pump and Cage Being Repaired
Sept. 8	12:55 a.m.	B-4	Sample and LiCl	Submersible Pump Clogged

[1] Intensity-duration information for many of the missed storms can be found in Tables E-1, E-2, and E-3 of Appendix E.
[2] Combined Sewer District G-4.
[3] Separated Sewer District - Good Hope Run
[4] Combined Sewer District B-4.

FIGURE 12

MONITORING EQUIPMENT

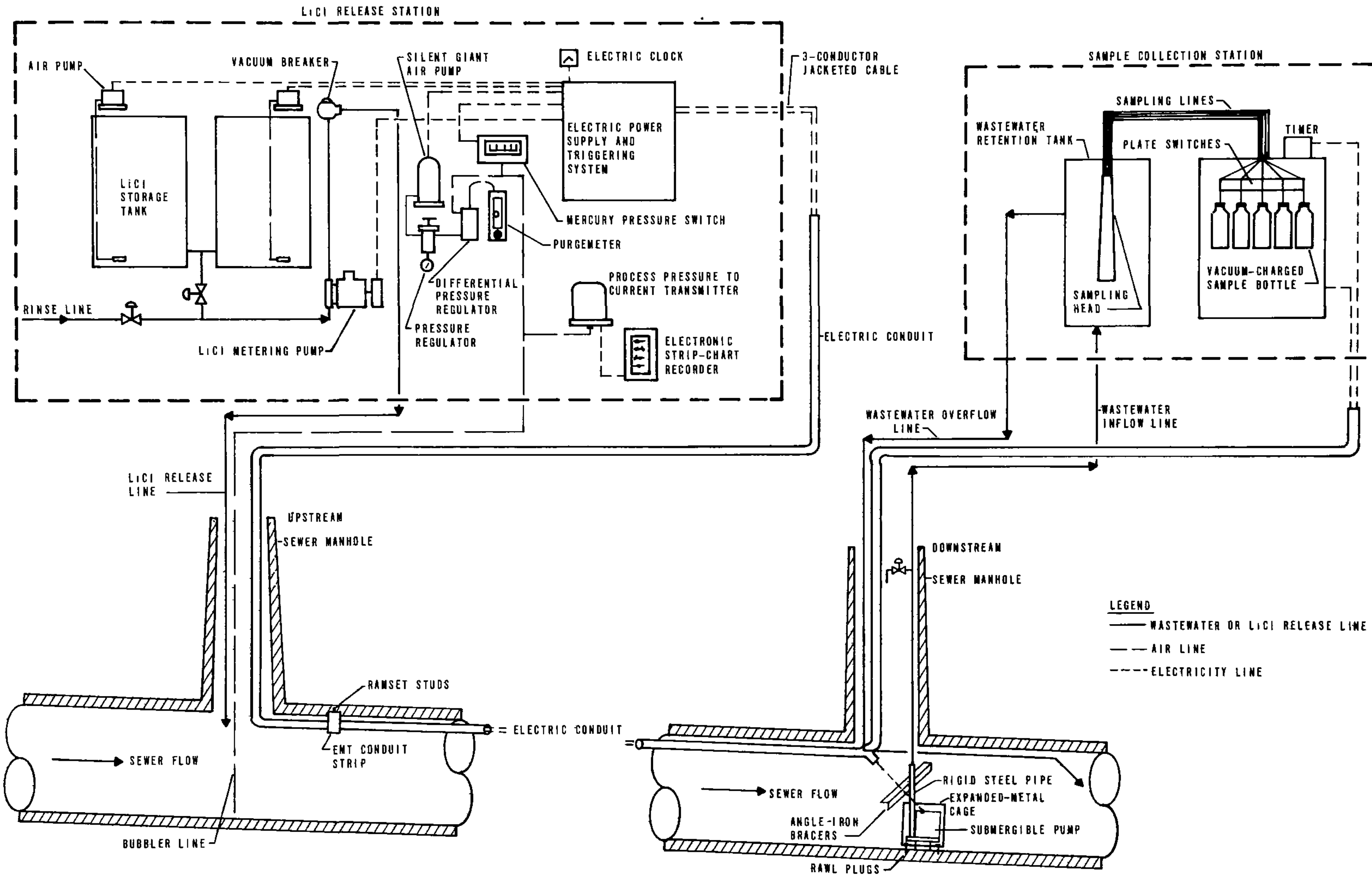


FIG. 12

can be obtained. It should be pointed out that the method has never been verified for runoff flow measurements; however, because the tracer dilution method is based on continuity of mass rather than energy, unsteady-state conditions should not introduce errors.

Flow rate estimates based on depth-of-flow measurements and the Manning formula were compared to the results of the tracer method. The depth-of-flow estimates showed only a general correlation, having a significant spread, in comparison with the tracer results. Three possible sources of error explain these differences:

1. Incomplete mixing of tracer solution
2. Inaccurate measurement of depth-of-flow
3. Assumption of steady-state conditions in using Manning's formula.

Sufficient steps were taken to assure practically complete mixing of the tracer solution, and therefore, the other two sources of error explain the differences in flow estimates and the relative inaccuracy of the depth-of-flow procedure.

During dry-weather periods, the operating effort was minimal, because the stations operated only during periods of precipitation. Thus, dry-weather periods provided adequate time for maintenance work, routine operating checks, and preparation of equipment for forecasted storms. The triggering depth at each site was set according to the minimum depth requirement for operation of the submersible pump, the maximum dry-weather flow depth, and the expected magnitude of the impending storm; the metering rate for lithium chloride was also pre-set on the basis of expected storm magnitude.

For the short intense storms, the concentration of each waste constituent was observed to increase with the discharge rate in the sewer; the peak concentrations of many waste constituents were concurrent with the peak flow. The concentrations were significantly high and remained so throughout most of the monitoring period.

The time variation of the quality and quantity of wastewater generated by a long-duration, high-intensity storm shows that the flow rate of the wastewater generated by this long, intense storm is much higher than that of short storms. However, the concentrations of various constituents were observed to be lower, which is an anticipated result of high dilution by storm water.

Chemical oxygen demand and suspended solids concentrations of wastewater from a long, intense storm are reduced to approximately one-third of the comparable values for a short, intense storm. The biochemical oxygen demand for the long, intense storm wastewater were about one-seventh of that in short-storm wastewater. This also implies that a higher fraction of surface material was eroded by the long, intense storm. However, the ratio of COD to BOD_5 was essentially the same as that for short-storm wastewater.

The characteristics and quantities of wastewater generated by four consecutive storms during July 27-28, demonstrated the initial flushing effect in all cases, even when the storms were only a few hours apart. However, the average concentrations of specific contaminants were observed to follow a decreasing trend with consecutive storms. For example, the average concentration of chemical oxygen demand (COD) decreased from 307 mg/L for the first storm to 154 mg/L for the fourth storm. This decrease may be attributed to the reduction of waste material accumulation on the surface and in the collection system. The ratio of COD to BOD_5 was essentially the same as the comparable ratios for short storms and for long-duration, high-intensity storms. The characteristics of long-duration, low-intensity storm wastewater probably will be similar to that observed for consecutive storms.

The average organic and nutrient concentrations in separated storm water discharges were observed to be approximately one-third of those in combined sewers, but the solids concentrations (especially non-volatile solids and settleable solids) were much higher than those in combined sewer overflow. However, this phenomenon may be attributed to the differences in surface development in the monitored areas. Silt was found to be the dominant factor in separated storm water discharges.

In general, the time variation in characteristics of wastewater from separated storm sewers is very similar to that from combined sewers. One significant difference observed for separated storm sewer discharge is the broader range of COD to BOD_5 ratio (1.8 to 32).

The bacteriological examinations were made on selected individual samples of combined sewer flow and on storm-composite samples. The range of variation and the mean values of different bacteria species are summarized in Table 9. The bacteriological counts varied with the flow rate of combined sewer overflow during each storm and the initial flushing had a significant effect on various bacteria counts. The variation of bacteriological data for the separated storm sewer discharge was similar to the variation in the combined sewer discharge. Bacteria counts were high at the beginning of the storm and then decreased as the storm progressed. Also, bacteria were present in storm runoff in fairly significant amounts although less than in combined sewage, as cited in Table 9.

To assess the relative contribution to pollution throughout the duration of a storm, the total waste materials discharged were expressed in units of pounds per unit time. As expected, these waste loading expressions indicate that most waste materials were carried by the initial flushing and scouring of the sewer. Waste loadings carried by the secondary flushing in a prolonged storm were limited.

The rain gauges were operated by a local Washington, D.C. subcontractor. The waste-level recorder charts required changing only once every 30 days, but the readings were observed and the equipment was checked on a routine weekly basis. Equipment installed in the sewer was inspected at least once a week.

Detailed discussions of monitoring practices and techniques are presented in Appendix D.

Monitored Wastewater Flows and Characteristics

As shown in Table 7, storms of varying intensities and durations were monitored during the six-month program. Wastewater flows and characteristics under storm conditions were studied both in the combined sewer districts and in the separated storm sewer district. The effects of short-duration storms, long-duration, high-intensity storms, and consecutive storms were observed and evaluated. The range and mean value of each of the waste constituents in storm wastewater (for both combined sewer and separated storm sewer flow) are presented in Table 9.

Appendix E presents a detailed comparison of the characteristics of the combined sewer and separated storm sewer flows generated by storms representative of the wide range

Table 9

Comparison of Characteristics of Combined Sewer Overflow and Separated Storm Water Discharge

Waste Constituents[1]	Combined Sewer Overflow[2]		Separated Storm Water Discharge[3]	
	Range	Mean	Range	Mean
Chemical Oxygen Demand	80 - 1,760	382	29 - 1,514	335
Biochemical Oxygen Demand	10 - 470	71	3 - 90[4]	19[4]
Total Solids	120 - 2,900	883	338 - 14,600	2,166
Total Volatile Solids	40 - 1,500	344	12 - 1,004	302
Suspended Solids	35 - 2,000	622	130 - 11,280	1,697
Volatile Suspended Solids	10 - 1,280	245	0 - 880	145
Settleable Solids	0 - 1,308	229	0 - 7,640	687
Total Phosphate	0.8 - 9.4	3.0	0.2 - 4.5	1.3
Total Nitrogen	1.0 - 16.5	3.5	0.5 - 6.5	2.1
Orthophosphate	0.1 - 5.0	2.0	Not Sampled	Not Sampled
Ammonia Nitrogen	0 - 4.7	1.5	Not Sampled	Not Sampled
pH[5]	5.6 - 6.7	6.3	7.2 - 6.0	6.5
Total Coliform[6]	420,000 - 5,800,000	2,800,000	120,000 - 3,200,000	600,000
Fecal Coliform[6]	240,000 - 5,040,000	2,400,000	40,000 - 1,300,000	310,000
Fecal Streptococcus[6]	1,000 - 49,000	17,200	3,000 - 60,000	21,000

[1] mg/L unless otherwise noted.
[2] Based on analysis of 94 samples
[3] Based on analysis of 64 samples
[4] Excluding sample from June 8 storm, which had a BOD concentration of 600 mg/L.
[5] pH units.
[6] Counts per 100 ml.

of durations and intensities. Appendix E includes figures that relate measured concentrations and loadings of various contaminants with time, cumulative rainfall, and sewer flow rate. Although Appendix E presents a detailed and complete discussion of the results of the monitoring program, it is convenient to highlight the more significant results.

Impact of Storm Water Discharges

Annual Discharges

Results of the 1956-1958 studies of the Washington Sewer System indicated that an average of 3.31 million gallons per day (mgd) of sanitary sewage had overflowed from the combined-system areas during rain storms and 0.34 mgd during dry weather. In terms of percentages, about 3 percent of the annual sanitary sewage produced was discharged directly to the streams during rain storms and about 0.31 percent during dry weather. Estimates were also made for the future, when the program of improvements (Project C as previously discussed) would be completed. At that time, the overflow from rain storms would amount to an average of 0.38 mgd, equivalent to 0.42 percent of the sewage flow from the areas which would still have combined sewers.

The approach to assessment of the effects of overflows used in the present study is slightly different. The effect on receiving streams was determined in terms of pollution load discharged into the streams. The parameters used were BOD, suspended solids, total phosphates, and total nitrogen. The amounts of these elements discharged into the streams from storm water and combined sewer overflow were calculated separately.

From a probability plot of the average rainfall for the period 1900-1968 based on Weather Bureau records, the annual rainfall with a 50 percent probability of occurrence was derived. This value, 40.2 inches, was used to calculate the volume of total storm water reaching the streams from the entire area of the District. After obtaining the median annual rainfall, certain basic assumptions were made to determine the quantity of storm water and sanitary sewage that would overflow into the streams. The parameters used in the calculations were obtained from prior reports, from literature review, from current study data, or from conservative estimates based on general technical knowledge.

The first assumption was that 90 percent of the runoff will reach the streams. Other assumptions, based on the field monitoring program were made for concentrations of pollution parameters. The following are the values of these parameters used for runoff in this assessment of pollution effects, as taken from Table 9.

BOD	19 mg/L
Suspended Solids	622 mg/L
Total PO_4	1.3 mg/L
Total Nitrogen	2.1 mg/L

The suspended solids concentration of 1,697 mg/L suggested for storm water runoff by Table 9 was not used. In comparison with the suspended solids concentrations measured in the two combined sewer districts and with the concentrations reported in other studies, the value of 1,697 mg/L seems extremely high and unreasonable. A value of 622 mg/L, an average measured for the two combined sewer districts, should be more valid.

The overall runoff coefficient for the entire District had to be determined in order to estimate the total volume of storm water runoff reaching the streams. The overall runoff coefficient value was derived by calculating a weighted average of the runoff coefficients obtained for individual drainage districts. The annual volume of storm water runoff was then calculated by multiplying 0.9 (only 90 percent was assumed to reach the streams) by the overall runoff coefficient, the total area in acres of the entire district, and the median annual rainfall. The average values of pounds of BOD, Suspended Solids, Total Phosphate, and Total Nitrogen were then calculated on the basis of this annual volume and the listed concentrations of pollution parameters.

Pollution load data for sanitary sewage discharged through the overflow structures were calculated separately. One of the assumptions made was that approximately 95 percent of the sanitary sewage in the combined sewers would overflow into the streams during periods of overflow. The data for average flow per capita and for sewage characteristics were obtained from the February 1969 (3) report on the District of Columbia's sewage treatment plant. Duration of overflow for an average year for various interceptor sewers was derived from the Board of Engineers' 1957 report (1).

The volume of sanitary sewage discharged directly into the stream through the overflow structures was then calculated to be an average of 545 million gallons in 1970 and 346 million gallons in 1975 (when less overflow would result because of improvements in the sewer system). In veiw of the delays encountered in the sewer improvement program, it appears to be more realistic to use the 1970 estimate to assess the pollution load to the river. In comparison with the value obtained in the prior studies, the current values are approximately 1.0 percent of the total annual sewage flow to the treatment plant or about 3.4 percent of the total annual sewage flow from the combined-system areas.

The average annual quantities of BOD and other pollution elements discharged to the streams through dry-weather overflow were calculated from the volume of overflow and concentrations of these elements present in sewage. Upon obtaining the data on pollution loads from storm water and sanitary sewage overflows, the total quantities of BOD, SS, PO_4 and Total Nitrogen discharged into the streams annually were calculated. The values obtained initially were for the 50 percent probability of occurrence. However, once these values were obtained, similar data for other probabilities were calculated by applying appropriate factors from the probability plot of average annual rainfall. Probability plots for BOD, SS, Total PO_4 and Total Nitrogen discharged annually to the streams in million pounds were plotted. These plots are shown on Figure 13, and the data for 50 percent probability are summarized in the first part of Table 10 (Annual Loads).

Individual Storm Loadings

A definition of the combined sewer problem based solely on annual pollution loadings is incomplete. Overflows from combined sewers do not occur as continuous steady discharges, but rather as slug loadings. This characteristc, in combination with the long, effective residence times of estuarine waters, explains the particularly serious impact combined sewer overflows have on water quality.

The sources of pollutants in combined sewer overflow are the storm runoff from the urban area, the sanitary sewage mixed with the runoff, and the initial flushing action in the sewers. The average concentrations of pollutants in storm runoff from an urban area have been established in this study. It is reasonable to assume that pollutant concentrations of sanitary sewage included in combined sewer overflows are typical of normal domestic sewage. An independent study (6) of the flow in combined sewers B-4 and G-4 during storm conditions estimated that flushing action results in a total added BOD loading of 3 pounds per acre of drainage area. An extension of this calculation to phosphate and nitrogen pollution indicates that flushing action results in total phosphorus and total nitrogen loadings of 0.2 lbs. and 0.3 lbs. per acre, respectively.

FIGURE 13
ANNUAL POLLUTION LOAD DISCHARGED TO STREAMS FROM STORM WATER

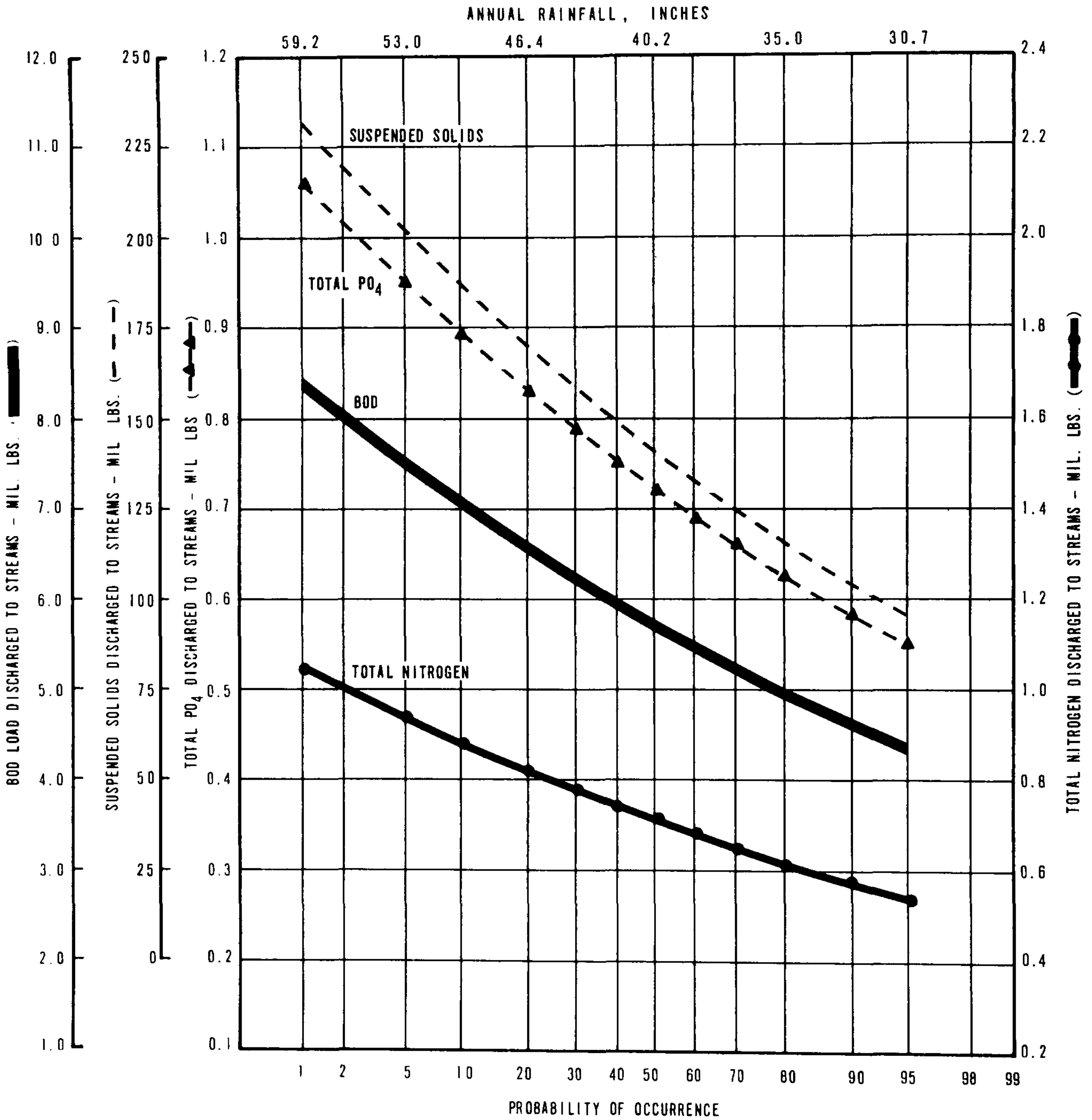


FIG. 13

Table 10

Expected Pollution Loads
from Combined and Separated Storm Sewers
in District of Columbia
Based on Annual Rainfall of 40.2 Inches

	Volume mil.gal./yr.	BOD mil.lb./yr.	SS mil.lb./yr.	TP[2] mil.lb./yr.	TN[3] mil.lb./yr.
Overflows from Combined Sewer Areas[1]					
Storm Water	11,000	1.8	57	0.1	0.2
Sanitary Sewage	1,000	1.4	2	0.4	0.3
Sub-Total	12,000	3.2	59	0.5	0.5
Direct Discharge from Separated Storm Sewers	16,000	2.5	83	0.2	0.2
TOTAL	28,000	5.7	142	0.7	0.7

24-Hour Pollution Loads from
Various Return Frequency Storms

	Volume million gallons	BOD lb.	TP[2] lb.	TN[3] lb.
Discharge from D.C. Combined Sewer District for 2-Year, 24-Hour Storm	616	160,000	12,000	19,000
Discharge from D.C. Combined Sewer Districts for 5-Year, 24-Hour Storm	874	200,000	13,000	24,000
Discharge from D.C. Combined Sewer Districts for 15-Year, 24-Hour Storm	1,190	250,000	14,000	29,000
Discharge from D.C. Combined Sewer Districts for 25-Year, 24-Hour Storm	1,396	280,000	15,000	33,000
Average Daily Load from D.C. Combined Sewer Districts[4]	33	8,000	1,400	1,400
Recommended Maximum Allowable Daily Loadings in Metropolitan Washington Area		16,500	740	8,000

[1]Combined Sewer Overflows occur approximately 50-60 times per year.
[2]Total Phosphorus.
[3]Total Nitrogen.
[4]Determined by dividing annual loads by 365 days/year.

The previous analysis of rainfall-runoff relationships to determine volume of overflow provides a sufficient basis to determine pollutant loadings. The total pollution load overflowing from the combined sewer district during a storm is estimated by adding the following values:

1. Volume of storm water runoff times the average pollutant concentrations previously established.

2. Acreage of the drainage district times the flushing unit loadings.

3. Typical sanitary sewage pollutant concentrations times the average dry-weather flow generated in the district.

The 24-hour pollution loads calculated for the combined sewer system for storms of various return frequencies are presented in the second half of Table 10. Also included for purposes of comparison are the maximum allowable contaminant loadings if water quality objectives in the Potomac are to be met. Examination of these figures shows that the BOD and phosphorus discharges, even from a 2-year, 24-hour storm are from ten-fold to twenty-fold greater than the recommended loadings in the entire metropolitan Washington area. In fact, even the average daily loadings (determined by dividing annual loadings by 365 days/year) in combined sewer overflows account for a good portion of or exceed the recommended loadings. It is pointed out that there is no uniform daily discharge of combined sewer overflow; the annual loadings are concentrated unevenly in the fifty to sixty overflows that occur each year. It is this characteristic combined with the long residence times of estuarine waters that explains the particularly serious impact of combined sewer overflows on the water quality of the Potomac.

SECTION V

INVESTIGATION OF POTENTIAL ABATEMENT MEASURES

Review of Approaches Tried at Other Cities

Many municipalities and some private companies have been and are continuing to investigate methods for dealing with the problem of combined sewers. The methods being considered, separately or in combination, can be classified under four headings: 1) sewer separation; 2) off-system storage; 3) treatment processes; and 4) miscellaneous.

Sewer Separation

Complete separation of sanitary and storm sewers is an enormous project. It requires the following:

1. Separate storm and sanitary sewers
2. Separate roof drains and downspouts
3. Separate yard and areaway drains
4. Separate air conditioning and cooling system drains
5. Separate foundation drains
6. Separate catch basin inlets

The costs of complete separation of any extensive existing combined sewer system are prohibitively high. In a recent study (7), the American Public Works Association estimated that it would cost 30 billion dollars to separate all the combined public sewers in the country, and another 18 billion to make the related plumbing changes on private properties. Costs for partial separation, involving separation of only the most troublesome sewers and the related storm water drains would obviously be less, but still would be substantial. According to District of Columbia Department of Sanitary Engineering figures, the typical cost encountered within the District of Columbia in 1957 was $18,000 per acre.

There are other complications. Complete separation projects would take many years to complete, even if concentrated construction effort were applied. Communities and businesses would suffer inconveniences and losses during construction, when streets would be closed to traffic. Extensive policing would be required to ensure the separation of all storm water connections (e.g. roofs, yards, etc.). Furthermore, recent studies (8, 9, 10) reveal that storm water runoff itself is polluting the streams, and, in many situations, should receive treatment. This has also been confirmed in this study.

Even with these complications, sewer separation is by far the most commonly used remedial method in the 900 communities surveyed by the APWA; however, separation has generally been confined to portions of sewer systems and has not very often been applied to an entire system. Only rarely have all roof drains, air-conditioning drains, etc., been separated. The method of separation which appears to be the most practical is to construct a sanitary sewer within the larger existing combined sewer. This was accomplished in Ottawa, Canada using 15-inch cast iron pipe at a cost of $20-25 per foot in place (11). Minneapolis installed a flattened 42-inch corrugated steel sanitary sewer along the invert of a 102-inch tunnel, at a low but undisclosed cost (12). A different approach is used in the ASCE Combined Sewer Separation Project, which involves pumping comminuted sewage from individual buildings through pressure tubing to pressure conduit installed within existing combined sewers (13).

Off-System Storage

The use of storage (surface or sub-surface) in connection with the problem of combined sewers is being applied in two general ways:

1. Temporary storage, with return of the retained flow to the sewer system for conventional treatment when the storm subsides; and

2. Temporary storage, followed by discharge of the retained flow directly to the watercourses.

The latter approach may be acceptable in certain situations, because some degree of solids removal will occur while the wastewater is stored; however, application is limited to situations where water quality requirements are not severe and/or when other pollutant concentrations (dissolved materials) are low.

Chicago Underflow and Deep Tunnel Plans (14) (15)

Several cities have or plan to have facilities for temporarily storing overflow for later release to a conventional waste treatment plant. The Chicago Underflow and Deep Tunnel Projects are the biggest and best known of these plans. For years, Chicago has experienced basement and underpass flooding, as well as storm and beach pollution resulting from combined sewer overflows. The Underflow-Storage Plan and the Deep Tunnel Plan represent viable alternatives for solving the problems of flooding and water pollution by handling the runoff from a 100-year storm.

"The Underflow-Storage Plan proposes the construction of a pattern of large tunnels in the dense Niagaran limestone rock formation, 200 to 300 feet below the surface waterway system. These tunnels would be sized to provide a linear distribution of storage volume and conveyance capacity in a pattern which would intercept all of the approximately 400 outfalls of the existing combined sewers. The tunnels would be sloped down to low points and pumping facilities opposite the existing sewage treatment plants. Overflow from the combined sewers, during storm periods, would drop through shafts to the large storage tunnels. In the post storm period, the tunnels would be dewatered by pumping directly to the existing treatment works."

"The Underflow-Storage Plan takes advantage of the lower water level to be established in the Illinois Waterway at Lockport, Illinois, for improvement of navigation and flood control of the waterway system. The new water level, 70 feet or more below the level of Lake Michigan, will allow the construction of tunnels with large underflow conveyance capacity to Lockport and provide flood protection for the largest storm of record."

"Storage of 18,000 acre-feet or 1.12 inches of runoff in the tunnel system will provide 98.5 percent reduction of pollutants entering the waterway from combined sewer spillages." (This represents sufficient capacity to handle the 100-year frequency storm.)

"During the study of the Underflow-Storage Plan, it was decided to modify a large relief sewer proposed by the City of Chicago, as an Underflow Sewer similar to the Metropolitan area-wide plan but on a much smaller scale. The Underflow Sewer would be constructed in solid rock, 250 feet below the ground surface. This sewer is now under construction with a portion being funded by a demonstration grant from EPA-WQO. Two additional Underflow Sewers are also under construction by the Metropolitan Sanitary District at widely separated locations in the Chicago area in the same dolomitic limestone rock formation. Each of the three Underflow Sewers are being mined by a machine of different manufacture. The construction of these Underflow Sewers has confirmed the structural integrity and the dense impermeability of this underlying rock blanket throughout the entire Chicago area."

"Prices per cubic yard of rock excavation vary from $60.00 for a 10-foot diameter single tunnel with two headings to $5.65 per cubic yard for cavern (room and pillar) excavation with multiple headings. The principal governing factor appears to be the size or face area of the headings. For the combined Underflow-Storage tunnels, with 26-foot wide by 50-foot high tunnel faces, the estimates are $8.81 per cubic yard for single tunnels and $8.03 per cubic yard for twin tunnels."

"The Deep Tunnel Plan is a multi-purpose plan, including hydroelectric power development, with a "pumped-storage" scheme, now widely used throughout the world as adjuncts to hydro-power developments on surface streams or to thermal power plants. In the Deep Tunnel Plan, storage for hydro-power would be provided in rock caverns, 600 feet or more below the surface and in surface reservoirs above ground in the vicinity of the underground caverns. Reversible pump-generator units would be used intermittently to move water upward and to develop power during downflow. Power would be generated and sold daily during the hours of peak demand for electricity. Power would be purchased for pumping daily, during the periods of low demand for other uses in the Metropolitan area. Based on an estimated net revenue, in excess of cost of operation, revenue bonds would be sold by the Metropolitan Sanitary District to provide capital for a portion of the multi-purpose project."

"The underground caverns and the surface reservoir would be over-sized beyond the needs for power development to provide for entrapment and storage of excess spillage from the combined sewer outlets. Primary sedimentation would be provided underground at the entrance to the caverns, and the sediment pumped to the existing treatment works. Controlled outflow from the surface storage would also be directed to the existing major treatment works."

"The total volumes of the proposed multi-purpose storage is 35,000 acre-feet below ground and 45,000 acre-feet above ground, or a total in the system of 80,000 acre-feet, of which 20,000 acre-feet was considered to be normally needed for power development, leaving 60,000 acre-feet normally available for pollution and flood control."

"The tunnel system to deliver the combined sewer spillage to the storage and power development site or sites would be generally of the same pattern as for the Underflow Plant."

The underflow tunnels under construction are compatible with future extensions along the conceptual lines of the Deep Tunnel Plan.

Other Storage Applications

Most of the applications of storage have been for treatment followed by release to receiving waters rather than by return to the sewer systems. The treatment may be removal of solids, or possibly some degree of stabilization, with or without disinfection by chlorination.

This type of storage was originally used at points along a sewer system where overflows would normally enter streams that have relatively low flows. Columbus, Ohio has been employing uncovered standby surface storm water tanks for 35 years. The tanks hold only part of the combined overflow, with the excess released to nearby streams (16). Because no means for frequent removal of settled solids was originally provided, odor problems were experienced when anaerobic decomposition occurred. Also, periodic cleaning of the settled sludge resulted in a serious overload on the activated sludge treatment plant, unless the cleaning operation could be stretched out over a week or so. However, facilities for regular removal of solids have been installed, and the odor and treatment plant overload problems have been minimized.

Halifax, Nova Scotia began the construction of two 1,000,000-gallon surface storage tanks in 1965 at a cost of $400,000 each (17). In contrast, complete separation was estimated at 4.3 million dollars, and partial separation (which would not prevent overflows in large storms) was estimated to cost 1 million dollars. These storage tanks, designed to provide 15 minutes retention at peak flow, will reduce the average frequency of overflow during the 4-month swimming season from fifteen overflows to two, and the volume of total overflow will be reduced by about 85 percent. Chlorination facilities will provide a dosage of 30 ppm for flows up to 40 cfs and a constant chlorine flow for the more diluted, higher flows. The city is now considering partial separation as a supplement to surface storage. This could be accomplished for one million dollars, and the retention tanks would then have capacity for the 5-year storm.

Other applications of storage for treatment are Grosse Point Woods, Michigan (18) (7.5 minutes detention of 1-year storm in surface tank, sludge removed by flushing with water); and Johnson County, Kansas (7) (30 minutes detention at maximum design rate). Planned applications are Milwaukee (19, 20) (underground, concrete tank with 3,900,000-gallon capacity providing 15 minutes of detention, screening of influent, and chlorination of effluent before discharge to river; Boston (19) (10-minute minimum sedimentation-chlorination, with chlorinated effluent discharged to river); and New York City (19) (system of basins designed to contain 25 of 40 summer storms, with overflow chlorinated to protect beaches). The reports of these planned applications did not identify the design storm.

For many years, British cities have been employing surface storm water tanks to store and treat combined sewer overflows. Their general practice is as follows (21), using British terminology:

1. Flows up to three times the normal dry-weather flow are treated at the municipal treatment plant.

2. Flows between three and six times the normal dry-weather flow are diverted to storm water tanks for sedimentation prior to discharge to the receiving stream. Any flow in excess of six times the normal dry-weather flow is discharged to the stream without any treatment.

3. Influent to the storm water tanks is screened (6-inch and 1-inch openings). Effluent is not chlorinated.

4. Minimum number of storm water tanks is two, and capacity must be sufficient to provide at least a two-hour detention period at the maximum flow (not disclosed) or must equal one half of a day's average dry-weather flow.

5. Settled solids and remaining wastewater are returned to the sewer systems for treatment when the storm subsides. Sludge can be removed by mechanical equipment or by flushing with municipal water.

Current studies supported by EPA/WQO research grants are investigating the use of polyelectrolytes and tube-type clarifiers to improve sedimentation in storage tanks (19). A study in Cleveland involves the feasibility of using storage basins for stabilization of combined sewer overflows (19); secondary effluent and flow from polluted streams would also be diverted to these basins. Little information is available now but there is enough to indicate that a higher degree of pollution abatement can be obtained with stabilization-retention basins than with sewer separation, and at approximately one-third of the cost of separation (22). The amount of land required, however, prohibits this application in many cities.

Effectiveness of Storage in Abating Pollution

Studies (10) at the EPA/WQO Robert A. Taft Sanitary Engineering Center (Cincinnati Water Research Laboratory) on sedimentation and concurrent sedimentation-disinfection of urban storm water runoff from a separately sewered residential and light-commercial area indicate that removal of 55 percent of suspended solids (normally expected of sanitary sewage in primary treatment) is not obtained within 20 minutes, but takes one hour of settling. The variations in removal appeared to be independent of the seasonal occurrence of the storms.

Studies in England and Canada (21, 23, 24) indicate similar results with combined sewer overflow. One consulting firm in Toronto, Canada maintains that a one-hour detention of the one-year storm overflow can effect a BOD removal of 30 percent and a suspended solids removal of 60 percent (23). For storms of less intensity, BOD reduction could be as high as 45 percent, and suspended solids removal could be 65 percent. None of these studies used polymeric flocculants or any other chemical additives.

Eliassen (25) investigated the aftergrowth of coliforms resulting from average discharge of combined sewer overflows into a brackish water tidal basin. He reported that, "Chlorination of the overflows to the 15-minute chlorine demand...will limit the peak aftergrowth to approximately 500,000 per 100 ml in 40 hours...Chlorination to the chlorine demand will result in average MPN aftergrowth values in the basin of from 10 percent to 30 percent of those which would develop if unchlorinated overflow were discharged to the river in the normal ranges of summer dilutions."

All studies stressed that their results, in general, cannot be applied to other sewerage systems. The composition of combined sewer overflow is extremely variable, varying from drainage area to drainage area as well as with intensity, duration, and frequency of storms and probably with the time of day. No data could be obtained which would correlate variations in composition with intensity and duration of rainfall. Suspended solids and BOD removal by sedimentation depend on concentration as well as on the period of detention. This further complicates establishing a pattern for chlorination efficiency, because it depends in part on the nature and concentration of the suspended solids.

Combined Sewer Overflow Treatment Processes

There are approximately 15 current EPA/WQO-funded R and D projects for investigation and/or demonstration of varied, non-conventional processes for treating combined sewer overflows; however, these projects have not been completed, and the available information is not conclusive (19). Very few of these treatment methods involve biological processes; most are concerned with screening or filtration. Some of the more interesting projects are listed in Table 11.

Microstraining, a form of simple filtration using specially-woven wire fabric mounted on the periphery of a revolving drum, is one of the more promising approaches. In this process, wastewater passes through a partially submerged drum from inside to outside. Downstream water is used to flush any solids that build up on the inside.

Preliminary results of microstraining at a demonstration site in Philadelphia indicated that a 30 to 60 percent removal of both BOD and suspended solids could be obtained with a 35-micron fabric; filtering rates were approximately 10 gpm per square foot (submerged). The contractor feels that better results are obtainable, because the overflows tested were pumped to the Microstrainer, and pumping should be avoided since it tends to break up any fragile solids, thereby reducing the efficiency of their removal.

The advantages of microstraining are: compactness; continuous and automatic operation (backwash is continuous and employs a non-clogging jet); the low head-loss of 12 to 18 inches; and economical operation. For best performance, a holding tank providing 10 to 20 minutes detention should precede the Microstrainer; this tank would tend to equalize the flow rates and would catch any heavy material, but it should not be designed as a sedimentation tank.

Miscellaneous Solutions

Besides sewer separation, off-system storage and in-line treatment, there are other possible solutions, which can be categorized as in-system storage, land-use improvements, and monitoring and regulations.

In-system storage is concerned with using the storage capacity of the sewer conduits themselves to hold the storm water and deliver it to the treatment plant over an extended period. This can be accomplished by reducing infiltration or friction to increase sewer capacity. It can also be accomplished by adding inter-connecting branch sewers or relief sewers. In-system storage will serve to prevent overflows from smaller storms, but is not practical for larger storms.

Some problems associated with combined sewers result from changes in zoning and land use, which in turn change surface characteristics to conditions for which existing sewer systems were not designed. Advanced urban planning on an area-wide basis and stricter control of land-use practices would assure continued efficient use of a sewer system and should lower the incidence of pollution in urban storm water runoff. A study to develop such a relationship has been conducted in Tulsa, Oklahoma (19).

In-system monitoring, with regulation and diversion, is one method being explored by Seattle, Detroit, and the Minneapolis-St. Paul Sanitary District (19). It assures maximum utilization of available capacity and minimizes the pollutional effects of any overflows. With proper instrumentation, the large flows could be diverted to those sewers with low flows; this is essentially a refinement of in-system storage. Instrumentation required includes flow measurement, rain-gauge telemetering, conduit liquid-level sensing, and remote operation of diversion gates. Computer-controlled operation would be required for rapid transmittal, recording, and feedback of data.

Table 11

WQO Combined Sewer Overflow Treatment Projects

Contractor	Project Site	Description of Treatment
City of San Francisco		Short-term sedimentation followed by dissolved air flotation and chlorination
Rand Corp.	Cleveland, Ohio	Percolation through a shallow bed of coal to filter coarser materials. Coal-solids mixture is later incinerated.
American Process Equipment Corporation	Los Angeles, California	Ultrasonic filtration.
Cornell, Howland, Hayes & Merryfield	Portland, Oregon	High-rate, fine-mesh, vibrating screens for removal of solids.
Autotrol	Milwaukee, Wisconsin	Biological treatment by a rotating biological contactor.
Rex-Chainbelt	Milwaukee, Wisconsin	Fine screening and dissolved air flotation for solids removal.
Crane Co.	Philadelphia, Pennsylvania	Microstraining, ozonation, and chlorination
Fram Corp.	Providence, Rhode Island	Strainer followed by self-cleaning diatomaceous earth filter.
Hercules, Inc.	Cumberland, Maryland	Self-cleaning filter of a flexible, filament-wound structure that will flex during storm flow to become permeable.
Hydrotechnic	New York, New York	Multi-media filtration
Battelle-Memorial	Richland, Washington	Activated carbon adsorption, chemical coagulation, sedimentation

Water quality monitoring of such characteristics as turbidity could assure that less-polluted flows are selected for discharge. This concept can be further refined by including techniques to provide selective discharge of overflows at different points in a manner which minimizes their pollutional effects on a waterway. Such a system is being studied in Seattle.

Another alternative method is to flush the sewerage system intermittently during dry-weather conditions with municipal water. This would flush out any solid matter which may have settled along the bottom of the sewer during the slow, dry-weather flows. The flushing flow must be controlled so as to not overload the hydraulic capacity of the sewage treatment plant. This method is being studied by FMC (19).

Summary of Previous Approaches

It is highly improbable that a community would have the economic capability to completely separate its combined sewers. Furthermore, complete separation could be accomplished only over a long period of time at great inconvenience to urban existence. Partial separation is less costly, but it probably would not provide the abatement required in many situations.

Extensive land areas are required to provide surface storage sufficiently large either to hold the larger overflows for later return to the sewerage system or to detain the flow long enough for proper sedimentation. Sub-surface storage appears to be far more promising, and it can also provide additional benefits beyond those of pollution control. It, too, would be an enormous project and feasible only if the bedrock is suitable.

Some treatment processes appear promising, but there is a dearth of basic information. EPA/WQO is supporting continuing studies in an effort to develop basic data which can be used to help work out economical solutions; however, few of these projects have yet produced definitive results. Even so, the extremely variable character of combined sewer overflows will inhibit general application of these data.

No particular method of solution has been proven to be the least costly for general application. Each city must evaluate its local situation and determine the method or combination of methods (and their priority) which will reduce the polluting impact of this problem to the desired level at the lowest cost and with the least inconvenience to the citizenry.

Ultra-High-Rate Filtration

General Discussion

High-rate filtration has been extensively applied for the removal of suspended impurities from raw water or wastewater, especially when the impurities are primarily non-volatile discrete particulates, such as the wastewater from steel mills. In combined sewer overflow and storm water discharge, large fractions of the suspended waste constituents are recognized to be non-volatile discrete solids; thus, high-rate filtration may be an effective treatment method. Despite the many studies that have been undertaken, the status of filtration development is still in transition from an art to a science. The practical design parameters for application of the filtration process to treatment of a specific wastewater must still be determined from results of specific laboratory or pilot-scale investigations. Moreover, ultra-high-rate filtration (greater than 15 gpm per square foot) must be applied in order to cope economically with the unique hydraulic characteristics of the combined sewer overflow or storm water discharge--high discharges within a short time period. This adds another dimension of uncertainty to the development of a feasible filtration process to treat the excess urban wastewater derived from intense storms. Therefore, a filtration study was conducted with the following objectives:

1. To evaluate the applicability of ultra-high-rate filtration to the treatment of combined sewer overflows.

2. To determine the flocculation effects of chemical additives on the removal of solids and organic material.

3. To provide a conceptual design basis for pilot-scale or full-scale treatment units.

The principal process variables evaluated in the laboratory program were:

1. Filter media, including type, depth, size, and arrangement.

2. Filtration rates.

3. Effects of addition of flocculants and flocculant.

4. Variation of solids concentration in the wastewater.

5. Backwash rate and quantity.

6. Air-souring rate, duration, and sequence in the backwash procedure.

7. Effluent quality characteristics, including suspended solids, COD, total five-day BOD, and soluble five-day BOD.

8. Length of filter run.

9. Head loss requirements.

Detailed discussion and results of ultra-high-rate filtration studies are presented in Appendix F and are summarized as follows.

Summary of Findings

The laboratory test program indicated that ultra-high-rate filtration, at rates of 15 or more gallons per minute per square foot, is a technically feasible process for the removal of solids and non-soluble BOD from combined sewer overflow. While actual combined sewer overflow was not used in these tests, a synthetic waste was made using an appropriate mixture of sanitary sewage, silt from the District area, and lake water. Of the three filter systems tested, the fiberglass filters performed best, achieving at least 90 percent removal of suspended solids and 70 percent removal of non-soluble BOD_5 at filtration rates of 15-30 gpm/sq.ft. and with reasonably long filter runs (1-3 hours). Comparable effluent quality was not achieved in tri-media filter runs at filter rates above 10 gpm/sq.ft. Upflow filtration through a garnet bed was unsatisfactory, largely on the basis of poor effluent quality.

Soluble BOD removal was negligible in all three filter systems. Even the addition of activated sludge to the influent wastewater did not significantly improve soluble BOD removal. The organic content (BOD_5, COD) of the influent wastewater appeared to have a greater impact on head-loss building than did the suspended solids content. The addition of flocculants and flocculant aids was not effective in improving the performance of the fiberglass filters.

The economic feasibility of the fiberglass filter process for ultra-high-rate filtration will probably require extending the useful life of the fiberglass medium beyond the limits indicated by the laboratory tests. Improvement of the backwash operations through modification of underdrain design, stagewise removal of backwash effluent, the use of air-scouring driven backwash, etc., and development of improved fiberglass bed designs and fiberglass filter regeneration techniques appear to be promising approaches to extension of filter life.

SECTION VI

DEVELOPMENT OF FEASIBLE ALTERNATIVES

Alternative Approaches

After thorough review and interpretation of the pertinent information, four methods of abating pollution from combined sewer overflows appeared to offer sufficient promise to justify consideration as alternative approaches for the District:

1. Sewer Separation.

2. Storage Reservoirs (with treatment at the Blue Plains Plant or other centralized facilities after the storm subsides).

3. Treatment at Overflow Points.

4. Tunnels and Mined Storage (with treatment at the Blue Plains Plant or other centralized facilities after the storm subsides).

Sewer Separation

As the name implies, this approach consists of complete separation of storm and sanitary sewers. The initial steps of this program involve continuing the sewer separation program of Project C as modified by the City of Washington. The details of Project C are contained in the 1957 report (1) of the Board of Engineers. Storm water would be discharged to the surface streams through what is now the combined sewer system, and sanitary wastewater would be conveyed through a new sewer system to the Blue Plains sewage treatment plant for treatment.

Storage Reservoirs

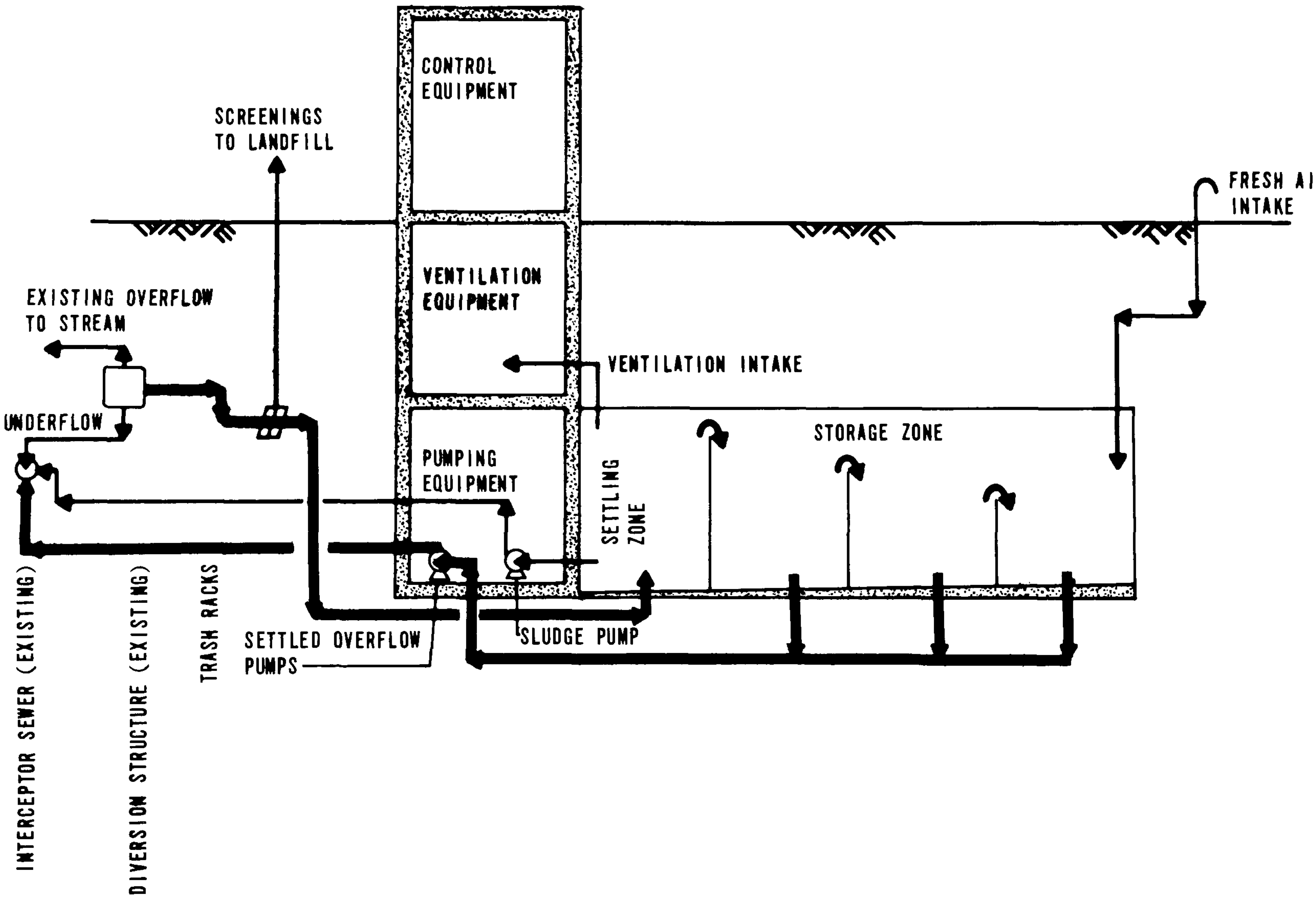
The concept in this approach is to provide sufficient underground storage volume to hold the combined sewer overflows caused by each storm until the storm subsides and then to pump the stored wastewater back into the sewerage system for conveyance to a centralized treatment plant. This would be accomplished by the construction of shallow, multi-cell, concrete tanks located five to ten feet underground at reservoir top. Depth of each reservoir would depend on land availability and volume to be stored; however, there are various technical, economic, and aesthetic factors that prevent the use of storage reservoirs in certain locations.

Each reservoir would be compartmented. The initial compartment would function as a settling chamber to remove grit and heavy solids, with the overflow going to other compartments for storage. The number of compartments used during any storm would depend on the rainfall intensity, the duration of the storm and the amount of stored water remaining from any previous storm A schematic of a typical installation is shown in Figure 14.

Each reservoir would be equipped with trash racks, a system for flushing sediment from the reservoir bottom, a ventilation system, pumps for returning stored wastewater into the regular sewer system and special pumps for transferring accumulated sludge back into the system or to tank trucks for disposal.

FIGURE 14
STORAGE RESERVOIRS

STORAGE RESERVOIR SCHEMATIC (TYPICAL)



Due to the extensive urban development within the District, little vacant land is available near overflow points for storage reservoirs. In certain areas of the District scheduled for redevelopment (e.g., Georgetown Waterfront), storage reservoirs could be constructed under planned open spaces and coordinated with the razing phase. In most other overflow areas, the only available vacant land is park land under the jurisdiction of the National Park Service; full cooperation of the Park Service would be necessary to use these sites for reservoir purposes. Since the reservoirs themselves would be underground, only relatively small pump houses would extend above grade, and the surface area could be used for parks and playgrounds. Sufficient care to preserve the natural park landscape by proper architectural design of both the pump houses and maintenance-access structures would tend to offset objections of the National Park Service.

Treatment at Overflow Points

In this method, there would be a treatment facility at each existing overflow point, except where conditions either prevent this or dictate that certain overflow points be combined.

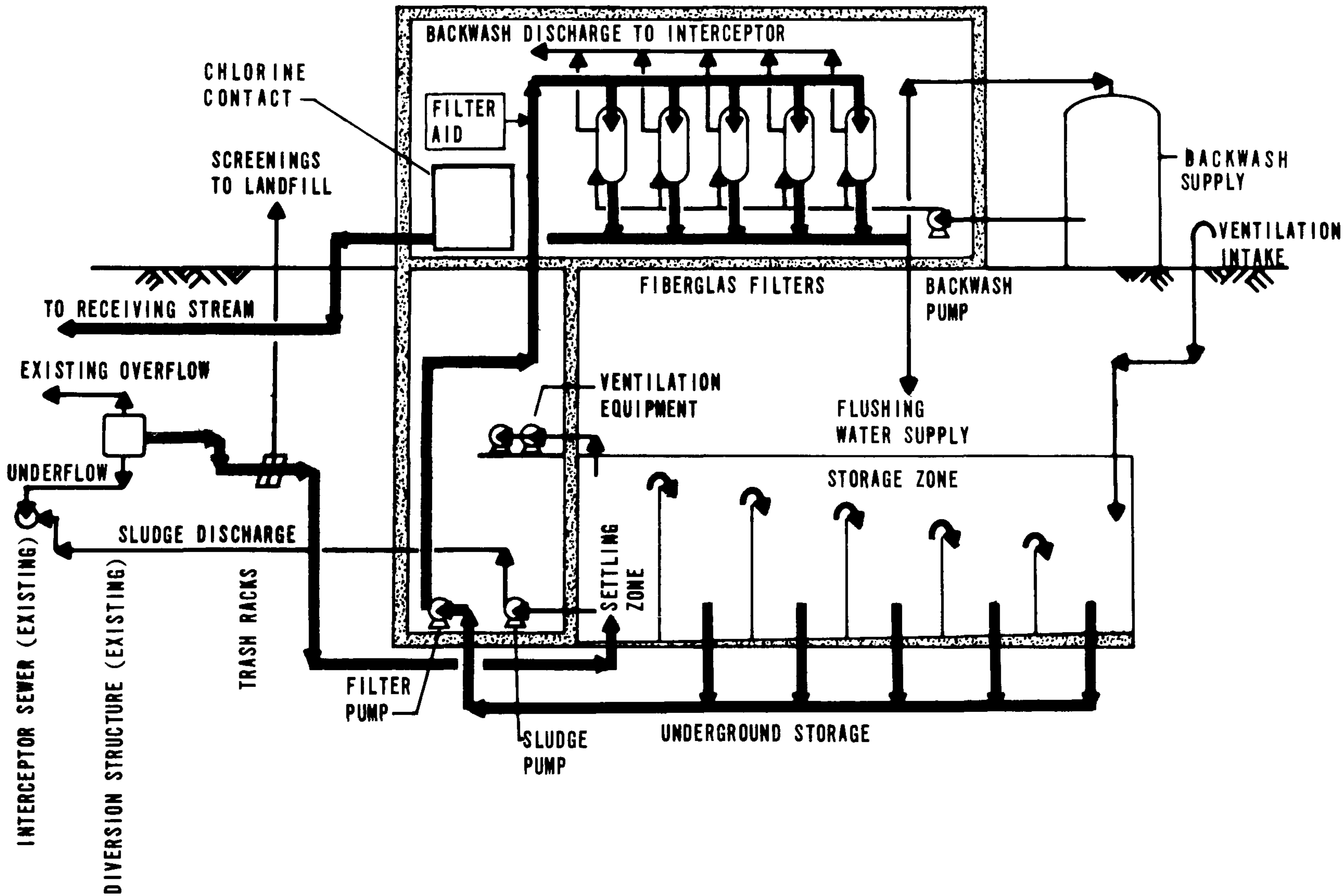
The treatment sequence would be sedimentation in the storage reservoirs, followed by ultra-high-rate filtration and chlorination. A schematic flow diagram for a typical treatment facility is presented in Figure 15. The individual filters would be ultra-high-rate pressure filters similar to those used in the treatment of industrial water supplies. Each installation would include a battery of filters, the number of which would vary depending on the design flow rate at each location. In addition to the filters, various pumping, chemical-mixing, and flocculant-aid equipment would be needed.

The filtered wastewater would be disinfected (by chlorination) and discharged to the surface streams. However, a portion of the filtrate would be stored for use as backwash, initiated when a predetermined head is reached at any individual filter. The backwash from the filters would be pumped back into the regular system for conveyance to the Blue Plains treatment plant. Backwash requirements were estimated to be approximately eight percent of the forward flow.

In addition to actual treatment facilities, each installation would require equalization storage because of the extremely high flow rates that would be encountered. (The smallest sewer district would require a treatment facility with a capacity of several million gallons per day if no equalization storage were provided.) Even with the high rates (15 gpm/sq.ft.) involved in ultra-high-rate filtration, the capacity required for equalization would be essentially the same as that required to capture the entire storm overflow. Therefore, the actual treatment facilities in this alternative would have to be supplemented by the same storage capacity required in the storage reservoir method.

The capacity of any treatment facility, since it determines the drawdown rate, is related to the probability of overflow from a subsequent storm exceeding the capacity of a storage facility. Regardless of any selected capacity of a storage facility or treatment facility, it is possible that the combined effect of two or more storms, occurring within a relatively short period of time, will result in runoff in excess of the reserve storage capacity, even though the separate runoff from each storm is less than the design capacity. The capacity of the treatment facility should be sufficiently large so that the probability of two consecutive storms overflowing a storage facility is reasonably low. A preliminary statistical analysis suggests that the maximum capacity that can be considered reasonable is one that draws down a filled storage facility within five days. A drawdown rate of five days, as compared to 10 days, certainly reduces the possibility of malodors resulting from long residence times.

FIGURE 15
TREATMENT AT OVERFLOW POINTS
TREATMENT FACILITY SCHEMATIC (TYPICAL)



Tunnels and Mined Storage

In this concept, combined sewer overflow would drop through a vertical shaft down to an underground system of tunnels and mined storage. The tunnels would convey the overflow at high velocity to mined storage. After the storm subsides, the retained overflow would be pumped back into the regular sewers for conveyance either to the Blue Plains sewage treatment plant or to a separate, centralized facility constructed specifically to treat combined sewer overflow.

Vertical Shaft Considerations

In a tunnel-mined storage system, each of the overflow points would have a vertical shaft for dropping the overflow to the underground system. Certain vertical shafts may be used for purposes other than the primary function to convey overflow to the tunnels. The shafts used for access of men and equipment and for removal of drilling muck would be larger than those to be used for conveyance of overflow.

The larger-diametered (20 and 30-feet) access shafts would be constructed by conventional techniques (i.e., by drilling and blasting, and by lining with either jacking or slip-form construction). The smaller (5-foot diameter) shafts would be constructed by simply augering down from grade to the storage tunnels or by raise-boring of a pilot hole from the tunnel, followed by pulling a larger-diameter auger up from the tunnel.

Each shaft would be concrete-lined (for structural purposes and to prevent ground water from entering the system) and would have a baffle to permit the escape of entrained air to the atmosphere. The tunnel bottom at the junction with each vertical shaft would be designed to dissipate the energy of the falling water and to provide transition from the vertical shaft to the conveyance tunnels.

Land requirements for shaft construction (and, therefore, tunnel construction) would be limited to the land needed for storing construction equipment and a small amount of excavated material. Preliminary site investigation has shown that sufficient land is available at nearly all reasonable vertical shaft sites and that only slight modifications to the existing sewer system would be required to connect the overflow points to the appropriate vertical shafts.

Tunnel Considerations

The tunnels would be concrete-lined and constructed in bedrock at sufficient depths to assure the structural integrity of the foundations of all existing bridges, buildings, or monuments. They would also be located so as to avoid interference with any planned underground facilities of the Washington Metropolitan Area Transit Authority.

In general, the bedrock geology of the area involved in the proposed storage tunnel construction meets the significant criteria for successful rock tunneling. As depth increases, the rock types tend to become less weathered, and the secondary openings become tighter and less frequent. The Washington area is relatively stable tectonically, and no problems traceable to rock deformation or faulting are anticipated. The geology of the area and its impact on tunneling are discussed in more detail in Appendix A.

Material excavated during tunneling would be transported to the surface through the vertical shafts. Depending on its quality, this material could be used for landfill or as concrete aggregate. If so used, it could substantially reduce the construction cost of this alternative, but in the cost comparisons to be presented in a later section of this report, the excavated material will be considered as being hauled away as waste.

Recent developments in the tunnel construction and the tunneling equipment fields contribute to the feasibility of this approach and to the realistic consideration of available maximum tunnel diameters. Tunneling equipment, or "moles", capable of boring 28-foot diameter tunnels are already in use, and those providing 32-foot diameter bores are in the design stage and should be ready within a few years. Therefore, the size of the conveyance tunnels would present no added problem in the drilling operations.

The Conveyance Tunnel/Mined Storage approach has many of the conceptual features of the Chicago Deep Tunnel Plan. The Chicago Plan also incorporates a pumped storage hydroelectric generating system to provide additional electric power during the peak consumption periods. However, preliminary investigation has indicated that such supplementary power generation is not economically feasible for the Washington D.C. area. Therefore, power generation has not been included in the Conveyance Tunnel/Mined Storage alternative.

Mined Storage Considerations

The mined storage would consist of a network of criss-crossed chambers at atmospheric pressure. Conventional deep-mining methods (i.e., drilling and blasting to remove the rock) and mine-railroad or rubber-tired earth-moving equipment to convey the excavated material to the access shaft would be used in construction of this mined-storage area. Storage area would be compartmented, with some compartments used as settling basins to facilitate solids removal. The chambers would be concrete-lined only in areas where faults or fractures would otherwise permit excessive inflow of ground water. Final design, as with the final design of tunnels, would depend on data collected during sub-surface geological investigation.

Pumping Station Considerations

A pumping station would be included with each storage system either at the bottom of one of the vertical shafts, or in a specially-excavated chamber nearby. The shaft involved would have elevators, discharge piping, ventilation, and electrical conduits. The design of the pumping system would be based on the capability of the existing sewer system to handle the stored overflow after the storm subsides. This capability in turn would be influenced by variations in the dry-weather flow, and the pumping of the stored overflow would have to be programmed to coordinate with the off-peak hours of normal dry-weather flow.

Solids Removal Considerations

If the tunnels are constructed on a reasonable slope, sufficient flow velocity will result to convey most of the solids to the mined-storage area. The initial chambers in the mined-storage area will provide sedimentation and will concentrate the deposition of solids there rather than uniformly over the entire bottom of the mined-storage area. This should permit effective removal of at least the lighter solids by pumping. If the removal of heavier solids in this manner presents a problem, their removal may be effected at the point of overflow by using vortex separators (compact, cyclone-type equipment that remove grit with centrifugal force). The separated grit would be discharged as a slurry to a classifier which washes and dewaters the grit for pickup by truck.

Application of Alternative Approaches to D.C.

General

It is especially fitting to evaluate each of the four approaches by individual sewer drainage basins because the method selected to deal with the combined sewers in one part of the District does not have to be the same method used in another part. Actually, the appropriate solution to the total combined sewer problem in the District may include features from each of these four methods.

Although it is possible to apply each of these four approaches at any sewer drainage district, physical and economic factors dictate that only one or two methods be employed for each district. The very large volume of overflow from certain districts in combination with existing land use requires the use of mined storage rather than reservoir storage if a storage method is used. The proximity of some overflow points or existing land use suggests that certain districts be interconnected. Because the available or proposed interceptor capacity is quite large in places, it is appropriate to use this capacity to convey combined sewer flow from certain districts during storm conditions to a more centralized storage facility at a subsequent point in the sewer system.

For the purpose of this evaluation, the District was divided into two large drainage basins, Anacostia and Rock Creek-Upper Potomac. The basic layout of the alternative systems was based on retaining and treating the overflow from the 15-year, 24-hour storm (see Table 5); however, the same design should apply to other major return frequencies with the exception that certain dimensions (e.g., tunnel diameter) will change.

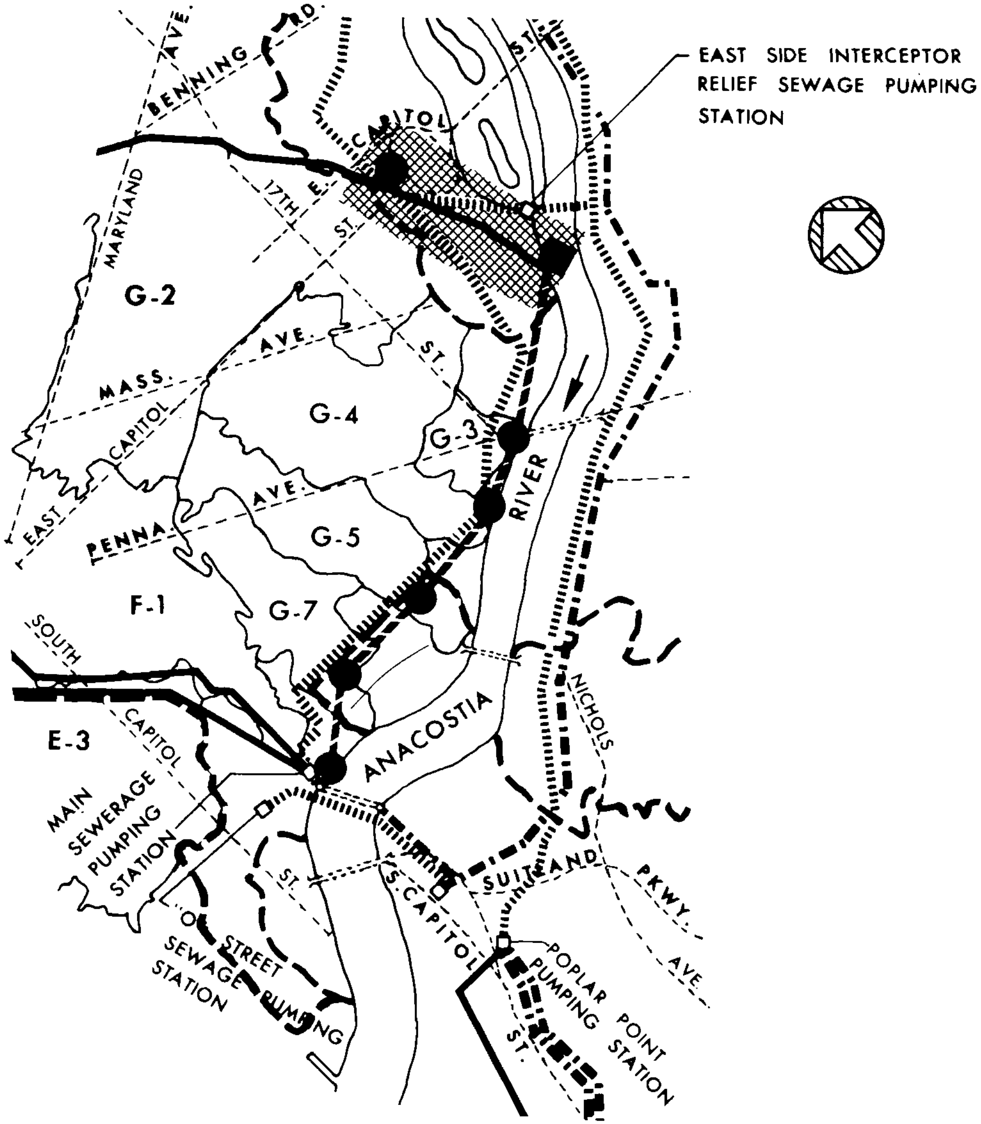
Anacostia River Area

Conveyance Tunnels and Mined Storage

The basin layout for this approach is shown in Figure 16. Essentially the plan proposes the dropping of all overflows through vertical shafts into conveyance tunnels that empty into a single mined-storage area near the Robert F. Kennedy Stadium. As mentioned in the description of the present D.C. sewer system (in Section IV of this report) and as shown in Table 2, the Northeast Boundary Trunk Sewer does not have sufficient capacity to convey the runoff from a 15-year storm. The District of Columbia currently has preliminary plans to construct a relief sewer in the Northeast Boundary area at a cost of $33,600,000 (1968 dollars, ENR = 1117); however, the relief sewer would behave as a combined sewer for a considerable period of time until sewer separation was completed. Even after complete separation, the relief sewer would discharge with each storm a considerable pollution load represented in the runoff from a drainage area one half as large as the G-2 district.

The construction of storage facilities in the upper and central reaches of the G-2 sewer system would not only provide additional storage capacity, but more importantly, could also provide sufficient relief to ease surcharging during the 15-year storm. In contrast with the relief sewer, the relief/storage system has the added feature of preventing discharge of untreated runoff and sanitary sewage to surface streams.

A recent study (6) proposed a combination relief/storage tunnel in the central part of the sewer district as presented in Figure 17. This tunnel would reduce most of the



LEGEND

- POLITICAL BOUNDARY
- COMBINED SEWERS
- SANITARY SEWERS
- OTHER EXISTING SEWERS (COMBINED AND SEPARATED)
- A-10 SEWER DISTRICT NUMBER
- LIMITS OF COMBINED SEWER SYSTEM IN 1970
- SEWER DISTRICT BOUNDARIES
- PUMPING STATIONS
- TUNNEL
- VERTICAL SHAFT
- PUMPING STATION
- MINED STORAGE

NO.	NAME
E-3	B ST.-NEW JERSEY AVE. TRUNK SEWER D/S PORTION
F-1	TIBER CREEK
G-2	NORTHEAST BOUNDARY
G-3	BARNEY CIRCLE
G-4	14TH ST.-PENNA. AVE.
G-5	12TH ST.-9TH ST.
G-7	6TH ST.-7TH ST.

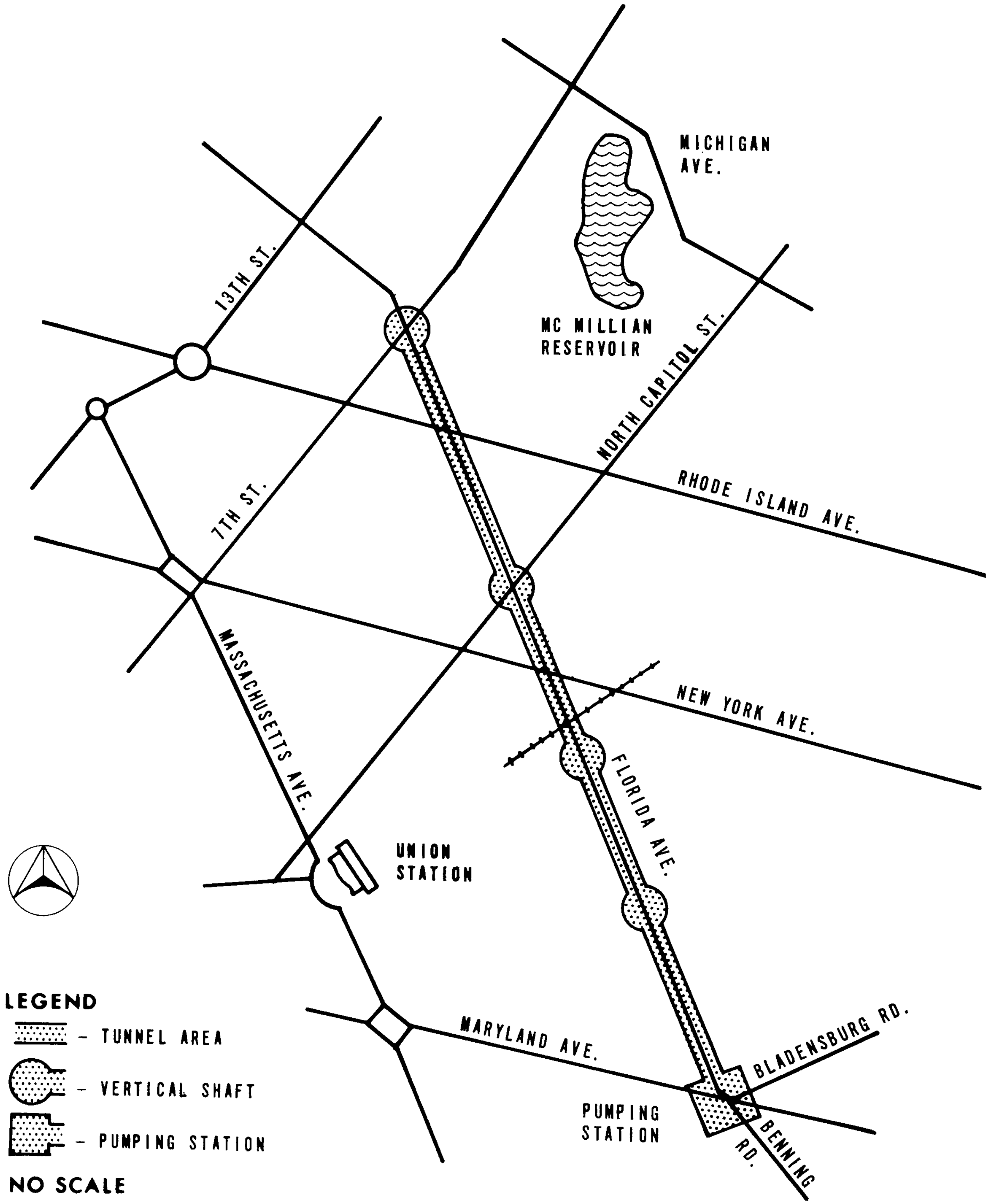
1500 0 1500 3000 4500 6000
SCALE IN FEET

DISTRICT OF COLUMBIA

TUNNELS AND MINED STORAGE IN ANACOSTIA RIVER BASIN

FIGURE 16

FIGURE 17
LOCATION OF NORTHEAST BOUNDARY RELIEF/STORAGE TUNNEL



surcharging in this sewer district; however, it would not provide total relief. It is conceivable that construction of only part of the proposed relief sewer or some strategically located storage reservoirs, in combination with the relief/storage tunnel, could prevent surcharging at a lower cost than the entire relief sewer. It is also possible that a different tunnel layout, with tunnels running perpendicular to the trunk sewer but beneath critical branch sewers, may also prevent surcharging at a lower cost.

The other basic components of the tunnel and mined-storage approach include:

1. A 20- to 22-foot I.D. tunnel extending from the Main Sewerage Pumping Station on the west bank of the Anacostia River northeastward (parallel to the river) to the vicinity of the Robert F. Kennedy Stadium. The total length of the Anacostia conveyance tunnel would be approximately 10,600 feet. The first 3,900 feet (starting from the Main Sewerage Pumping Station) would be 20 feet I.D., and the remainder would be 22 feet I.D. Figure 18 provides a profile of this tunnel.

2. A 3,700,000-cubic yard mined-storage area in bedrock near the Robert F. Kennedy Stadium. This underground storage facility, to be located beneath the parking lot of the Robert F. Kennedy Stadium, would eliminate the need for a separate tunnel or other facility to convey the extremely large flow from the Northeast Boundary Trunk Sewer. This location for the mined storage would also relieve portions of the Northeast Boundary Trunk Sewer during periods of high flow.

3. An underground pumping station and a series of vertical overflow and access shafts.

Storage Reservoirs

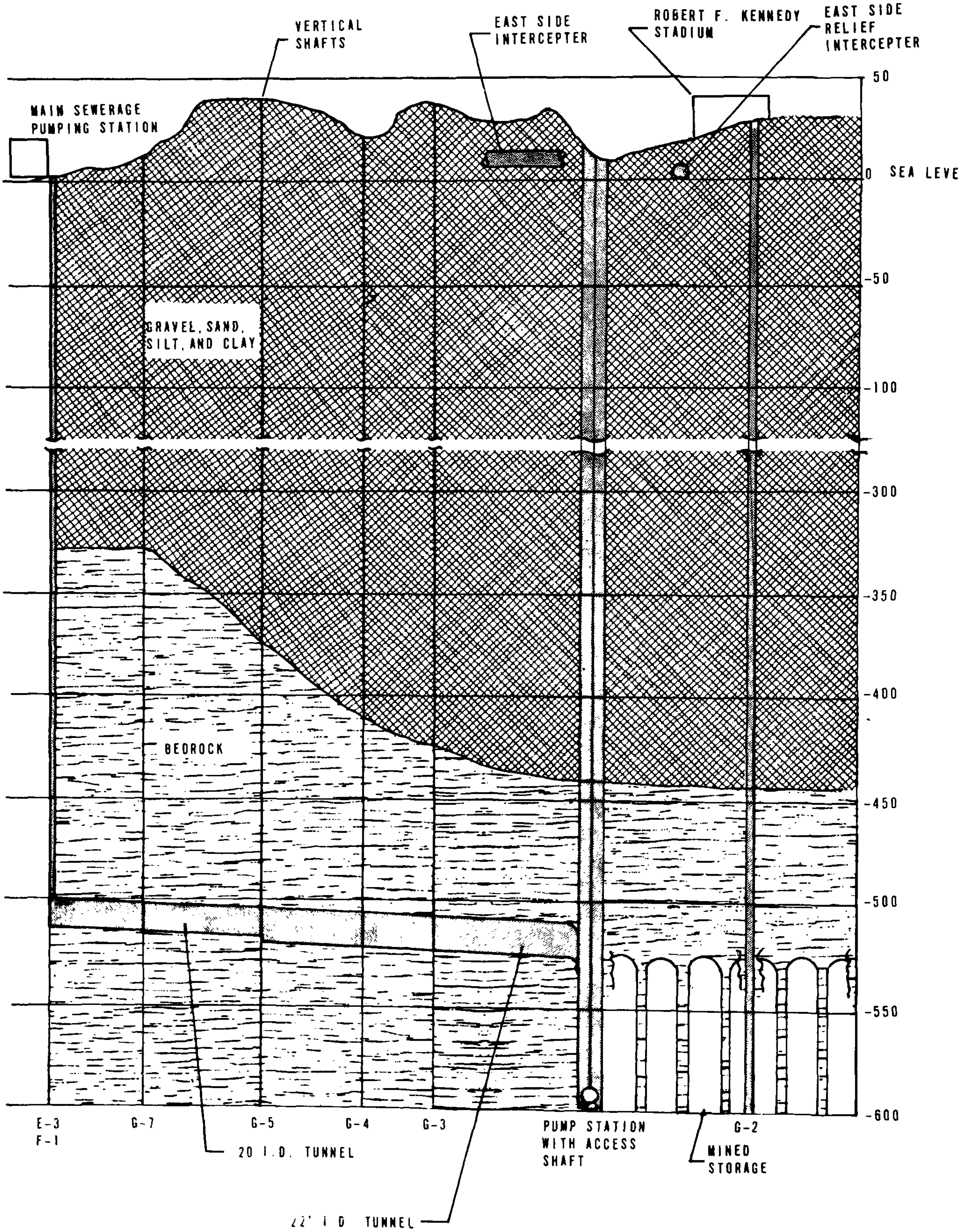
Figure 19 shows the basic layout of a storage system incorporating shallow underground reservoirs wherever they are reasonable. Storage reservoirs are not applicable throughout the Anacostia River area; they would be attractive only for the Sewer Districts with smaller flows, namely G-3, G-4, G-5, and G-7. The required capacities for these concrete tanks range from 800,000 gallons to 28,000,000 gallons.

The very large volume (490 million gallons) and overflow rate (5.5 billion gallons per day) from sewer district G-2 dictate the use of an underground mined-storage chamber beneath the overflow point near Robert F. Kennedy Stadium as the most effective means of capturing the overflow from the Northeast Boundary Trunk Sewer. This mined-storage area is similar to the mined-storage area proposed before, except that the capacity is not as large.

For Sewer Districts F-1 and E-3, existing land use and the high volume (230 million gallons) of overflow for the outfall at the Main Sewerage Pumping Station dictate that the overflow at these two structures be conveyed to the east side of the Anacostia River for retention in a mined-storage facility. These mined-storage areas are suggested only if storage reservoirs are utilized in the Anacostia River; otherwise, a single larger mined-storage area near the stadium is suggested.

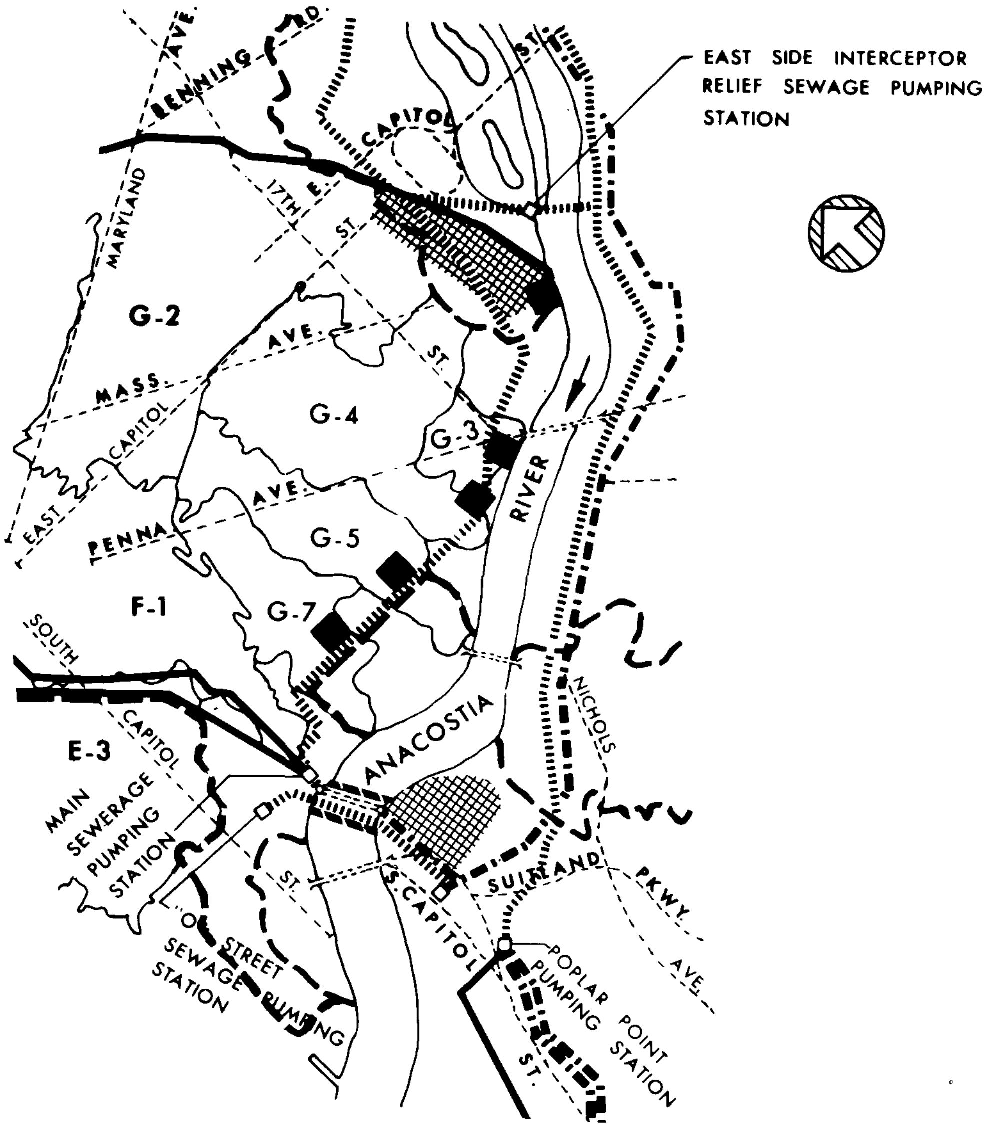
FIGURE 18

CONVEYANCE TUNNELS AND MINED STORAGE
ANACOSTIA RIVER TUNNEL PROFILE (SEE FIGURE 16 FOR PLAN)



FIG

18



LEGEND

- POLITICAL BOUNDARY
- COMBINED SEWERS
- SANITARY SEWERS
- OTHER EXISTING SEWERS (COMBINED AND SEPARATED)
- A-10 SEWER DISTRICT NUMBER
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- STORAGE TUNNELS
- MINED STORAGE

NO.	NAME
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G-5	12TH ST.-9TH ST.
G-7	6TH ST.-7TH ST.

1500 0 1500 3000 4500 6000
SCALE IN FEET

DISTRICT OF COLUMBIA

MAXIMUM USE OF STORAGE RESERVOIRS IN ANACOSTIA RIVER BASIN

FIGURE 19

The relief/storage tunnel in the central reach of the Northeast Boundary Trunk Sewer is required in view of the surcharging frequency of this sewer; however, in this approach, total use of storage reservoirs rather than a tunnel may be desirable.

An independent study (6) of the G-2 Sewer District has identified a feasible location for a large storage reservoir near the Robert F. Kennedy Stadium. This location could be developed into a storage capacity of 235 million gallons. While this is not sufficient for the overflow from the 15-year, 24-hour storm, this storage capacity plus the capacity of the relief tunnel would be more attractive than mined storage for design storms of less overflow volume.

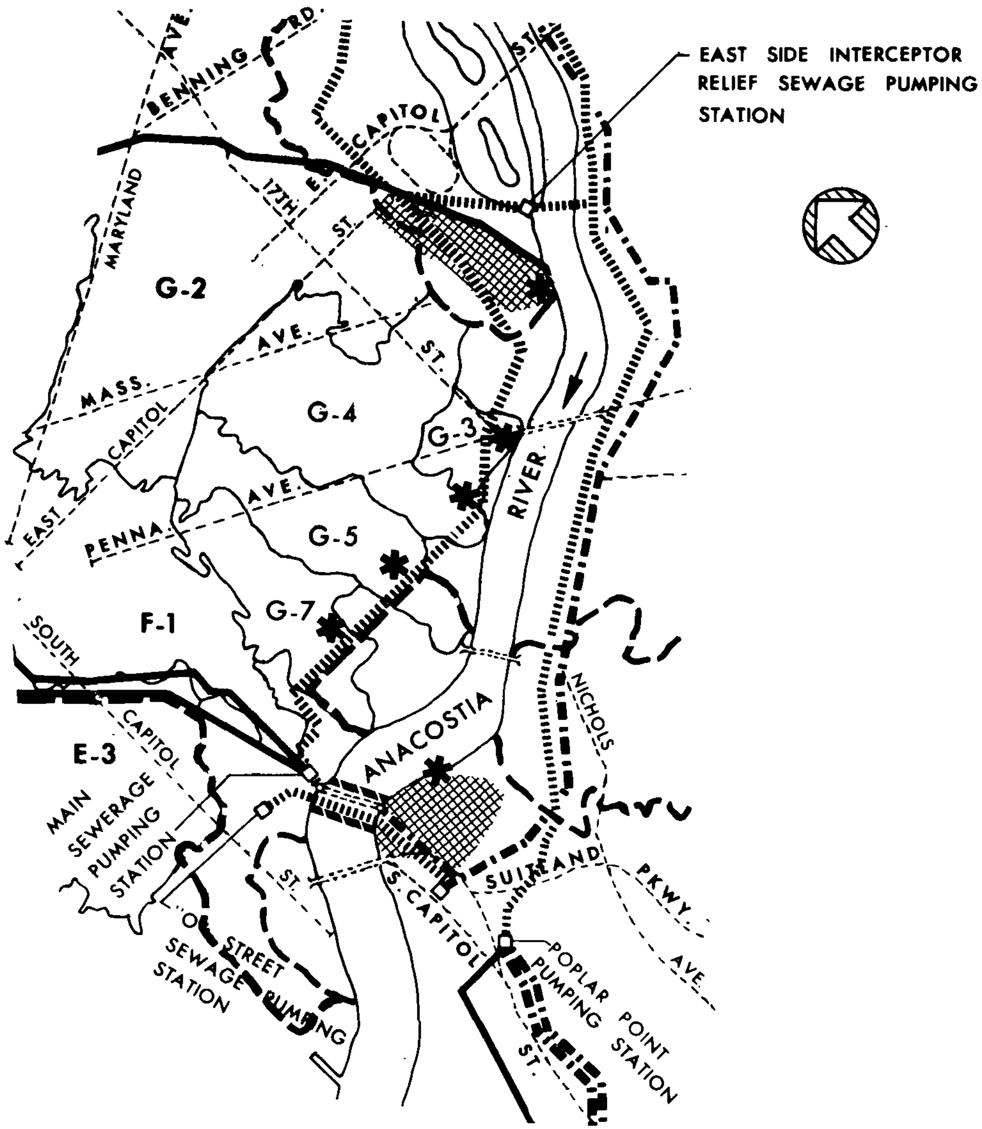
Treatment at Overflow Points

The conceptual design of this approach essentially follows the layout for the storage reservoir approach, except that treatment plants are required near points of storage. The locations of the treatment facilities are shown in Figure 20. As stated before, the treatment plant capacities are sufficiently large to draw down each filled storage facility within five days.

The most attractive feature of this method is that is provides facilities to treat stored overflow. As mentioned in the discussion of the D.C. sewage treatment facilities, the average dry-weather flow to the only sewage treatment facility within the District will exceed the ultimate capacity at this plant in 1977. While the Blue Plains plant can handle for short durations flow in excess of this ultimate capacity, the plant will not be able to fully treat all increased flows when stored overflow is returned to the sewer system unless some of the dry-weather flow to this plant is diverted and treated elsewhere. Hence, treatment facilities at overflow points will probably be needed.

A recent study (6) has suggested an approach for the Northeast Boundary Trunk Sewer District (G-2) similar to the method of treating at overflow points, except that the project would serve many purposes other than pollution abatement. The project is referred to as the Kingman Lake Project. It envisions the retention of overflow from the G-2 sewer district in a surface reservoir, followed by treatment of the stored overflow and reuse for recreational purposes, e.g., swimming and fishing. The degree of treatment required at the Kingman Lake treatment facility is obviously higher; but other than this, the features of the Kingman Lake project follow exactly the basic concepts suggested in this study, i.e., relief tunnel and surface storage, possibly augmented with mined storage. The EPA/WQO, the National Park Service, and the District of Columbia have shown considerable interest in this project since it would bring a much needed recreational park to the urban dweller and would rid the urban area of a serious source of pollution.

Another attractive feature of local treatment, is that treatment plants can be located underground near some of the artificial pools in Washington that sometimes are polluted by storm water runoff. Treatment of the pool water will restore their beauty rather than let debris and biotic growths accumulate.



LEGEND

- POLITICAL BOUNDARY
- COMBINED SEWERS
- SANITARY SEWERS
- OTHER EXISTING SEWERS (COMBINED AND SEPARATED)
- A-10 SEWER DISTRICT NUMBER
- LIMITS OF COMBINED SEWER SYSTEM IN 1970
- SEWER DISTRICT BOUNDARIES
- PUMPING STATIONS
- TREATMENT FACILITIES
- STORAGE TUNNELS
- MINED STORAGE

NO.	NAME
E-3	B ST.-NEW JERSEY AVE. TRUNK SEWER D/S PORTION
F-1	TIBER CREEK
G-2	NORTHEAST BOUNDARY
G-3	BARNEY CIRCLE
G-4	14TH ST.-PENNA. AVE.
G-5	12TH ST.-9TH ST.
G-7	6TH ST.-7TH ST.

1500 0 1500 3000 4500 6000

SCALE IN FEET

DISTRICT OF COLUMBIA

TREATMENT AT OVERFLOW POINTS IN
ANACOSTIA RIVER BASIN

FIGURE 20

Rock Creek Upper Potomac Area

Conveyance Tunnels and Mined Storage

The basic layout for this approach is shown in Figure 21. Essentially the plan proposes the dropping of all overflows through vertical shafts into conveyance tunnels that empty into a single mined-storage area near Water Gate. The basic components of this system include:

1. A 15- to 18-foot I.D. tunnel extending from the outfall of the Piney Branch Trunk Sewer in a southerly direction (approximately parallel to Rock Creek) to the vicinity of the Water Gate. The total length of the Rock Creek conveyance tunnel would be approximately 19,800 feet, of which the first 4,600 feet (starting from D-4) would be 15 feet I.D. The profile of this tunnel is shown in Figure 22.

2. A 2- to 7.5-foot I.D. sewer/tunnel extending from the intersection of 36th Street NW and the Potomac River eastward to intersect with the Rock Creek Tunnel near New Hampshire Avenue NW (see Figure 23). The Upper Potomac tunnel would be 5,300 feet long, of which the first 1,600 feet would be 24-inch sewer pipe and the remainder 7.5-foot I.D. tunnel.

3. A 1,100,000-cubic yard mined-storage area in bedrock beneath Water Gate plus a pumping station.

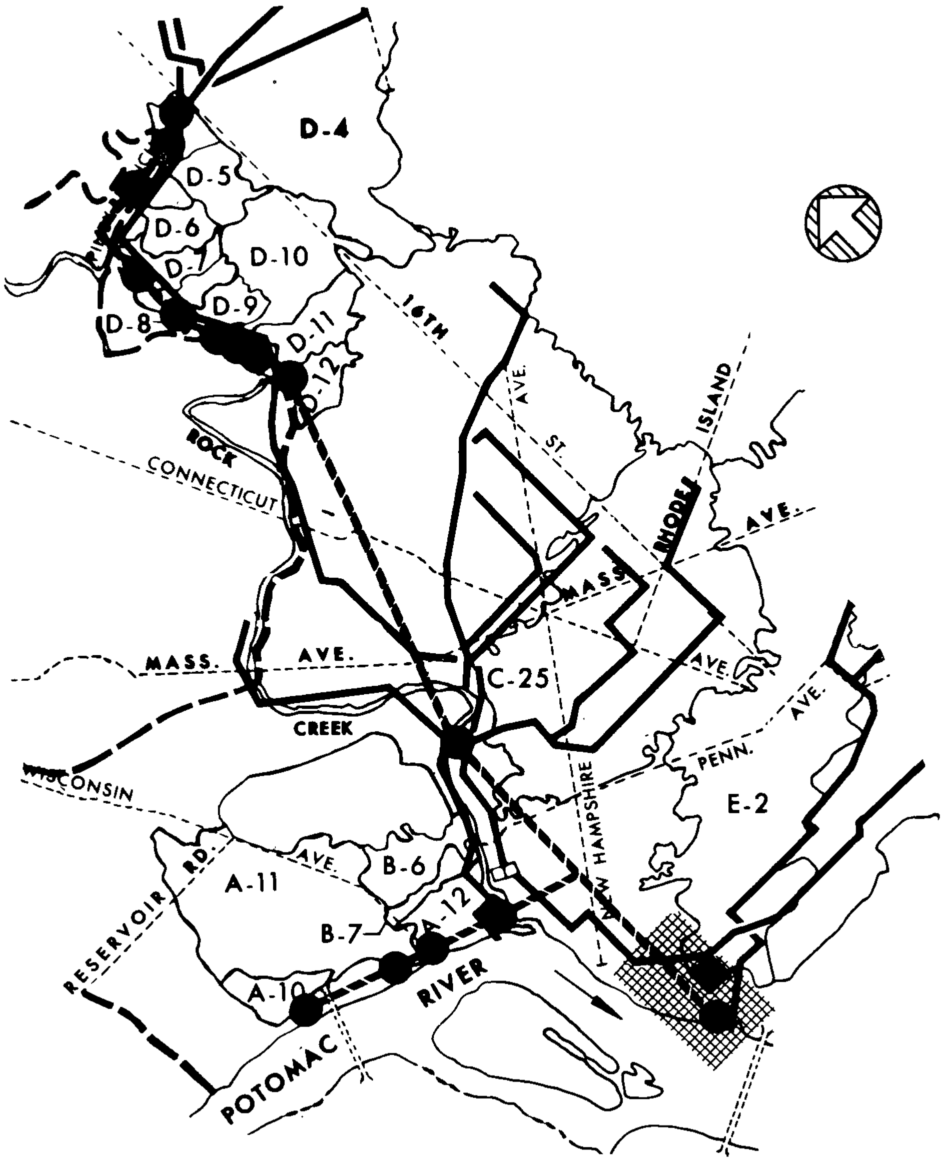
4. A vertical shaft at each point of overflow. Three 30-foot diameter access shafts are contemplated, one each in sewer districts A-10, D-4, and E-2. The E-2 shaft would have the additional function of providing space for the pumping equipment. There will be no overflow from a number of sewer districts along the eastern lower part of Rock Creek (e.g., Districts D-13 through D-16 and part of C-25) due to the large interceptor capacity available and the relief resulting from capture of flow upstream. Any plan in the Rock Creek Area should propose routing this flow directly to the Blue Plains plant rather than retain it in any storage facility.

Storage Reservoirs

Figure 24 shows the basic layout of a storage system incorporating as many underground storage reservoirs as reasonable. Storage reservoirs offer promise in the vicinity of Georgetown; however, in other parts of the Rock Creek-Upper Potomac area, tunnel tunnel storage and mined storage are indicated.

Storage reservoirs would be attractive only for the sewer districts with smaller flows: A-10, A-11, A-12, B-6, B-7, and C-25. Because of the present land use in the vicinity of the A-11 overflow point, it would be necessary to convey the overflow to the A-10 overflow point for reservoir storage or treatment. Likewise, districts B-6 and B-7 should be combined. The required capacities for these concrete tanks range from 3,000,000 to 21,000,000 gallons.

The large volume (160 million gallons) and high flow rate (2.7 billion gallons per day) from the overflow point at Piney Branch (D-4) indicate that mined storage or a network of underground tunnels would be the most effective method of handling the storm



LEGEND

POLITICAL BOUNDARY
COMBINED SEWERS
SANITARY SEWERS
OTHER EXISTING SEWERS (COMBINED AND SEPARATED)
A-10 SEWER DISTRICT NUMBER
LIMITS OF COMBINED SEWER SYSTEM IN 1970
SEWER DISTRICT BOUNDARIES
PUMPING STATIONS
TUNNEL
VERTICAL SHAFT
PUMPING STATION
MINED STORAGE

NO.	NAME
A-10	37TH ST.-GEORGETOWN
A-11	GEORGETOWN
A-12	K ST.-WISCONSIN AVE.
B-6	M ST. -27TH ST.
B-7	28TH ST.-WISCONSIN AVE.
C-25	SLASH RUN (PART)
D-4	PINEY BRANCH
D-5	OAK ST.-MT.PLEASANT ST.
D-6	INGLESIDE TER.
D-7	PARK RD.
D-8	LAMONT ST.
D-9	KENYON ST.
D-10	IRVING ST.
D-11	QUARRY RD.
D-12	ONTARIO RD.
E-2	EASBY POINT

1500 0 1500 3000 4500 6000
SCALE IN FEET

DISTRICT OF COLUMBIA

TUNNELS AND MINED STORAGE IN UPPER POTOMAC-ROCK CREEK BASIN

FIGURE 21

FIGURE 22
CONVEYANCE TUNNELS AND MINED STORAGE
ROCK CREEK TUNNEL PROFILE (SEE FIGURE 21 FOR PLAN)

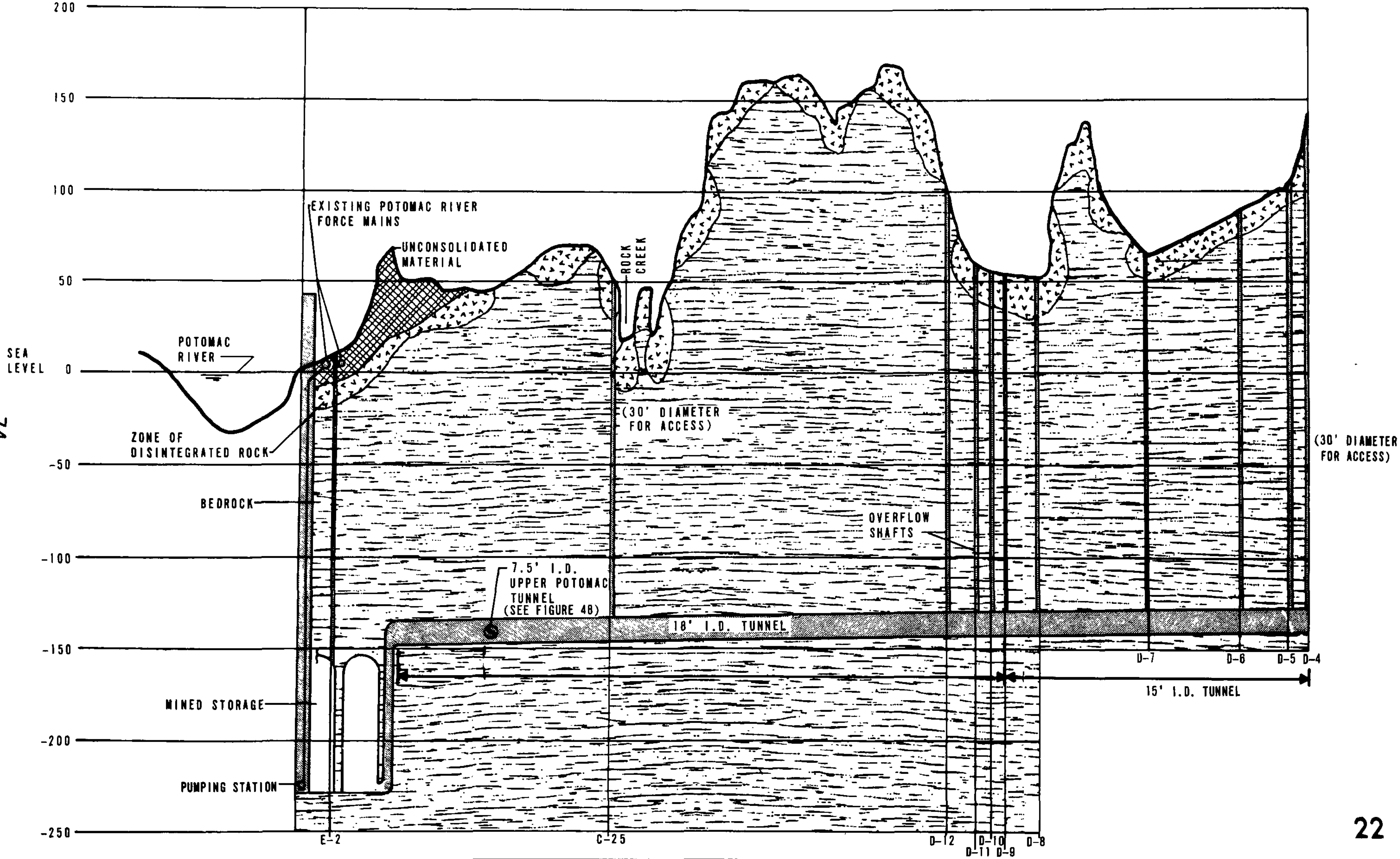
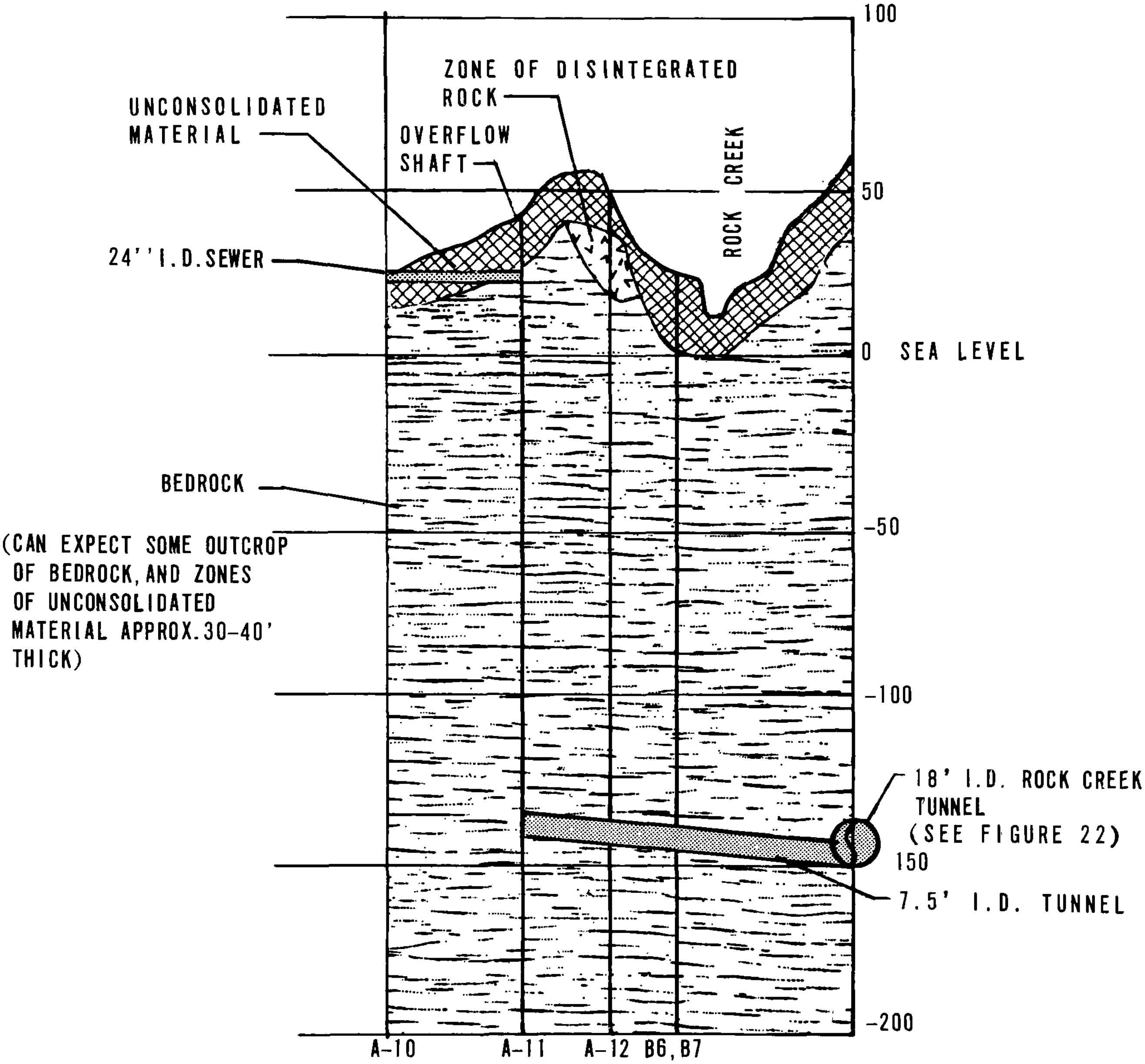
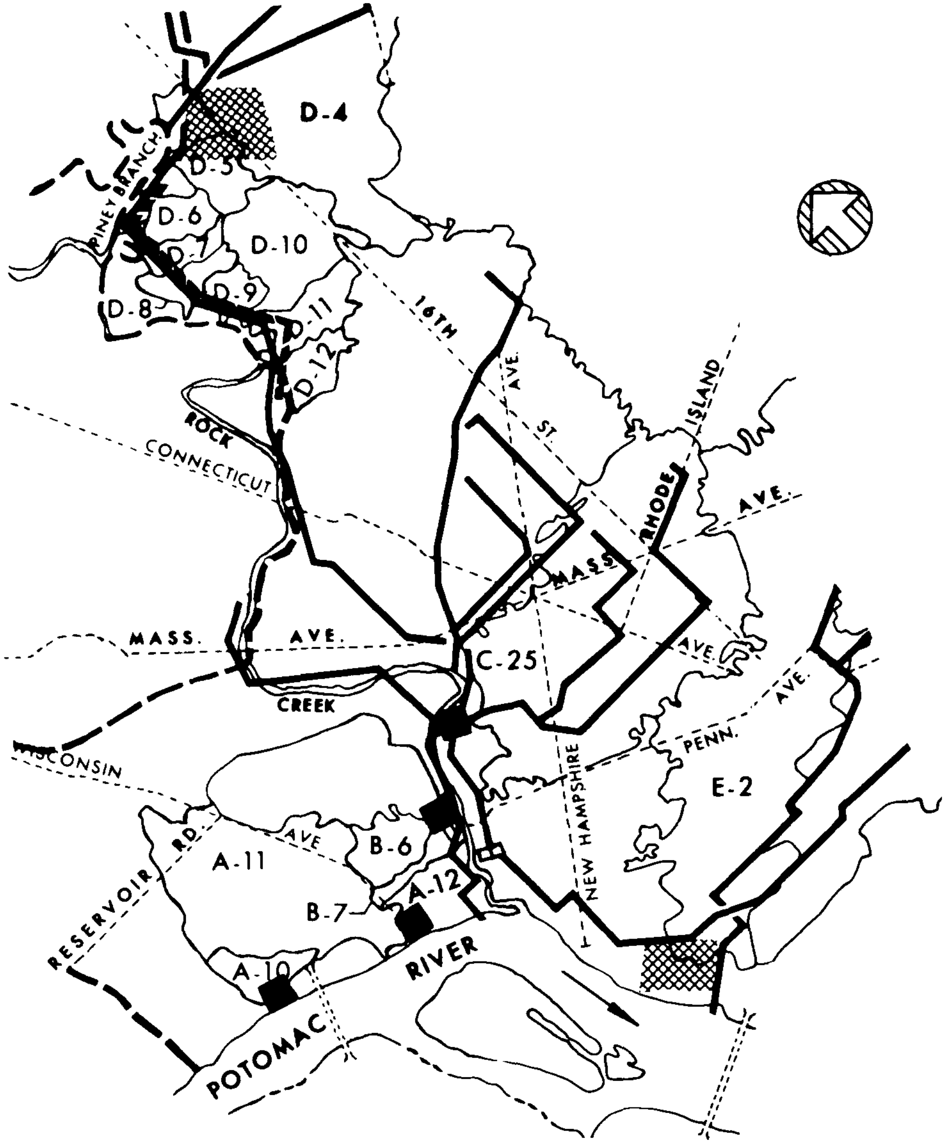


FIGURE 23

CONVEYANCE TUNNELS AND MINED STORAGE
UPPER POTOMAC TUNNEL PROFILE (SEE FIGURE 21 FOR PLAN)





LEGEND

- POLITICAL BOUNDARY
- COMBINED SEWERS
- SANITARY SEWERS
- OTHER EXISTING SEWERS (COMBINED AND SEPARATED)
- A-10 SEWER DISTRICT NUMBER
- LIMITS OF COMBINED SEWER SYSTEM IN 1970
- SEWER DISTRICT BOUNDARIES
- PUMPING STATIONS
- STORAGE RESERVOIRS
- STORAGE TUNNELS
- MINED STORAGE

NO.	NAME
A-10	37TH ST.-GEORGETOWN
A-11	GEORGETOWN
A-12	K ST.-WISCONSIN AVE.
B-6	M ST. -27TH ST.
B-7	28TH ST.-WISCONSIN AVE.
C-25	SLASH RUN (PART)
D-4	PINEY BRANCH
D-5	OAK ST.-MT.PLEASANT ST.
D-6	INGLESIDE TER.
D-7	PARK RD.
D-8	LAMONT ST.
D-9	KENYON ST.
D-10	IRVING ST.
D-11	QUARRY RD.
D-12	ONTARIO RD.
E-2	EASBY POINT

1500 0 1500 3000 4500 6000
SCALE IN FEET

DISTRICT OF COLUMBIA

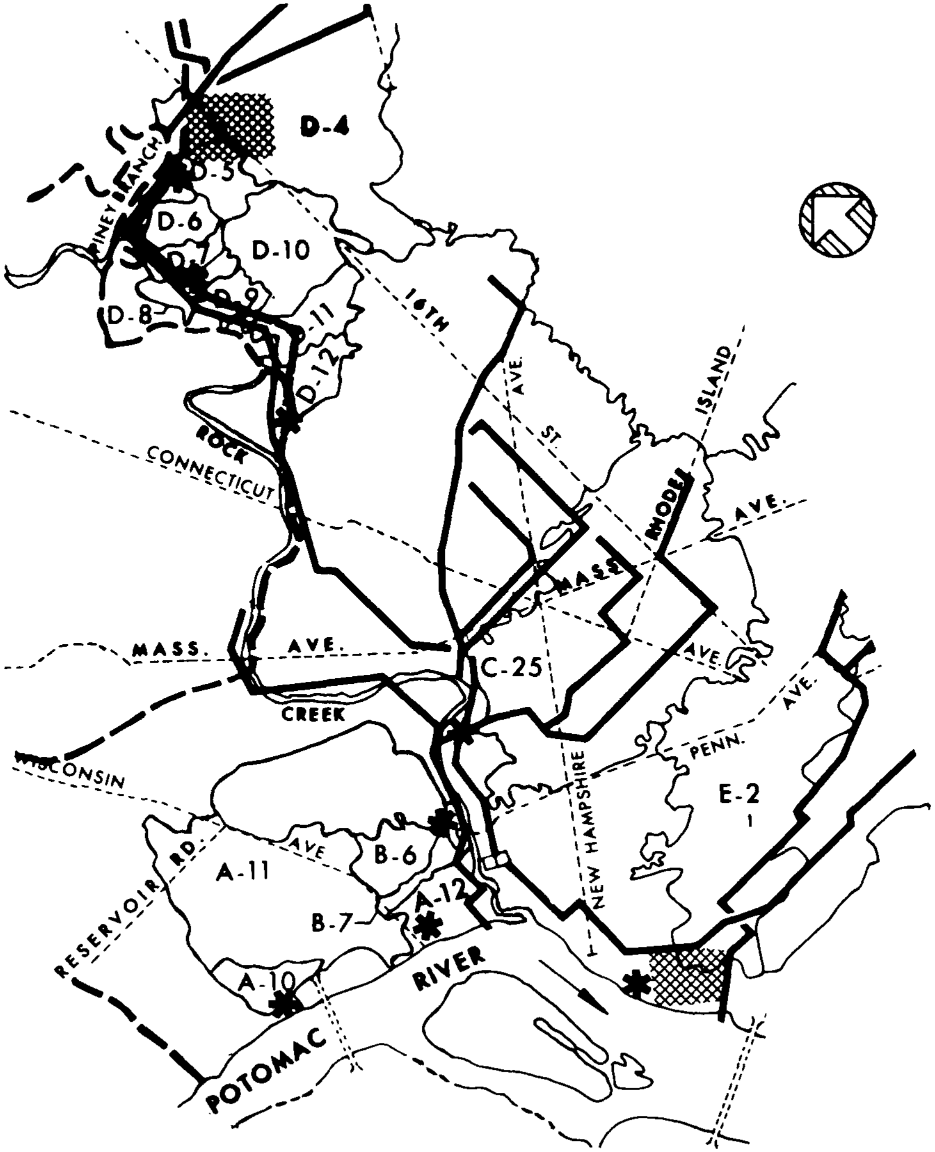
MAXIMUM USE OF STORAGE RESERVOIRS IN
UPPER POTOMAC-ROCK CREEK BASIN

FIGURE 24

overflow. Since the D-5 outfall is only about 300 feet downstream of D-4 and since the D-5 overflow volume would be small, it is logical to combine the overflows from these two districts and collect them in the underground storage required for D-4. Likewise, mined storage is indicated for sewer district E-2 by itself if storage reservoirs are used in the Georgetown area. The close proximity of the points of overflow from sewer districts D-8 through D-12 and the relatively low hydraulic characteristics make it advantageous to connect these overflow points by means of tunnels (or conventional cut-and-fill sewer construction). Likewise, districts D-6 and D-7 should be connected in a similar manner. Having connected these districts, simply expanding the connecting system into shallow tunnel storage is probably less expensive than construction of storage reservoirs. Nevertheless, the small volume of overflow (less than three million gallons) from these seven districts indicates that the costs to control their overflow is minor in comparison to other sewer districts.

Treatment at Overflow Points

Again, the conceptual design of this approach follows the layout for the storage reservoir method, except that treatment facilities are required near points of storage. The locations of the required treatment facilities are shown in Figure 25. Again, the most attractive feature of this approach is that it provides facilities for treating stored overflow.



LEGEND

- POLITICAL BOUNDARY
- COMBINED SEWERS
- SANITARY SEWERS
- OTHER EXISTING SEWERS (COMBINED AND SEPARATED)
- A-10 SEWER DISTRICT NUMBER
- LIMITS OF COMBINED SEWER SYSTEM IN 1970
- SEWER DISTRICT BOUNDARIES
- PUMPING STATIONS
- TREATMENT FACILITIES
- STORAGE TUNNELS
- MINED STORAGE

NO.	NAME
A-10	37TH ST.-GEORGETOWN
A-11	GEORGETOWN
A-12	K ST.-WISCONSIN AVE.
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B-7	28TH ST.-WISCONSIN AVE.
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D-6	INGLESIDE TER.
D-7	PARK RD.
D-8	LAMONT ST.
D-9	KENYON ST.
D-10	IRVING ST.
D-11	QUARRY RD.
D-12	ONTARIO RD.
E-2	EASBY POINT

1500 0 1500 3000 4500 6000
SCALE IN FEET

DISTRICT OF COLUMBIA

TREATMENT AT OVERFLOW POINTS IN
UPPER POTOMAC-ROCK CREEK BASIN

FIGURE 25

SECTION VII

EVALUATION OF FEASIBLE ALTERNATIVES

General

The preceding sections of this report have defined the variability and impact of combined sewer overflows in the District and have presented four approaches for dealing with the problem. From these approaches, it is possible to develop numerous alternative strategies within the District for collecting, storing, and treating overflow. For example, one strategy may involve conveyance tunnels and mined storage for the overflow from the 2-year, 24-hour storm in the Anacostia River area while calling for storage reservoirs and treatment for the overflow from the 25-year 24-hour storm in the Rock Creek-Upper Potomac area.

To provide a sound basis for decision-making, it is necessary to define the consequences of promising alternatives in detailed and comparative terms. Costs and impact on water quality demand the most attention, but non-quantifiable aspects, such as public convenience, also influence the selection of the appropriate solution. The four District-wide approaches previously presented in Figures 16 thru 25 are considered sufficiently varied to offer a wide choice of strategies throughout the District. The approaches presented were:

1. Maximum use of storage reservoirs
2. Treatment at points of overflow
3. Conveyance tunnels and mined storage
4. Sewer separation

Again, it is pointed out that the approach selected for one sewer basin does not have to be used in the other basins; the appropriate District-wide solution may incorporate features from each of these four approaches.

The following sections present the costs and facilities associated with individual sewer districts for each approach. The indicated reduction in pollutant loadings for alternative design-storm frequencies is also indicated. Finally, to permit a balanced assessment, non-quantifiable factors (those other than cost and pollutant reductions) are discussed and compared.

Cost Analysis

Construction Costs

Cost is an essential factor in selecting the appropriate alternative for abating combined sewer overflows. Estimates have been made for the costs involved in the alternatives which have been specifically discussed. A construction cost and escalation contingency of 15 percent has been included in all estimates, as well as 6 percent for engineering and 15 percent for resident inspection, bonding, etc.; but it should be noted that the cost estimates do not include Surveying, Soils Investigations, Land Acquisition, or Rights-of-Way.

If it will take the responsible agencies two years from the time of release of this report to act on appropriation of funds for any alternative (which is a reasonable assumption in view of the magnitude and complexity of the project), construction would probably start about the middle of 1973. To assure that the cost estimates are consistent with that date, the total cost estimates reflect an Engineering News-Record Construction Cost Index of 1800, the projected index for June, 1973. The construction costs specific to any date can be determined by multiplying the costs presented herein by the ratio of the then current ENR index to 1800.

General Procedure

The same general procedure was used to estimate the construction costs of the alternative methods of storage reservoirs, treatment at overflow points, and tunnel and mined storage. First, a detailed cost estimate was made of the facilities required to handle the 15-year, 24-hour storm at each sewer district. This was done for each of the three approaches illustrated in Figures 16 through 25. This cost analysis included preliminary on-site inspection to consider land availability, terrain, etc. in selecting sites for facilities. All the unit cost estimates are based on December 1969 costs for similar construction, with the total costs increased by the ratio of the 1800 ENR index to 1300, the national average during December 1969.

Following the analysis of the 15-year, 24-hour storm graphs were made of total costs versus capacity relationships for the following components:

1. Storage reservoirs and appurtenant equipment
2. Tunnels
3. Mined storage
4. Vertical shafts
5. Treatment plants
6. Pumping stations

The many different capacities required for the various sewer districts provided a suitable span of plotting points. These costs curves were then used to estimate the costs for handling the overflow from each sewer district for the 2-year, 5-year, and 25-year frequency, 24-hour duration storms.

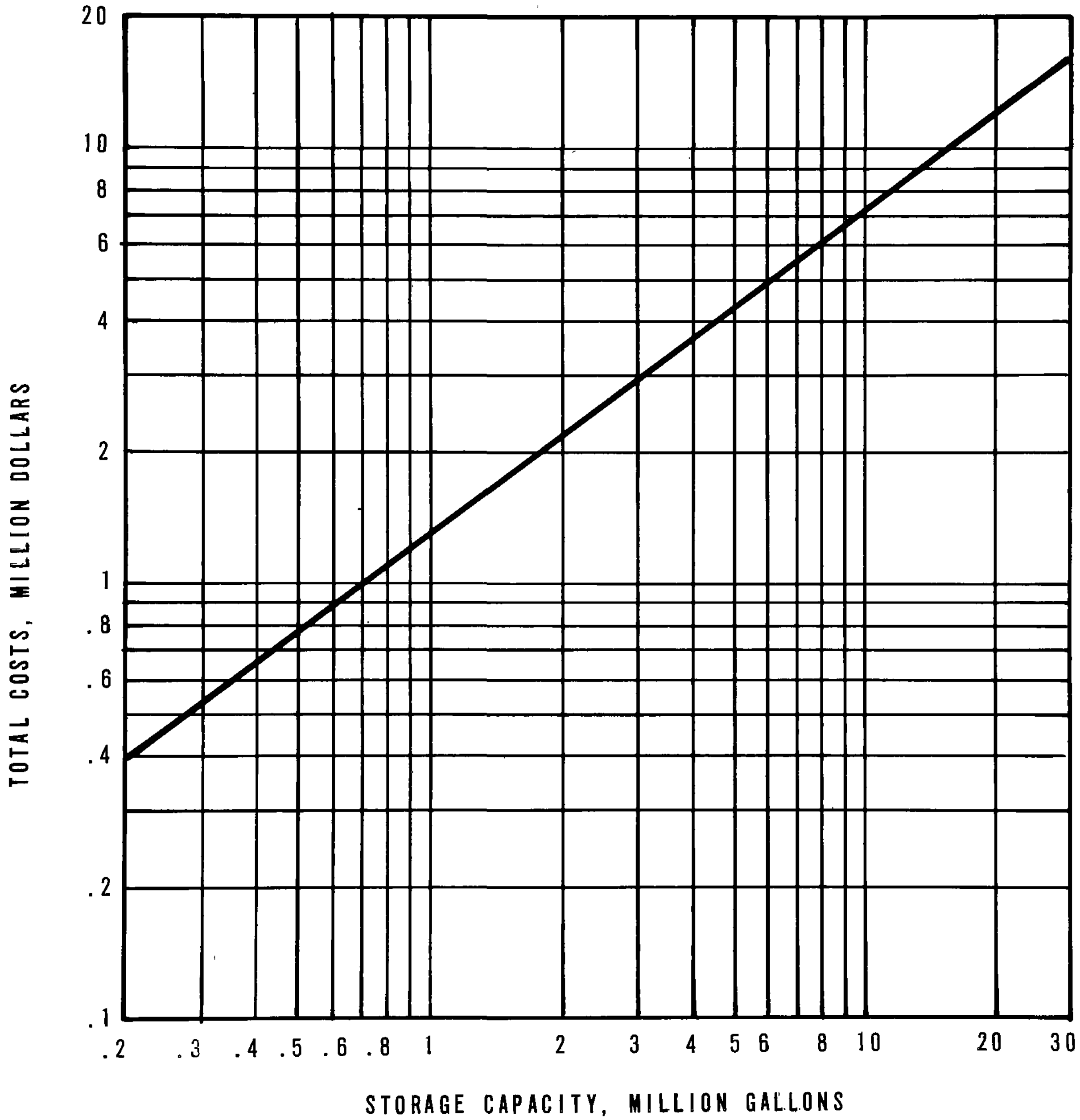
Storage Reservoir Construction Costs

The storage reservoirs would be shallow-underground, multi-cell, concrete tanks. Figure 26 shows the relationship for construction costs of reservoirs and appurtenant equipment versus capacity. Typical costs range from $900,000 for a 600,000-gallon reservoir to

FIGURE 26

TOTAL CONSTRUCTION COSTS OF STORAGE RESERVOIRS AND APPURTENANT EQUIPMENT

ENR= 1800



for a 32,000,000-gallon reservoir. The construction costs include the costs of site preparation, excavation, concrete, backfill, buildings, landscaping, roadways, and mechanical and electrical construction.

Costs are based on simultaneous construction of all reservoirs. Some advantage may be gained by building only a portion of the reservoirs at the beginning of the program. The knowledge and experience gained during the design and construction of the initial reservoirs could be used to reduce the construction costs of subsequent reservoirs.

Tunnel Construction Costs

These costs were based on up-dated bid prices of current tunnel construction projects in Chicago and California in which the boring method is utilized. Information obtained from one of the largest manufacturers of drilling equipment indicated that the rapid development of boring equipment will tend to moderate increases in these prices even in the face of rising costs. Figure 27 defines the unit cost versus diameter relationships for tunnel construction. The following important assumptions have been made with regard to construction costs for tunnels:

1. One machine would be used to construct the Upper Potomac-Rock Creek tunnels, and a separate machine for the Anacostia River tunnel.
2. All tunnels would be concrete lined. (12" lining at an unit cost of $100 per cubic yard of concrete).
3. Material excavated from the the tunnel would be hauled away as waste at a cost of $5.00 per cubic yard.
4. Boring would be at an average rate of 36 feet per day.
5. There would be no salvage value for the boring machine or mucking equipment.

If the quality of material excavated from tunnel and mined-storage construction is such that it could be used as a building material, the construction costs would be reduced substantially. The structural characteristics of the material would be determined during the sub-surface investigation, and the physical properties (size and shape) of the aggregate would be determined at the time of construction.

Although boring rates of approximately 200 to 300 feet per day have been considered possible in rock similar to that underlying Washington, an average of 36 feet per day over the construction period seems more reasonable for purposes of cost estimating.

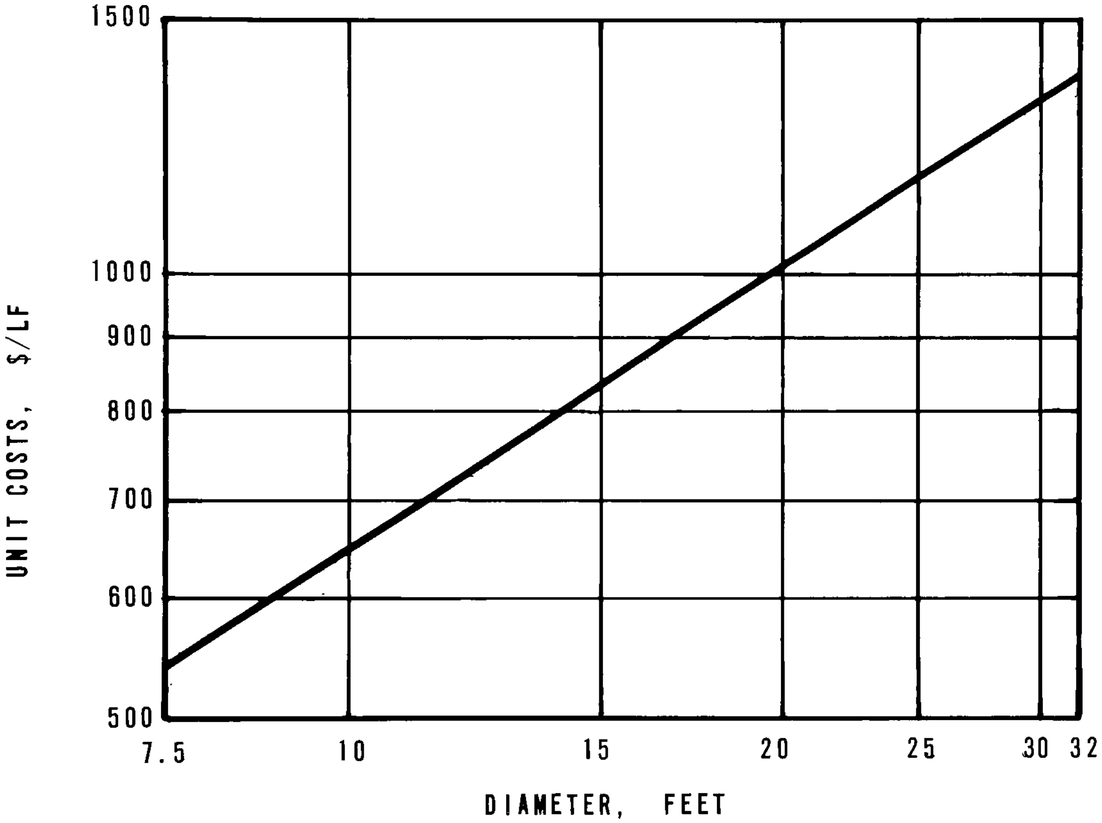
Boring machines are generally custom built for a particular project and may not be usable on a different project. For this reason, no salvage value has been allowed. Other equipment--muck cars, locomotives, dust, cable, etc.,--will have some small salvage value, but this will be negligible compared to total construction costs.

The construction costs also include power and labor. Labor costs and concrete costs are the most significant, accounting for over 60 percent of all costs.

FIGURE 27

UNIT CONSTRUCTION COSTS OF TUNNELS

ENR= 1800



Mined Storage Construction Costs

Mined storage construction costs are based on similar costs for underground mining and include labor, equipment, and handling of excess material. The unit cost for mined-storage construction is about $20 per cubic yard but varies depending on the volume to be excavated. Figure 28 illustrates the relationship between construction costs and mined-storage capacity. This curve accounts only for the cost of mining and does not include pumping stations and vertical shafts.

Vertical Shaft Construction Costs

Three basic types of vertical shafts are expected:

1. Relatively small shafts (for the smaller sewer districts) drilled from the ground surface to the tunnel.

2. Slightly larger shafts, five to six feet in diameter, (for the larger sewer districts) to be constructed by the raise-boring method. The unit cost of these shafts is about $200-$250 per vertical foot.

3. Large-diameter (30 feet) shafts, to be used for access by men and equipment as well as for diverting flow to the tunnel. Construction techniques would be conventional slurry trench or jacking methods. Large-diameter shafts would also be used for pumping and appurtenant equipment. The unit cost of these shafts is about $5,000-$6,000 per vertical foot.

Pump Station Construction Costs

Pump station construction cost estimates include the cost of pumping equipment, controls, sludge facilities, superstructures, heating, ventilating and electrical work, instrumentation, metering, inside piping and valves, and sitework. The cost of the access shaft for pumping equipment is included in the vertical shaft construction costs.

Treatment Plant Construction Costs

The treatment process recommended is filtration preceded by sedimentation and followed by disinfection. Three cost estimates were developed from the costs of the major components of 1-mgd, 10-mgd, and 100-mgd plants. Figure 29 illustrates the relationship of total construction costs to capacity as suggested by these three estimates. For a 10-mgd plant, the costs (ENR = 1300) of various plant components were as follows:

Component	Construction Costs
Filters	$ 500,000
Chlorinator and Contact Tank	90,000
Pumps	40,000
Chemical Feed and Mixing	90,000
Instruments, Electrical, Piping	270,000
Building	250,000
	$1,240,000
Contingency	190,000
TOTAL	$1,430,000

FIGURE 28

TOTAL CONSTRUCTION COSTS OF MINED STORAGE
ENR= 1800

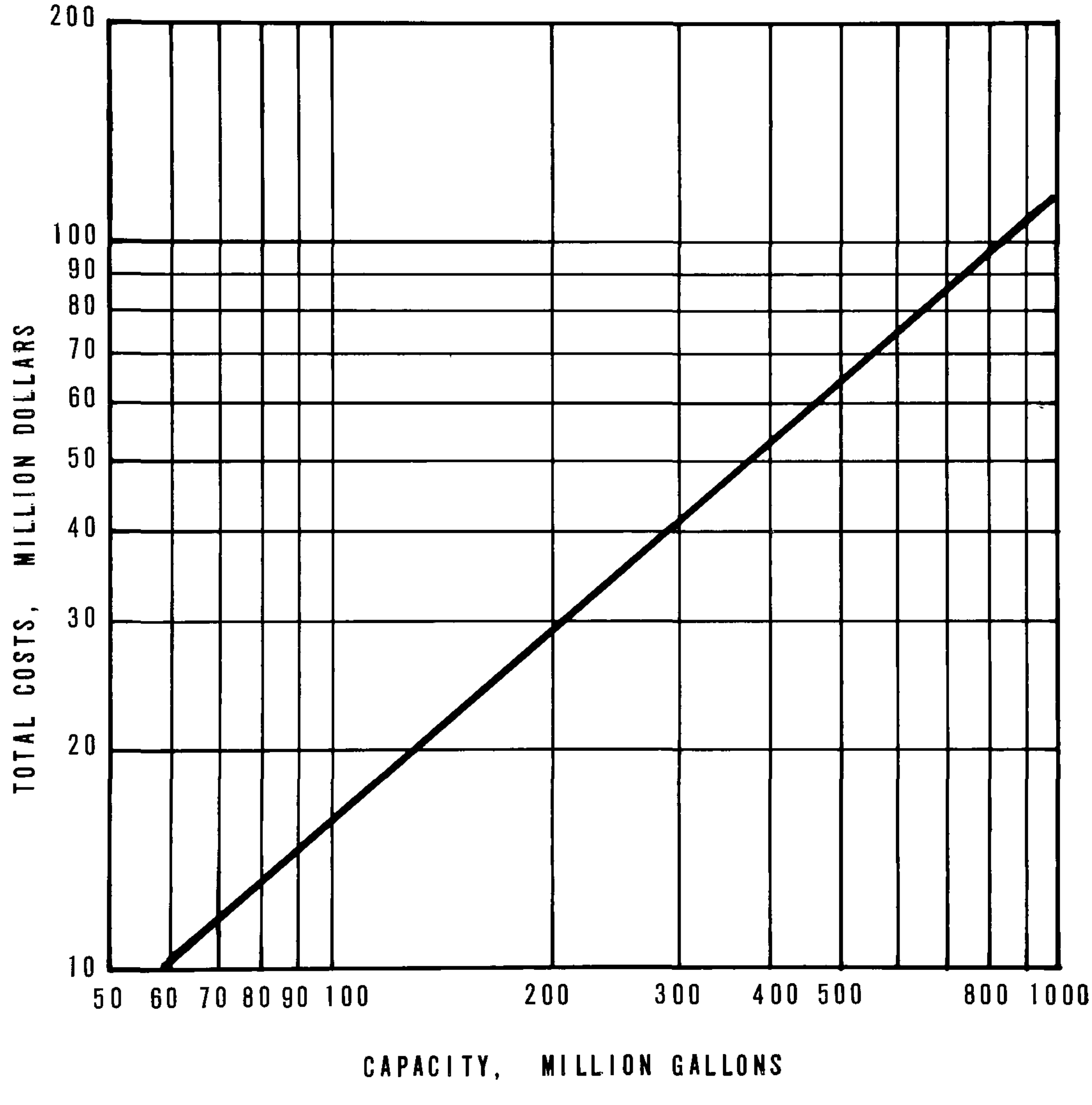
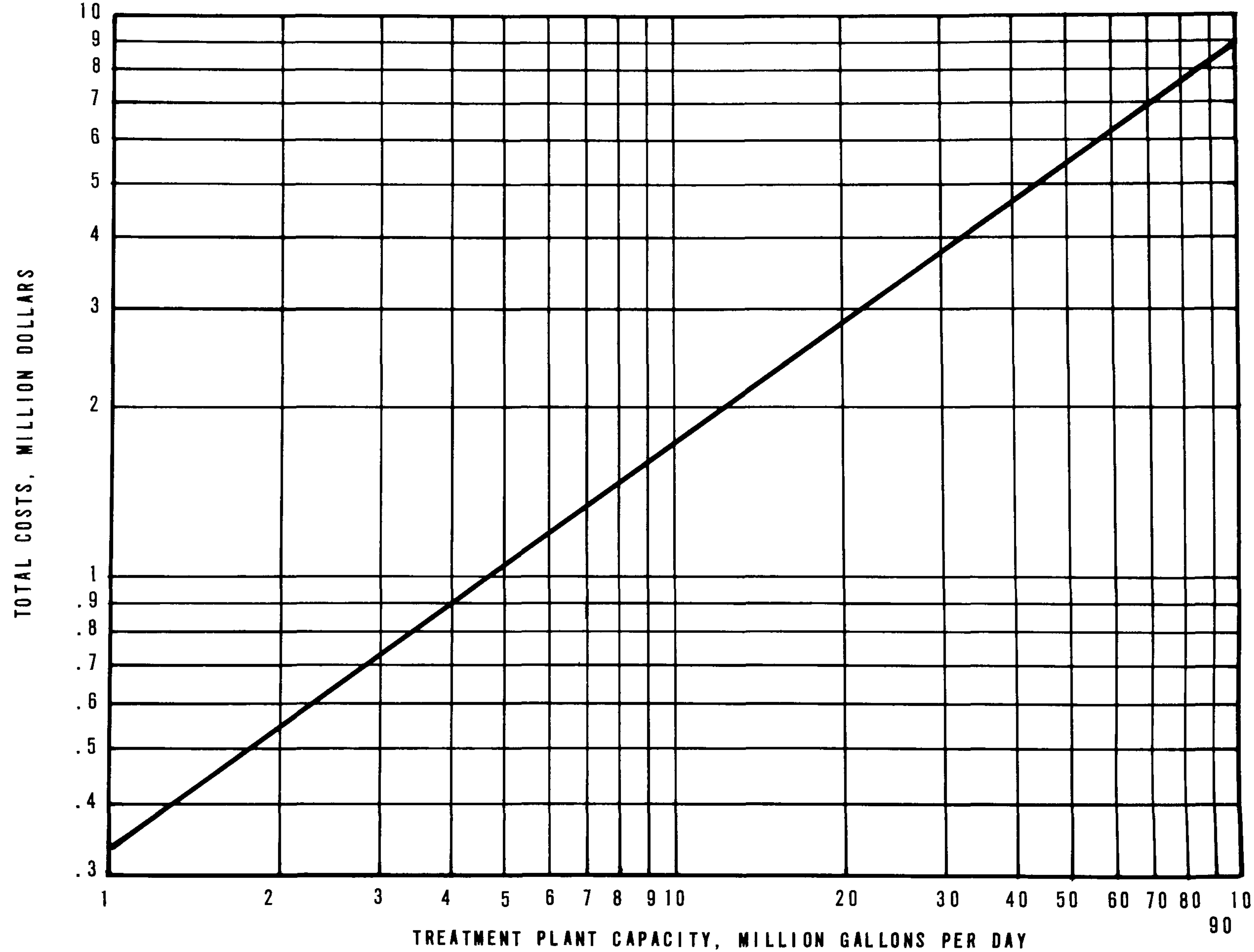


FIGURE 29
TOTAL CONSTRUCTION COSTS OF TREATMENT PLANTS VERSUS CAPACITY
ENR= 1800



Two important assumptions have been made in determing the flow rate of the treatment facility to be located at each overflow point:

1. The maximum residence period in a storage structure will be five days.

2. The minimum capacity of a treatment facility will be 0.2 mgd.

The relationship illustrated in Figure 29 was used to determine the costs of the centralized facilities required to treat stored overflow, in addition to being used to define the costs of treatment facilities at points of overflow.

Total Construction Costs

The four alternative District-wide approaches are as follows:

1. Maximum use of storage reservoirs
2. Treatment at points of overflow
3. Conveyance tunnels and mined storage
4. Sewer separation

Tables 12, 13, 14, and 15 present the construction costs by individual sewer drainage basins for the four approaches. It must be emphasized again that the appropriate District-wide solution may include features from each of these approaches. To facilitate the selection of this solution, the costs and facilities associated with each approach are identified by sewer district.

As stated before, these costs correspond to an ENR index of 1800. Also, these costs exclude the costs of dealing with those sewer districts scheduled for separation prior to 1975. However, budgeting problems have stopped the program of separating those sewer districts: therefore, in the final analysis, the total costs in Tables 12 to 15 will have to be increased. The incremental costs attributable to those sewer districts is probably slight because their total area is about 400 acreas, representing less than 4 percent of the collective area of all combined sewer districts. The added costs would be from $6,000,000 to $12,000,000, depending on the design storm, for projects not involving separation, and $27,000,000 for the separation project.

In the approach proposing treatment at overflow points, equalization storage in the amount equal to the total volume of overflow would be required because the rate of overflow (even from the smallest drainage area) is extremely high. The cost of constructing treatment facilities is, therefore, in addition to the reservoir storage construction costs. The difference in costs between this approach and the reservoir approach is explained by the economies of scale resulting from constructing a single centralized treatment facility. Inclusion of the Kingman Lake project in the approach proposing treatment at points of overflow would increase the costs of the G-2 sewer district from $45,000,000 to $56,700,000 for a 2-year, 24-hour design event. The relatively small increase in cost for incorporating an approach that proposes complete treatment is explained by the fact that, for the 2-year design storm, it is feasible to construct a sufficiently large storage reservoir. In contrast, the cost estimate listed in Table 13 for District G-2 was based on more costly mined storage.

Table 12

Maximum Use of Storage Reservoirs[1]

Estimated Capital Costs
(ENR Index = 1800)

Sewer District	Type of Storage	2 Years - 24-Hour Volume million gallons	2 Years - 24-Hour Cost	5 Years - 24-Hour Volume million gallons	5 Years - 24-Hour Cost	15 Years - 24-Hour Volume million gallons	15 Years - 24-Hour Cost	25 Years - 24-Hour Volume million gallons	25 Years - 24-Hour Cost
				Rock Creek - Upper Potomac Basin					
A 10, A-11	Reservoir	11.	$ 7,600,000	15.	$ 9,600,000	21	$ 12,200,000	24	$ 13,800,000
A-12	Reservoir	2	2,200,000	3	2,900,000	4	3,600,000	4	3,600,000
B-6, B-7	Reservoir	---		0.6	900,000	3	2,900,000	4	3,600,000
C-25	Reservoir	1	1,300,000	4	3,600,000	8	6,000,000	11	7,600,000
D-4, D-5	Mined	65	15,200,000	104	21,500,000	168	30,900,000	190	33,300,000
D-6, D-7	Tunnel/Reservoir	1.7	2,000,000	2	2,200,000	4	3,600,000	4	3,600,000
D-8, D-9, D-10, D-11, D-12	Tunnel/Reservoir	4.4	3,900,000	8.3	5,600,000	11	7,600,000	14	9,100,000
E-2	Mined	97	18,300,000	158	27,700,000	214	35,500,000	260	41,700,000
				Anacostia River Basin					
E-3, F 1	Mined	128	23,300,000	175	30,500,000	230	38,300,000	270	43,900,000
G-2	Mined	280	45,000,000	370	57,000,000	490	73,000,000	560	82,000,000
G-3	Reservoir	0.2	400,000	0.4	700,000	0.8	1,100,000	1	1,300,000
G-4	Reservoir	15	9,600,000	20	12,000,000	28	15,000,000	32	17,000,000
G-5	Reservoir	6	4,800,000	9	6,600,000	13	8,600,000	15	9,500,000
G-7	Reservoir	5	4,300,000	7	5,400,000	10	7,200,000	12	8,000,000
Storage Total			$138,000,000		$186,000,000		$246,000,000		$278,000,000
Northeast Boundary Relief/Storage Tunnel			27,000,000		27,000,000		27,000,000		27,000,000
Centralized Treatment Plant			10,000,000		10,000,000		16,000,000		18,000,000
Total Construction Cost			175,000,000		$226,000,000		$289,000,000		$323,000,000
Engineering Design at ± 6%			11,000,000		14,000,000		17,000,000		19,000,000
Other Costs (resident inspection, legal, bonding, and administrative) at ± 15%			26,000,000		34,000,000		43,000,000		48,000,000
Total Project Costs[2]			$212,000,000		$274,000,000		$349,000,000		$380,000,000

[1]Basic layout illustrated in Figures 19 and 24.

[2]In the final analysis, these values will have to be increased by $6,000,000-$12,000,000, to reflect the costs of dealing with those sewer districts scheduled for separation before 1975.

Table 13

Treatment at Overflow Points[1]

Estimated Capital Costs
(ENR Index = 1800)

Sewer District	Storm Frequency and Duration: 2 Years, 24-Hour		5 Years, 24-Hour		15 Years, 24-Hour		25 Years, 24-Hour	
Rock Creek - Upper Potomac Basin								
A-10, A-11								
a. Storage	$ 7,600,000		$ 9,600,000		$12,200,000		$13,800,000	
b. Treatment	600,000	$ 8,200,000	780,000	$ 10,380,000	960,000	$ 13,160,000	1,100,000	$ 14,900,000
A-12								
a. Storage	2,200,000		2,900,000		3,600,000		3,600,000	
b. Treatment	170,000	2,370,000	220,000	3,120,000	280,000	3,880,000	280,000	3,880,000
B-6, B-7								
a. Storage	---		900,000		2,900,000		3,600,000	
b. Treatment	---		100,000	1,000,000	220,000	3,120,000	280,000	3,880,000
C-25								
a. Storage	1,300,000		3,600,000		6,000,000		7,600,000	
b. Treatment	100,000	1,400,000	280,000	3,880,000	400,000	6,400,000	560,000	8,160,000
D-4, D-5								
a. Storage	15,200,000		21,500,000		30,900,000		33,300,000	
b. Treatment	2,100,000	17,300,000	3,000,000	24,500,000	4,100,000	35,000,000	4,400,000	37,700,000
D-6, D-7								
a. Storage	2,000,000		2,200,000		3,600,000		3,600,000	
b. Treatment	150,000	2,150,000	170,000	2,370,000	280,000	3,880,000	280,000	3,880,000
D-8, D-9, D-10, D-11, D-12								
a. Storage	3,900,000		5,600,000		7,600,000		9,100,000	
b. Treatment	300,000	4,200,000	460,000	6,060,000	570,000	8,170,000	690,000	9,790,000
E-2								
a. Storage	18,300,000		27,700,000		35,500,000		41,700,000	
b. Treatment	2,900,000	21,200,000	4,000,000	31,700,000	4,900,000	40,400,000	5,600,000	47,300,000
Anacostia River Basin								
E-3, F-1								
a. Storage	23,300,000		30,500,000		38,300,000		43,900,000	
b. Treatment	3,400,000	26,700,000	4,200,000	34,700,000	5,200,000	43,500,000	5,800,000	49,700,000
G-2								
a. Storage	45,000,000		57,000,000		73,000,000		82,000,000	
b. Treatment	5,800,000	50,800,000	7,300,000	64,300,000	8,400,000	81,400,000	9,500,000	91,500,000
G-3								
a. Storage	400,000		700,000		1,100,000		1,300,000	
b. Treatment	100,000	500,000	100,000	800,000	100,000	1,200,000	100,000	1,400,000
G-4								
a. Storage	9,600,000		12,000,000		15,000,000		17,000,000	
b. Treatment	730,000	10,330,000	900,000	12,900,000	1,300,000	16,300,000	1,500,000	18,500,000
G-5								
a. Storage	4,800,000		6,600,000		8,600,000		9,500,000	
b. Treatment	370,000	5,170,000	540,000	7,140,000	680,000	9,280,000	750,000	10,250,000
G-7								
a. Storage	4,300,000		5,400,000		7,200,000		8,000,000	
b. Treatment	360,000	4,660,000	440,000	5,840,000	580,000	7,780,000	650,000	8,650,000
Storage and Treatment Construction Cost		$155,000,000		$209,000,000		$273,000,000		$309,000,000
Northeast Boundary Relief/Storage Tunnel		27,000,000		27,000,000		27,000,000		27,000,000
Total Construction Cost		$182,000,000		$236,000,000		$300,000,000		$336,000,000
Engineering Design at ± 6%		11,000,000		14,000,000		18,000,000		19,000,000
Other costs (resident inspection, legal, bonding and administrative) at ± 15%		27,000,000		35,000,000		45,000,000		50,000,000
Total Project Costs[2]		$220,000,000		$285,000,000		$363,000,000		$405,000,000

[1]Basic layout illustrated in Figures 20 and 25.
[2]In the final analysis, those values will have to be increased by $6,000,000-$12,000,000, to reflect the costs of dealing with those districts scheduled for separation before 1975.

Table 14

Conveyance Tunnels and Mined Storage[1]

Estimated Capital Costs
(ENR Index = 1800)

	Storm Frequency and Duration			
	2-Year, 24-Hour	5-Year, 24-Hour	15-Year, 24-Hour	25-Year, 24-Hour
Overflow Volume, million gallons				
Rock Creek	169	275	404	487
Upper Potomac	13	18	24	29
Anacostia River	434	581	772	890
Rock Creek - Upper Potomac Area				
Costs				
1. Tunnel Construction				
a. Rock Creek	$ 14,900,000	$ 16,800,000	$ 18,200,000	19,000,000
b. Upper Potomac	1,800,000	1,900,000	2,000,000	2,100,000
2. Vertical Shafts				
a. Rock Creek	4,400,000	4,400,000	4,400,000	4,400,000
b. Upper Potomac	2,100,000	2,100,000	2,100,000	2,100,000
3. Mined Storage	26,500,000	40,000,000	57,000,000	67,000,000
4. Pumping Station	2,600,000	4,300,000	6,200,000	7,600,000
Anacostia River Area				
Costs				
1. Tunnel Construction	9,400,000	10,400,000	11,100,000	11,500,000
2. Vertical Shafts	5,800,000	5,800,000	5,800,000	5,800,000
3. Mined Storage	58,000,000	74,000,000	94,000,000	108,000,000
4. Pumping Station	6,100,000	8,300,000	10,900,000	12,600,000
5. Northeast Boundary Relief/Storage Tunnel	27,000,000	27,000,000	27,000,000	27,000,000
Sub-Total	$159,000,000	$195,000,000	$239,000,000	$267,000,000
Centralized Treatment Plant	10,000,000	13,000,000	16,000,000	18,000,000
Total Construction Costs	$169,000,000	$208,000,000	$255,000,000	$285,000,000
Engineering Design at 6%	10,000,000	12,000,000	15,000,000	17,000,000
Other Costs (resident inspection, legal, bonding and administrative) at ± 15%	25,000,000	31,000,000	38,000,000	43,000,000
Total Project Costs	$204,000,000	$251,000,000	$308,000,000	$345,000,000

[1] Basic layout illustrated in Figures 16 and 21.
[2] In the final analysis, these values will have to be increased by $6,000,000-$12,000,000, to reflect the cost of dealing with those sewer districts scheduled for separation before 1975.

Table 15

Sewer Separation Costs
(Based on the 15-Year Storm)

Estimated Capital Costs
(ENR Index = 1800)

Sewer District	Separation Cost
A-10	$ 900,000
A-11	9,400,000
A-12	700,000
B-3	500,000
B-4	4,300,000
B-5	600,000
B-6	1,500,000
B-7	300,000
C-23	200,000
C-24	500,000
C-25 (Small Part)	4,100,000
C-25 (Large Part)	20,500,000
C-26	600,000
C-28	1,500,000
C-29	6,900,000
D-4	97,000,000
D-5	1,400,000
D-6	1,200,000
D-7	1,000,000
D-8	800,000
D-9	1,000,000
D-10	5,600,000
D-11	2,500,000
D-12	1,900,000
D-13	1,500,000
D-14	2,300,000
D-15	2,200,000
D-16	35,000,000
D-17	300,000
E-2	28,000,000
E-3	18,000,000
F-1	49,000,000
G-2	177,000,000
G-3	1,300,000
G-4	13,000,000
G-5	7,700,000
G-7	6,600,000
Total Construction Costs	$507,000,000[1]
Other Costs (resident inspection, legal, bonding, and administrative) at ± 15%	76,000,000
Total Projects Cost	$583,000,000[2]

[1] Includes Engineering Costs

[2] In the final analysis, this value will have to be increased by $27,000,000, to reflect the costs of separating those sewer districts scheduled for separation before 1975.

The construction costs for sewer separation have been estimated from cost information in the 1957 Board of Engineers Report (1). The estimated costs, updated to an ENR index of 1800, for separating those sewer districts scheduled after 1975 is $583 million.

Table 16 and Figure 30 summarize the total construction costs of each District-wide approach. The costs of sewer separation is considerably higher than the costs of the three other approaches, which are about the same. The alternative of total use of tunnels and mined storage represents about a 10 percent cost savings over the next lowest approach.

Table 16

Preliminary Estimate
Comparison of Total Project Costs

Districtwide Alternative	Design Storm Frequency 2-Year	5-Year	15-Year	25-Year
Maximum Utilization of[1] Storage Reservoirs	$212,000,000	$274,000,000	$349,000,000	$380,000,000
Treatment at Points of[1] Overflow	220,000,000	285,000,000	363,000,000	405,000,000
Total Use of Conveyance[1] Tunnels and Mined Storage	204,000,000	251,000,000	308,000,000	345,000,000
Sewer Separation[2]			583,000,000	

[1] In the final analysis, these values will have to be increased by $6,000,000-$12,000,000, to reflect the cost of dealing with those sewer districts scheduled for separation before 1975.
[2] In the final analysis, this value will have to be increased by $27,000,000, to reflect the cost of separating those sewer districts scheduled for separation before 1975.

Operating Costs

Preliminary estimates of annual operating costs have been prepared for each of the various alternatives and are presented in Tables 17 through 20. These estimates are prepared for the 15-year, 24-hour storm only. The annual operating costs for various alternatives are similar except for a significant difference in maintenance costs. The maintenance costs for the reservoir approach and the local treatment approach are higher because there are fourteen storage or storage-treatment facilities to be maintained, as compared to only two central storage facilities for the tunnel approach. The treatment plant cost for the local treatment approach is higher because this approach requires many local plants rather than a single centralized facility.

The handling and disposal of sludge represents a significant cost for all the approaches except sewer separation. As listed in Table 10, combined sewer overflows will contain approximately 30,000 tons of dry suspended solids per year. In any system utilizing storage or in-line treatment, most of these solids will be removed, and consequently will require handling and disposal.

FIGURE 30
INVESTMENT VERSUS DESIGN STORM RETURN FREQUENCY
FOR ALTERNATIVE DISTRICTWIDE APPROACHES

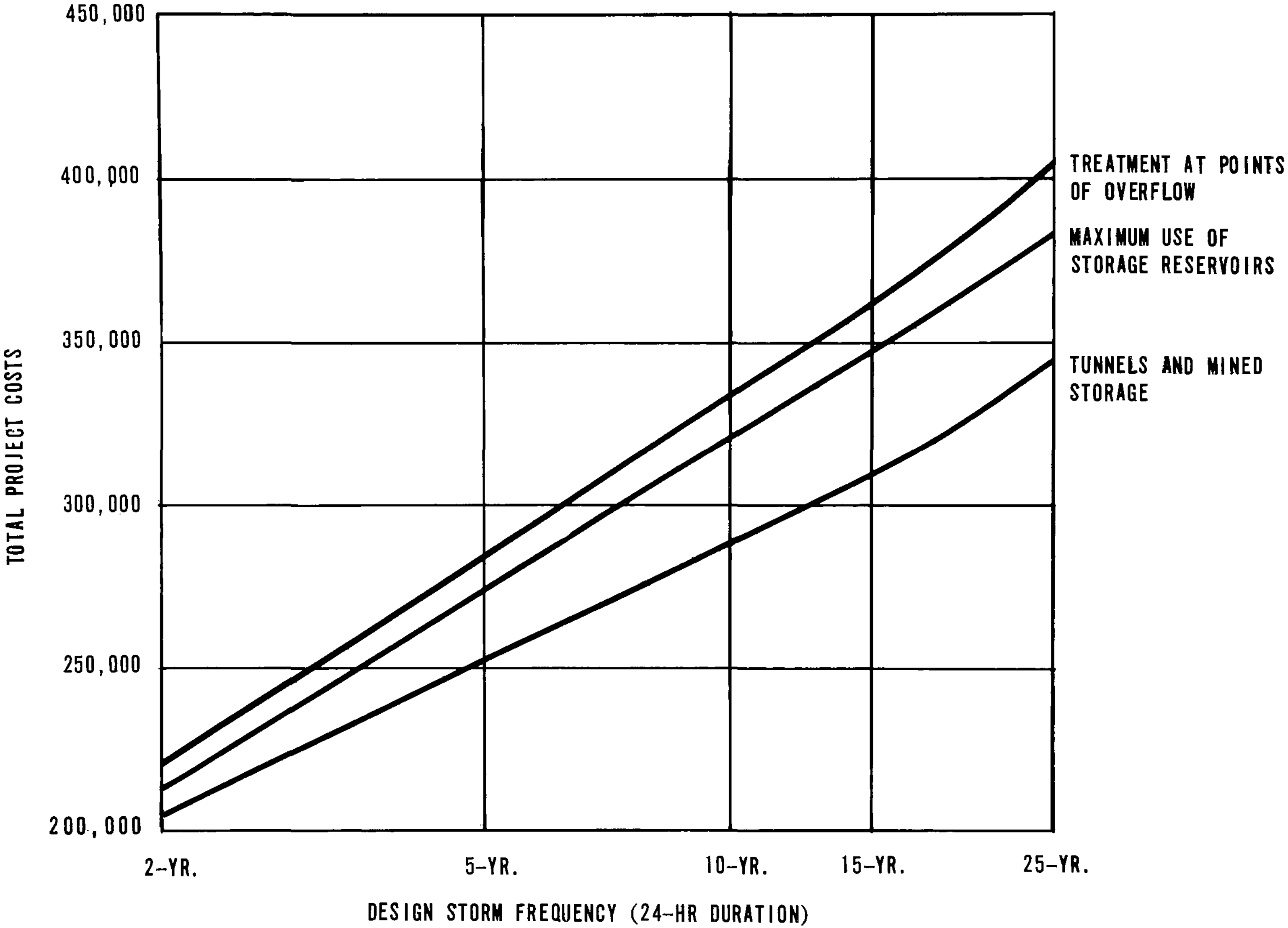


Table 17

Preliminary Estimate

Summary of Annual Operating Costs
for
Maximum Use of Storage Reservoirs
for 15-Year, 24-Hour Storm

Administration		$ 290,000
Labor		
Permanent Staff	$ 200,000[1]	
Auxiliary Staff	160,000[1]	360,000
Maintenance		
Structures	$ 370,000	
Mechanical	2,000,000	
Electrical & Instrumentation	730,000	3,100,000
Utilities		
Electrical 20,520 x 0.746 x 952 x $0.010/KWH		150,000
Treatment Plant (Additional Facilities to treat storm water flows at Blue Plains Plant or other location)		
Sludge Handling and Disposal		750,000
Operation and Maintenance - 4% of Capital Cost		640,000
Sub-Total		$5,290,000
Operating Contingency at 5 Percent		270,000
TOTAL		$5,560,000

[1] Includes overhead and benefits

Table 18

Preliminary Estimate

Summary of Annual Operating Costs
for
Treatment at Overflow Points
for 15-Year, 24-Hour Storm

Administration		$ 300,000
Labor		
Permanent Staff	$ 170,000[1]	
Auxiliary Staff	200,000[1]	370,000
Maintenance		
Structures	$ 370,000	
Mechanical	2,100,000	
Electrical & Instrumentation	750,000	3,220,000
Utilities		
Electrical - 20,520 x 0.746 x 952 x $0.010/KWH		150,000
Filter System (per 1,000 gals.)		
Filter Replacement and Installation Cost	$0.08	
Operation and Maintenance	0.10	
Power and Backwash	0.02	
975,000 x $0.20 = 195,000		200,000
Treatment Plants		
Sludge Handling and Disposal		750,000
Operation and Maintenance - 4% of Capital Cost		1,500,000
Sub-Total		$6,490,000
Operating Contingency at 5 Percent		330,000
TOTAL		$6,820,000

[1] Includes overhead and benefits.

Table 19

Preliminary Estimate

Summary of Annual Operating Costs
for
Conveyance Tunnels and Mined Storage
for 15-Year, 24-Hour Storm

Administration		$ 250,000
Labor		
Permanent Staff	$140,000[1]	
Auxiliary	60,000[1]	200,000
Maintenance		
Structures	$400,000	
Access Shafts	60,000	
Mechanical	450,000	
Electrical & Instrumentation	370,000	1,280,000
Utilities		
Electrical - 23,850 x 0.746 x 952 x $0.010/KWH		170,000
Treatment Plant (Additional Facilities to treat storm water flows at Blue Plains Plant or other location)		
Sludge Handling and Disposal		750,000
Operation and Maintenance - 4% of Capital Cost		680,000
Sub-Total		$3,330,000
Operating Contingency at 5 percent		170,000
Total		$3,500,000

[1] Includes overhead and benefits.

Table 20

Preliminary Estimate
Comparison of Annual Operating Costs
Based on 15-Years, 24-Hour Storm

	Maximum Use of Storage Reservoirs	Treatment at Overflow Points	Conveyance Tunnels and Mined Storage
Administration	$ 290,000	$ 300,000	$ 250,000
Labor	360,000	370,000	200,000
Maintenance	3,100,000	3,220,000	1,280,000
Utilities	150,000	150,000	170,000
Sludge Handling and Disposal	750,000	750,000	750,000
Treatment O & M Costs	640,000	1,500,000	680,000
Sub-Total	$5,290,000	$6,490,000	$3,330,000
Operating Contingency at 5 Percent	270,000	330,000	170,000
TOTAL	$5,560,000	$6,820,000	$3,500,000

This quantity is greater than the quantity of solids generated at the entire Blue Plains plant (26); therefore, the quantity of sludge from combined sewer overflows probably could not be handled entirely at the Blue Plains plant. However, in view of the general problems of solid waste disposal in the Washington area, perhaps the same order of sophistication will be required to handle the combined sewer sludge as is employed to handle the sewage treatment plant sludge. To handle and dispose of one ton of dry suspended solids at the Blue Plains plant costs about $25.00 (26). This suggests that annual costs of sludge disposal for a combined sewer pollution abatement approach (other than separation) will be $750,000.

Impact on Water Quality

Indicated Requirements

As previously presented in this report, the Water Quality Office of the Environmental Protection Agency has determined the maximum allowable contaminant loadings to the Potomac if water quality objectives are to be met. These recommended loadings are quite stringent, requiring, at all treatment plants, the removal of at least 96 percent of 5-day BOD, 96 percent of phosphorus, and 85 percent of nitrogen. These removals are based on existing flows to these plants; as population increases, even greater degrees of treatment will be required.

These reductions assume no other source of pollutants to the Potomac and no allowance for combined sewer overflow loadings. Even if these reductions are achieved at the treatment plants, the combined sewer overflows (occuring fifty to sixty times a year) will in themselves result in loadings greater than recommended. At times, these overflow loadings will be from ten to thirty times greater than the recommended loadings. It is apparent that some measures must be taken to reduce the discharge of untreated combined sewer overflow, or the water quality objectives will almost certainly fail to be achieved.

These pollutant reduction percentages should not apply directly to combined sewer overflows. A statistical analysis (6) indicates that a system of storage and treatment facilities sized for the overflow from the one-year, 24-hour storm will retain over 99 percent of the long-term-averaged annual volume of overflow. Nevertheless, infrequent storms will at times have an overflow volume far in excess of this storage capacity. This will result in the discharge of a significantly high shock loading to the Potomac, enough to interfere with many uses of the river, possibly enough to result in the mass destruction of a desirable fish population which may have reappeared with the cleansing of the Potomac. The primary decision focuses not on selecting a reduction percentage but on selecting the storm event and degree of control for which storage and treatment facilities will be designed.

Reduction in Pollutant Loadings

The selection of the design frequency is properly made only in the context of basin-wide water quality management. The Potomac pollution problems to which combined sewers contribute are interstate in scope, and require evaluation on a regional basis. Many factors beyond the scope of this study, e.g. flow augmentation, financing, population growth, have too great an influence to be ignored. To provide part of the information basis upon which this decision is made, this section of the report indicates the reduction in pollutant loadings expected from storage and treatment facilities based on various design storm frequencies.

Although sewer separation will stop the discharge of untreated sanitary sewage into the Potomac, a significant loading will still occur with each rainfall in the form of storm water runoff. Table 10 shows that, on an annual basis, storm water runoff from the combined sewer districts accounts for more organic loading than the sanitary sewage in the overflow. This effect, in combination with the extremely higher costs, the long implementation period, and the public inconveniences associated with sewer separation, tends to disqualify the sewer-separation approach as a feasible District-wide solution. Therefore, the remaining discussion of impact on water quality focuses on the other three approaches.

A two-year frequency is suggested as the minimum design frequency. While a two-year design frequency may seem small, it will retain and treat over 99 percent of the expected annual volume of overflow on a long-term basis. Two reasons explain this seemingly high value:

1. Practically all of the fifty to sixty overflows occurring yearly have volumes less than the volume of the overflow from the 2-year, 24-hour storm.

2. The volume of overflow does not differ greatly among significant storms, e.g. a facility designed to retain the overflow from the 2-year, 24-hour storm could retain 70 percent of the overflow from the 5-year, 24-hour storm.

Regardless of the selected design-storm frequency, there exists the possibility, however remote, of occurrence of a storm which would result in an overflow in excess of the design storage and treatment capacities. Nevertheless, implementation of a program based on a design storm whose recurrence interval is less than 25 years may represent an injudicious use of public funds. On the other hand, the very high storm recurrence intervals (50 years and more) are generally associated only with projects involving immediate risk of human life and/or catastrophic property damage.

The abatement of pollution from combined sewers requires storage facilities. Since any stored flow is to be treated, the quantity of retained overflow represents a practical measurement of the reduction in the pollution load. Figure 31 illustrates the reduction in BOD, SS, PO_4, or N corresponding to various retention capacities. It was assumed the percentage reduction in any pollution load is equal to the percentage of the total overflow collected, multiplied by the percentage removal efficiency of the subsequent treatment method (assumed to be 85 percent). Examination of Figure 31 shows that facilities designed to handle a 2-year, 24-hour storm (and provide 85 percent treatment) will provide 60 percent treatment for a 5-year, 24-hour storm but only 38 percent for a 25-year, 24-hour storm. Table 21 summarizes some of the information illustrated in Figure 31.

Figure 31 and Table 21 illustrate the necessity of evaluating combined sewer overflows in terms of their true impact as shock loadings. For example, effective storage capacity equal to the overflow from the two-year, 24-hour storm probably will reduce the expected annual loadings by just under 85 percent (the maximum possible). Nevertheless, with a storage capacity equivalent to the two-year, 24-hour storm it is expected that once in five years a loading equal to or exceeding 40 percent of the total loading in the five-year, 24-hour storm will be bypassed to the river. In terms of BOD, this represents a loading of 80,000 pounds, almost five times the maximum allowable daily loading in the entire metropolitan Washington area. Again, it is pointed out that this shock form of loading in combination with the long effective residence times of estuarine waters explains the serious impact of combined sewer overflows in the Washington area.

In making the final analysis of alternative strategies, Figure 31 can be used with the information in Tables 5 and 10 (in the Problem Definition section) to determine the pollutant loadings from any sewer district. While this procedure requires some simplifying assumptions, it will provide a sufficiently accurate estimate of pollutant loadings for comparison with other factors such as costs. Figure 32 represents the relationship between investment and the discharge of BOD for three intense-storm return frequencies. The

FIGURE 31

PERCENT REDUCTION IN POLLUTION LOAD FROM VARIOUS STORMS
VS
ALTERNATIVE DESIGN EVENTS

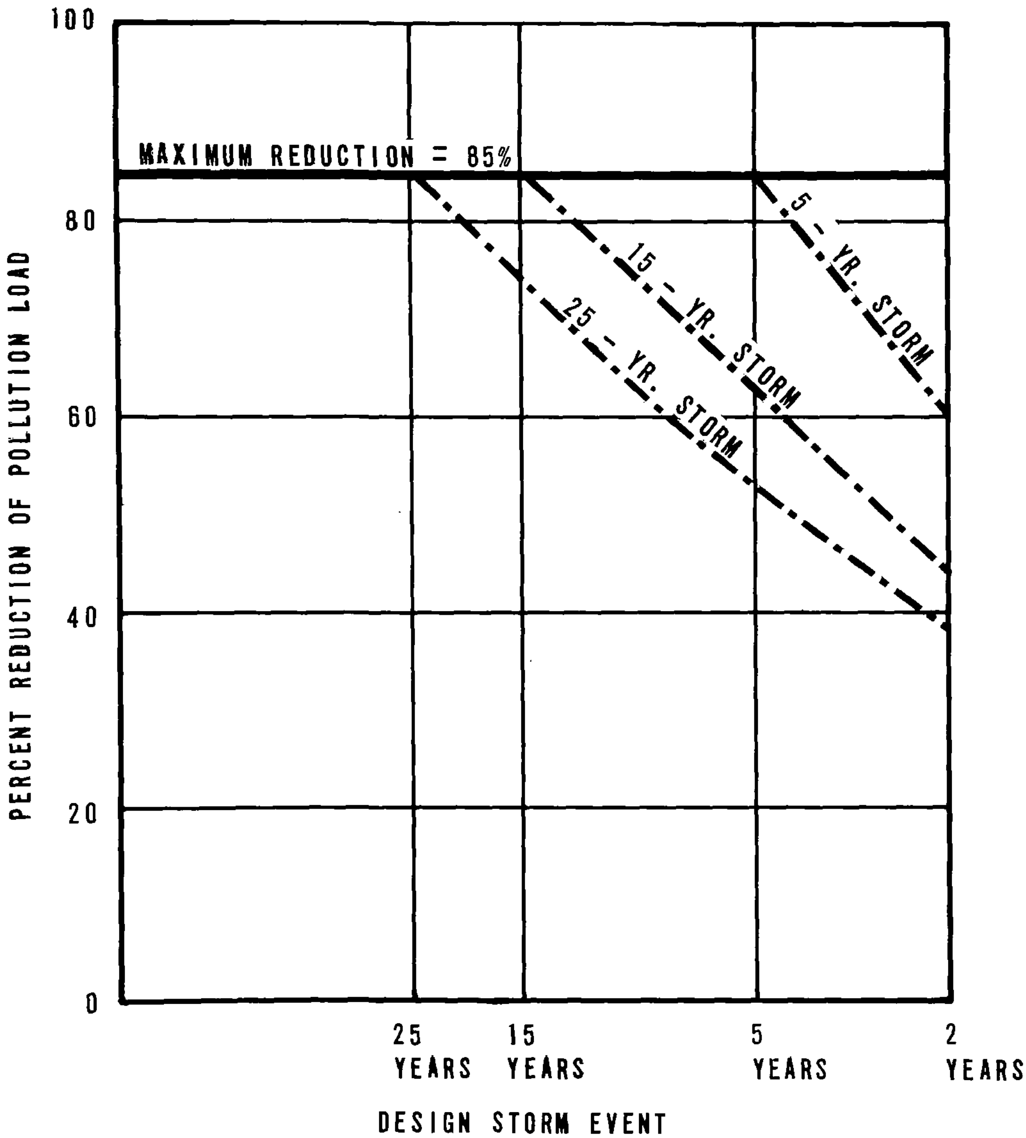
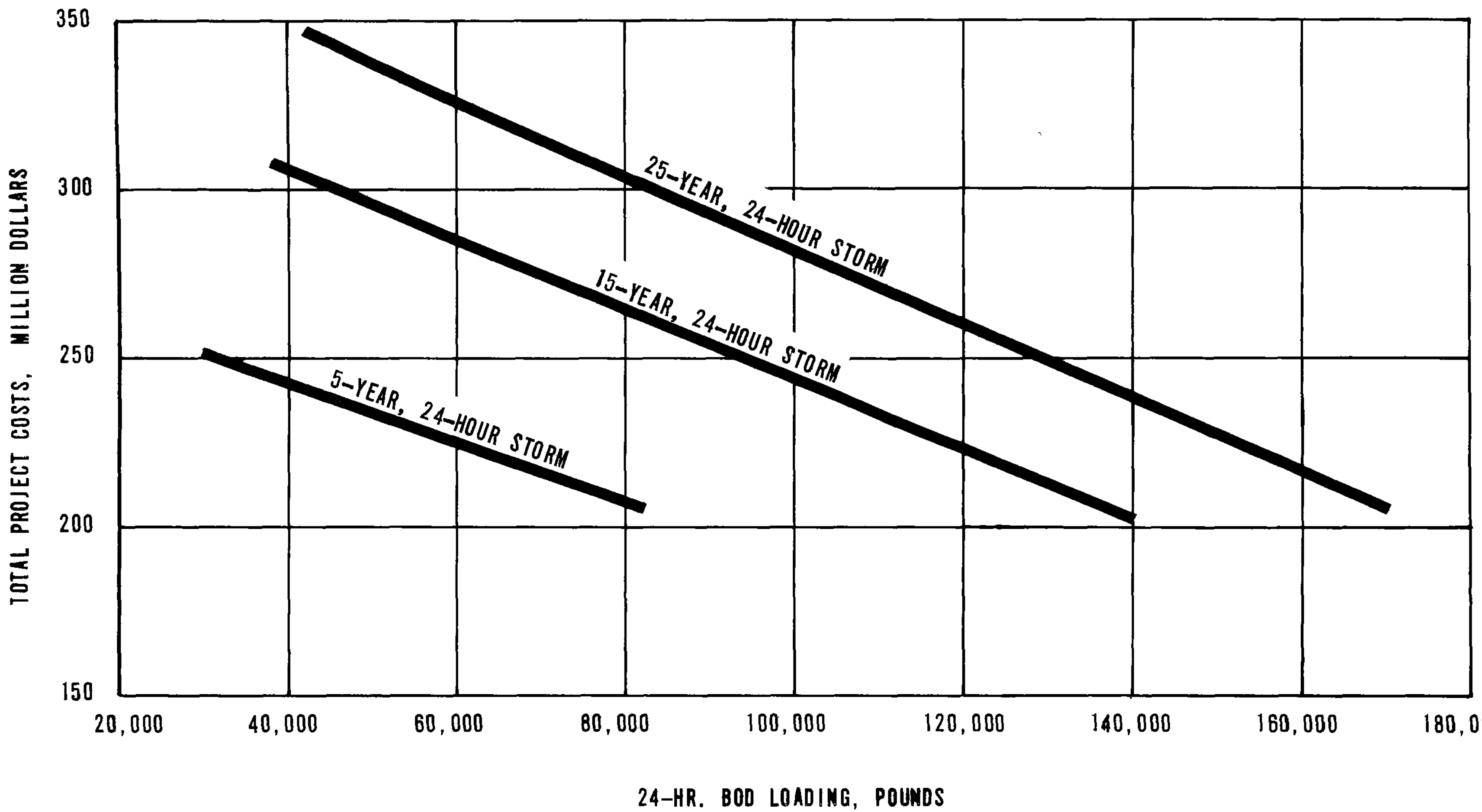


Table 21

Comparison of 24-Hour Pollution Discharges from Various Return Frequency Storms for Various Design Frequencies

Storm	No Treatment	Design Storm 2-Year, 24-Hour	5-Year, 24-Hour	15-Year, 24-Hour	25-Year, 24-Hour
2-Year, 24-Hour Storm					
BOD, lbs.	160,000	24,000	24,000	24,000	24,000
TP, lbs.	12,000	1,800	1,800	1,800	1,800
5-Year, 24-Hour Storm					
BOD, lbs.	200,000	80,000	30,000	30,000	30,000
TP, lbs.	13,000	5,000	2,700	2,700	2,700
15-Year, 24-Hour Storm					
BOD, lbs.	250,000	140,000	93,000	38,000	38,000
TP, lbs.	14,000	8,000	5,000	2,100	2,100
25-Year, 24-Hour Storm					
BOD, lbs.	280,000	170,000	130,000	76,000	42,000
TP, lbs.	15,000	9,000	7,200	4,000	2,300

FIGURE 32
EFFECT OF INVESTMENT ON DISCHARGE OF BOD LOADING FOR VARIOUS STORM RETURN FREQUENCIES



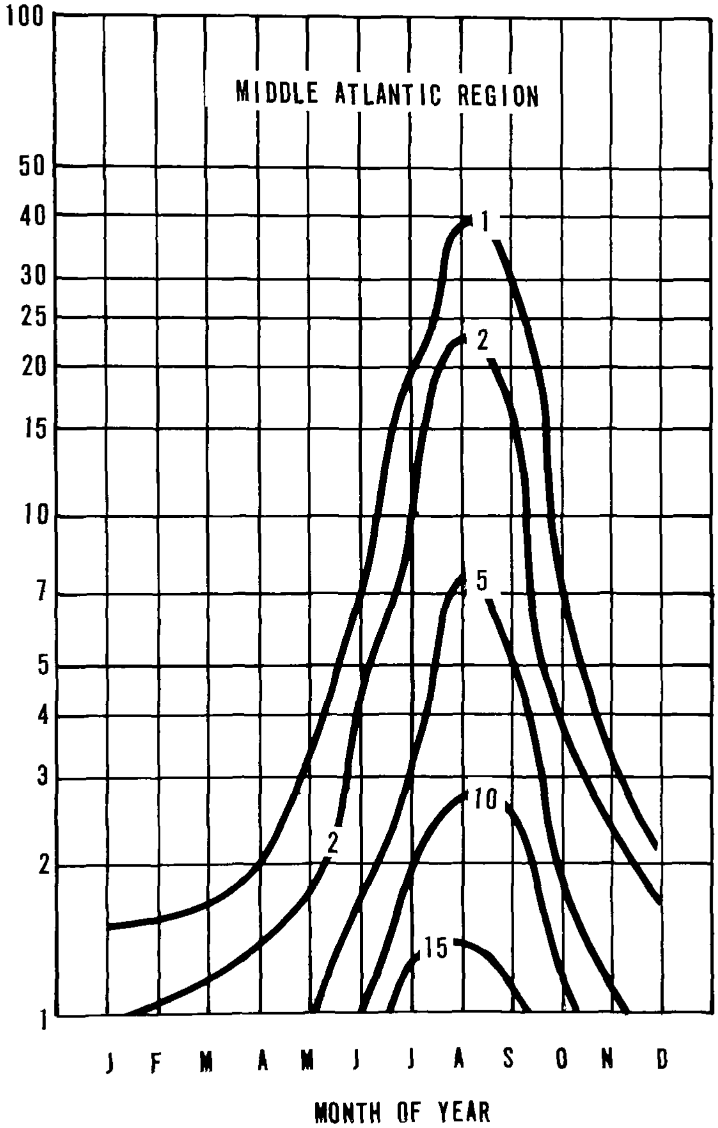
construction costs correspond to the approach which employs conveyance tunnels and mined storage District-wide. Note that each even-increment increase in investment results in a far greater proportionate decrease in BOD loading from intense storms. For example, an increase of investment from $250,000,000 to $300,000,000 results in a decrease of BOD loading, due to the 25-year, 24-hour storm from about 130,000 pounds to about 84,000 pounds.

Besides examining the costs and possible pollutant loadings when determining the design frequency, consideration must be given to the probability of an intense storm occurring during critical stream conditions. The recommended maximum pollutant loadings for the metropolitan Washington area were determined for summer conditions, when fresh water flow is low (less dispersion of waste), temperature is high (higher temperatures increase biological activity and reduce the dissolved oxygen saturation concentration), and plant respiration overnight has somewhat depleted the available dissolved oxygen. Water quality conditions usually are not as critical during other seasons of the year. A graph in a U. S. Weather Bureau Technical Paper (27) is reproduced as Figure 33 to provide insight as to the seasonal probability of intense storms. Note that in the months of July, August, and September there is a 15 percent probability of obtaining a rainfall equal to or exceeding that corresponding to a 5-year return period, while the probability of a similar occurrence during the remaining nine months is only 5 percent.

The preceding discussion has focused on the removal of those contaminants that deplete dissolved oxygen levels. However, the overflow from combined sewers also contributes to high bacteria levels and repulsive floating matter in the Potomac and its tributaries. While only 85 percent BOD removal can probably be effected from any retained overflow, total removal of floating matter and almost total disinfection is possible. Therefore, facilities based on a 15-year design frequency would rid the Potomac of high bacteria levels and repulsive floating matter from combined sewers for 14 years out of a 15-year period, and for practically all of the recreational season in the fifteenth year.

FIGURE 33

SEASONAL PROBABILITY OF INTENSE 24-HOUR RAINFALL



PROBABILITY IN PERCENT OF OBTAINING A RAINFALL IN ANY MONTH OF A PARTICULAR YEAR EQUAL TO OR EXCEEDING THE RETURN PERIOD VALUES.

SOURCE: U.S. WEATHER BUREAU TECHNICAL PAPER NO.40

Comparison of Non-Quantifiable Factors

To determine which of the feasible alternative approaches would offer the best opportunity for abatement of the combined sewer overflow problem requires careful consideration of many factors. Cost comparison obviously is essential, but there are other factors which also have a significant bearing on the selection of the most favorable plan. The most important of these other factors are Reliability, Flexibility, Land Requirements, Public Convenience, Implementation, and Solids Deposition and Gas Production. In general, the effects of these factors on the various alternatives are either non-quantifiable or are difficult to quantify. The relative impact on each alternative is discussed in the following paragraphs.

Reliability

An important consideration is the reliability of any system to perform its designed function whenever a storm occurs; there is no time to get things ready after the onset of a storm In regard to collection of the storm waters, all four approaches are equally (and strongly) reliable, because all rely primarily on gravity for capture and collection.

However, efficient capture and collection presupposes sufficient storage capacity, which means that the wastewaters collected from previous storms must have been moved out of the storage spaces to the treatment facilities or to the streams. This obviously is dependent on the readiness and capacity of the pumping facilities (except for sewer separation). In this regard, there is a difference, however small, between the various alternatives. Sewer separation would be the most reliable overall, because it would involve no dependence on pumping equipment. Conveyance tunnels and mined storage would be the next most reliable. Its requirement for two large pumping stations in comparison with many smaller pumping stations for the reservoir and local treatment approaches constitutes a reliability advantage, because of the inherent higher reliability of a larger installation and the relative ease of routine preventive maintenance.

In addition to the same number of pumps required for the reservoir approach, treatment at overflow points would involve equipment at treatment facilities, and this equipment would also have to be constantly maintained and operated to assure reliability. This dependence on additional equipment makes this approach not quite as realizable as the others. Nevertheless, no severe problems are anticipated with regard to reliability.

Flexibility

In addition to random occurrence of storms and the variations in storm duration and intensity for the Washington, D. C. area as a whole, there will be variations within the area. These local variations could be significant.

Each storage reservoir (in the storage reservoir approach) would serve a discrete sub-drainage area, and the probability of occurrence of a storm of such duration and intensity as to cause overflow is greater for any of the smaller discrete areas than for the overall area. Treatment at overflow points has reservoirs based on the same discrete drainage areas and is subject to the same degree of inflexibility as the reservoir approach, and the same is true for sewer separation.

A network of tunnels and mined storage would provide the greatest flexibility. Variations in storm occurrence, intensity, and duration within the Washington D. C. area would not per se consititute a problem, because flow from all sub-drainage districts would be dropped to the mined-storage chambers and equalized over the whole study area.

Land Requirements

Site investigations have been made to determine whether land is available at outfall points for construction of facilities required under the various approaches. In general, there is vacant land available at each of the overflow points. However, it should be noted that most of the potential sites are under the jurisdiction of the National Park Service, and their full cooperation would be necessary to obtain the sites.

Since construction at storage reservoirs would be underground, the only land required would be for access and maintenance. The ground above the reservoirs could be used for parks and playgrounds, with only relatively small pump houses extending above grade. Architecturally, the pump houses could be built to preserve the national park landscape.

The land required would be greater for the approach proposing treatment at overflow points. The additional housing for the pressure filters and appurtenances would require construction of buildings at each overflow point. Again, the structures could be built to preserve the aesthetics of the land under the jurisdiction of the National Park Service.

Land required for tunnel and mined-storage construction would be minimal. Only the land necessary for temporary occupancy during shaft construction would be required. Upon completion of construction, the work area would be landscaped to restore the original condition.

Sewer construction under a separation program would be in the City streets and rights-of-way. Therefore, for purposes of this report, no land is required for this approach.

Public Inconvenience

The disruption of public travel in Washington during the construction period is a major factor in the selection of an alternative approach. In fact, this may be the most important factor, possibly carrying more weight than costs or water quality. The present construction of the Washington Metropolitan Transit Authority subway illustrates the severe impact of a construction program that requires the closure of streets to traffic and the erection of barricades to pedesterians. Many businesses have had to close for indefinite periods, since customers refuse to frequent establishments on the disrupted streets. Traffic problems worsen as fewer avenues of travel and fewer parking spaces remain available.

Reservoir construction would be off the public streets. Traffic disruptions would be caused only by the construction vehicles required to move equipment and materials from the reservoir site.

For the local treatment approach, traffic disruption again would be only for movement of construction equipment and materials. However, due to additional equipment and more complex construction, the construction period and attendant traffic disruption would be for a longer period of time.

The tunnel and mined-storage approach would require virtually no disruption or inconvenience to residents and visitors. All construction would be underground and off the City streets. The only movement of vehicles would be to remove drilling muck. This operation could be done during the off-peak traffic hours; however, if moling progresses at a fast rate, continuous muck removal may be required.

Sewer separation would be the most disruptive to public travel. Virtually every street in the study area would be excavated at some point in time. The ramifications of such disruption would be difficult to assess completely, but it is certain that the disruption would be much more serious than with any of the other approaches. Besides this, sewer separation would require considerable work on private property to separate roof drains and other plumbing.

Implementation

Each of the four approaches can be implemented, but some will be easier and/or quicker than others. Considerations affecting the relative ease of implementation are:

1. Public attitudes and reaction.
2. Cooperation of other District and Federal Agencies.
3. Political implications and restraints.
4. Staging of construction and possible benefits with phased construction.
5. Financial schedules and monetary assistance programs.

Construction schedule, i.e. time required to complete each of the alternatives, is one of the critical elements involved in evaluating the adaptability of each approach to the establishment of stricter requirements, particularly in view of the strong pressures to improve the environment at the earliest possible date.

Before final design of storage reservoirs could be started, additional studies would have to be made to determine their exact location and the design parameters of each. The necessity to design reservoirs unique to each drainage district complicates the design; and the number of facilities required extends the construction period. It would take approximately eight years to complete the facilities required if this approach is applied District-wide where feasible.

Additional studies and individual design would also be necessary for treatment at overflow points. Due to the necessity to design special equipment, and taking into account the need to fabricate and install the special equipment, the time for completion would be longer than for the storage reservoir approach. It is estimated that a District-wide approach of treatment at overflow points could be completed in ten years.

The District-wide tunnel and mined-storage approach would take about eight years. However, the boring techniques are sufficiently advanced to allow relatively rapid construction.

Sewer separation would take the longest time, by far, to complete. On the basis of data presented in the 1957 Board of Engineers Report, it is estimated that the sewer separation program would be completed sometime after 2000.

Solids Deposition and Gas Production

Another major consideration in the selection of an approach is the problem of solids deposition and concomitant gas production. Solids deposition would have a three-fold effect:

1. Deposition of solid material could reduce storage capacity.

2. Organic solids may decompose to form malodorous compounds thus causing a nuisance condition.

3. Organic solids may decompose to form explosive organic gases.

Settleable solids deposited in the settling zone of each reservoir could be pumped back to the existing system for ultimate disposal at Blue Plains. Solids carried over to the storage zone would be pumped to Blue Plains through the raw sewage pumps. If the reservoirs are emptied relatively soon after the storm subsides and regular maintenance by flushing is provided, neither septicity nor gas production is expected to be a problem. To insure that gas production is not a problem each of the reservoirs would be equipped with adequate ventilating and air-pollution control devices.

The solids deposition and gas production would have about the same impact on the approach using treatment at overflow points as for the reservoir approach. In addition to solids removed in the settling zone of the storage structures, backwash from the fiberglass filters would be discharged to the existing system and conveyed to Blue Plains.

The field studies presented in the first part of this report have determined that a high concentration of solids occurs during the first few minutes of the storm Therefore, the major portion of the solids will be carried into the tunnels (under this approach) during the first few minutes. This flushing action will carry the solids to the low point and facilitate solids removal. The mined storage would be compartmentized and the first few compartments used as settling basins to facilitate removal of solids. However, the great depths and vast extent of these tunnels may present problems in regard to effective solids removal.

With a separation program, solids carried into the storm sewer would be discharged to the receiving stream. As determined during the field studies, the amount of solids delivered to the stream would be significant.

Evaluation of Factors Other Than Cost

The results of the evaluation of comparative reliability, flexibility, land requirements, and other non-cost factors are summarized in Table 22. In view of the difficulty in properly assessing the relative worth of each factor in quantitative terms, no quantifiable measurement is employed in this presentation. Instead, a measurement scale of X's indicates the degree of the problems possible with each approach. An overall evaluation indicates a clear advantage for conveyance tunnels and mined storage over the other three approaches, provided the solids question can be answered.

Table 22

Evaluation of General Factors

Factor	Maximum Use of Storage Reservoirs	Treatment at Overflow Points	Conveyance Tunnels and Mined Storage	Sewer Separation
Reliability	X	X	X	–
Flexibility	X	X	–	X
Land Requirement	XX	XX	–	–
Public Inconvenience	X	X	–	XXX
Implementation	XX	XX	X	XXX
Solids Deposition and Gas Production	X	X	XXX	–

Key:
– No Problem
X Possible Minor Problem
XX Possible Moderate Problem
XXX Possible Major Problem

As previously pointed out in the comparison of the cost estimates presented in Tables 15 though 20, the conveyance tunnel-mined storage approach would also involve the lowest construction cost of any of the four approaches and lower operating costs than either storage reservoir or treatment at overflow points (Operating costs, which would be primarily maintenance, were not calculated for sewer separation). Thus, the evaluation of general factors reinforces the selection (on a cost basis) of conveyance tunnels and mined storage as the approach to be adopted.

Indicated Appropriate District-wide Solution

The preceding sections of this report illustrate how it is possible to develop within the District numerous strategies for collecting, storing, and treating combined sewer overflow. Each strategy offers a different level of costs, pollutant loadings, and benefits. While the selection of the appropriate strategy must rest, in part, on factors beyond the scope of this study, the information presented in this report indicates that the appropriate District-wide solution may be one which centers on a single approach, the use of mined storage and conveyance tunnels.

The recreational and other values of the Potomac River and its tributaries demand that these waters be protected from pollution. Further study is needed to predict the effect of shock contaminant loadings to the Potomac and its tributaries. Therefore, the selection of the appropriate design frequency requires a somewhat arbitrary decision at this time. It is common practice to use a 10-year minimum flow recurrence frequency as the design basis for treatment facilities on fresh water streams. While the situation for estuarine waters is different, this basis can be interpreted to mean that all combined sewer overflows from storms-of-lower magnitude than the design storm must be entirely intercepted, stored, and treated. Quite possibly, extending this to the 15-year design frequency represents a reasonable and conservative decision in view of the small increase in costs (see Figure 30) and the significant decrease in pollution loadings from the intense storms (see Figure 32). Selection of the 15-year, 24-hour design event is further supported by consideration of the water-contact activities proposed for the Potomac River and its tributaries. The indications are that facilities based on this design event would permit all the desired recreational activities with practically no interruption.

Table 10 in this report shows that the expected annual volume of combined sewer overflow from the District is 12 billion gallons. The Blue Plains plant can completely treat 289 mgd above the average dry-weather flow for 400 hours per year. This is equivalent to an annual reserve capacity of 5 billion gallons. While the Blue Plains plant can partially treat some of the remaining 7 billion gallons, it is obvious that treatment at some other location is required. The conceptual engineering study (6) of the Kingman Lake project has identified the feasibility of constructing a multiple-purpose treatment plant in that area. Actually, as brought out in that study (see Appendix G), this location for a treatment facility offers a wide number of benefits, probably more than any other location in the District. The original concept of a 50-mgd plant in this area could probably be expanded to a 150-mgd plant (the 100-mgd increase could be directed simply to filtration and chlorination rather than to complete treatment). This would handle in about five days the 772 million gallons of overflow from the Anacostia River area for the 15-year, 24-hour storm. This single facility, in combination with the reserve hydraulic capacity of the Blue Plains plant, would provide sufficient capacity to treat the annual volume of combined sewer overflow from the entire District.

Concentrating treatment capacity at these two locations, rather than at many more locations, affords another advantage. Even with complete retention of overflow from the 15-year, 24-hour storm, the assumed 85 percent treatment efficiency would still allow a 24-hour discharge of about 38,000 pounds of BOD to the Potomac. This loading equals twice the recommended maximum daily loading in the Metropolitan Washington area, and perhaps this would deplete dissolved oxygen below the minimum acceptable levels. However, the treatment provided at the Blue Plains advanced plant and at the Kingman Lake reclamation plant would result in an overall treatment efficiency higher than 85 percent, and less than 38,000-pound loading would result.

A network of conveyance tunnels and mined storage is suggested because no other approach offers capacity for the overflow from the 15-year, 24-hour storm at a lower cost or at a higher level of benefits. The total project costs of the indicated appropriate solution is about $318,000,000, as shown in Table 23. Note that this cost includes the costs of dealing with those sewer districts scheduled to be separated before 1975 whose separation has been delayed indefinitely. A solution incorporating mined storage in the Anacostia River area and with both storage reservoirs and mined storage in the Rock Creek-Upper Potomac area (see Figure 24) comes somewhat close to this figure, with a cost of $325,000,000. By inspection of Tables 12 and 15, it is possible to reduce this figure by $4,800,000 if the method of separation rather than storage reservoirs is applied to Sewer Districts A-10, A-11, and A-12.

Actually, a cost less than $318,000,000 is possible with a solution designed for the 2-year, 24-hour storm in one area and the 20-year, 24-hour storm in another. This represents both overdesigning and underdesigning, and produces the same reduction in pollutant loadings from the 25-year storm as a uniformly applied 15-year design frequency. Economies of scale explain this cost difference. However, this scheme would allow a higher loading from the 5-year storm than would a uniformly applied 15-year design frequency.

Although the selection of the appropriate solution must consider some factors beyond the scope of this study, the information developed herein does point to a particular solution. <u>In summary, this solution is one that provides a network of tunnels and mined storage, with treatment at the Blue Plains plant and at a facility near Kingman Lake. A 15-year, 24-hour storm event (or one based on a longer recurrence interval) is suggested at the design basis, in the public's best interest.</u>

Table 23

Estimated Capital Costs of Indicated Appropriate Solution
(ENR Index = 1800)

Feature	Costs
Rock Creek Upper Potomac	
1. Tunnel Construction	
a. Rock Creek	$ 18,200,000
b. Upper Potomac	2,000,000
2. Vertical Shafts	
a. Rock Creek	4,400,000
b. Upper Potomac	2,100,000
3. Mined Storage	57,000,000
4. Pumping Station	6,200,000
Anacostia River Area	
1. Tunnel Construction	11,100,000
2. Vertical Shafts	5,800,000
3. Mined Storage	94,000,000
4. Pumping Station	10,900,000
5. Northeast Boundary Relief/Storage Tunnel	27,000,000
6. 150-MGD Treatment Plant at Kingman Lake[1]	12,000,000
Sewer Districts Scheduled for Separation prior to 1975	10,000,000
Total Construction Costs	261,000,000
Engineering Design at 6%	16,000,000
Other Costs (resident inspection, legal, bonding, and administrative at 15%)	41,000,000
Total Project Costs	$318,000,000

[1]Costs based on 150-MGD plant incorporating processes depicted in Figure 15 (filtration and chlorination) rather than complete water reclamation processes.

SECTION VIII

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SECTION IX

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SECTION X

APPENDICES

APPENDIX A

GEOLOGICAL AND OTHER NATURAL CONDITIONS

Introduction

Information on the geologic and other natural conditions is essential to a complete evaluation of the various possible approaches to the abatement of pollution caused by the inadequacies of combined sewers, including the feasibility of tunneling beneath the City of Washington to provide for storage of combined overflows.

Data were obtained from published geologic reports, from engineering reports prepared under the auspices of the Metropolitan Area Rapid Transit Authority, and from verbal communication with personnel from the Authority. The published geologic data are fairly broad in coverage, although sufficiently detailed to provide information helpful in forming conclusions in regard to the feasibility of tunneling. The previously-mentioned engineering reports are extremely detailed and provide an abundance of data concerning the soils, bedrock, and rock mechanics along certain restricted areas for the purpose of underground tunneling for rapid transit. Although somewhat restricted as to area-wide application, the data included in these reports provide enough coverage to be representative of the overall Washington area.

Geologic Setting

The District of Columbia lies within portions of two physiographic provinces; the southeastern portion is located within the Coastal Plain Province, which consists of relatively flat-lying sediments overlying deep bedrock, and the northwestern portion is in the Piedmont Province, which in general is characterized by a thin layer of overburden covering crystalline bedrock. The Fall Line separating the two provinces extends roughly southwest from Blair Park in the northeast through Farragut Square and on toward the Pentagon. Figure A-1 presents the generalized geology of the District of Columbia.

Previous subsurface investigations throughout the District have resulted in grouping the materials into five major categories: bedrock, Cretaceous sediments, Pleistocene terrace deposits, recent river alluvium, and drainage channels and man-made fills. These major categories of materials in various parts of the District are found in the following five vertical profiles:

1. Recent alluvium over bedrock or Pleistocene terrace deposits,

2. Overburden of Pleistocene terrace and Cretaceous coastal plain soils above deep bedrock,

3. Comparatively thick cover of Pleistocene terrace and Cretaceous coastal plain soils above deep bedrock,

4. Thin to moderately thick cover of Cretaceous coastal plain materials above decomposed rock and bedrock,

5. Relatively thin cover of man-made fill and decomposed rock and bedrock at shallow to moderate depths.

Principal Unconsolidated Formations

Cretaceous Sediments

Lower Cretaceous sediments belonging to the Potomac group overlie weathered bedrock throughout most of the downtown area of Washington. These sediments dip gently to the southeast, thickening in that direction. In general, they vary in lithology from weathered soft clayey materials to dense sand, silt, and gravel. In other areas along the Atlantic Coastal Plain, the Potomac group is overlain by upper Cretaceous and Tertiary sediments. However, in downtown Washington, these younger formations plus a considerable thickness of the Potomac group have been removed by past erosion. The sediments occur over sufficient areas and at elevations appropriate for tunnel storage.

Pleistocene Terrace Deposits

Pleistocene terrace deposits are a succession of river deposits, which generally overlie the Cretaceous sediments or decomposed rock. They consist of a heterogeneous mixture of interbedded sandy clays, sand, and gravelly sands.

Previous investigations have determined that the terraces occur at several characteristic elevations in the Washington area, e.g. the "25-foot terrace", "50-foot terrace", and the "90-foot terrace". The terraces characteristically show a change in gradation (in vertical profile) from coarse-grained soils at their base to fine-grained sands, silts, and clays at shallower depths. In general, the Pleistocene deposits occur at elevations which are considered too high for storage tunnels.

Recent Alluvium

In comparison to the other unconsolidated formations in the Washington area, the recent alluvium is relatively restricted in areal extent. It occurs primarily along the Potomac River as far west as the mouth of Rock Creek, and along the flats bordering the Anacostia River. The alluvial material consists of fine-grained organic sand and silty to sandy clay with lenses of peat, and may have a thickness of as much as 25 feet.

Artificial Fills

Cutting and filling of irregular natural topography has been quite extensive throughout the Washington area. Reference to old maps indicates that extensive filling has taken place along the low areas bordering the Potomac and Anacostia Rivers, and in East Potomac Park, National Airport, the Navy Yard, and the Southwest Mall area. In addition, the drainage systems of the Tiber and St. James Creeks and of Slash Run, which were originally located in downtown Washington, have been covered with fill.

Several hills composed of Pleistocene terrace materials have been removed and the material utilized to fill nearby low areas. Extensive cuts and fills have been made along Connecticut Avenue north of Rock Creek. Due to the lack of continuity of the fills and to the elevation at which they occur, they are not considered suitable of tunneling.

Bedrock

Bedrock in the Washington area consists of crystalline metamorphic schists and gneisses of Paleozoic age. It originated in upgrading by metamorphism of wide areas of sandy and clayey sediments to form the lithologic complex known as the Wissahickon formation, which is a part of the extensive band of crystalline rocks that extend from New England to Alabama. The general category is schistose gneiss, which is essentially a metamorphosed sedimentary rock with zones of various mineral compositions. In many areas of Washington, rock is overlain by compact residual soil which is derived from weathering and decomposition of the underlying parent rock. Northwest of the Fall Line, remnants of residual soil over 60 feet thick have been encountered, while southeast of the Fall Line erosion has removed much of this material leaving an average thickness of only five feet.

General Depths of Bedrock

The bedrock surface southeast of the Fall Line dips southeastward at a rate of approximately 60-125 feet per mile, thus occurring at increasing depths toward the southeast. In the area of the Anacostia River, it occurs between 300 and 400 feet below mean sea level. Northwest of the Fall Line, bedrock begins to outcrop at the surface, although in many areas it is overlain by a cover of unconsolidated material. In this area, bedrock generally occurs between the surface and a depth of about 75 feet. Figure A-2 depicts the general configuration of the bedrock surface underlying the City of Washington.

Depth of Rock Weathering

A study of the logs of bore holes drilled for the Metropolitan Area Transit Authority indicates that the upper surface of the bedrock generally is weathered to some extent. The extent of weathering is largely dependent on rock type and on the frequency of jointing. The logs indicate the general depth of weathering to be on the order of 5 feet, although weathered thicknesses up to 17 feet were noted in several of the borings.

Rock Types

Through subsurface investigations, three interfingered general bedrock types have been identified along Connecticut Avenue. These include schistose gneiss, chlorite schist, and quartz-diorite gneiss; these are described below, since each has different characteristics in regard to tunnel construction. These rock types are considered to be generally representative of the rock and conditions underlying Washington.

Schistose Gneiss

The schistose gneiss includes three rocks of distinctive mineral composition: hornblende gneiss, quartz-hornblende gneiss, and quartz-biotite gneiss. The hornblende gneiss is characterized by its dark color and high content of hornblende crystals, combined with lesser amounts of quartz and feldspar. Compared with the other two rock types, schistose gneiss is more subject to weathering and lower in compressive strength and modulus of elasticity. The quartz-hornblende gneiss is distinguished by its "salt and pepper" appearance, and has the highest compressive strength and elastic modulus of the schistose gneisses. The quartz-biotite gneiss is similar in character to the quartz-hornblende gneiss, but has slightly less compressive strength and modulus of elasticity.

Chlorite Schist

The chlorite schist is a gray-green to dark green rock, which corresponds to the soapstone that has been previously mapped from outcrops northwest of the Fall Line. The occurrence of talc and the presence of joint planes in which the chlorite schist has weathered to clay make the formation structurally the weakest of the bedrock types. It has been encountered in the valley of Rock Creek and in other areas lying to the north. Because of the weakness of the chlorite schist, tunnels would probably require structural support whenever this rock type is encountered. The existing data indicate that this rock type is of relatively limited extent.

Quartz-Diorite Gneiss

The quartz-diorite gneiss (also referred to as granite gneiss or biotite gneiss) is a light-colored, coarse-grained gneiss with prominent flakes of biotite mica. Eighty to ninety percent of the rock is composed of feldspar and quartz. It has been encountered north of Klingle Creek, along Connecticut Avenue. In general, the quartz-diorite gneiss is the most structurally favorable bedrock from the standpoint of tunnel construction. It is the strongest and least weathered of the bedrock types and has the highest compressive strength and modulus of elasticity. Tunnels constucted in this rock type would probably require the least amount of structural support.

Structure of Bedrock

Faults and Folds

The crystalline bedrock floor underlying the Coastal Plain sediments has been folded and faulted to some extent during the geologic past. Since early Cretaceous time, much of the region has been uplifted and depressed many times, accompanied by folding and faulting of the bedrock surface. However, due to the thickness (and in some cases, partial removal of the overlying sediments), data on the directional trend and amount of movement of these folds and faults are extremely difficult to obtain. The predominant trend of these movements and resultant structures has been along northeast-south-west axes, although localized variations, both in trend and amount, do exist. Several faults in bedrock and the overlying sediments have been observed in the northwestern part of Washington, but the displacements appear small and the faults short. One fault, which was exposed in a trench on 18th Street near California Avenue, showed a displacement of 40 feet where Potomac group sediments and crystalline bedrock had been thrust in contact with each other. Another fault, on Adams Mill Road, has a vertical displacement of only 8 feet.

Joints and Foliation

Previous investigations have been made in regard to determining the general foliation pattern and the primary and secondary jointing pattern of the bedrock underlying Washington. Foliation is a layering within a rock caused by segregation of various constituent minerals due to metamorphism. It is an important factor to consider in regard to tunnel construction in that rock breakage may be controlled by the attitude of foliations. Joints are fractures or partings which interrupt the physical continuity of a rock mass. The attitude and frequency of joints are extremely important criteria in the construction of tunnels, and zones of intense jointing are areas which may require special structural treatment. A detailed examination of outcrop in Rock Creek Park was made by R. E. Fellows in 1950. Another investigation made by the Metropolitan Area Transit Authority along Connecticut Avenue utilized bore hole photography to determine the attitude of foliation and jointing. The results of both investigations show a good correlation of attitudes and indicate a definite consistency throughout the study areas, with little effect from lithologic variations. Based on the results of the above-mentioned investigations, the prinicipal conclusions regarding the attitude of foliation and jointing are as follows:

1. The general foliation pattern trends directionally between the true north and N20°E, and dips at angles from the horizontal between 45° and 75°, with an average angle of 60° to the northwest.

2. The primary or major jointing pattern trends directionally between N70°W and N80°W, with the joint surface dipping to the northeast between 45° and 75° from the horizontal.

3. The secondary jointing pattern trends directionally between N20° and N40°E, with dips ranging between 30° and 60° to the southeast.

Frequency of Secondary Openings

For the purpose of this study, secondary openings are considered to be joints which are defined as fractures or partings which interrupt the physical continuity of a rock mass. Joint frequency is a criterion by which the quality of a rock mass can be judged; the greater the frequency of joints, the lower the quality of the rock mass. In brief, frequency is based on the number of joints occurring within a given interval of rock core or bore hole, computed on the basis of joints per foot. Frequency of joints was studied during the Metropolitan Area Transit Authority investigations, which included the use of bore hole photographic logs and of core samples recovered from the bore holes. The logs indicated a frequency of joints of one for every two feet of depth. Examination of core samples showed a frequency range of 1.3 to 1.9 joints per foot of depth. Concentrations of closely spaced joints in broken zones were omitted in making the frequency counts. It should be assumed that localized shear zones with more intense jointing may be encountered, particularly in contact zones between different rock types. In regard to tunnel concentration, zones of intense jointing might necessitate structural support or sealing off of the zone to prevent inflow of ground water.

Hydrology

Present and Potential Ground-Water Use

Ground water in the Washington area is obtained from both the unconsolidated Coastal Plain sediments and from the crystalline rock of the Piedmont sediments and from the crystalline rock of the Piedmont province. In general, the upper portion of the Coastal Plain sediments are clayey and contain few highly productive aquifers; the older and deeper strata are sandier and furnish moderate yields to wells. As of 1960, approximately 2 million gallons per day were being pumped from wells located within Washington; this is lower than the pumpage rates of earlier years. Most of the well water pumped in 1960 was from wells located at the railroad terminal and at other industrial and commercial sites in the area east of North and South Capitol Streets. Since the Piedmont section of Washington is largely residential and is supplied with surface water, very little ground water is utilized there. However, ground-water studies of nearby Montgomery and Howard Counties in Maryland indicate that the crystalline rock of the Piedmont is capable of yielding as much as 180 gallons per minute per well.

It is estimated that several additional million gallons per day of ground water can be withdrawn from the aquifers underlying Washington without appreciably affecting the present hydrologic balance. The general water supply for Washington is from surface sources, in particular the Potomac River, and is considered adequate for present and future demands. However, serious consideration must be given to protecting the valuable ground-water resource from pollution and depletion, in case further demands and changes in the water budget necessitate its use.

Ground-Water Levels

Ground-water levels in the Washington area are dependent on several factors, such as seasonal changes of infiltration, presence of storm sewers to carry water away, temporary or permanent pumping associated with construction, and variation in river levels. Three observation wells used by the U. S. Geological Survey indicate that the typical yearly variation, from a high in February or March to a low in August or September, is approximately 5 feet. A study of well data and water levels in the borings made by the Metropolitan Area Transit Authority shows that the ground water level in Washington generally occurs at depths between 16 and 38 feet.

Permeability of Rock and Unconsolidated Sediments

Permeability tests performed on the various lithologies encountered indicated that, in general, the overall permeability decreases with depth and increasing age of the deposit, although local exceptions to this should be expected.

The overall media permeability coefficients for the various lithologies are as follows:

1. Pleistocene sands and gravelly sands 5×10^{-4} feet/minute (fpm)
2. Cretaceous sands and gravelly sands 3×10^{-4} fpm
3. Decomposed rock 1×10^{-4} fpm
4. Bedrock of all types 4×10^{-5} fpm

Tunneling

Types of Tunnels

Two general tunnel types, earth and rock, have been considered for the purpose of storage. The earth tunnel, which would be located in the area of coastal plain sediments, would require a complete lining. It is anticipated that the rock tunnel type would be constructed in areas where depth to bedrock is not too great. This type would probably require a minimum of lining and structural support.

Certain desirable geological criteria reguarding the feasibility of rock tunneling have been established as follows:

1. Rock type capable of being tunneled with a minimum of structural support.

2. Bedrock with a minimum number of shear and fault zones, which would require additional structural support.

3. Bedrock of high stability and relative insensitivity to tectonic disturbances.

4. Bedrock of low porosity and permeability, to minimize the amount of water seepage into tunnels.

5. Location below the water table, to prevent contamination of ground water.

6. Sufficient depth, to avoid any effects of overlying structures or nearby existing tunnels.

7. Construction in fresh unweathered rock.

8. Location as shallow as possible, yet conforming to the above criteria, to keep the economics of construction and operation at a minimum

Undesirable conditions for rock tunneling are as follows:

1. Bedrock types with thick or irregular zones of weathering.

2. Bedrock susceptible to rapid weathering.

3. Rock types of high permeability.

4. Areas of variable rock types.

5. Easily deformable bedrock.

6. Bedrock that has been subject to much faulting and shearing.

7. Unstable areas subject to tectonic disturbances.

Experience has shown that the rock types underlying the Washington area tend to become less weathered with increasing depth. In addition, as depth increases, secondary openings become tighter and occur with less frequency. Tunnels constructed for the storage of the combined flow will probably be located at such a depth that most of the weathered zones will be avoided.

Existing Tunnels

Earth Tunnel

An earth tunnel was recently constructed in southeast Washington for the District Sanitary Engineering Department. It is a 12-foot diameter sewer tunnel extending from L and Half Streets, S.E. to the District pumping station at the Navy Yard. The tunnel was excavated through recent alluvium and terrace deposits utilizing a shield-driving method. During construction of the tunnel, dewatering was provided by deep wells spaced at minimum intervals of 300 ft. Although the quantity pumped was small, it was felt that sloughing of cohesionless sand was significantly reduced.

Rock Tunnel

A rock tunnel was completed in 1966 for the National Park Service on Rock Creek and Potomac Parkway. The tunnel is located near the Zoo and about 1/2 mile from Connecticut

Avenue. It is 780 feet long with a 30-foot diameter, and extends through a rock ridge at a distance of 80 to 100 feet from ground surface to tunnel base. The weathered condition of the rock encountered during construction necessitated that a concrete lining be used for the entire length. Problems from heavy overbreak in the rock occurred during construction. Seepage of ground water caused little difficulty, except where bore holes had not been grouted prior to construction to provide storage of the combined sewer flows, because most of the construction will be in massive unweathered bedrock.

A more typical example of rock tunneling was encountered in the construction of the Lydecker Water Tunnel (1888-1902). This tunnel extends from Georgetown Reservoir to MacMillan Park Reservoir, and runs at depths ranging from 60 to 170 feet. The tunnel has a horseshoe-shaped section and is brick-lined for most of its length; however, the section underlying Rock Creek is circular and steel supported. Except for some minor cracks and bulges that developed near the Georgetown end of the tunnel (which subsequently needed additional support), no evidence has been reported of swelling or breaking rock.

Rock Drillability

Laboratory tests, performed on the different rock types for the purpose of estimating the practicability of using tunnel excavation machines in the Washington area, measured various factors relating to rock hardness and strength. A combination of rebound hardness and abrasion hardness, known as "total hardness", is thought to be the most closely related to machine drillability. Past performance indicates that rock with total hardness values less than 120 has been successfully drilled by machines presently in use. The results of the tests are summarized in the following tabulation:

	Compressive Strength p.s.i.	Modulus of Elasticity p.s.i.	Total Hardness
Chlorite Schist	5,000	2×10^6	20 - 30
Schistose Gneiss	10,000	5×10^6	90
Quartz-diorite Gneiss	15,000	8×10^6	110 130

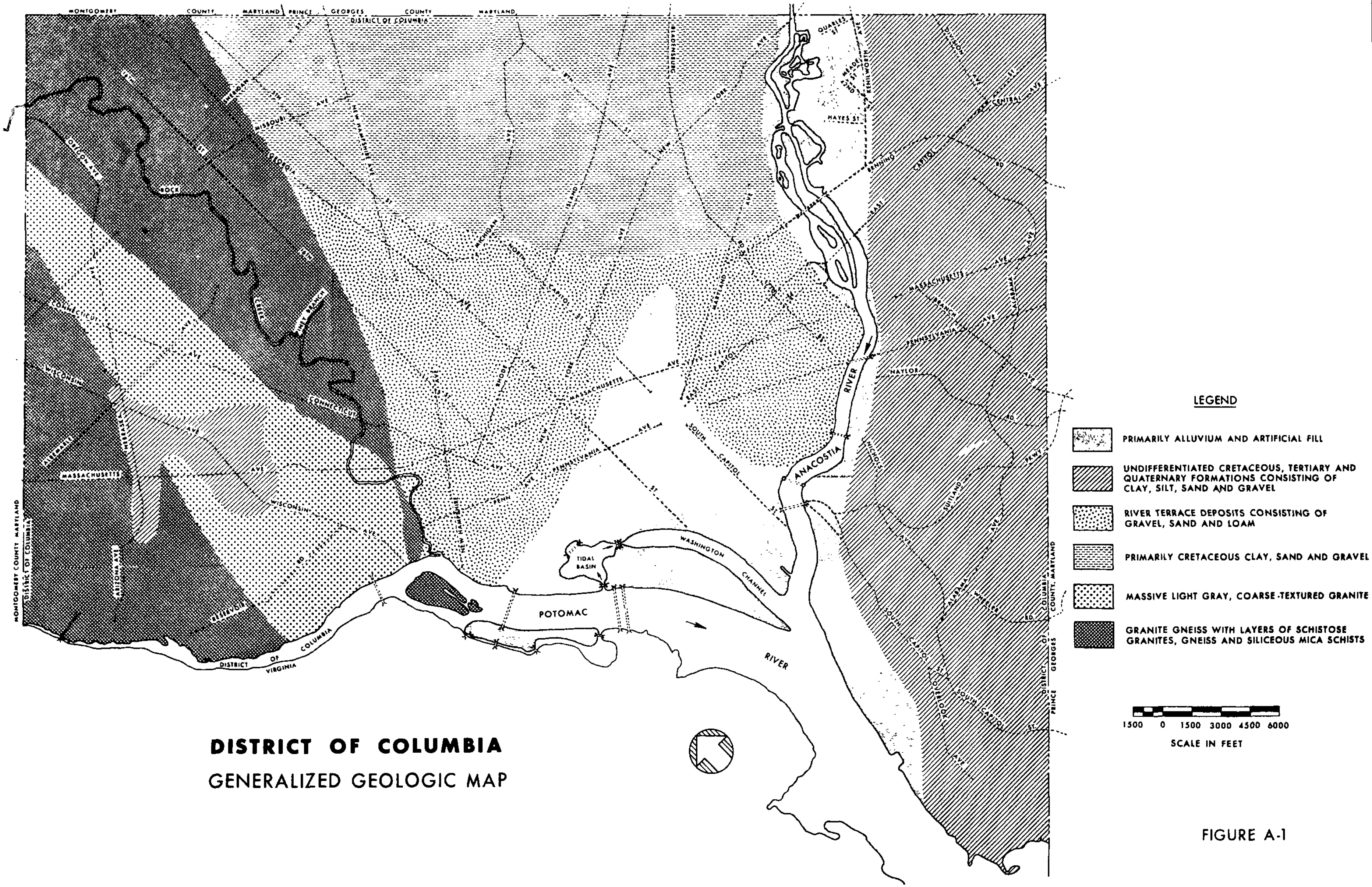
It appears that the chlorite schist and schistose gneiss can be quite easily tunneled by the machines now in use. On the other hand, the quartz-diorite gneiss would probably present difficulty with respect to maintenance, drilling progress, and cost.

Estimates of the rate of drilling, based on laboratory tests and on a measurement index known as "Reed" drillability, indicate the rate of mechanical boring in feet per hour. The following rates have been estimated for the three general rock types: quartz-diorite-gneiss 3 ft/hr.; schistose gneiss 3-1/2 4-1/2 ft/hr.; chlorite schist 8 10 ft/hr.

Feasibility of Tunneling

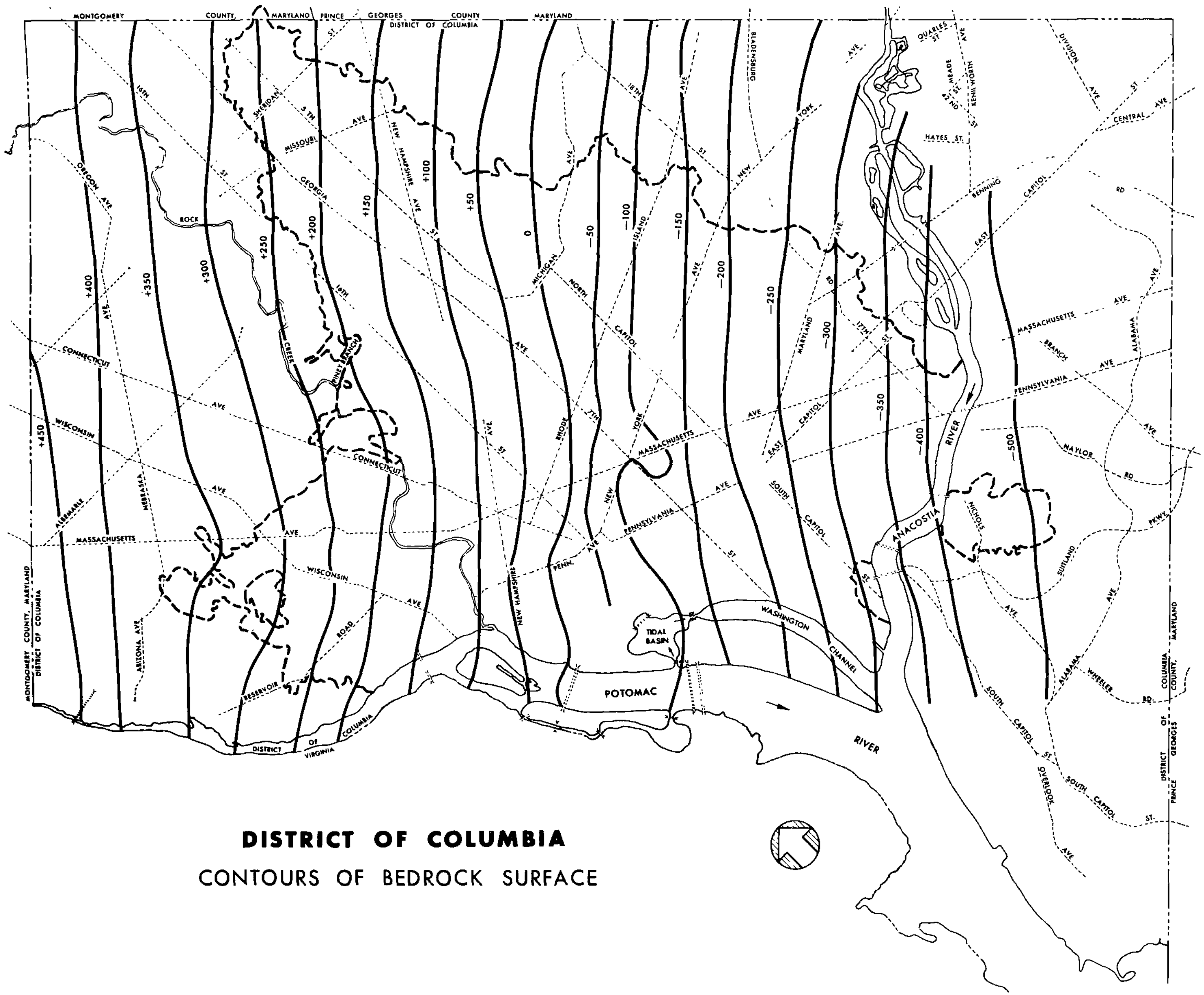
Based on existing geologic literature and the Transit Authority investigations, tunnels for storage of combined sewer flows appear to be feasible. The bedrock geology of the Washington area in general meets most of the criteria necessary for successful rock tunneling. Although there are areas where the condition of the bedrock will require much additional structural support, most of the bedrock should be capable of being tunneled with a minimum of lining and structural support. It is probable that the principal mode of support would be a combination of rock bolting and several layers of pneumatically-applied "shotcrete" covering the roof area of the tunnel. Because of the low permeability of the bedrock, seepage of ground water will probably be insignificant except in areas where the rock has been intensely sheared. In these cases, the shear zones will have to be sealed to prevent the inflow of large quantities of ground water into the tunnel. In general, the Washington area is relatively stable tectonically; therefore, no particular problems are anticipated in regard to rock deformation and faulting due to tectonic activity.

Earth tunnels are considered feasible from a geologic standpoint. At this stage of the investigation, the best sediments in the Washington area in which to construct lined-earth tunnels appear to be the more dense Cretaceous clays, sands, and gravels. They are considered best because of their greater horizontal continuity and their occurrence at more suitable elevations. Even though earth tunnels are feasible geologically, the fact that a complete lining is required may inhibit their use for sub-surface storage, for economic reasons.



DISTRICT OF COLUMBIA
GENERALIZED GEOLOGIC MAP

FIGURE A-1



1500 0 1500 3000 4500 6000
SCALE IN FEET

DISTRICT OF COLUMBIA

CONTOURS OF BEDROCK SURFACE

FIGURE A-2

APPENDIX B

A LIST OF PERTINENT STUDIES AND REPORTS REVIEWED

1. Metcalf and Eddy, "Report Upon Investigation of Sewerage System", 1955.
2. Board of Engineers (Greeley, S. A., et al), "Report to District of Columbia Department of Sanitary Engineering on Improvements to Sewerage Systems", 1957.
3. Greeley, S. A., et al, "Report to District of Columbia Department of Sanitary Engineering on Improvement to Sewage Treatment Plant", 1964.
4. Metcalf and Eddy, Engineers, "A General Plan for Development of the Water Pollution Control 1969-1972), (1969).
5. Moorehead, G. J., "Overflows from Combined Sewers in Washington, D.C.," *JWPCF*, July 1961, p. 711.
6. Johnson, C. F., "Equipment, Methods, and Results from Washington, D.C., Combined Sewer Overflow Studies", *JWPCF*, July 1961, p. 721.
7. The Potomac Interceptor Symbol of Metropolitan Cooperation.
8. Project C. Improvements to the Sewerage System of the Nation's Capital.
9. Storm Sewer Preparation Program.
10. Water Pollution Control Plant of the District of Columbia, 1967.
11. Eddy-Gregory-Greeley Report, 1933.
12. Sherman-Horner Report, 1935.

APPENDIX C

INVESTIGATION OF RAINFALL RUNOFF RELATIONSHIPS

Rainfall Data

Rainfall data specific to the Washington, D.C. area were used in this investigation. Intensity-duration-frequency relationships have been formulated and plotted in a previous study using rainfall data collected during a 60-year period from 1894 to 1954. These relationships are defined in the following equation for return frequencies from 2 years to 100 years.

$$i_{avg} = \frac{a}{(t_d + b)^c}$$

where:

t_d = duration of storm corresponding to a period of maximum rainfall, minutes

i_{avg} = average intensity during particular duration, in./hr.

a, b, c = constants for particular return frequencies

In addition, a plot of these relationships has been updated by the Special Studies Branch of the Office of Hydrology to reflect additional data collected at the National Airport for 1951-1969. These updated relationships have only been plotted and not formulated in the form of equation (C-1).

Peak Rate of Overflow

The rational method was used to determine the peak rate of runoff in each sewer district and employed the following equation:

$$Q = CIA$$

where:

Q = rate of runoff in cu.ft./sec.
C = runoff coefficient
I = rainfall intensity in in./hr.
A = drainage area in acres

The areas used were the net sewage-producing areas of each basin. An average runoff coefficient was determined for each sewer district from zoning maps and runoff coefficients applicable to each type of zoning. This required planimetering the maps of the Zoning Regulations of the District of Columbia (effective May 12, 1958) to determine the area occupied by each type of zoning. Coefficients for individual types of zoning were provided by Washington, D.C. The overall coefficient for each drainage basin was then determined by a summation of the products of the appropriate average coefficient for each type of zoning multiplied by the fraction of the products of the appropriate average coefficient for each type of zoning multiplied by the fraction of the total area occupied by that type.

Different intensities were used in equation (C-2) together with a constant area and runoff coefficient to determine the rate for runoff produced in each district for different storm return frequencies. The intensity used for each storm was the average intensity in equation (C-1) which corresponded to the period of maximum rainfall equal to the time of concentration of each district.

The rational method determines the peak rate of runoff, not the peak rate of overflow. To determine this, a cursory examination was made of the hydraulic characteristics, control structures, and average dry-weather flow at each point of overflow. The rate of flow not overflowing, but continuing on in the sewer system was subtracted from the peak rate of runoff to determine the peak rate of overflow. At overflow points with complete diversion of all flow, the rate of sewage flow had to be added to determine peak overflow rates. The specific information relative to capacity of interceptors, operation of diversion structures, etc. was obtained from the District officials, as well as from data included in previous reports. Table 5 of the main report lists the peak rate of overflow at various diversion structures.

Volume of Overflow

Hyetograph Method   Volume of Runoff

The volume of the surface runoff of rainfall from a particular area is determined by subtracting from the total amount of precipitation the losses due to interception by vegetation, infiltration into permeable soils, retention in surface depressions, and evaporation. Of these, infiltration and retention in depressions are the only losses of significance in urban drainage. Numerous runoff coefficients accounting for the combined effect of these losses have been widely reported; however, the use of these coefficients calls for extensive judgment because the coefficient will vary throughout the duration of the storm. It is more precise to account for these losses separately and to examine their variability through time.

The methododogy employed to account for these losses involves the development of hyetographs. Using differential and integral calculus, the equation for the hyetograph is derived from the equation (C-1) to be as follows:

$$i_b = a \frac{1}{\left(\frac{t_b}{r} + c\right)^b} \quad \frac{b - \frac{t_b}{r}}{\left(\frac{t_b}{r} + c\right)^{b+1}} \qquad \text{(C-3a)}$$

$$i_a = a \frac{1}{\left(\frac{t_a}{r} + c\right)^b} \quad \frac{b - \frac{t_a}{r}}{\left(\frac{t_a}{r} + c\right)^{b+1}} \qquad \text{(C-3b)}$$

where:

t_b = time before the peak intensity, minutes

t_a = time after the peak intensity, minutes

i_b = instantaneous intensity before the peak intensity, in./hr.

i_a = instantaneous intensity after the peak intensity, in./hr.

r = portion of any duration of maximum rainfall occurring before the peak intensity
Note: $t_b = rt_d$, t_a $(1-r)t_d$

a,b,c constants from equation (C-2)

A study (28) of the storms occurring in the Chicago area showed that r in equation B-2 has a weighted average value of 3/8. A detailed examination of twelve consecutive months of rainfall as recorded at the Washington National Airport Station indicates that a value of 3/8 is reasonable for the Washington, D.C. area. Hyetographs for the relevant storm frequencies were developed on the basis of this value and of the values for a, b, and c in equation (C-1). As an example, Figure 5 in the main report is the hyetograph for various rainfall frequencies.

Research (29) indicates that the capacity which a soil exhibits for infiltration varies little with surface slope and depends primarily on soil porosity, ground cover, and antecedent rainfall. Infiltration capacity curves have been developed for various soil and surface conditions, and two of the mostly commonly used curves are shown in Figure 6 of the main report.

The capacity for infiltration is relatively high at the beginning of precipitation and decreases rapidly to a rather definite minimum value. During periods when the rate of precipitation is less than the infiltration capacity, all precipitation is infiltrated; when precipitation is greater than the infiltration capacity, excess runoff is produced in an amount equal to the difference between the precipitation rate and infiltration capacity. Consequently, an analysis of runoff requires the simultaneous examination of precipitation rates and infiltration capacities as they vary through the duration of the storm.

Figure 7 of the main report is a graph of the accumulated mass of rainfall and the accumulated mass of infiltration for residential type pervious surfaces versus time from beginning of significant rainfall. For storms in which the precipitation rate is initially less than the infiltration capacity, it is reasonable to assume the same amount of infiltration will occur as if the infiltration capacity was exceeded, but at some later time in the course of the storm. Therefore, the time of the start of excess is found by shifting the accumulated mass of infiltration curve along the time axis until it is tangent to the accumulated mass of rainfall curve. The actual mass infiltrated follows the trend of the accumulated mass of rainfall from zero time to the point of tangency, and from there it follows the offset curve of accumulated mass of infiltration up to the point when infiltration again equals the rate of precipitation. To define the latter point, the infiltration capacity curve is plotted with the hyetograph but at a time-offset equal to the offset of the accumulated mass of infiltration curve. The point at which infiltration again equals precipitation is defined as the second intersection of their curves. This procedure is illustrated in Figure 8 of the main report.

After the precipitation rate again drops below the infiltration capacity, the deficit is satisfied in part by infiltration from runoff traveling overland. It is conservative, but reasonable, to assume that for an urban area of moderate grade and intense development such as the Northeast Boundary Trunk Sewer basin, the time of overland travel on a pervious surface is so short that this type of infiltration is insignificant. On this basis, the rainfall in excess of infiltration is defined as the difference between the accumulated mass of the rainfall curve and the actual accumulated mass infiltration curve at the point when the rate of precipitation equals, for the second time, the infiltration capacity. The rainfall in excess of infiltration is shown in Figure 5 of the main report.

In this study, the rainfall in excess of infiltration was determined for pervious surfaces characteristic of residential and commercial-industrial areas. It is assumed in this study that infiltration is negligible for impervious surfaces and that runoff from an impervious onto a pervious surface is insignificant, or at least the effect of such is negligible. A past study by the D.C. Department of Sanitary Engineering provides a sound basis for determining the area of pervious and impervious surfaces. In the study, the pervious and impervious areas of typical blocks of the city were reported as follows:

Block Type	Pervious	Impervious
Residential, single house	54.1 percent	45.9 percent
Residential, row house	35.8 percent	64.2 percent
Commercial, neighborhood	25.8 percent	74.2 percent

The overall surface characteristics of the drainage basins were determined by assuming that zoning categories R-1 and R-2 correspond to the single house residential block type; R-3, R-4, and R-5 correspond to row house residential block type; C-1, C-2, C-M, and M correspond to neighborhood and commerical block type; park area is totally pervious; and C-3 and C-4 are totally impervious. On this basis, the percent impervious, percent residential-type pervious, and percent commercial-type pervious was determined for each sewer district.

It should be noted that any hyetograph is asymptotic to the time axis. However, in this study the hyetograph was not extended beyond the point at which rainfall intensity is less than 0.2 inches/hour. For pervious surfaces, the minimum infiltration is greater than 0.2 inches/hour and there is no need to examine rainfall past this point. Nevertheless, prior to the peak of a storm the mass of rainfall preceding an intensity of 0.2 inches/hour may be significant. For impervious surfaces, the total amount of precipitation for each rainfall frequency examined was the 24-hour duration value read from the updated Washington, D.C. intensity-duration-frequency curves published by the Special Studies Branch of the Office of Hydrology of the U. S. Weather Bureau.

A significant volume of the rainfall in excess of infiltration is retained in surface depressions, and it either evaporates or infiltrates after the storm subsides. Observations made during periods of heavy rainfall for the flat topography of Chicago suggest that the overall average depth of depression storage is 0.25 inches on pervious areas and 0.06 inches on paved areas (5). In a widely accepted design manual (30), various investigators have observed that in urban areas of moderate grade the overall average depth of surface depressions is about 0.05 inches for impervious surfaces and about 0.10 inches for pervious surfaces. The values reported in the design manual were assumed to apply to the Washington, D.C. area and were subtracted from the mass of rainfall in excess of infiltration.

The application to a large drainage area of rainfall data collected at one point outside the area requires careful interpretation of two phenomena:

a. The difference in prevailing physical conditions at two separate, but proximate, locations may result in different extreme rainfall data.

b. Average depth of rainfall over a large area is less than maximum point rainfall.

As for the former, the atmospheric forces of extreme storms are greater than the local effects of terrain and thermal patterns in the Washington area, and there should be no difference in extreme rainfall data (i.e. intensity versus frequency) at two separate sites in the area. To account for the latter, a correction factor of 98 percent was extrapolated from the area-depth relationship used by the D.C. Department of Sanitary Engineering. The volume of runoff from each drainage basin is determined by multiplying the mass of rainfall (less abstractions) by the appropriate surface area, summing, and reducing by the appropriate percent value to account for the depth-area relationship.

Hydrograph Routing   Volume of Overflow

The hyetograph method provides reliable values for the volume of runoff from any sewer district; however, this value may differ from the volume of overflow. To determine the volume of overflow, a method of hydrograph routing was used.

For hydrograph routing studies, the hydrographs obtained by the rational method were then further simplified by treating them as triangular hydrographs with their peaks located at the time of concentration of each drainage basin. Some of the critical time-of-concentration values were obtained from the District officials, while the remaining were calculated by considering the drainage area characteristics of each drainage district.

A time offset method of routing was utilized to route the hydrographs down the interceptor sewer to ascertain the cumulative effect at any point along the interceptor. This permitted development of a volume and peak rate relationship. A velocity of three feet per second was assumed along the interceptor, and the appropriate hydrographs from the tributary districts were plotted. Each drainage district hydrograph was offset an amount equal to the period of time required to reach the point in question. The resulting hydrograph was then found by summing up the ordinates of the individual hydrographs at the required points.

The hydrograph developed from hydrograph routing was compared with the volume of runoff determined by the hyetograph method. Using a heuristic procedure, the two types of information were combined to develop a synthetic hydrograph equal in volume to the runoff determined by the hyetograph method but smoothed and extended to follow the general shape of actual hydrographs. The dry-weather flow was added to the synthetic hydrograph to provide a plot of flow rate versus time at each point of overflow

To determine the volume of overflow, a cursory examination was made at each point of overflow. This examination practically duplicated the examination made to determine the peak rate of overflow, i.e. interceptor capacity and type of diversion structure operation were accounted for. Table 5 lists the volume of overflow (by point of overflow) corresponding to four different storm frequencies.

APPENDIX D

MONITORING EQUIPMENT, INSTRUMENTS, AND PROCEDURES

Discussion of Facilities and Operating Procedure

At each monitoring site, two fenced metal sheds, 400 to 800 feet apart, were erected along the sewer trunk line in the vicinity of the overflow point of the monitored sewer district. The upstream shed was set up as the lithium chloride release station and the downstream shed as the sample collection station. Each pair of stations was integrated electrically into an operating system by an underground conduit installed in the sewer.

During each storm, the system was actuated by the back pressure of a bubbler system installed at the manhole near the lithium chloride release station. Lithium chloride was injected as a tracer for accurate measurement of the unsteady storm discharges in the sewers. With the lithium chloride continuously released at a specified metering rate into the sewer, wastewater samples were collected intermittently at predetermined sampling intervals varying according to the duration of storm. All the collected samples were refrigerated at 3°C until they were ready for analysis. Discharge flows were computed on the basis of the lithium concentration detected in the collected samples and on application of the sample mass-conservation relationship. Major equipment and instruments installed at each site are shown in Figure D-1 as a schematic flow diagram.

Descriptions of Principal Equipment and Instruments

Lithium Chloride Release Station

The lithium chloride release station included a continuously-operated air bubbling system, a lithium metering system, and an electricity triggering system A metal shed 6'4" x 3'4" x 6'11" high housed the equipment and instruments. To provide additional protection, a transformer-cage type of structure with a 9-gauge cyclone fence on four sides and top was constructed outside the metal shed.

Air-Bubbling System

The air-bubbling system consisted of an air pump, a rotameter, a pressure regulator, a mercury pressure switch, and a stainless steel bubbling-tube assembly. Air from the pump continuously bubbled at the bottom of the sewer through the pressure regulator, flow rotameter, and the 1/4"-stainless steel tubing system. The back pressure of the bubbling system actuated a mercury pressure switch, which sends proper signals to the relay of the electrical triggering system. During a storm an increase of the water depth in the sewer activated the mercury pressure switch which, through the relay, triggered the circuits for the lithium chloride metering system and the sample collection system.

Lithium Chloride Metering System

The lithium chloride metering system consisted of a metering pump, a vacuum breaker, and a polyethylene piping assembly. The metering pump was a piston-diaphragm pump with positive displacement, with output adjustable by turning the handwheel which in turn sets the length of the stroke. Maximum theoretical metering rates were 11.9 gallons per hour for the pump used at the B-4 Sewer District monitoring station and 26.0 gallons per hour for the pumps used at the other sites. The initial setting of each metering pump was determined by the expected intensity of rainfall and by the maximum dry-weather flow. The major moving parts of the metering pump were kept in an oil bath constantly. The level of the oil in the gear box and compression reservoir was always maintained at 3/4" below the top.

The vacuum breaker was an anti-siphon fixture (Watts Model No. 288A) installed for the prevention of back-siphonage of lithium chloride through the pump head after pumping action stopped. It contains a light-weight disc-float, which opened and closed the atmospheric vent according to the pressure downstream of the outlet of the metering pump. The minimum negative pressure head required in the discharge line was 6 inches; at each monitoring site, this requirement was met all the time. The vacuum breaker was located above the possible highest level of the lithium chloride in the storage tanks. In order to prevent caking of lithium chloride residue in the pump, a water-rinse line was attached to the piping ahead of the inlet to the metering pump. The lithium chloride injection line downstream of the vacuum breaker was 1/2"-diameter polyethylene piping. At each lithium release station, one electric clock wired into the storm time operating circuit provided an accurate estimate of total storm operation time.

Electricity Triggering System

The Electricity Triggering equipment included one magnetic contactor, two manual motor starters, underground conduit, and liquid-tight junction boxes, as shown in Figure D-2. The magnetic contactor provided a safe and automatic connection for the time-operated circuits. The magnet coil was wired to be energized by the current through the pressure switch.

The entire assembly was housed in a water-tight enclosure on the panelboard in the lithium chloride release station. Also on this panelboard were manual starters used for supplementary control of the lithium metering pump and water-level recording systems.

Underground conduit was installed to integrate the operations of the lithium release and sample collection systems; this consisted of a 3-conductor, neoprene-jacketed cable, pre-assembled in a 1-1/2"-diameter polyethylene pipe. The conduit was fastened along the center top of the sewer with power-charged Ramset studs and EMT strips.

Flow Depth Recording System

At one of the sampling sites in a combined sewer district, an electronic flow-depth recording system was installed. Integrated with the bubbler tubing assembly, the flow recording instruments included a process pressure-to-current transmitter and an electronic strip chart recorder.

The process pressure-to-current transmitter was an electronic instrument operating on the principle of force balance. With the bellows as the pressure element, the transmitter produced an electrical output signal in milliamps. The monitoring range of the pressure was fully adjustable between zero and the maximum pressure, which in turn could be adjusted between 15 and 150 inches of water. In the present instance, the transmitter was calibrated for a pressure range 0-100 inches of water.

The water-level recorder was a 3" x 6" strip-chart electronic recorder. It accepted electrical analog signals representing pressure as input, recorded the input on a 4" strip chart, and displayed the value on a vertical indicating scale. The input signal ranged between 4 and 20 milliamps DC. An internal power supply (117 volts AC) provided operating power for the transmitter. The accuracy of the recorder was ±0.5 percent of the range, and repeatability was ±0.2 percent of the range. Recording chart speed was 7/8" per hour, and the useful length of each roll of chart was sufficient to permit continuous operation for more than a month.

Sample Collection System

The sample collection system consisted of a submersible pump, a refrigerated vacuum-charged sampler, a wastewater retention tank, a portable vacuum pump, and the piping system associated with wastewater flow. During a storm the sample collection system and the lithium chloride release station were triggered by the increased back-pressure of the bubbler line resulting from the increased depth of combined sewer overflow or separated storm water discharge. The wastewater was pumped to the surface continuously by a submersible pump anchored onto the floor of the sewer and flowed through the wastewater retention tank (retention time normally less than one minute). The sample was collected by the vacuum-charged sampler according to a predetermined sampling interval. The wastewater retention tank, the refrigerated sampler, and the piping all were housed in a 7' x 5'3" x 6'6" metal shed, protected by a 9-gauge cyclone fence around its four sides and above its top.

Submersible Pump

The submersible pump was a heavy-duty, manually-controlled sewage pump. It could be operated for long periods with the motor housing out of water, without damage. In view of the impacts generated by the hydraulic surge and the heavy solids flowing down the sewer during a storm, a solid, rigid anchorage for the submersible pump was designed and constructed. To provide additional protection against large pieces of solid waste, an expanded metal cage was installed for the containment of the pump. The legs of the pump were fastened onto the bottom plate of the prefabricated cage with three 1/2-inch machine bolts, and the cage was anchored onto the bottom of the sewer with four 1/2-inch diameter Rawl plugs. To furnish additional stability for the anchorage, angle irons were installed to brace the rigid pipe extending from the outlet of the submersible pump. The capacity of the pump was 60 to 80 gallons per minute, depending on the pumping head required at the different sites.

Automated Sampler

The sampler installed at each site was an automated sampling device equipped with a portable vacuum pump and an electric timer to adjust the sampling interval to the desired period (5 minutes minimum). Twenty-four bottles, each with an individual sampling line and the control switch, were furnished for each sampler and installed in a refrigerated enclosure. The intake ends of sampling lines (sampling head) were in the wastewater retention tank, and the sample was drawn into the bottle by vacuum when the control switch was released by a tripper arm operated in conjunction with the timer. Photographs of the sampling components are presented in Figure D-3.

Rain Gauges

The rain gauges were spring-driven, weighing-recording instruments, with 12 inches of range and a 24-hour time scale. The precipitation was measured in a prebalanced collection system by a weighing mechanism, which guided the pen to trace on a paper chart. The chart drive was spring-wound, and it rotated a cylinder on which the chart paper was held. The accuracy of the rain gauges in the first traverse (0" to 6") was ±0.03", and ±0.06" in the second traverse (6" to 12").

A dashpot was provided to minimize irregularities in the chart trace. In view of the requirements for truly representative records and to protect against vandalism the gauges were installed on the roof of the public schools in the vicinity of the sampling points. Their locations were: 1) Francis Junior High School, at N and 25th Streets, NW; 2) Chamberlain Vocational High School, at 14th and Potomac Avenue, SE; and 3) Anacostia Senior High School, at R and 16th Streets, SE.

Each of the rain gauges was fastened to a solid foundation, and the base was carefully leveled. To minimize possible damage from snow drifts or floods, each gauge was set up on a wooden platform 15 inches above the roof. For each rain gauge, windbreakers of 2" x 12" x 1" white pine were installed to provide additional stability for the measurement of rainfall, as illustrated in Figure D-4.

Lithium Chloride Flow Measurement

General Procedures and Principles

The lithium dilution procedure provides an accurate and simple technique for the measurement of wastewater flows which may be applied when:

1. Flow measurement devices do not exist,

2. Temporary installation of flow measurement devices is difficult, and

3. Suitable conditions for application exist in the wastewater system

The lithium dilution procedure was a modification of the long-applied salt dilution procedure. The use of a lithium salt improves the technique because the background of lithium in wastewater is usually low, and because lithium concentrations at fractional parts per million levels can be accurately and conveniently determined by atomic absorption or flame emission spectroscopy.

The technique involves preparing a lithium salt at a known concentration and then delivering to the wastewater stream at a continuous precise rate. At a point downstream where the lithium salt has become homogeneously mixed with the flow of wastewater, the wastewater is sampled and the concentration of lithium is determined. The flow of wastewater is then calculated as:

$$Q_W = Q_1 \times \frac{C_1 \quad C_2}{C_2 \quad C_o}$$

where:

Q_W = the flow rate of wastewater in the sewer, in gallons per minute (gpm)

Q_1 = the metering rate of the lithium chloride solution, in gpm

C_1 = the lithium concentration in the lithium chloride solution, in milligrams/liter (mg/L)

C_2 = the lithium concentration in the collected sample in mg/L, and

C_o = the lithium concentration in the raw wastewater, in mg/L

Since repeated analyses showed that the lithium concentration in the raw wastewater was consistently at undetectable level, Equation D-1 was simplified by eliminating C_o, as follows:

$$Q_W = Q_1 \times \frac{C_1 \quad C_2}{C_2}$$

which rearranges to:

$$Q_W + Q_1 = Q_1 \times \frac{C_1}{C_2}$$

However, during a storm the metering rate of lithium chloride (Q_1) is negligible compared to the wastewater flow rate (Q_W), and thus it can be eliminated from the left-hand term of Equation 2 without affecting the accuracy of subsequent calculations. This permits further simplification of the equation to:

$$Q_W = Q_1 \times \frac{C_1}{C_2}$$

Precautions

The preparation of the lithium salt solution requires a high degree of mixing to insure that the lithium is homgeneously dispersed. Because of the density of the salt solution, stratification can occur, if mixing is inadequate. Once the solution is prepared, it will remain homogeneous.

Mixing in the wastewater stream after the lithium solution is added must adequately achieve a homogeneous dispersion of lithium throughout the wastewater stream before the downstream sample is taken. If the wastewater stream is essentially in laminar flow the salt dilution procedure cannot be employed.

Application to D.C. Combined Sewer Study

Lithium chloride solutions varying in strength from 60,000 to 78,000 mg/L were purchased from the Foote Mineral Company. The actual lithium concentration in each batch was determined by ROY F. WESTON personnel using an atomic absorption spectrometer. The lithium chloride solution was introduced into the sewers (by means of previously calibrated pumps) at flow rates as high as 260 milliliters per minute. Lithium chloride solution volumes were determined before and after each dosing period, to serve as a check on the dosing rate. Since lithium is detectable in concentrations as low as 0.005 mg/L, flows as high as 1,000,000 gallons per minute could be measured with reasonable accuracy.

Monitoring Practices

Storm-monitoring activity was triggered automatically by the increase of depth of water in the sewer. Lithium chloride release, submersible pump operation, and the sample-collection system were started by this initial impulse. However, a time lag of 5-10 minutes was built into the program for the start of actual sampling, to give enough time for the injected lithium chloride to mix with the wastewater and to flow to the downstream manhole. The operational metering rate for lithium chloride solution was calculated from the storage tank levels before and after release, and from electric clock measurements of initiation and completion of release.

After each monitored storm, the collected samples were immediately brought to the ROY F. WESTON laboratory in West Chester, Pennsylvania for analysis. The characteristics determined for each sample included: BOD, COD, settleable solids, total solids, volatile solids, suspended solids, volatile suspended solids, lithium chloride, total coliform fecal coliform, and fecal streptococci. Total phosphate, total nitrogen, and other nutrient analyses were performed by ROY F. WESTON personnel at Baltimore's Back River Sewage Treatment Plant.

The lithium concentration was analyzed in an atomic absorption spectrometer. All analyses of the other chemical and physical characteristics were conducted according to the appropriate procedures in "Standard Methods for the Examination of Water and Wastewater", Twelfth Edition, 1967. The bacteriological analyses were basically in accordance with the membrane filter technique suggested in "Standard Methods", with various equipment and procedural improvements as recommended by the filter manufacturer.

After each storm operation, the lithium chloride release lines, the metering-pump head, the wastewater retention tank, the sample collection piping, and the sampling lines were rinsed clean with tap water. The area around the submersible pump was cleaned of debris, rags, and large solids. Before the systems were set up for the next storm all equipment and instruments were turned on by hand to check for proper functioning.

During the monitoring program, several operating problems were encountered, the most significant of which were:

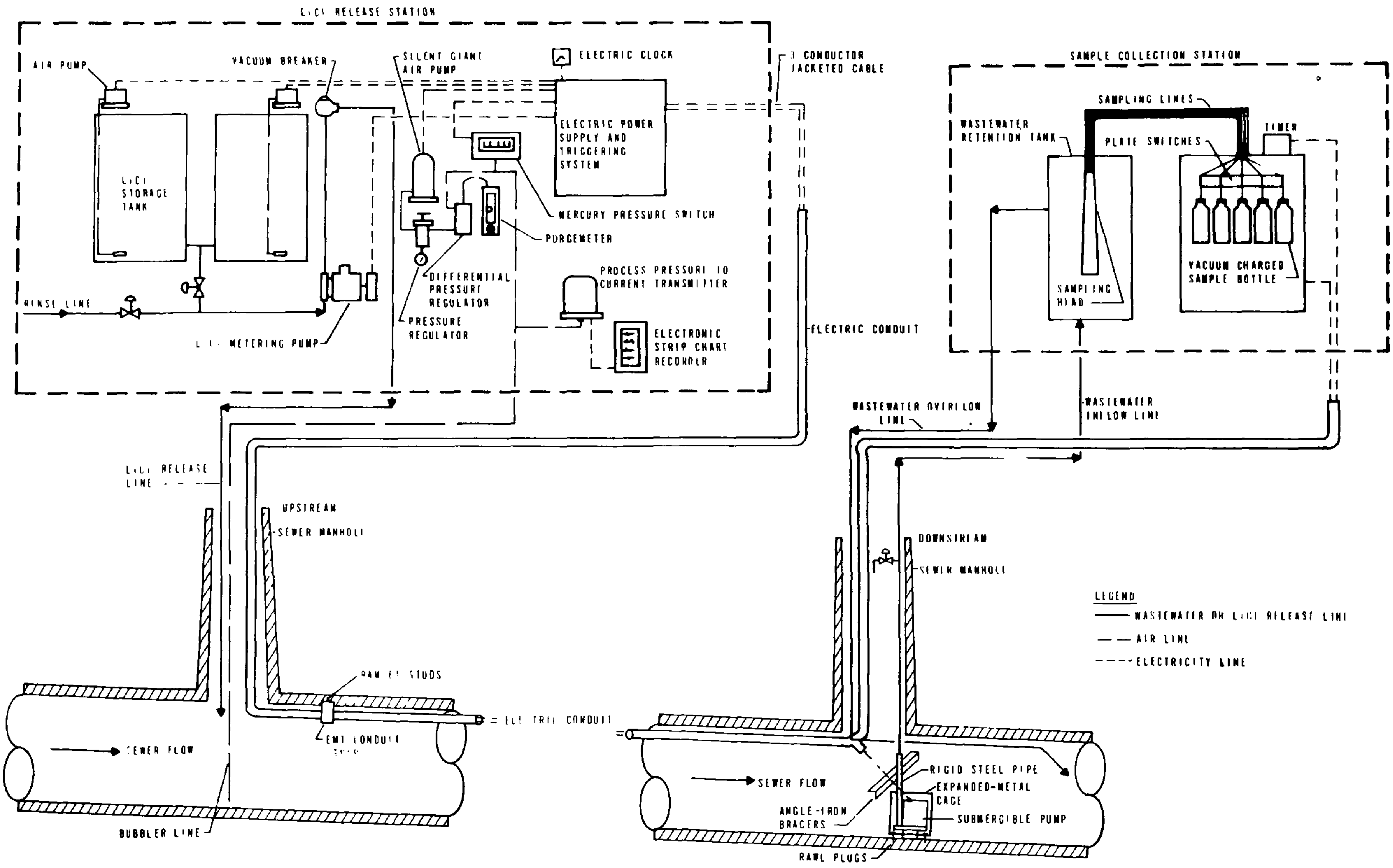
1. Difficulties in Pumping Wastewater Up from the Sewer--The submersible pump anchored to the bottom of the sewer was often clogged by solid wastes (such as cans, rags, wire, wood chips, tree stems, gravel, sand, etc.) and stopped working. There were also some pump stoppages during low-intensity storms, probably because of insufficient water depth in the sewer.

2. Physical Damage to Equipment Installed in the Sewer--During intense storms, heavy solid wastes (such as tires, concrete slabs, 55-gallon drums, mattresses, automobile radiators, chains, etc.) slammed into the protective cages of the submersible pumps and caused extensive damage to various equipment items. Bubbler lines were broken and torn loose; the protective cage was severely deformed and even disintegrated; pump braces were sheared off; pumps were washed away in the sewer; and the electric conduit was pulled out of its fastening studs.

3. Flooding of Lithium Chloride Release Station at Rose Park Playground--This station was sunk half way into the ground, at the request of the local residents. Consequently, the lower part of the structure was inundated by excess storm water runoff. This flooding caused minor damage to the bubbler instruments, the lithium chloride release system and the pressure-to-current transmitter. This problem was overcome by reinstalling the equipment above grade.

The equipment malfunctions and physical damage described above prevented complete coverage of all the storms that occurred during the monitoring period.

FIGURE D-1
MONITORING EQUIPMENT



D-1

FIGURE D-2

ELECTRIC POWER SUPPLY AND TRIGGERING SYSTEM

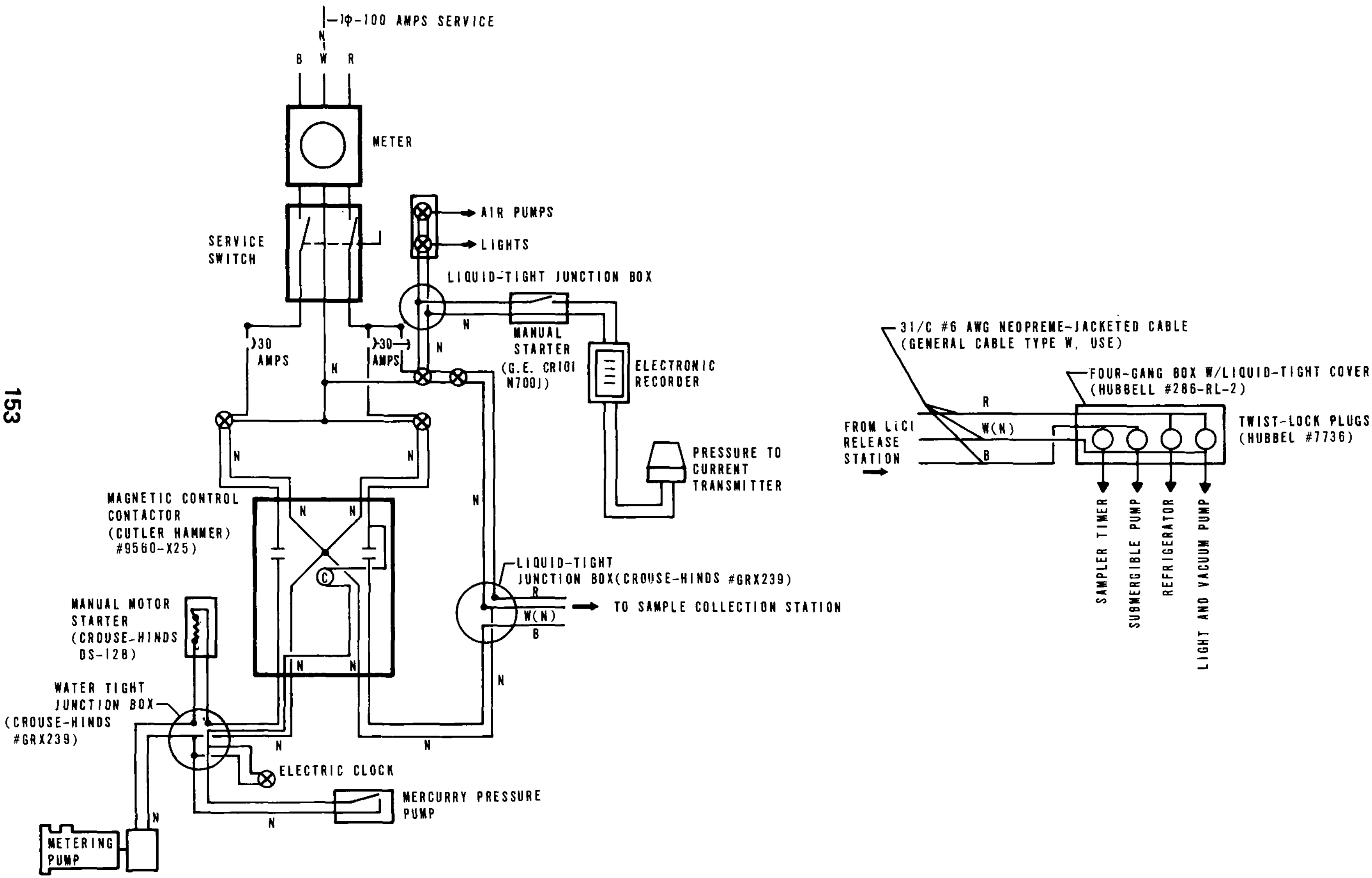
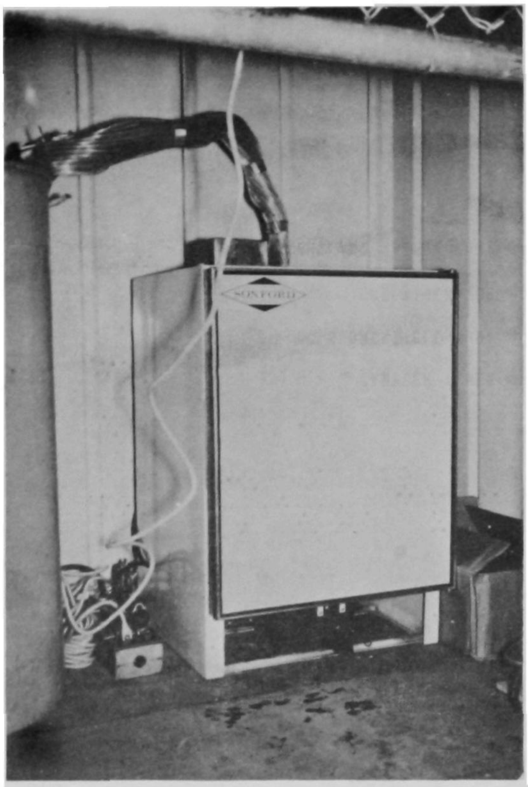
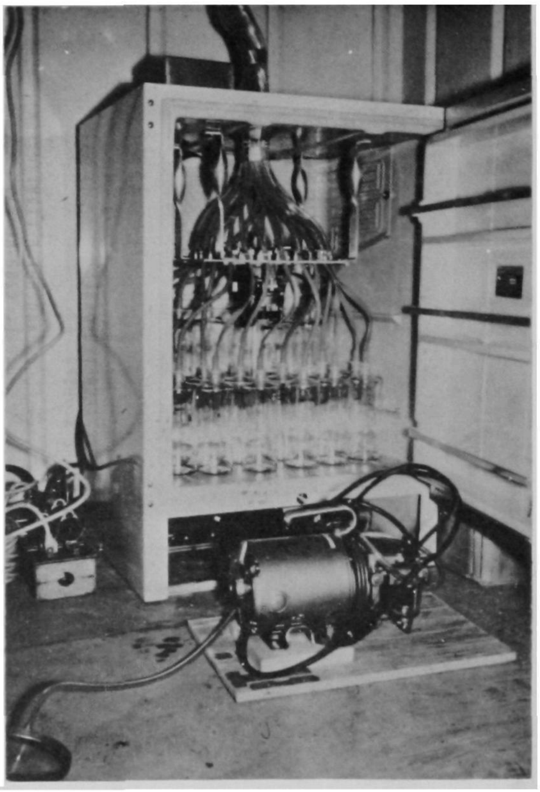


FIGURE D-3

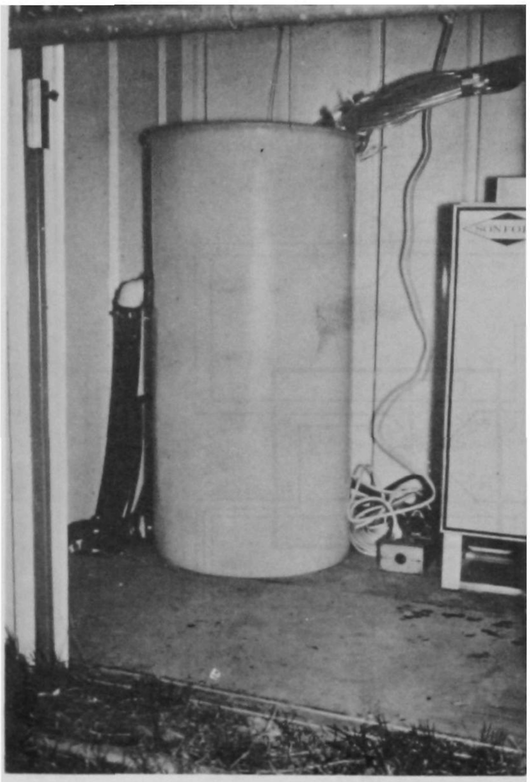
SAMPLING COMPONENTS



REFRIGERATED AUTOMATIC SAMPLER



INSIDE OF SAMPLER



WASTEWATER RETENTION
TANK AND PIPING

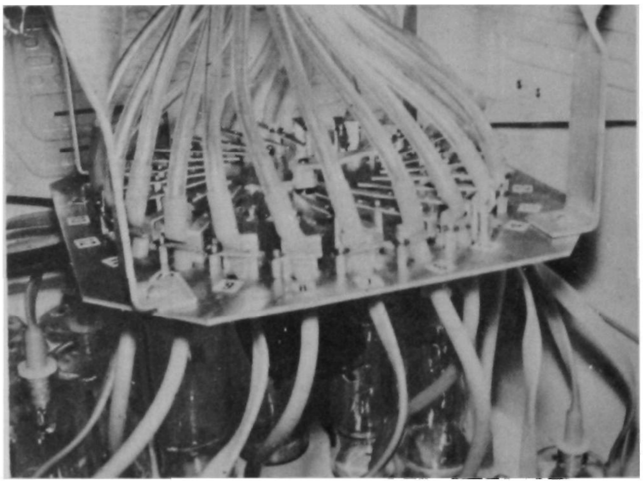
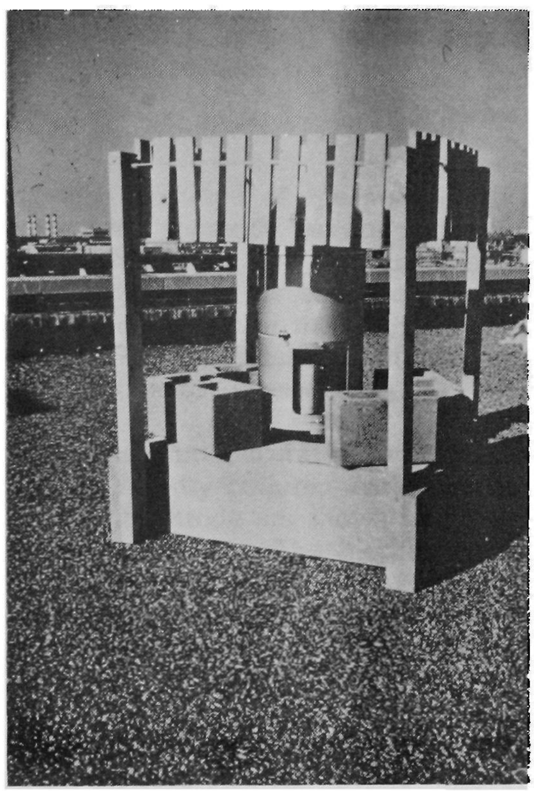


PLATE SWITCHES OF SAMPLER

FIGURE D-4

RAIN GAUGE



RAIN GAUGE AND WIND BAFFLE

APPENDIX E

MONITORED WASTEWATER FLOWS AND CHARACTERISTICS

Comparison of Representative Storms

The characteristics of the combined sewer overflows and of the separated storm water discharges generated by representative storms are described herein, according to the duration and intensity of the storms monitored.

A detailed examination of some graphs reveals an apparent discrepancy, at times, between rainfall intensity and flow rate. For example, flow may be decreasing while it is still raining at a significant rate. Inconsistencies such as this reflect the non-uniform spatial and temporal distribution of rainfall over the drainage basin. Although rainfall was measured at a location closest to the sampling point in each of the districts, the rainfall intensity may vary considerably between that measured at the point of sample collection and that which is actually falling in upper areas of the drainage basin.

Dry-Weather Conditions

In order to provide a more accurate definition of the pollution problem of combined sewer overflows, a 24-hour dry-weather flow study was conducted for Sewer District G-4 on September 22, 1969. The time variations of the characteristics and of the flow rate of the dry-weather flow from this combined sewer area are presented in Table E-1 and in Figure E-1. The range and mean value of each of the waste constituents in dry-weather sewage are presented in Table E-2. The characteristics of the dry-weather flow were consistent with those expected for municipal sewage.

The dry-weather flow for Good Hope Run Separated Storm Sewer District was also monitored; a very small flow of significantly polluted water was observed. Quality and flow data for a 24-hour dry-weather flow study are shown in Table E-3 and Figure E-2. Hydraulically, the variation of flow was minor. The biochemical oxygen demand was generally between 10 and 20 mg/L; comparative chemical oxygen demand concentrations were 40 to 100 mg/L. The bacteriological analyses show that the mean values of total coliform and fecal coliform were, respectively, 24,000 counts/100 ml and 21,000 counts/100 ml.

Whereas the dry-weather flow from the combined sewer district exhibited characteristics which were expected, the flow from the separated storm sewer district indicated relatively high BOD, COD, and fecal coliform levels. The BOD's and COD's could be explained by decaying organics in catch basins, inlets, manholes, and deposition in the pipe. The fecal coliform level is less than one percent of the corresponding value in the combined sewer district, and is an indicator that either some sanitary sewage or animal fecal matter is entering the storm sewer.

Short-Duration Storms

The data derived from the samples collected for the storms on June 8 and June 15, 1969 indicate the combined sewer overflow quality and quantity in Sewer District G-4 for storms of short duration. Figures E-3 and E-4 show the starting time, the duration, and the total rainfall of these storms. Because samples were collected only during the period of high flow in the sewer (the monitoring equipment automatically shuts-off when the flow subsides), the fact that samples were not obtained indicates that the flow rate is quite low at the end points of the graphs, with corresponding concentrations of pollutants. The total rainfall of each storm was the same, 0.7 inches, but the peak flow rate during the storm of June 8, 1969 was much higher than that of June 15, 1969. The characteristics (other than the settleable solids concentration) of the combined sewer overflow generated by these two storms were almost identical because both storms occurred during the same season and both had comparable waste accumulation times, as measured by the dry-weather period before precipitation.

For the short intense storms, as shown in Figures E-4 and E-5, the concentration of each waste constituent was observed to increase with the discharge rate in the sewer; the peak concentrations of many waste constituents were concurrent with the peak flow.

The concentrations were significantly high and remained so throughout most of the monitoring period. For example, for the intense storm of June 8, 1969, the flow rate was relatively constant during the 30-minute storm discharge period, but the concentrations of pollutants peaked in the early period of discharge and decreased to approximately one half of the peak value after two thirds of the discharge period had elapsed. The observed peak values for biochemical oxygen demand, chemical oxygen demand, and suspended solids were 405 mg/L, 1,358 mg/L, and 1,268 mg/L, respectively. The fraction of the fixed or inorganic solids was about one half of the total suspended solids, and this essentially constant relationship reflected the composition of surface flushings. Characteristically, the ratio of COD to BOD_5 in the wastewater during this short storm period varied between 3.0 and 20.

Long-Duration, High-Intensity Storms

The time variation of the quality and quantity of wastewater generated by a long-duration, high-intensity storm at Sewer District G-4 is indicated by the data for the storm on August 2, 1969 (Figure E-6). This shows that the flow rate of the wastewater generated by this long, intense storm is much higher than that of short storms. However, the concentrations of various constituents were observed to be lower, which is an anticipated result of high dilution by storm water.

Similar results were observed at Sewer District B-4, as shown in Figure E-7. In general, the concentrations of specific waste constituents measured at District B-4 were lower than those at District G-4. This may be explained by the fact that higher sewer flows were required before sampling could be triggered.

Concentration data in Figures E-3 and E-6 show that chemical oxygen demand and suspended solids concentrations of wastewater from a long, intense storm are reduced to approximately one third of the comparable values for a short, intense storm The biochemical oxygen demands for the long, intense storm wastewater were about one seventh of that in short-storm wastewater. This also implies that a higher fraction of surface material was eroded by the long intense storm. However, the ratio of COD to BOD_5 was essentially the same as that for short-storm wastewater.

Consecutive Storms

The characteristics and quantities of wastewater generated by consecutive storms can be represented by the data of the four storms during July 27-28, 1969, as presented in Figures

E-8 and E-9. The initial flushing effect was observed in all cases, even when the storms were only a few hours apart. However, the average concentrations of specific contaminants were observed to follow a decreasing trend with consecutive storms. For example, the average concentration of chemical oxygen demand (COD) decreased from 307 mg/L for the first storm to 154 mg/L for the fourth storm This decrease may be attributed to the reduction of waste material accumulation on the surface and in the collection system. The ratio of COD to BOD_5 was essentially the same as the comparable ratios for short storms and for long-duration, high-intensity storms. The characteristics of long-duration, low-intensity storm wastewater probably will be similar to that observed for consecutive storms.

Separated Storm Water Discharge

Nine storms, of varying size, were monitored at the Good Hope Run Sewer District. Table E-4 and Figures E-10, E-11, E-12 and E-13 illustrate the quality and quantity of the runoffs from a short intense storm a long intense storm and two consecutive storms. The average organic and nutrient concentrations in separated storm water discharges were observed to be approximately one third of those in combined sewers, but the solids concentrations (especially non-volatile solids and settleable solids) were much higher than those in combined sewer overflow. However, this phenomenon is attributed to the differences in the urban development on land surfaces in the monitored areas.

In general, the time variation in characteristics of wastewater from separated storm sewers is very similar to that from combined sewers. These figures also show the effects of the intensity, duration, and frequency of rainfall on wastewater characteristics. When the interval between storms is short, the accumulation of contaminants on the surface is less, and storm water is less contaminated.

One significant difference observed for separated storm sewer discharge is the broader range of COD to BOD_5 ratio (1.8 to 32). The higher ratio probably reflects a significant level of chemically oxidizable inorganic material introduced by erosion.

Bacteriological Quality of Storm Wastewater

Combined Sewer

The bacteriological examinations were made on selected individual samples. The range of variation and the mean values of different bacteria species are summarized in Table 9 in the body of the report. As presented in Figure E-14, the bacteriological counts varied with the flow rate of combined sewer overflow during each storm The initial flushing also had a significant effect on various bacteria counts.

The mean values for total and fecal coliforms were, respectively, 2,800,000 and 2,400,000 microorganisms/100 ml.

Separated Storm Sewer

The variation of bacteriological data for the separated storm sewer discharge was similar to the variation in the combined sewer discharge. Bacteria counts were high at the beginning of the storm and then decreased as the storm progressed. For instance, the average bacteria counts in the early part of the storm runoff on August 9, 1969 were 1,340,000/100 ml, 1,300,000/100 ml, and 31,000/100 ml, respectively for total coliform, fecal coliform, and fecal streptococci, whereas in the later part of the storm they averaged only 540,000/100 ml, 160,000/100 ml, and 17,400/100 ml, respectively.

Bacteriological examinations were made for selected samples, and their results indicated that microorganisms were present in storm runoff in fairly significant amounts although less than in combined sewage as cited in Table 9. The mean values of total and fecal coliforms in separated storm sewer discharge were 600,000 and 310,000 microorganisms/100 ml.

Waste Loadings from the Monitored Sewer Districts

Waste loading during a storm is the total waste material discharged, as pounds per unit time, and is computed from the measured flow rate and the concentration of each waste constituent. Expression of the waste loading in terms of pounds of specific waste constituent per minute provides an integrated assessment of the pollution potential associated with the wastewater generated by different storms. The range and mean value of waste loadings from the monitored storms and from the dry-weather flow in Sewer District G-4 are summarized in Table E-5.

Waste Loading Associated with Combined Sewer Overflow

The variation in waste loadings of combined sewer overflow was found to be highly dependent on the fluctuation of the flow rate in the sewer, which changed with the rainfall intensity during each storm. For an impulse-type short storm, the waste loading reached the peak value immediately and decreased very rapidly with time (see Figure E-15). For a long intense storm (with more than one discharge peak) the loadings of various constituents varied with the flow rate during the initial peak as anticipated (see Figure E-16). When a second peak occurred, the loadings increased with discharge only through the first few minutes and then dropped rapidly. Thus, most waste materials were carried by the initial flushing and scouring of the sewer. Waste loadings carried by the secondary flushing in a prolonged storm were limited.

The waste-loading time-variations in combined sewer overflow associated with the consecutive storms are presented together with the flow rate and the rainfall data in Figures E-17 and E-18. As anticipated from flow and concentration data, an initial impulse loading was always observed for the short intense storms even though they were only a few hours apart. Also as anticipated, the magnitude of the initial loading was found to be proportional to the length of the dry-weather period between the storms. The organic loading after the initial flushing of each storm was almost always low; for instance, the COD loading at the end of each consecutive storm was between 20 and 60 pounds per minute on the monitored areas, while the peak loadings varied between 105 and 346 pounds per minute. This implies that a certain base value may exist for the average organic loading in combined sewer overflow in each sewer district.

The mean waste loading of the dry-weather flow (sanitary sewage from District G-4) was calculated to demonstrate the relationship between the pollution loads of dry- and wet-weather flows. Because of the significant increase in flow rates during wet weather, the pollution load during such a time may be as much as three orders of magnitude higher.

Waste Loading Associated with Separated Storm Water Discharge

The waste loadings contained in separated storm water discharges varied with time in the same manner as combined sewer overflows. Generally, the peak organic loading in separated storm water discharges was lower than in combined sewer overflow, but the peak solids loading was higher (see Figures E-19, E-20, E-21 and E-22). However, these deviations may be attributed to the difference in surface runoff characteristics of the monitored sewer districts. Silt was found to be the dominant factor in storm water discharges from the Good Hope Run Sewer District.

Total Waste Load in Storm Water Runoff

The pollution constituents found in storm water runoff constitute inputs to the environment or disruptions of the environment. These include:

1. Depositions on the surface from air pollution,
2. Depositions on highways from vehicular traffic or dust control activities,
3. Practiced depositions from such activities as plant fertilization or insect control,
4. Natural degradation for flora and fauna,
5. Soil erosion, and
6. Air pollutants washed from air during rainfall.

The waste load in a particular storms depends upon: 1) the amount of material which has accumulated or developed on surfaces since the last storm and 2) the efficiency of the washing action accomplished by the storm

If the storm waters are transported through combined sewers, materials which have accumulated between storms may be flushed from the sewer. Any attempt to predict waste loads which will be generated by a specific storm must be based upon knowledge of the inputs to the drainage area and the washing efficiency that the storm will develop. Waste loadings generated by storms in combined and in separated storm sewer districts are presented in Table E-6. The total waste loading generated by each storm was computed by integrating the waste-loading time relationship of each storm Since the total waste loading carried by each storm could be a function of intensity of rainfall, quantity of rainfall, variation of the surface runoff characteristics, duration of the dry-weather period before the storm, and factors related to surface deposition, it would be very difficult to derive any definitive correlation between any type of total waste loading and the variables mentioned above. Examination of the data indicates that depositions from air pollution are very important in waste generation, because the waste loading is appreciably larger for a storm interval of even a few days (that is, larger than for a one-day or shorter interval). The major source of the rather rapid accumulation of materials is logically from air pollution; other sources are catch basins, inlet boxes, and street sweepings.

Two storms which provide a good comparison occurred on June 8 and 15, 1969 in combined Sewer District G-4 (refer to Figures E-3 and E-4). Rainfall in each of the two storms was the same (0.7 inches), the dry-weather period before each storm was about the same (6-7 days), and the storms were one week apart, which should eliminate any seasonal differences. Also, the dry-weather flow should have been the same since both storms occurred in the afternoon of the same day of the week. In addition to this, the characteristics of the rainfall event preceding the June 8th storm were approximately the same as those during the June 8th storm. However, the maximum intensities of the two storms were different, being 4.0 inches per hour on June 8 and 2.5 inches per hour on June 15. The more intense storm produced about twice the waste load, as measured by chemical oxygen demand. Maximum storm intensity and waste loading appear to correlate, which suggests that depositions to be carried away by the storm flow were equal and that the different results were due to washing efficiency. A potential fallacy in this reasoning is related to depositions on surfaces. Depositions which existed before the more intense storm may have been twice as large. The number of days of dry weather is not in itself an adequate criterion for estimating the deposition on a given area from air pollution. Wind direction and velocity as related to pollution sources are potentially more important.

Examination of the data in Table E-6 shows that there are general trends which are influenced by the length of dry-weather periods, total rainfall, and rainfall intensity. The variations of the total loadings of COD and suspended solids with total rainfall of each storm are presented in Figures E-23 and E-24. The relationships were fitted to the data using the method of least squares. Although the results tend to be inconclusive and there is considerable data scattering, Figures E-23 and E-24 show a recognizable difference between the separated and the combined sewer districts; however, they do not show the consistent pattern that might have been expected for the two combined sewer districts. Despite the inconsistencies in the available data, an attempt has been made to establish a correlation between various parameters. The average correlation between the total COD loading and total rainfall for Sewer District G-4 can be derived from Figure E-23, as follows:

$$L = 1{,}000 \times e^{1.47F} \quad , \text{ with } 0.5 > F > 3.0$$

where L = total COD loading, in pounds per storm, and

F = total rainfall of the storms, in inches

Similarly an average suspended solids correlation can be derived:

$$S = 600 \times e^{0.44F} \quad , \text{ with } 0.5 > F > 3.0$$

where S = total suspended solids loading, in pounds per storm

The variation of runoff with the progress of the storm showed a reasonably consistent relationship to the change in rainfall intensity. Figures E-25, E-26, and E-27 depict typical variations of wastewater depth in the sewer, rainfall intensity, and wastewater flow as the storm progresses; these plots show the respective effects of a short intense storm, a long intense storm, and consecutive storms in a combined sewer district. Rainfall and runoff data were also compared to develop the coefficient used in the calculation of volume of storm runoff.

In order to generalize the quality of both combined sewer overflow and separated storm water discharge, the concentrations of different wastewater quality parameters were compared and correlated. A comparison between BOD_5 and COD is shown in Figure E-28, in which the relationships were fitted to the data by inspection. The ratios between average BOD_5 and average COD are 0.20, 0.14, and 0.11 for Sewer Districts G-4, B-4, and Good Hope Run, respectively. This implies that the organic materials in separated storm water discharge do not degrade as rapidly or as completely as the organic materials in combined sewer overflow. The average ratio of BOD_5 to COD for the combined sewer overflow from Sewer District B-4 falls between the ratio for combined sewer overflow from Sewer District G-4 and the ratio for the separated storm water discharge from the Good Hope Run District, possibly because the combined sewer overflow monitored at Sewer District B-4 was more highly diluted by storm runoff than was the combined sewer overflow in District G-4. As shown in Figure E-29, the correlation (developed by fitting the relationships to the data by inspection) between volatile suspended solids and total suspended solids averaged 0.50, 0.30, and 0.052, respectively, for G-4, B-4, and Good Hope Run. This indicates that the organic proportion of the solids varied in much the same manner as the BOD_5-to-COD ratio for the wastewater collected at the different sewer districts. The ratios of the volatile suspended solids to total suspended solids also indicate that most of the solids contained in storm runoff are non-volatile silt and sand.

The results of the various comparisons and correlations indicate that there are definite overall trends within these relationships; however, they are very loose. Although it is believed that a relationship could be developed for surface characteristics and depositions to relate to waste loading characteristics, sufficient data were not collected to define this relationship.

Comparison of Flow Rate Measurements

Use of the steady-state formula and flow depth measurement has been the basic approach of many engineers to estimation of the flow rate of storm runoff. This method may be valid for the flow in streams or rivers, where flow variations generally are gradual. The flows of interest in the current study, however, vary drastically with time, especially during the surge period of a storm. Thus, a formula for calculation based on steady-state flow was considered not applicable. The lithium dilution tracer method as a direct measurement technique was selected, therefore, as the primary flow measurement method in this study.

Tracer dilution methods are recognized as reliable and practical flow measurement techniques in both laboratory and industrial operations. Application of the lithium dilution method to a variety of industrial wastewater stream flow measurements by ROY F. WESTON has shown an accuracy of ±4 percent (6). A detailed outline of the lithium chloride flow measurement technique is presented in Appendix D.

For purposes of comparison, flow estimations were made at Sewer District G-4 by assuming steady-state conditions (Manning equation) and measuring the depth of flow. A plot of the "depth" method versus the reference lithium chloride method is shown in Figure E-30. Although a detailed statistical evaluation was not made, the data were observed to be a general cluster around the 45^o line of exact correlation. The maximum predicted gravity flow using the Manning equation ($n = 0.015$, $s = 0.0459$ ft/ft, Diam = 5.5 feet) is approximately 280,000 gpm.

It is felt that the variability in the flow rate estimated by depth measurements is due to the lack of applicability of the steady state flow formula to the estimation of storm runoff. However, the depth-of-flow measuring instrument may have malfunctioned (leak in the line, insufficient air pressure etc.) even though this was not detected. Nevertheless, the tracer method may introduce some errors into the results unless feed rates, remote operations, degree of mixing, and concentration of tracer during storage are carefully checked or controlled. Besides this, the tracer method gives no indication of flow between samples.

In order to verify the unit hydrograph approach to the estimation of total runoff discharge, the flow data generated by the lithium chloride dilution method in District G-4 for various monitored storms were compared with unit hydrographs constructed for the same storms.

The actual total discharge of each storm was determined by integrating the measured flow rates with time, with peak rates taken directly from the field data. The unit hydrograph methodology was discussed in a preceding section and in Appendix C. Table E-7 summarizes the results of this comparison. For each storm compared, the total measured runoff was within ten percent of the amount established from the corresponding unit hydrograph.

Table E-1

Dry Weather Flow For Combined Sewer District G-4

Sampling Date	Schedule Time	Sampling Interval minutes	Flow gpm	pH	COD mg/L	BOD mg/L	Total Solids mg/L	Total Volatile Solids mg/L	Suspended Solids mg/L	Volatile Suspended Solids mg/L	Settleable Solids mg/L
Sept. 22, 1969	10:50 a.m.	58	1,000	6.5	360	204	680	330	170	130	90
	11:48 a.m.	58	1,000	6.7	340	119	540	160	130	130	30
	12:46 p.m.	58	940	6.7	330	----	610	180	130	120	22
	1:44 p.m.	58	955	6.6	360	108	570	180	170	156	70
	2:42 p.m.	58	900	6.8	360	111	830	210	170	170	88
	3:40 p.m.	58	882	6.8	370	128	540	210	190	182	110
	4:38 p.m.	58	940	6.8	420	126	600	200	190	182	90
	5:36 p.m.	58	985	6.6	440	135	650	230	210	190	102
	6:34 p.m.	58	1,000	6.4	400	117	550	300	170	168	50
	7:32 p.m.	58	1,015	6.7	510	171	680	250	240	230	100
	8:30 p.m.	58	985	6.6	560	192	660	280	220	178	100
	9:28 p.m.	58	955	6.8	490	183	720	300	260	230	140
	10:26 p.m.	58	940	6.4	430	153	680	290	200	154	60
	11:24 p.m.	58	711	6.4	340	117	580	200	160	160	60
Sept. 23, 1969	12:22 a.m.	58	872	6.6	320	188	650	250	190	174	110
	1:20 a.m.	58	604	6.6	320	----	610	260	190	168	130
	2:18 a.m.	58	542	6.6	320	84	610	220	190	190	118
	3:16 a.m.	58	516	6.7	260	71	740	230	200	200	160
	4:14 a.m.	58	477	6.8	170	49	470	210	90	88	50
	5:12 a.m.	58	438	6.6	184	54	550	220	120	120	100
	6:10 a.m.	58	457	6.4	275	58	780	760	240	200	200
	7:08 a.m.	58	542	6.4	255	86	540	180	200	180	168
	8:06 a.m.	58	780	6.8	357	103	570	300	160	140	92
	9:04 a.m.	58	1,156	6.7	418	91	610	240	180	156	70
	10:02 a.m.	58	940	6.8	439	168	620	240	160	158	12
	11:00 a.m.	58	956	6.6	337	123	590	200	110	110	
	11:58 a.m.	58	928	6.7	306	105	540	190	100	90	

Table E-2

Characteristics of Dry-Weather Flow

Waste Constituents, mg/L	Combined Sewer District G-4		Separate Storm Sewer in Good Hope Run Sewer District	
	Range	Mean	Range	Mean
Chemical Oxygen Demand	170 - 560	358	10 - 184	69
Biochemical Oxygen Demand	49 - 204	120	8 - 58	18
Total Solids	470 - 830	621	456 - 1,950	639
Total Volatile Solids	30 - 760	243	158 - 1,780	345
Suspended Solids	90 - 260	176	20 - 260	68
Volatile Suspended Solids	88 - 230	161	0 - 64	28
Settleable Solids	12 - 300	95	2 - 180	47
Total Phosphate	---	---	---	---
Total Nitrogen	---	---	---	---
Orthophosphate	---	---	---	---
Ammonia Nitrogen	---	---	---	---
pH	6.4 - 6.8	6.6	6.6 - 6.1	6.35
Total Coliform, counts/100 ml	---	3,900,000	---	24,000
Fecal Coliform, counts/100 ml	---	2,900,000	---	21,000
Fecal Streptococcus, counts/100 ml	---	64,000	---	800

Table E-3

Storm Sewer Dry-Weather Flow for Separated Sewer District - Good Hope Run

Location of Sampling Site - 17th St. - Minnesota Ave. S.E.
Drainage Area - 264 acres

Sampling Date	Schedule Time	Sampling Interval	Flow	pH	COD	BOD	Total Solids	Total Volatile Solids	Suspended Solids	Volatile Suspended Solids	Settleable Solids
		minutes	gpm		mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
Sept. 22, 1969	3:15 p.m.	60	92.0	6.5	41	37	730	240	170	50	50
	4:15 p.m.	60	90.0	6.4	82	----	630	300	60	40	16
	5:15 p.m.	60	86.0	6.4	51	13	630	310	60	34	12
	6:15 p.m.	60	83.7	6.4	------	11	590	230	70	30	34
	7:15 p.m.	60	116.0	6.4	10	10	640	270	260	64	180
	8:15 p.m.	60	90.0	6.4	61	8	610	180	130	50	62
	9:15 p.m.	60	90.0	6.4	42	8	540	190	50	10	14
	10:15 p.m.	60	90.0	6.3	71	13	620	280	80	50	32
	11:15 p.m.	60	90.0	6.4	61	9	550	290	200	64	152
Sept. 23, 1969	12:15 a.m.	60	88.1	6.4	61	12	560	280	90	60	42
	1:15 a.m.	60	90.0	6.4	51	20	560	320	52	44	50
	2:15 a.m.	60	91.0	6.4	82	14	1,950	1,780	60	18	40
	3:15 a.m.	60	91.0	6.4	82	17	534	264	68	40	50
	4:15 a.m.	60	91.0	6.5	82	12	550	334	52	6	6
	5:15 a.m.	60	89.2	6.2	51	11	564	314	60	0	20
	6:15 a.m.	60	91.0	6.2	82	18	560	308	28	14	14
	7:15 a.m.	60	92.0	6.2	51	15	514	192	30	10	2
	8:15 a.m.	60	92.0	6.1	51	31	630	270	98	18	90
	9:15 a.m.	60	138.0	6.2	82	19	456	232	30	20	22
	10:15 a.m.	60	92.0	6.2	184	58	920	764	46	16	46
	11:15 a.m.	60	109.0	6.2	102	22	470	250	40	20	20
	12:15 p.m.	60	88.0	6.4	71	18	464	158	20	0	20
	1:15 p.m.	60	88.0	6.4	71	18	546	296	32	22	12
	2:15 p.m.	60	91.0	6.4	102	16	554	308	36	16	36
	3:15 p.m.	60	89.0	6.6	102	20	610	258	30	10	12

Table E-4

Characteristics of Storm Runoff in Sewer District Good Hope Run

Location of Sampling Site - 17-Minn. and 16 S.E.

Storm Date	Storm Time	Total Rainfall	Sampling Interval	Flow	pH	COD	BOD	Total Solids	Total Volatile Solids	Suspended Solids	Volatile Suspended Solids	Settleable Solids	Total P	Total N
		inches	minutes	gpm		mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
July 28	1:20-2:00 p.m.	1.6	5	21,000	6.2	430	13	14,600	912	9,600	880	6,756	4.5	4.0
			5	65,600	6.2	400	15	12,560	996	11,200	860	7,640	2.8	2.8
			5	75,000	6.1	280	11	6,638	278	6,050	60	3,330	1.5	2.5
			5	57,900	6.0	170	16	5,830	268	5,520	40	2,660	1.8	2.5
			5	47,700	6.1	310	15	10,002	600	9,020	430	6,528	2.4	4.0
			5	43,300	6.0	300	15	10,632	484	10,010	370	6,906	2.0	2.5
			5	57,900	6.0	370	5	10,242	512	9,170	380	5,702	2.6	3.0
			5	15,400	6.2	240	8	8,676	488	8,150	410	6,662	1.6	2.5
			5	12,500	6.2	230	13	7,198	460	5,560	460	2,912	1.8	2.0
			5	10,200	6.3	210	15	6,092	390	5,900	210	2,332	2.2	3.2
			5	7,900	6.2	210	16	4,898	288	4,620	180	2,530	2.0	3.0
			5	6,090	6.3	230	4	4,598	378	3,920	280	3,616	1.6	2.0
			10	4,570	6.4	150	15	3,908	284	3,140	300	2,792	1.6	2.2
			10	5,000	6.3	120	17	2,898	228	2,160	180	1,016	1.5	1.8
			10	3,740	6.6	140	14	2,310	200	1,920	200	1,036	1.4	2.0
			10	3,620	6.8	120	14	1,670	110	1,020	50	360	2.1	2.0
			35	4,770	6.9	58	12	1,454	136	1,160	100	524	1.0	1.6
			10	2,020	7.0	48	4	1,140	136	640	120	–	1.0	1.4
			10	2,625	6.3	67	4	770	138	480	100	–	0.5	1.5
			10	2,190	7.0	77	5	944	136	720	120	396	1.0	2.4
			30	3,180	7.0	67	8	776	76	480	–	280	1.0	1.2
				1,140	7.1	29	3	778	142	520	100	200	0.4	1.6
July 28	5:00-5:30 p.m.	0.20	10	2,020	7.0	58	4	578	90	380	100	248	0.3	2.0
			10	2,500		96	6	488	90	320	100	144	0.4	1.6
			10	4,040	6.9	77	7	446	154	300	120	192	0.4	1.4
			10	2,640	7.0	77	5	530	96	340	100	157	1.8	3.4
			10	3,600	7.1	48	5	1,070	120	920	120	472	1.0	1.2
			10	4,200	7.1	106	5	1,842	136	1,500	180	812	0.8	1.2
			10	3,090	7.1	86	5	1,580	84	1,300	160	708	0.4	1.2
			10	1,640	7.2	77	4	1,984	12	1,740	180	964	0.2	1.2
				2,020	7.1	38	3	1,240	106	980	140	496	0.2	1.0
August 2	8:17-9:30 p.m.	2.9	10	34,400	6.3	400	16	10,346	538	9,568	524	5,353	2.0	4.0
			10	16,800	6.3	259	12	6,626	368	6,560	210	4,760	1.8	4.0
			10	10,100	6.2	216	36	4,290	250	4,210	250	2,370	1.5	2.5
			10	5,660	6.2	140	16	3,318	226	2,610	50	1,290	1.0	2.0
			10	4,400	6.5	184	36	2,478	188	1,200	70	710	1.0	2.0
			10	3,520	6.6	119	17	1,836	130	1,550	60	1,050	1.0	1.5
			10	2,470	6.8	108	12	1,090	40	1,278	232	490	1.0	2.0
			10	1,960	6.9	140	31	1,290	184	910	60	662	1.0	1.6
			10	1,995	7.0	129	13	1,342	178	840	20	700	1.4	1.0
			10	1,601	7.0	65	40	680	164	200	0	40	0.4	1.0
			10	1,410	7.0	86	14	1,130	200	548	40	400	0.2	1.0
				1,340	7.1	54	17	910	72	416	12	268	0.4	0.6
August 9	9:20-9:45 p.m.	1.5	10	2,205	6.3	180	20	456	190	100	10	80	1.4	1.4
			40	2,030	6.3	200	29	1,094	272	1,000	80	280	1.0	1.2
			10	50,000	6.1	160	19	2,374	126	2,020	92	396	1.0	0.3
			10	16,150	6.2	390	28	13,590	826	11,280	720	460	1.4	1.6
			10	7,500	6.1	270	20	10,674	604	8,500	450	1,676	1.0	1.4
			20	5,000	6.2	180	16	5,462	248	5,100	150	1,640	1.0	1.0
				2,860	6.2	150	17	3,988	260	3,400	150	1,212	0.6	0.8

Table E-5

Waste Loadings in Combined Sewer Overflow and Separated Storm Water Discharge

Waste Constituents	Combined Sewer Overflow from Sewer District G-4 (lbs./min.) Range	Mean	Separated Storm Water Discharge from Good Hope Run Sewer District (lbs./min.) Range	Mean
Chemical Oxygen Demand	0.2 - 1,359	157.4	0.2 - 219	19.0
Biochemical Oxygen Demand	0.6 - 298	28.3	<0.1 - 8.7	1.7
Total Solids	10.4 - 2,552	419.9	3.7 - 6,872	333.0
Total Volatile Solids	3.0 - 876	125.4	0.2 - 545	23.8
Suspended Solids	7.3 - 2,268	322.7	0.9 - 6,128	293.0
Volatile Suspended Solids	1.5 - 652	85.8	<0.1 - 471	16.5
Settleable Solids	3.4 - 1,996	165.7	<0.1 - 4,180	10.2
Total Phosphate	0.0 - 38.75	1.82	<0.1 - 1.53	.09
Total Nitrogen	0.1 - 15.37	1.56	<0.1 - 1.5	.11

Table E-6

Total Waste Loadings Generated by Different Storms

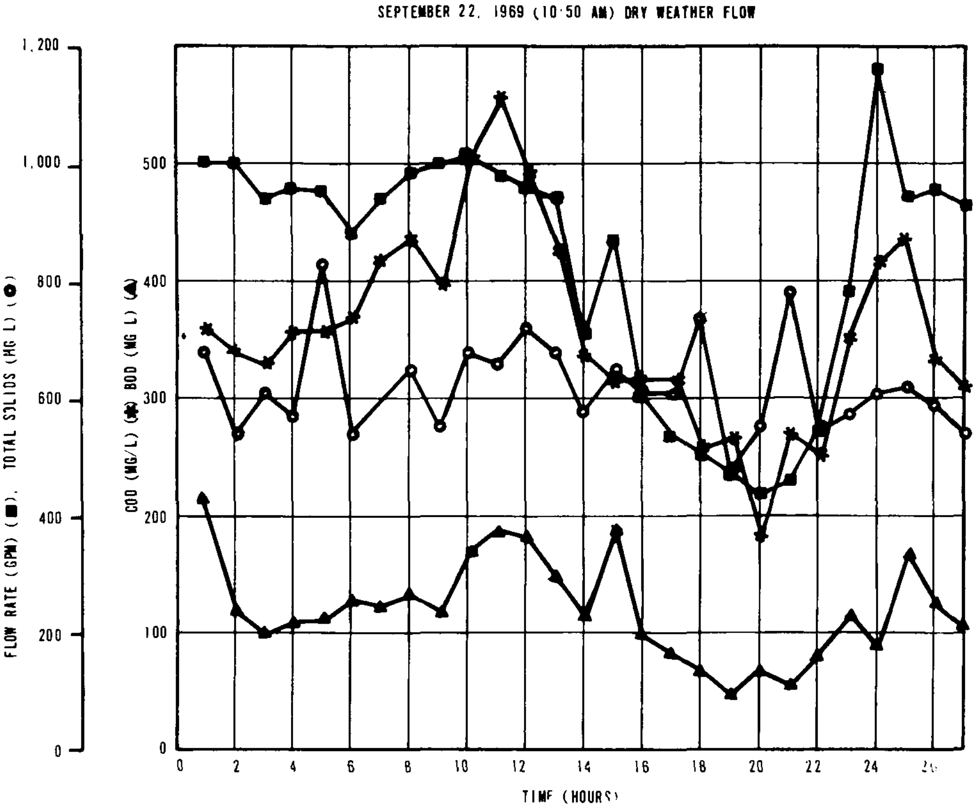
Storm			Total Loadings, pounds								
Date 1969	Time	Total Rainfall	COD	BOD	Total Solids	Total Volatile Solids	Suspended Solids	Volatile Suspended Solids	Settleable Solids	Total P	Total N
Combined Sewer District G-4											
May 19	1:42- 1:45 a.m.	0.4"	3,482.0	470	6,159	2,387	4,247	1,540	571	----------	----------
May 20	11:42-11:49 p.m.	0.6"	1,920.0	152	6,059	2,309	4,294	1,634	667	----------	----------
June 8	5:50- 6:03 p.m.	0.7"	2,890.0	894	3,947	2,250	2,521	1,390	880	11.13	20.52
June 15	2:20- 2:40 p.m.	0.7"	1,415.0	302	2,410	1,130	1,328	473	1,007	5.75	5.19
July 6	7:40- 8:20 p.m.	0.4"	838.0	160	1,304	637	1,060	561	----------	----------	----------
July 27	11:35-12:14 a.m.	2.1"	9,538.0	1,554	26,886	10,951	20,701	4,877	14,374	37.89	104.18
July 28	2:30- 2:52 a.m.	0.6"	2,009.0	326	4,777	2,075	3,187	974	2,107	9.11	13.66
July 28	11:30-11:45 a.m.	0.6"	2,696.0	526	5,539	2,071	3,844	1,200	2,969	13.40	18.21
July 28	1:20- 2:00 p.m.	1.3"	3,109.0	390	7,895	1,379	6,431	1,506	4,434	25.61	23.90
August 2	8:05- 9:15 p.m.	2.8"	44,815.0	10,109	101,817	24,010	75,657	14,399	61,705	421.44	382.48
August 9	9:20- 9:37 p.m.	1.1"	7,624.0	1,272	29,176	10,136	18,374	4,635	3,331	59.99	88.02
August 9	11:20-11:30 p.m.	1.6"	673.0	115	2,476	736	2,061	289	153	5.11	4.43
August 19	6:40- 6:53 p.m.	1.35"	3,213.0	354	14,218	3,569	11,667	1,089	9,052	13.30	61.29
Combined Sewer District B-4											
June 1	7:25- 7:45 p.m.	1.4"	3,120.0	316	6,335	1,392	4,306	1,249	4,205	7.86	15.37
June 2	7:45- 8:05 p.m.	0.9"	155.0	28	345	166	78	40	48	----------	----------
June 3	12:25-12:40 a.m.	0.95"	137.5	29	405	170	73	13	38	----------	----------
July 27	11:35-11:55 p.m.	1.3"	866.0	85	2,694	----------	2,004	1,011	1,822	21.87	15.41
August 2	8:10- 9:00 p.m.	3.9"	4,608.0	1,309	18,831	3,671	18,145	2,709	16,162	222.72	74.84
August 3	10:30-11:40 p.m.	0.4"	280.0	105	454	252	444	156	39	----------	----------
August 9	11:22-11:37 p.m.	1.6"	5,848.0	1,063	14,692	4,041	13,469	3,722	886	14.18	10.63
Separate Sewer District - Good Hope Run											
July 28	1:20- 2:00 p.m.	1.6"	5,423	252	160,253	9,415	142,958	6,621	90,845	41.60	54.69
July 28	5:00- 5:30 p.m.	0.2"	155	10	2,167	215	1,732	268	924	1.37	3.06
August 2	8:17- 9:30 p.m.	2.9"	2,474	160	57,556	3,180	55,000	2,604	30,503	13.47	25.91
August 9	9:20- 9:45 p.m.	1.1"	3,749	388	81,193	4,709	68,150	3,679	9,021	20.03	18.01
August 9	11:00-11:30 p.m.	0.8"	193	16	425	76	272	3	148	0.93	0.90
August 10	12:25-12:45 a.m.	0.65"	178	40	1,884	359	1,737	56	433	1.83	1.16
September 17	8:20-10:00 p.m.	0.6"	1,906	208	15,980	1,841	14,389	1,257	3,043	----------	----------

Table E-7

Comparison of Actual Total Flow Measurement and Volume Obtained by Hydrograph Analysis

Storm	Actual Measurement Volume million gallons	Volume From Hydrograph Analysis million gallons
	Combined Sewer District G-4:	
July 27-28, 1969 11:35 p.m. - 0:14 a.m.	4.36	3.93
July 28, l969 1:28 p.m. 2:03 p.m.	2.43	3.22
August 2, 1969 8:05 p.m. - 9:10 p.m.	21.10	20.20
July 6, 1969 7:40 p.m. - 8:20 p.m.	0.12	0.19
July 28, 1969 2:30 a.m. - 2:52 a.m.	1.09	0.96
	Combined Sewer District B-4:	
August 2, 1969 8:10 p.m. - 9:00 p.m.	4.40	5.12

FIGURE E-1
DRY WEATHER FLOW IN COMBINED SEWER DISTRICT G-4



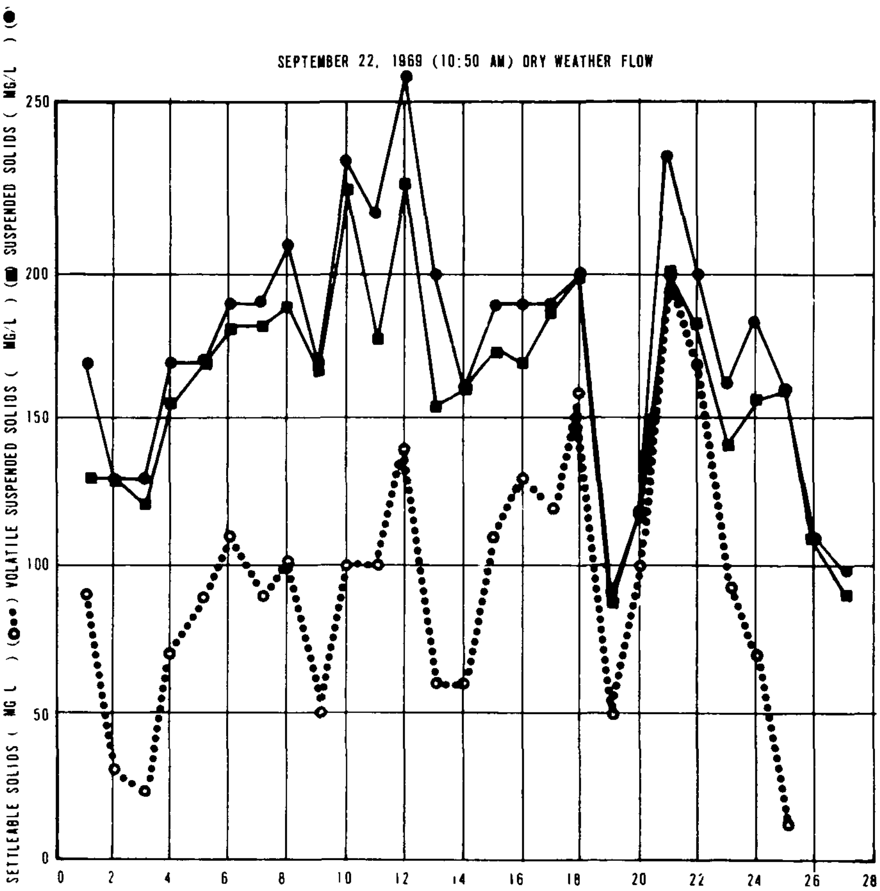
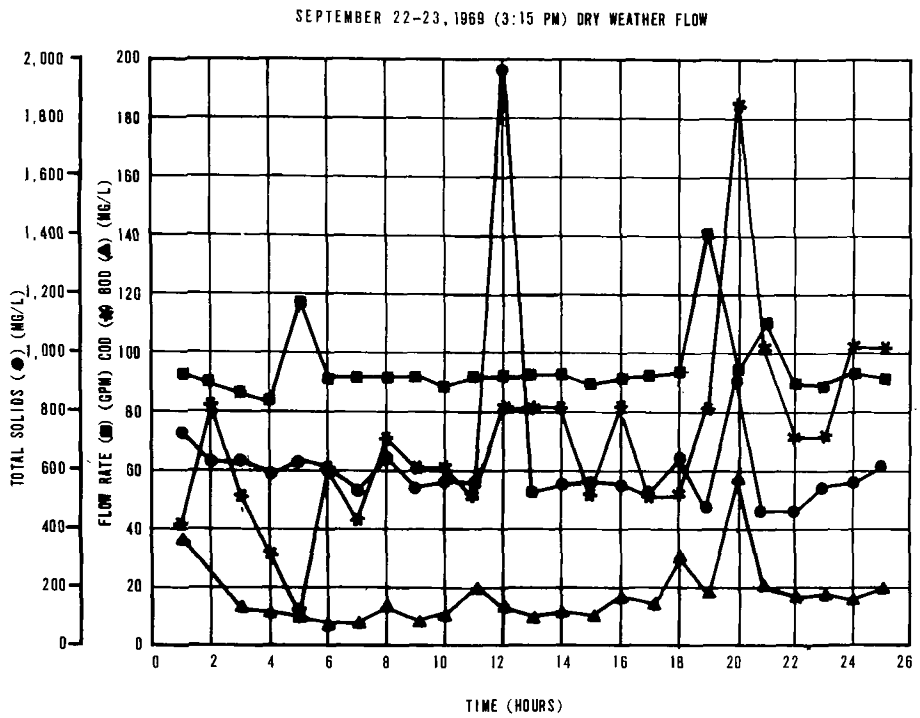


FIGURE E-2
DRY WEATHER FLOW IN GOOD HOPE RUN SEPARATED STORM SEWER



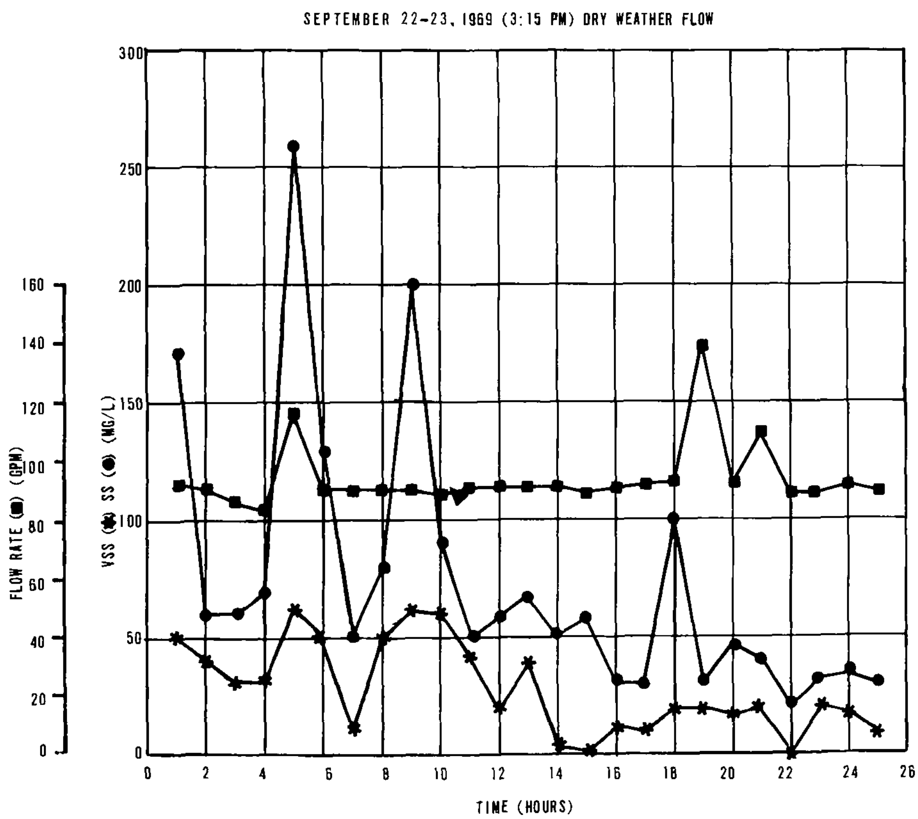
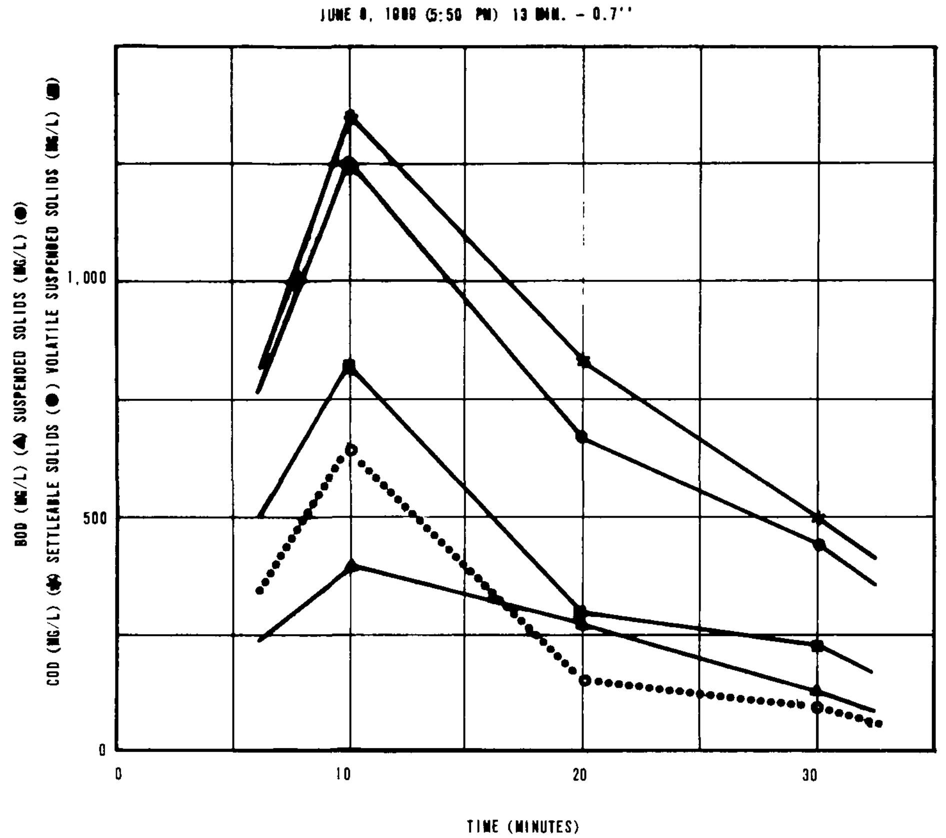


FIGURE E-3

SHORT INTENSE STORM IN COMBINED SEWER DISTRICT G-4



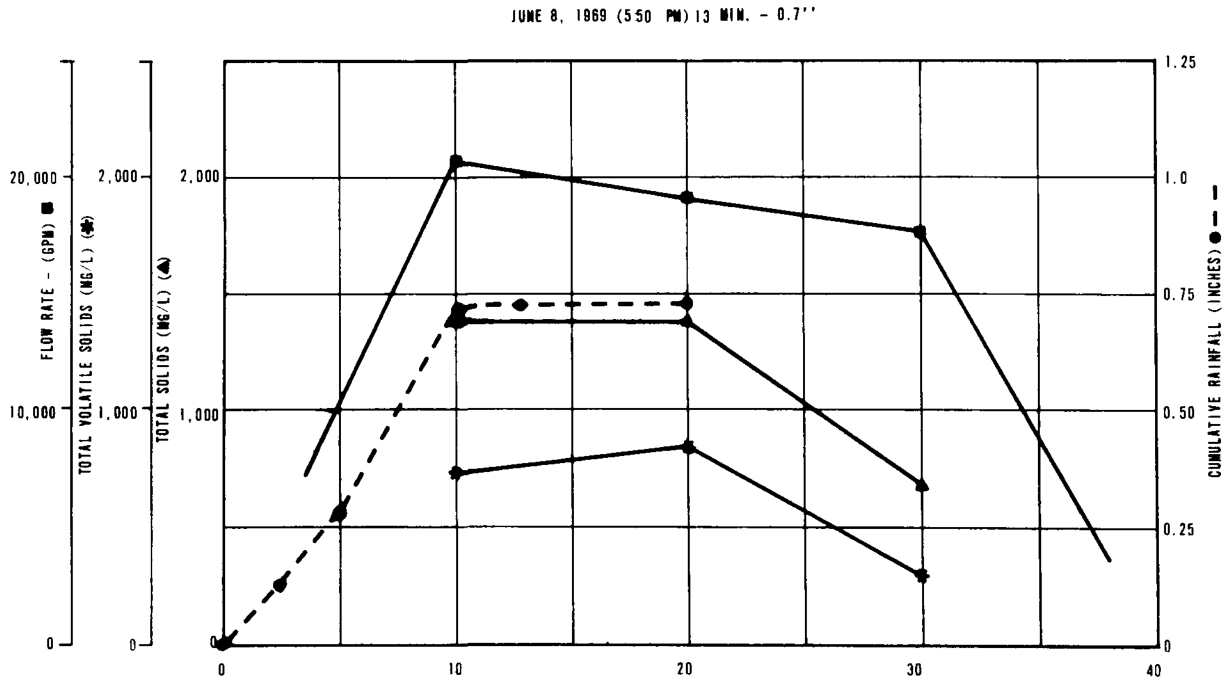
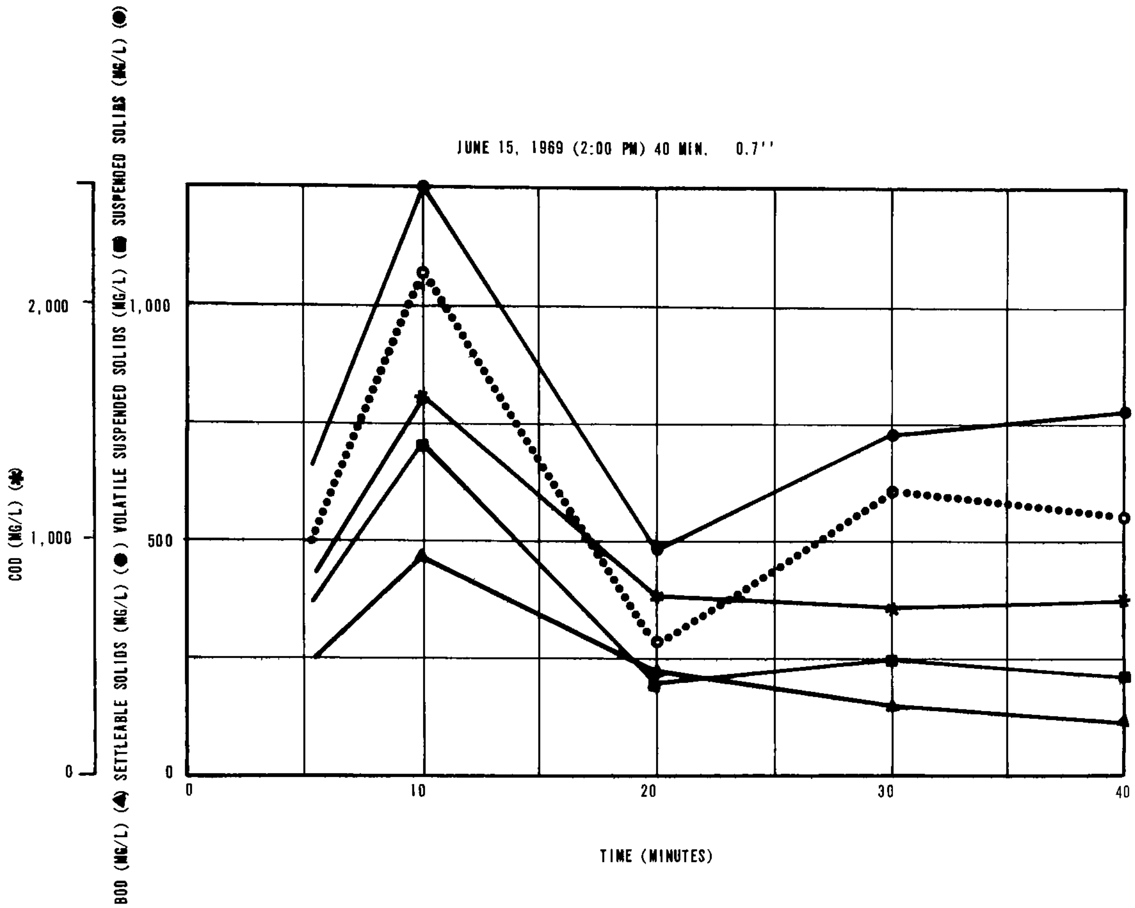


FIGURE E-4

LOW INTENSITY STORM IN COMBINED SEWER DISTRICT G-4



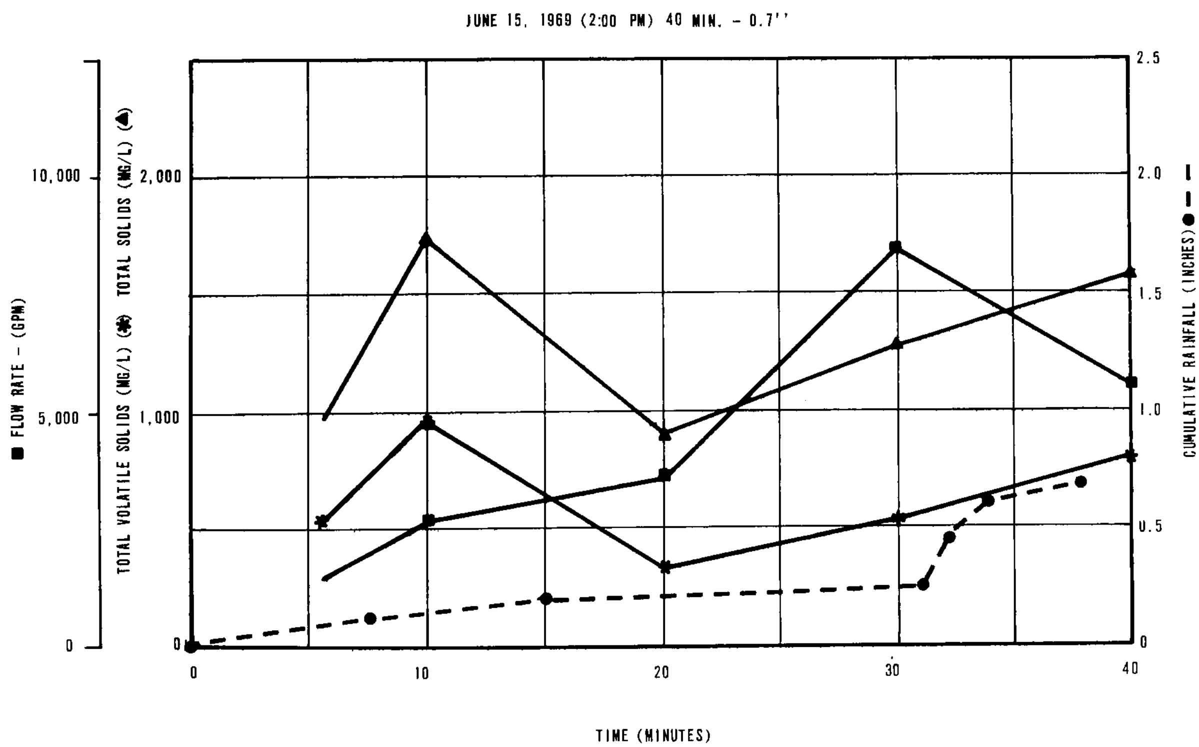
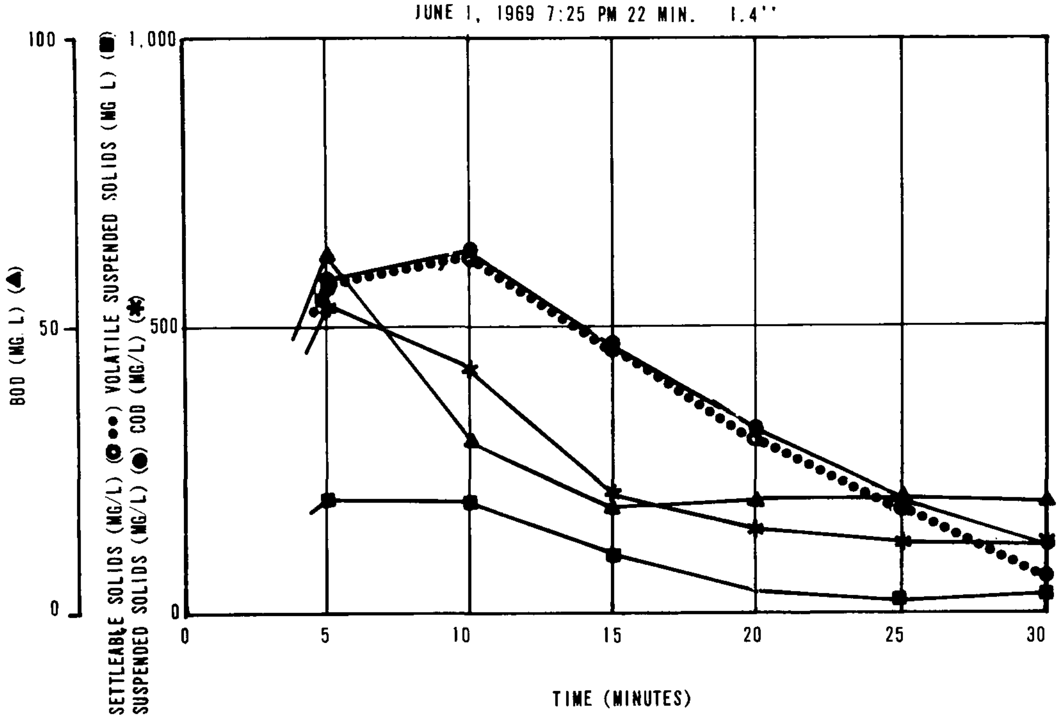


FIGURE E-5
SHORT INTENSE STORM IN COMBINED SEWER DISTRICT B-4



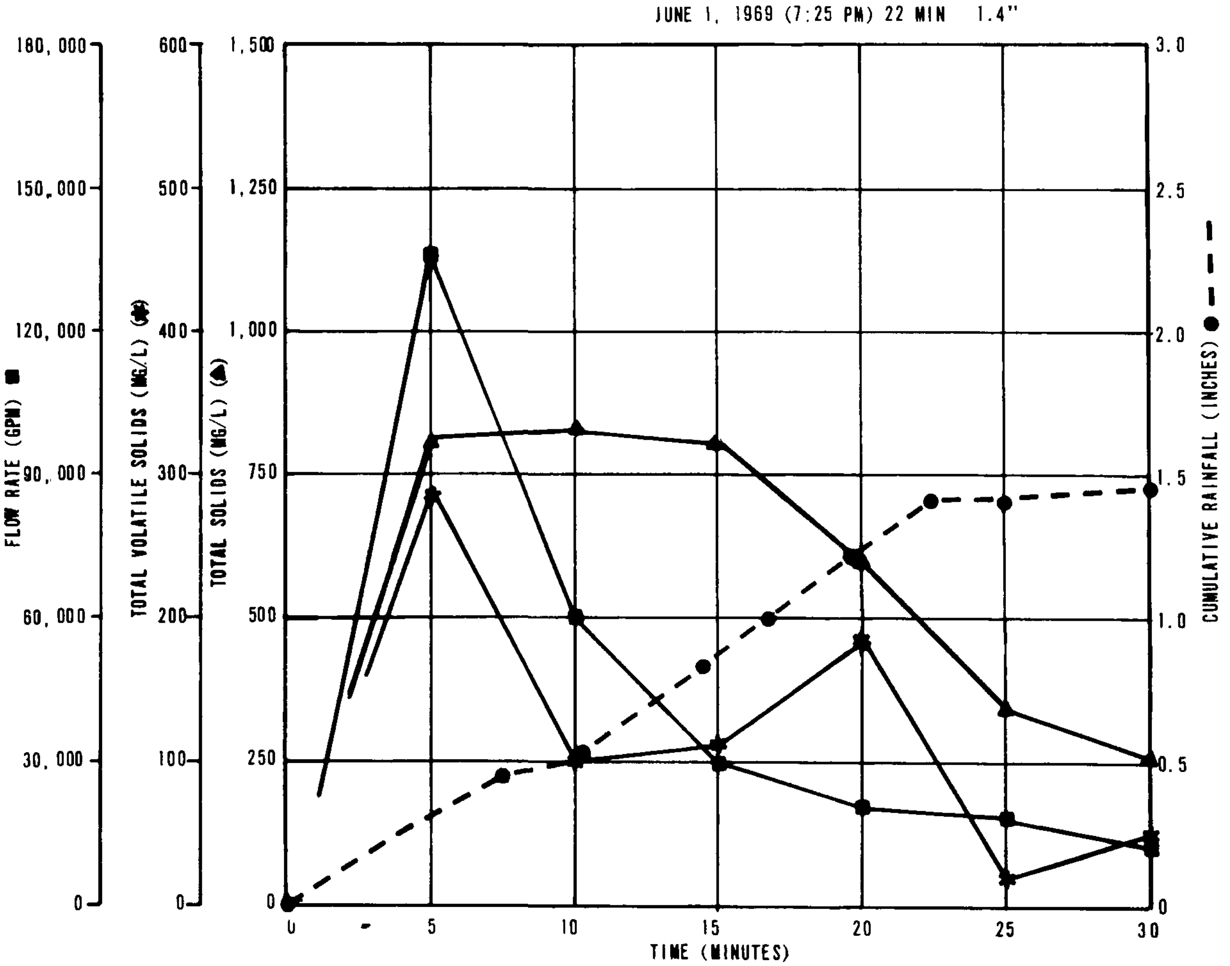


FIGURE E-6
LONG INTENSE STORM IN COMBINED SEWER DISTRICT G-4

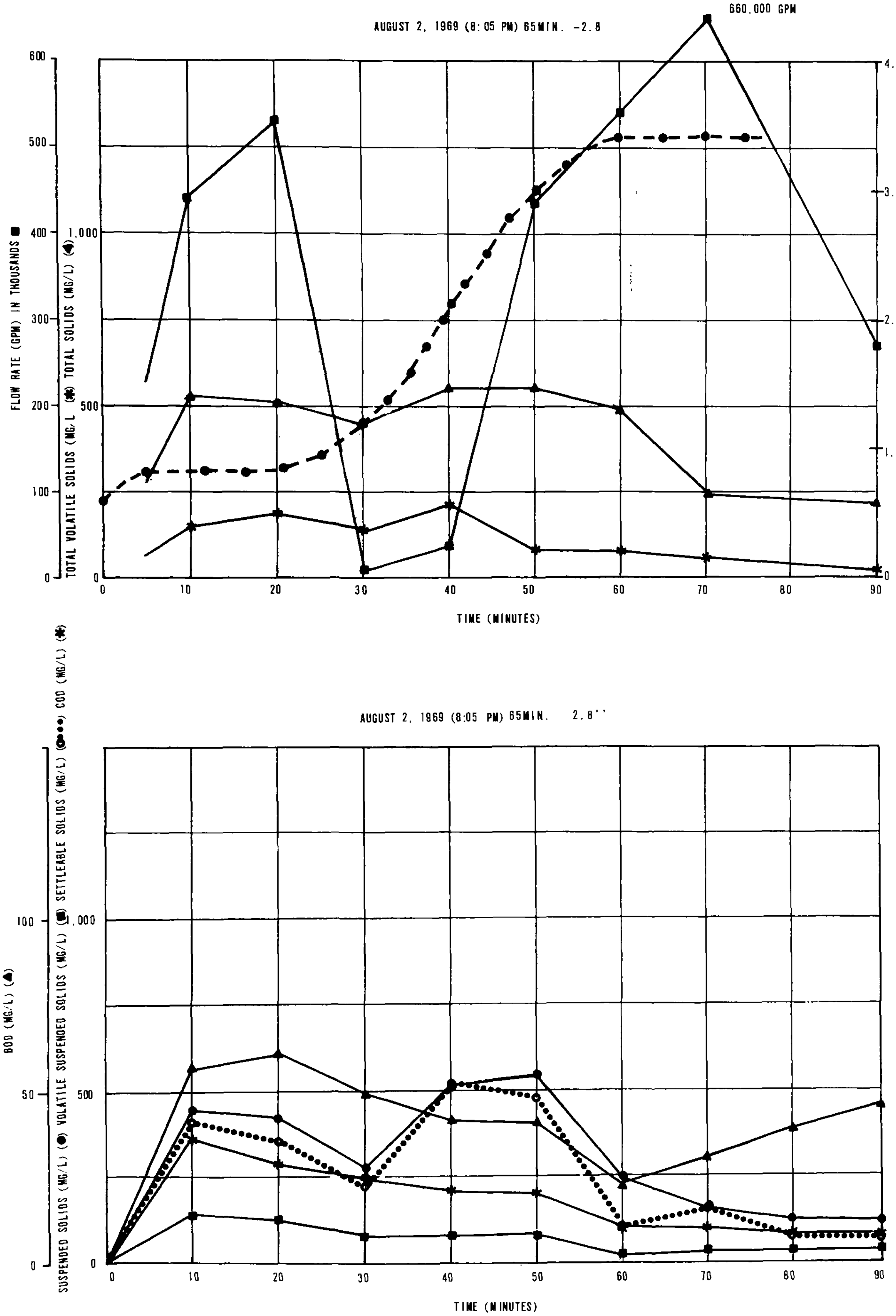
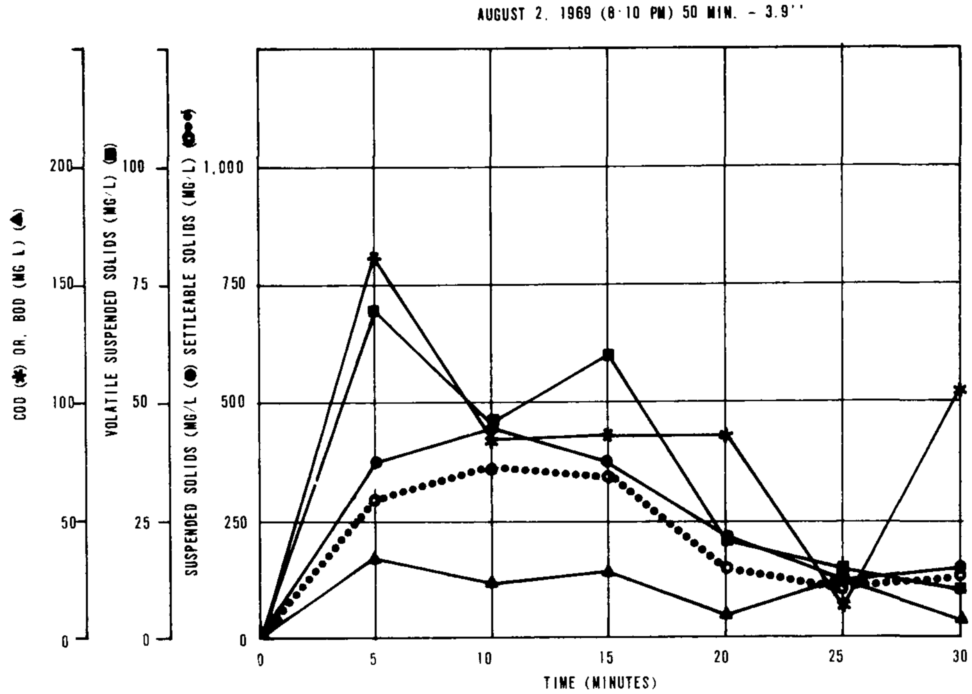


FIGURE E-7

LONG INTENSE STORM IN COMBINED SEWER DISTRICT B-4



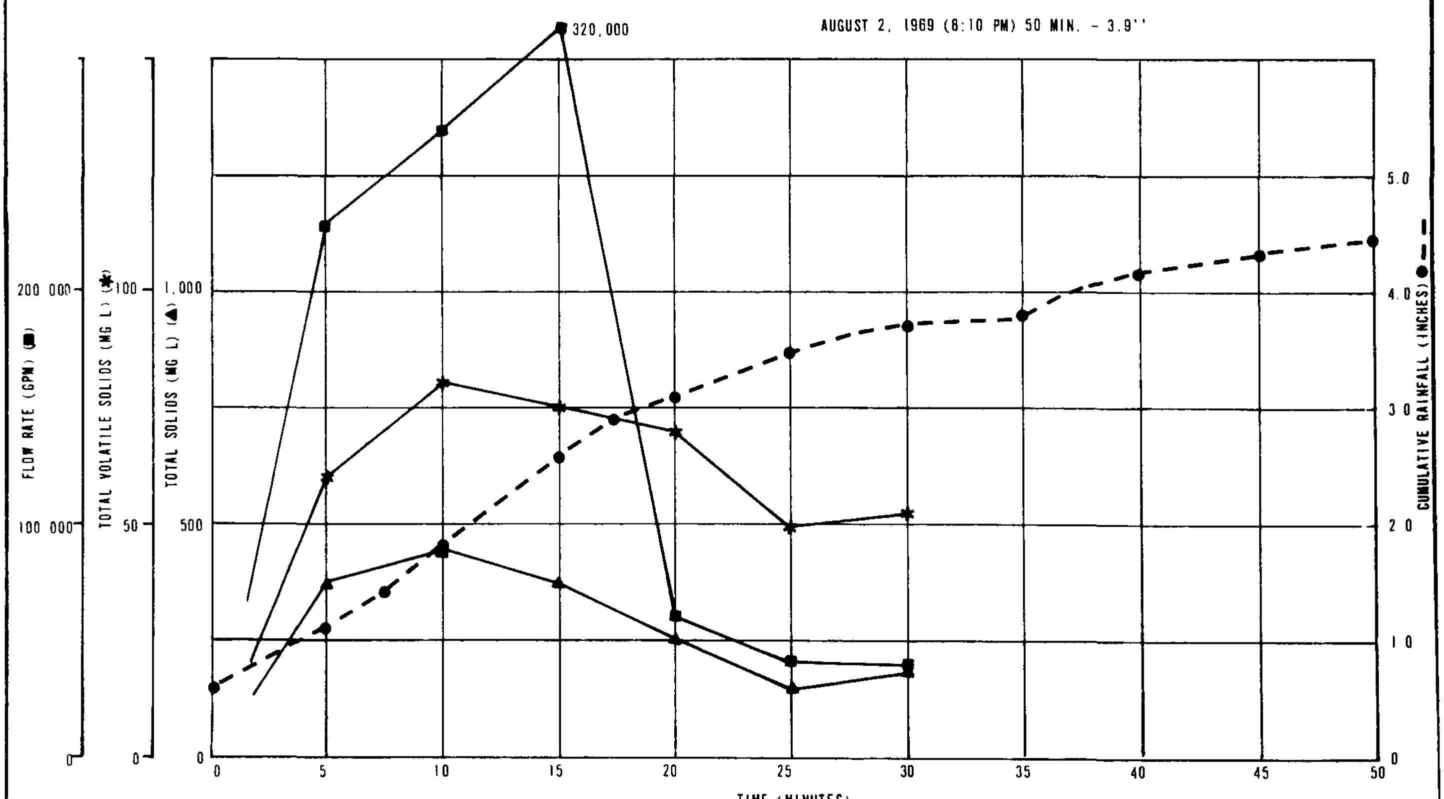


FIGURE E-8
CONSECUTIVE STORMS IN COMBINED SEWER DISTRICT G-4
JULY 27-28, 1969

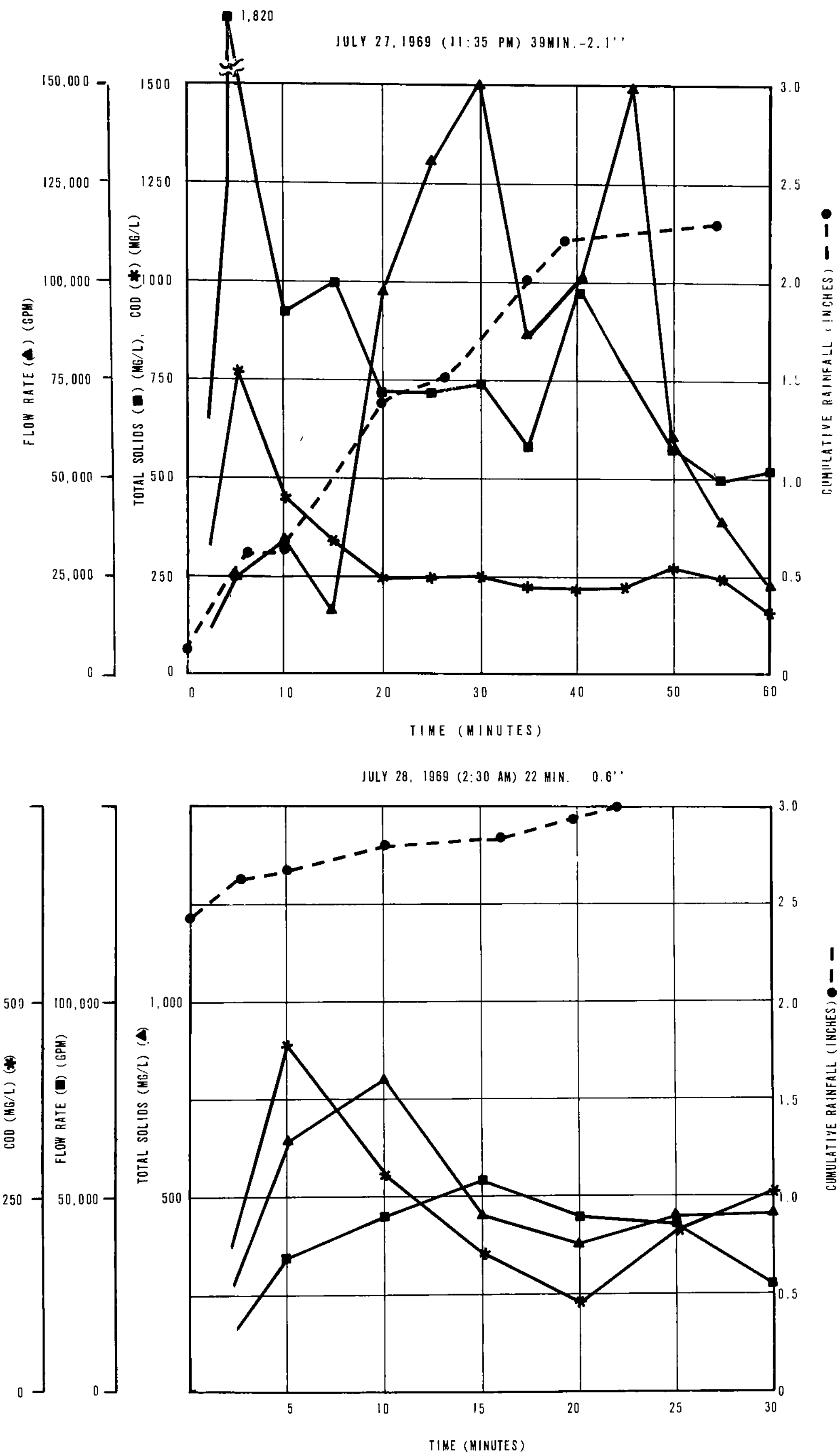
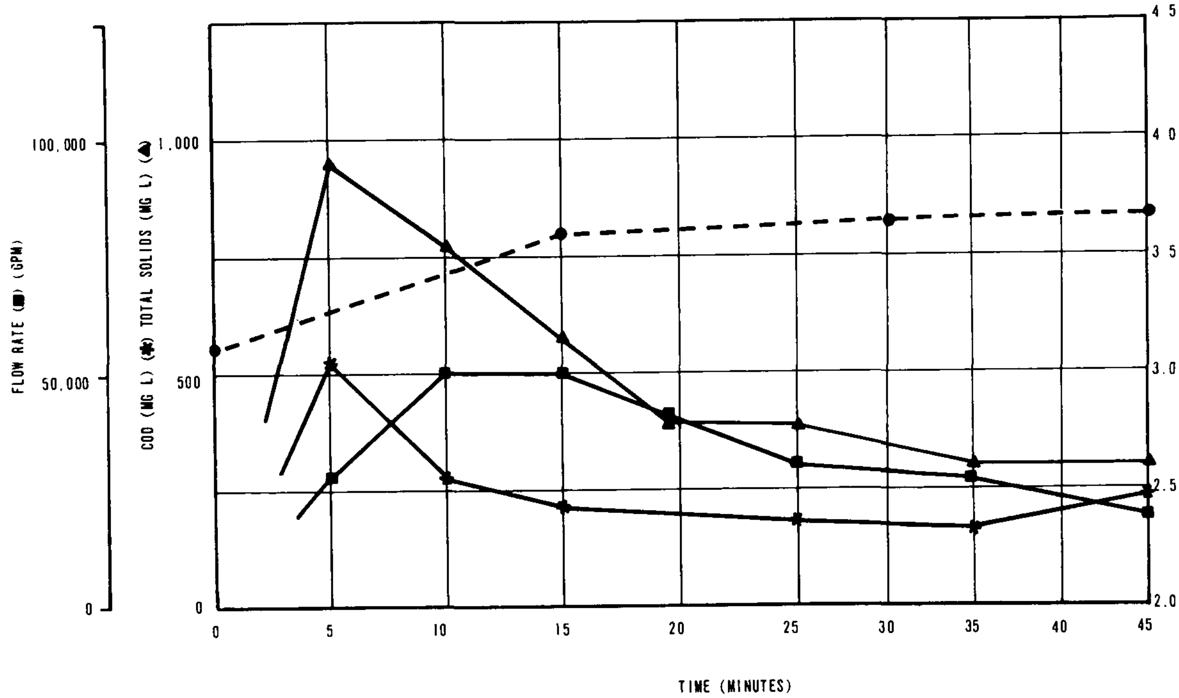


FIGURE E-9

CONSECUTIVE STORMS IN COMBINED SEWER DISTRICT G-4

JULY 28, 1969

JULY 28, 1969 (11:30 AM) 15 MIN - 0.6''



JULY 28, 1969 (1:28 PM) 35 MIN 1.3''

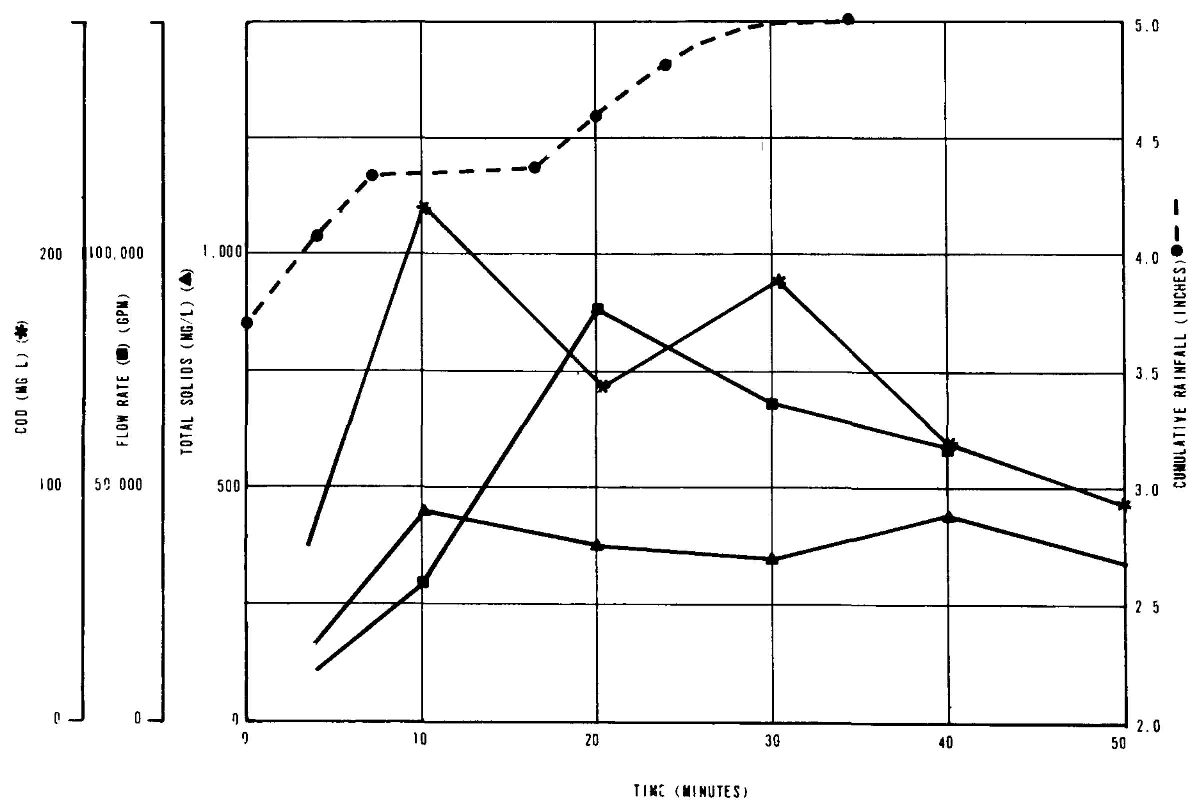
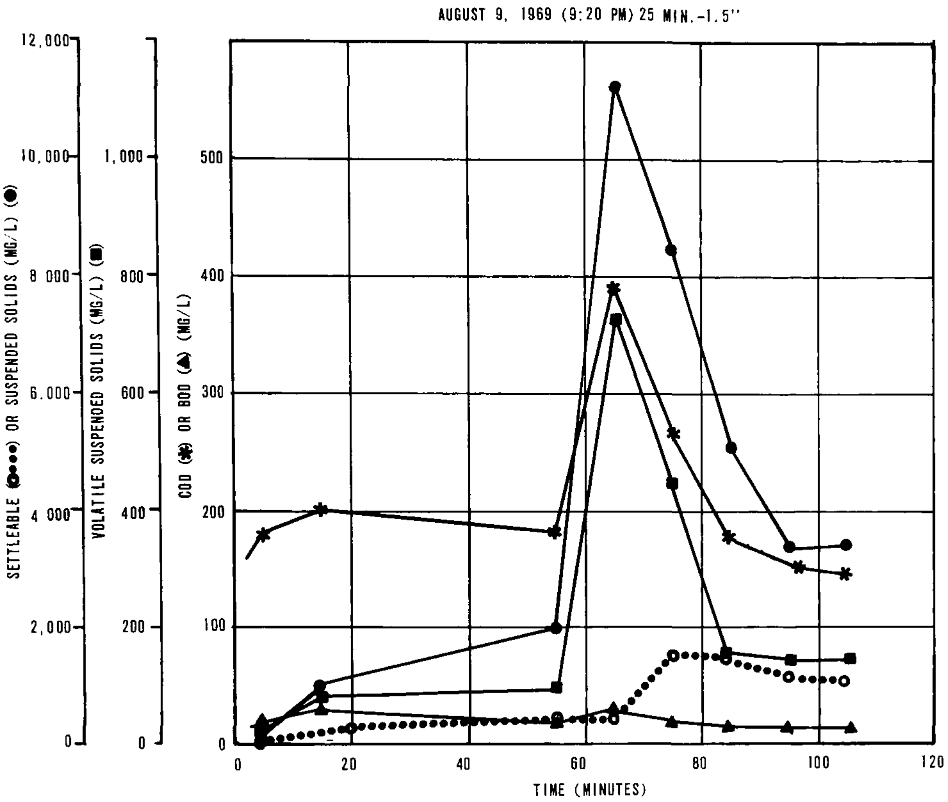


FIGURE E-10

SHORT INTENSE STORM IN SEPARATED SEWER DISTRICT



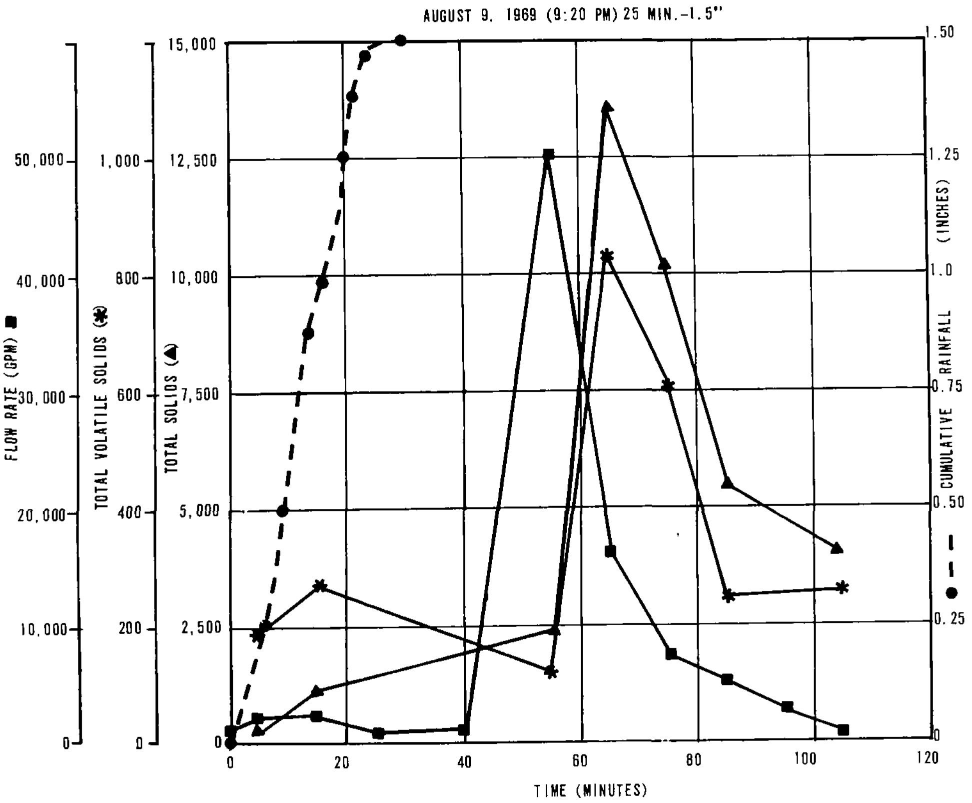
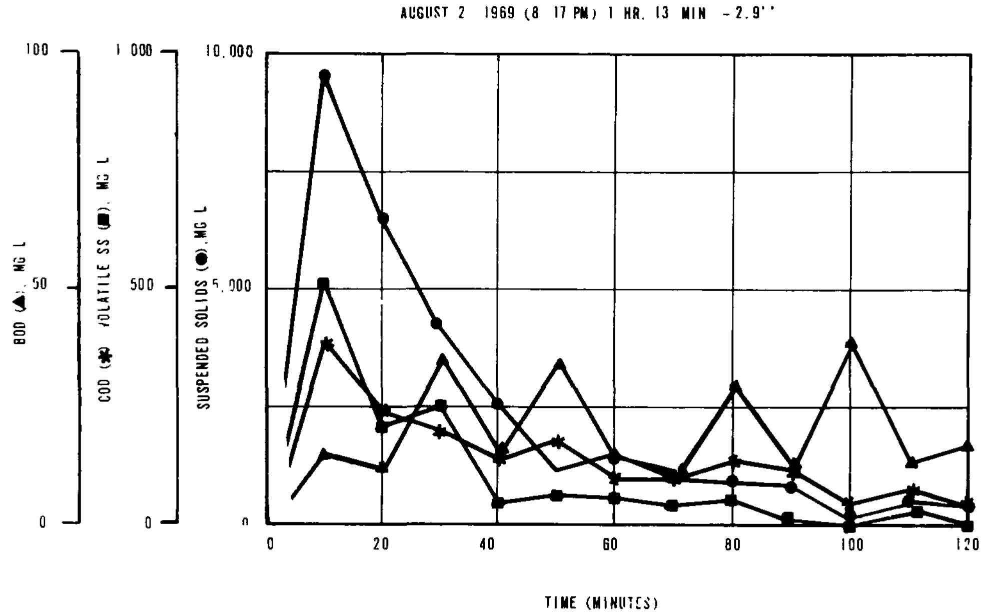


FIGURE E-11

LONG INTENSE STORM IN SEPARATED SEWER DISTRICT



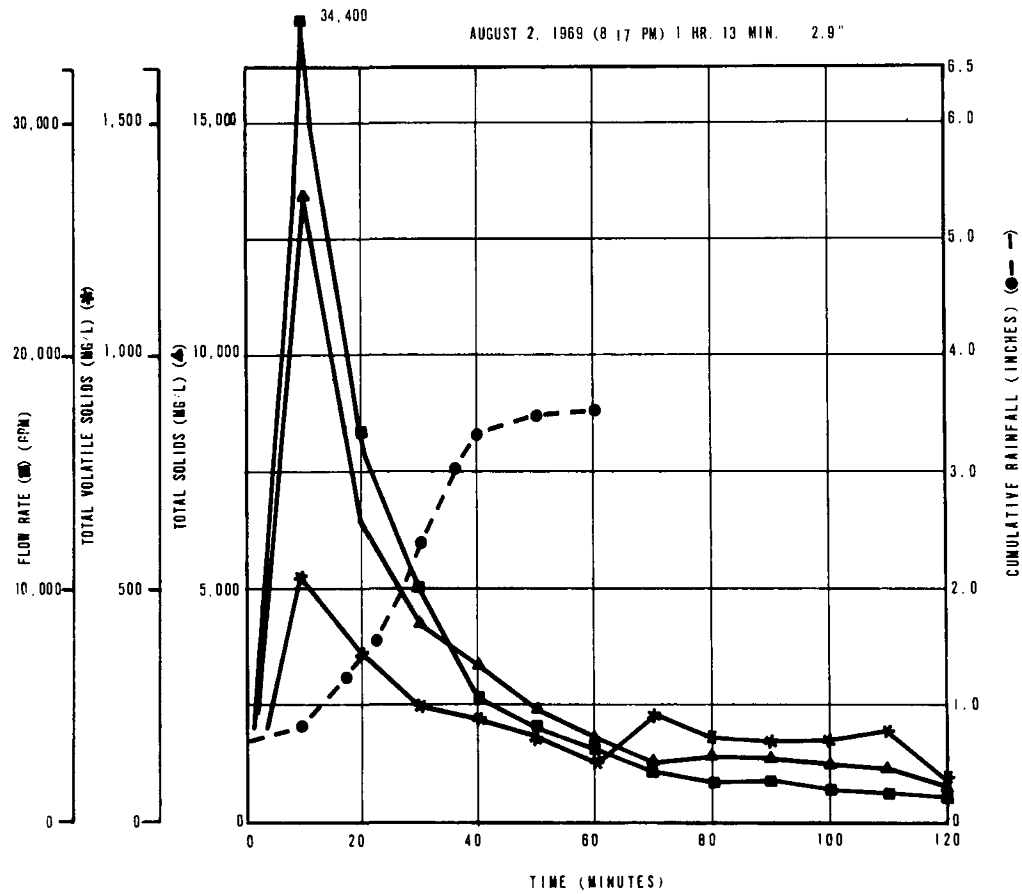
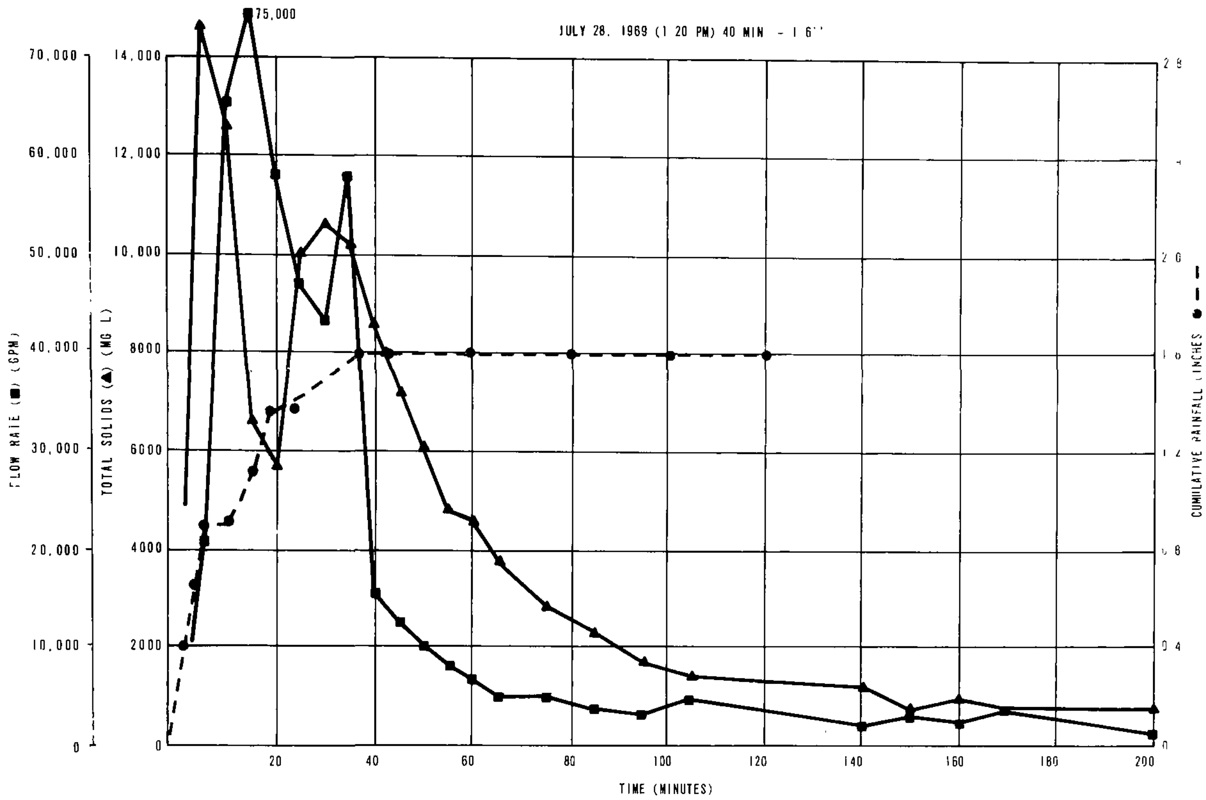


FIGURE E-12

CONSECUTIVE STORMS IN SEPARATED SEWER DISTRICT

JULY 28, 1969 1:20 PM



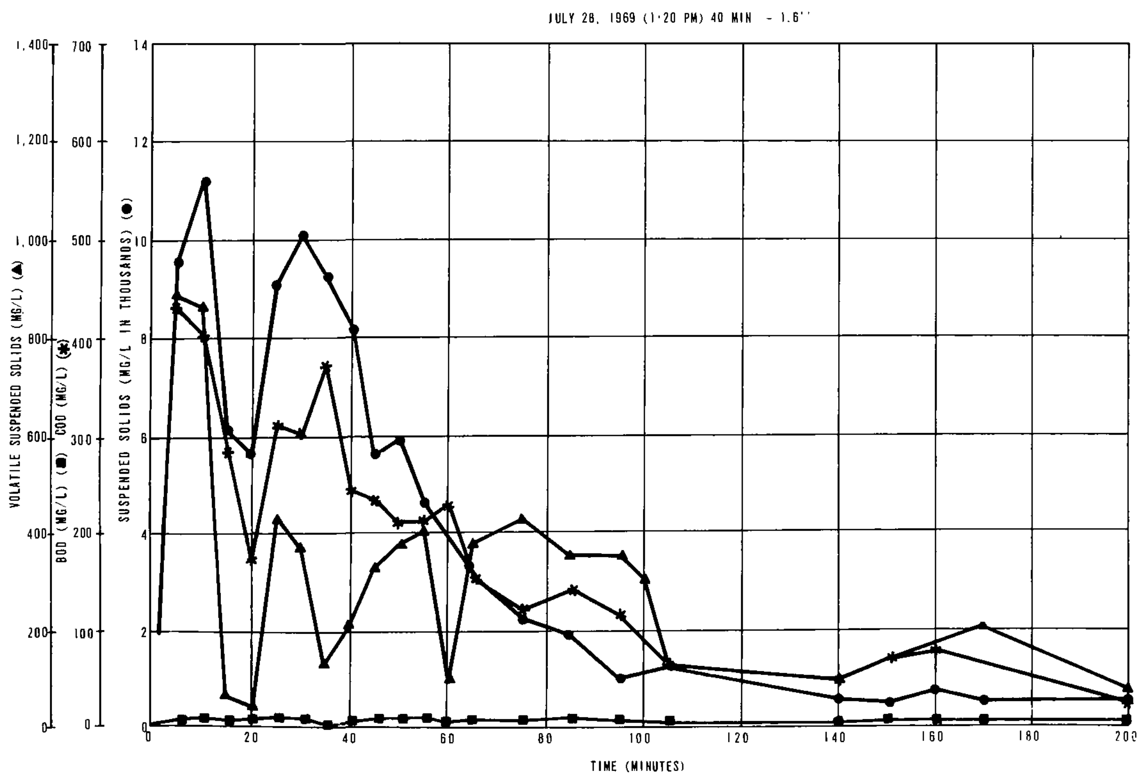
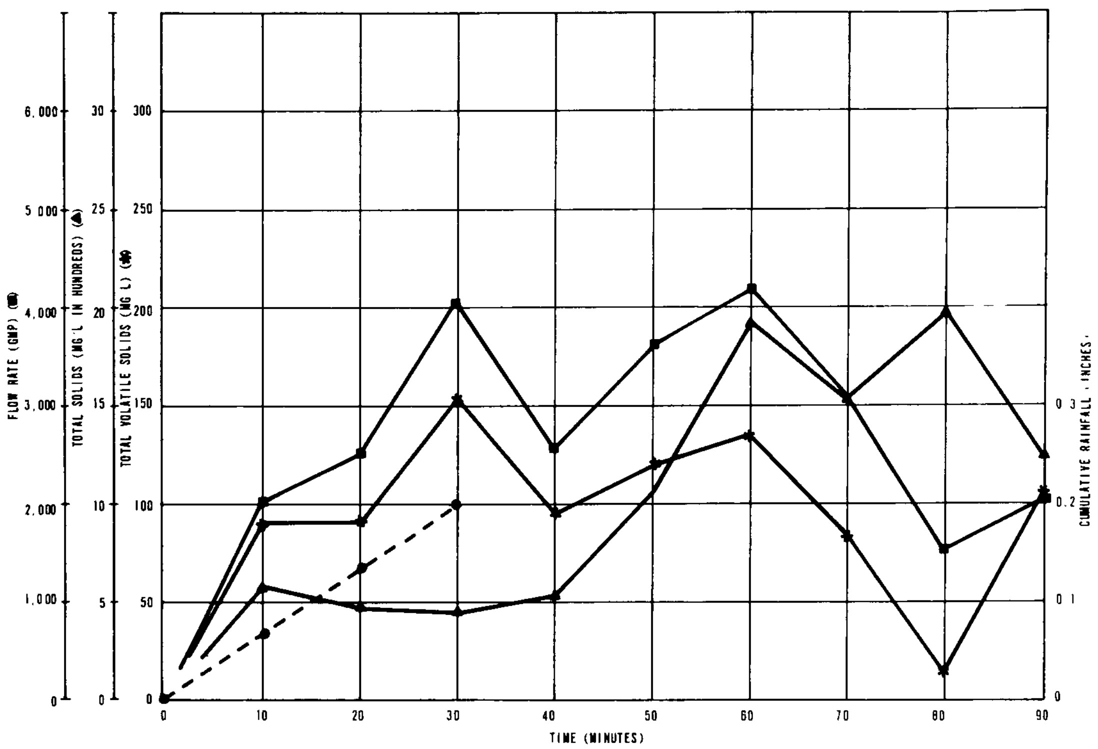


FIGURE E-13

CONSECUTIVE STORMS IN SEPARATED SEWER DISTRICT

JULY 28, 1969 5:00 PM

JULY 28, 1969 (5 00 PM) 30 MINS - 0 20"



JULY 28, 1969 (5 00 PM) 30 MINS - 0 20"

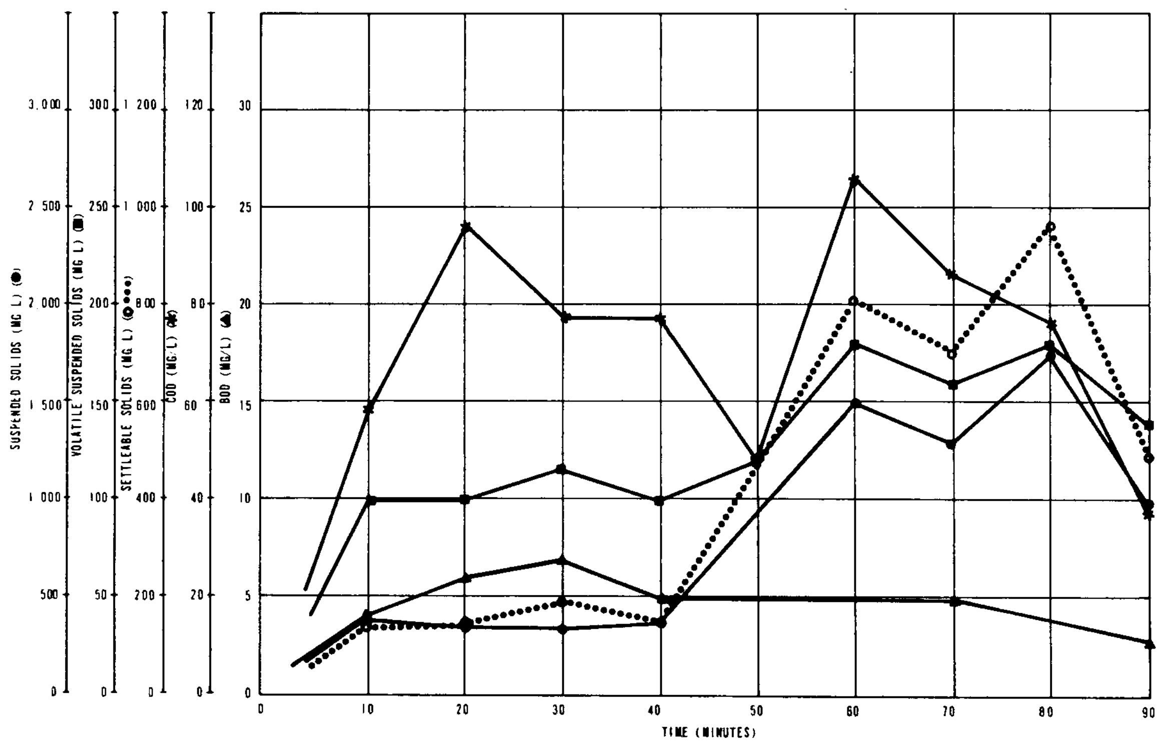
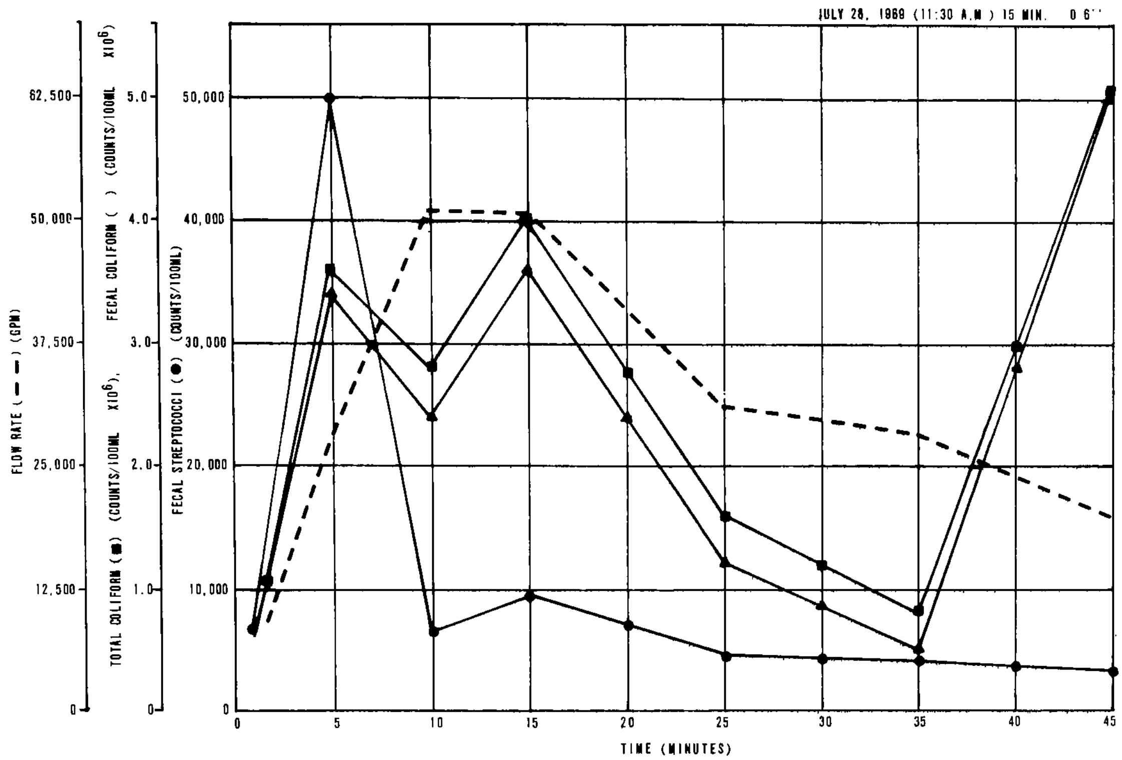


FIGURE E-14

REPRESENTATIVE BACTERIOLOGICAL DATA FOR COMBINED SEWER OVERFLOW IN WASHINGTON, D.C.



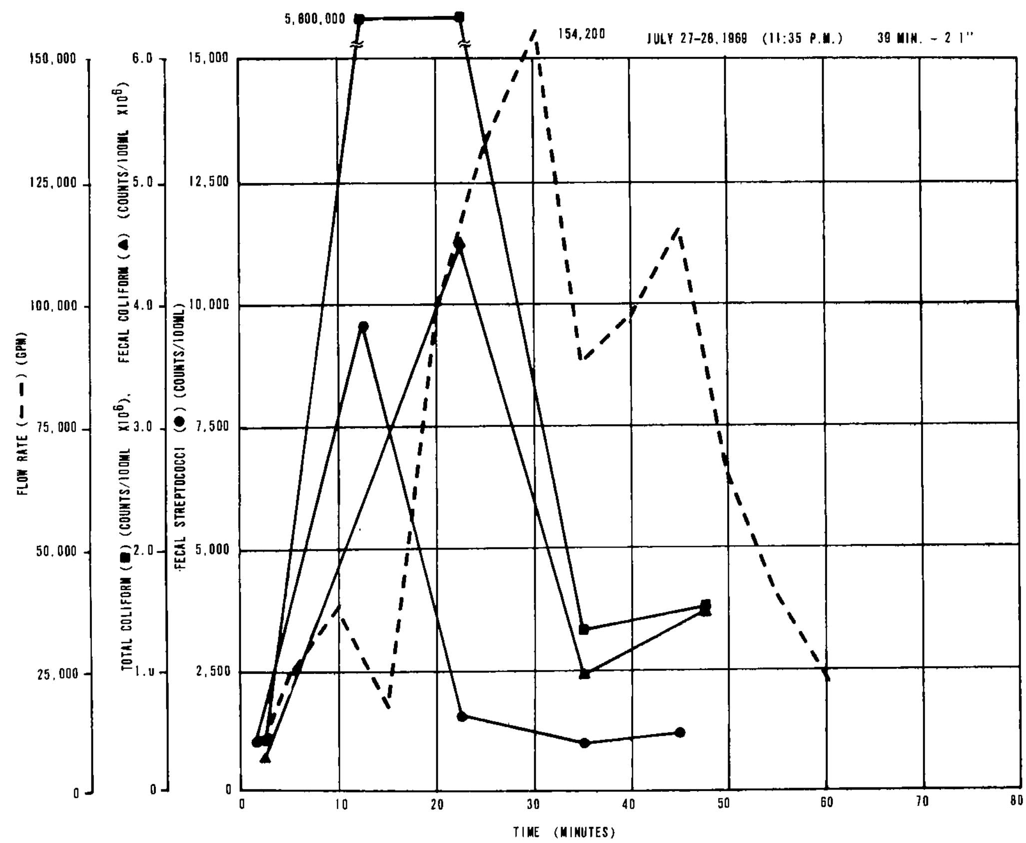
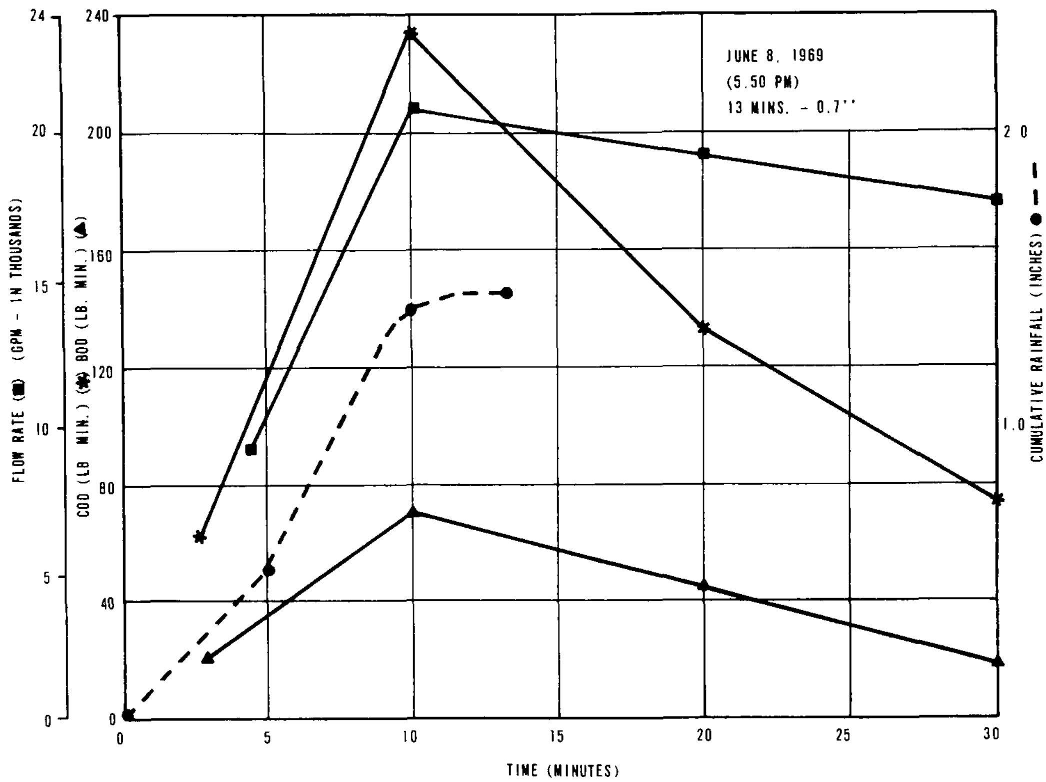


FIGURE E-15

WASTE LOADING ASSOCIATED WITH SHORT INTENSE STORM IN COMBINED SEWER DISTRICT G-4



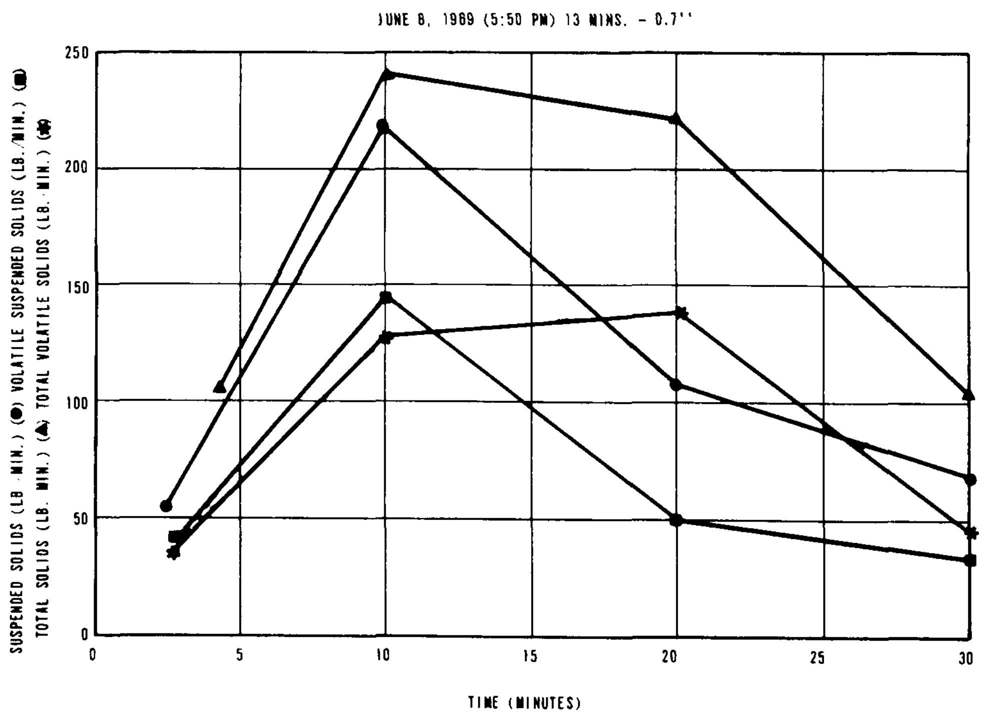
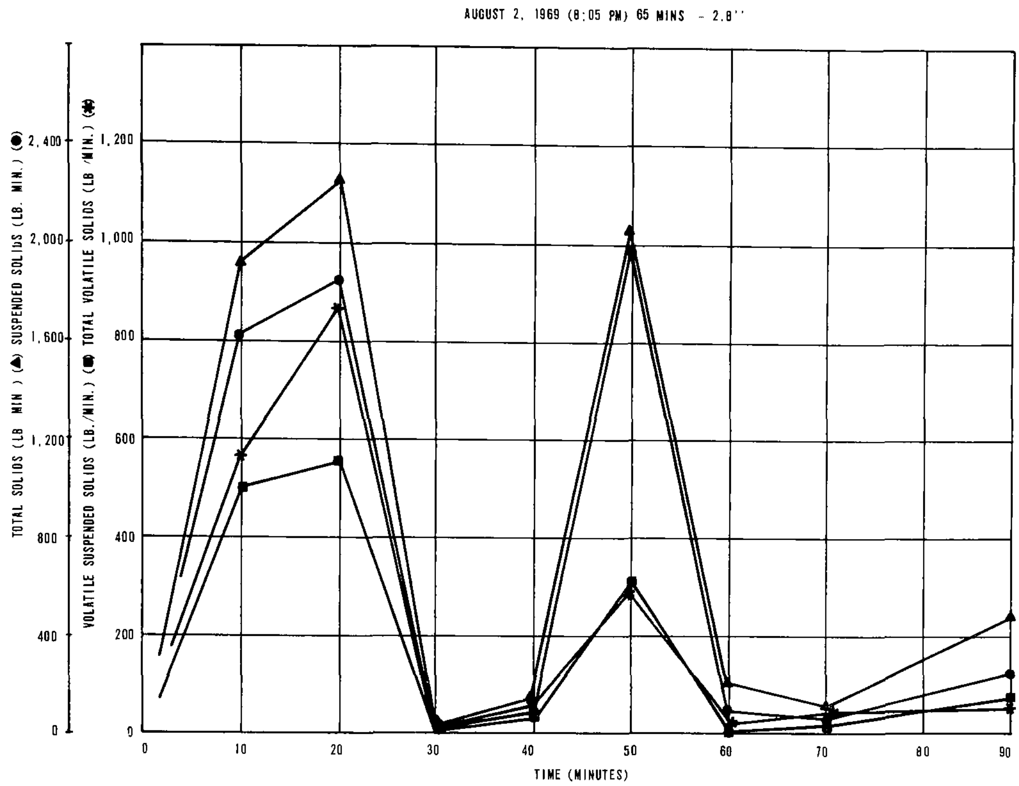


FIGURE E-16

WASTE LOADING ASSOCIATED WITH LONG INTENSE STORM IN COMBINED SEWER DISTRICT G-4



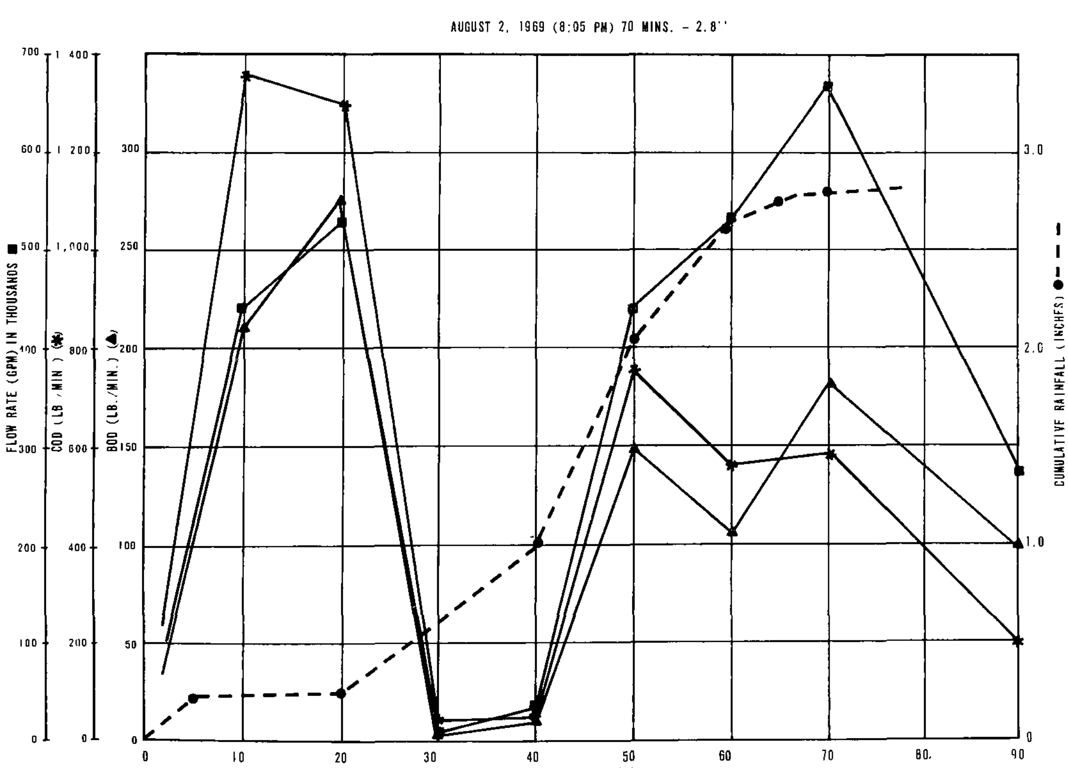


FIGURE E-17

WASTE LOADING ASSOCIATED WITH CONSECUTIVE STORMS IN COMBINED SEWER DISTRICT G-4

JULY 27, 1969 - CUMULATIVE RAINFALL, FLOW RATE, COD

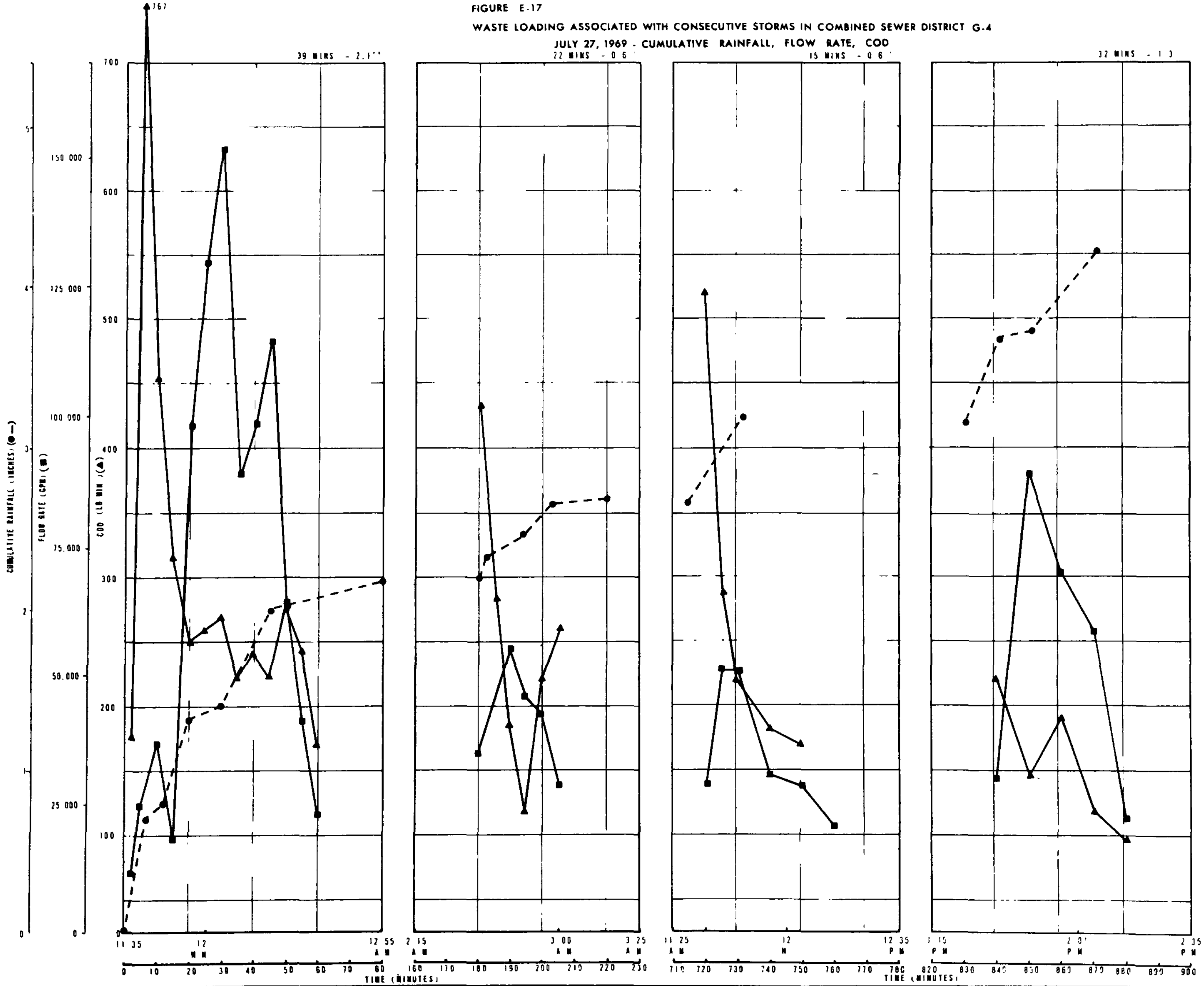


FIGURE E-18
WASTE LOADING ASSOCIATED WITH CONSECUTIVE STORMS IN COMBINED SEWER DISTRICT G-4
JULY 27, 1969 - SUSPENDED SOLIDS, TOTAL VOLATILE SOLIDS, VOLATILE SUSPENDED SOLIDS.

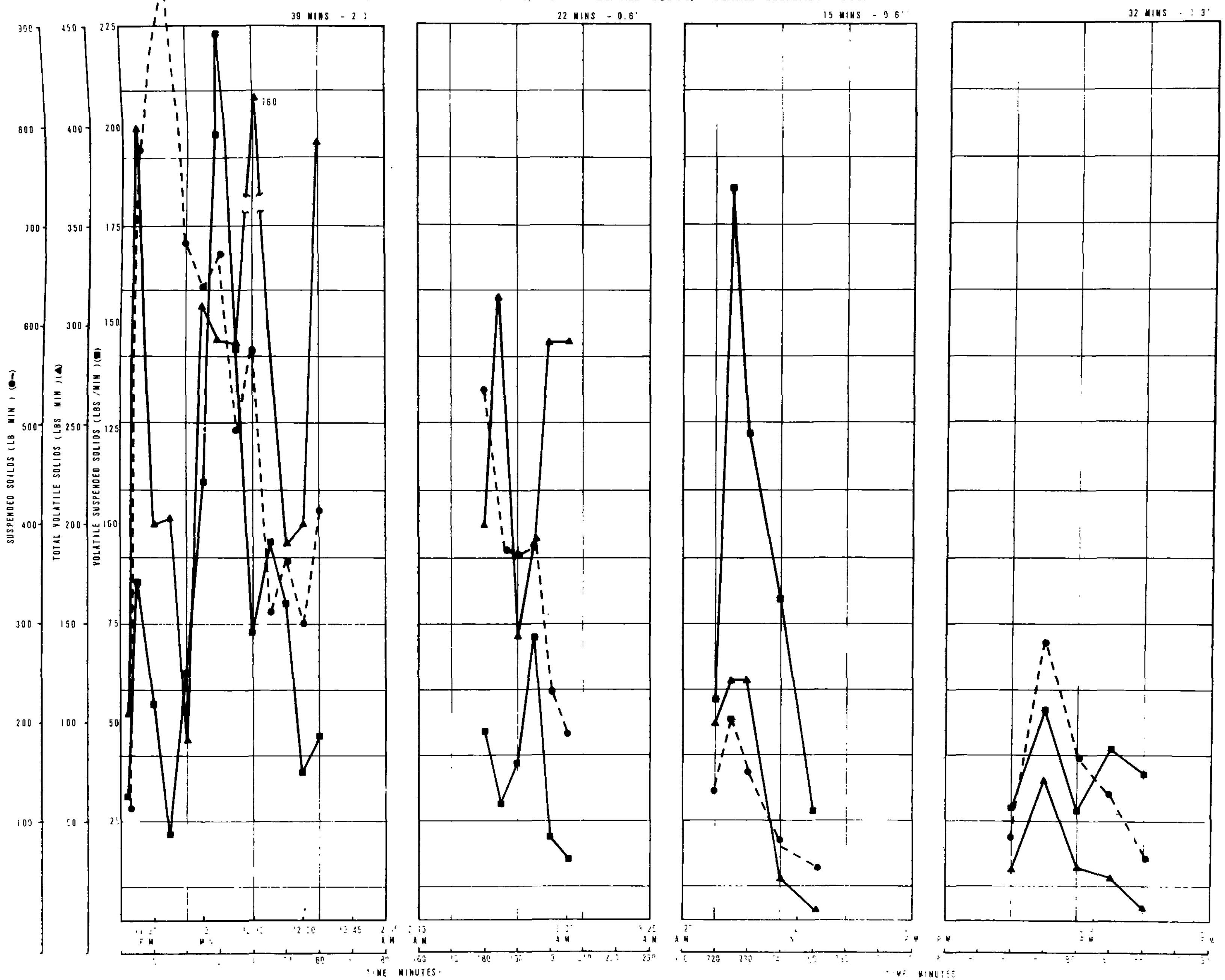
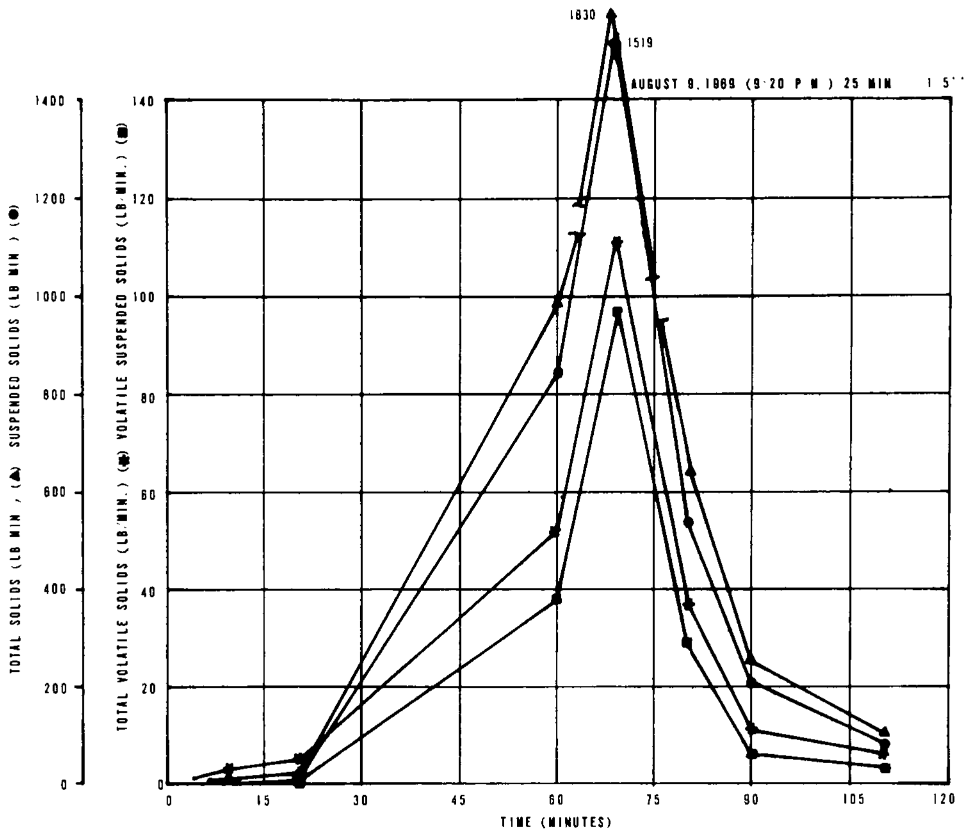


FIGURE E-19

WASTE LOADING ASSOCIATED WITH SHORT INTENSE STORM IN SEPARATED SEWER DISTRICT



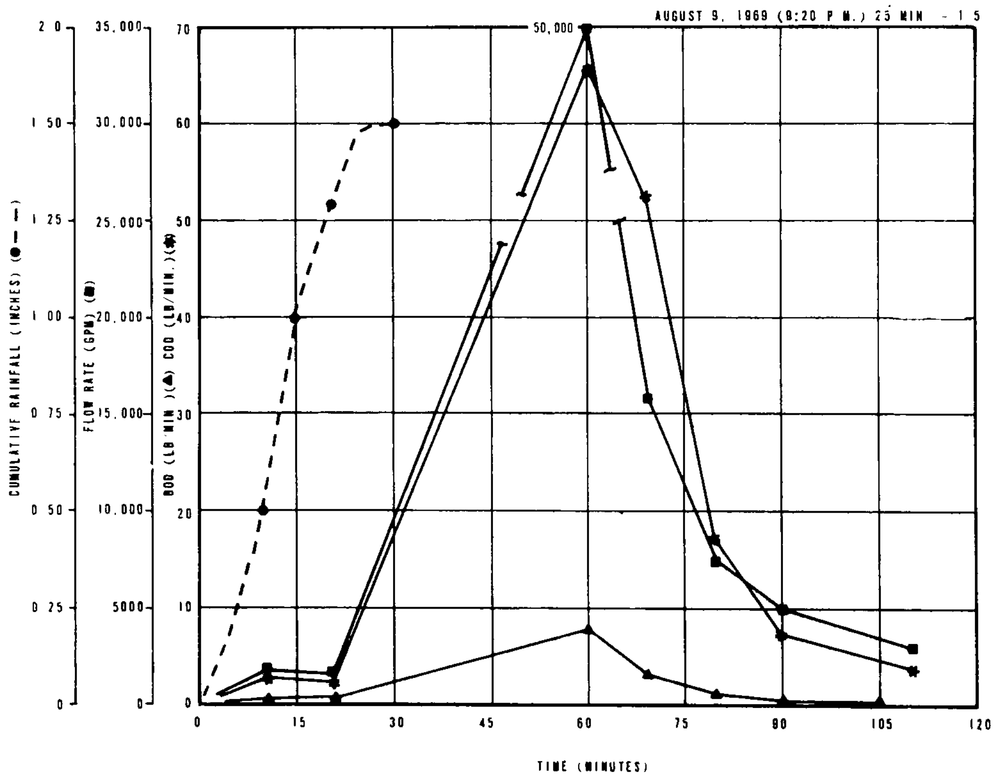
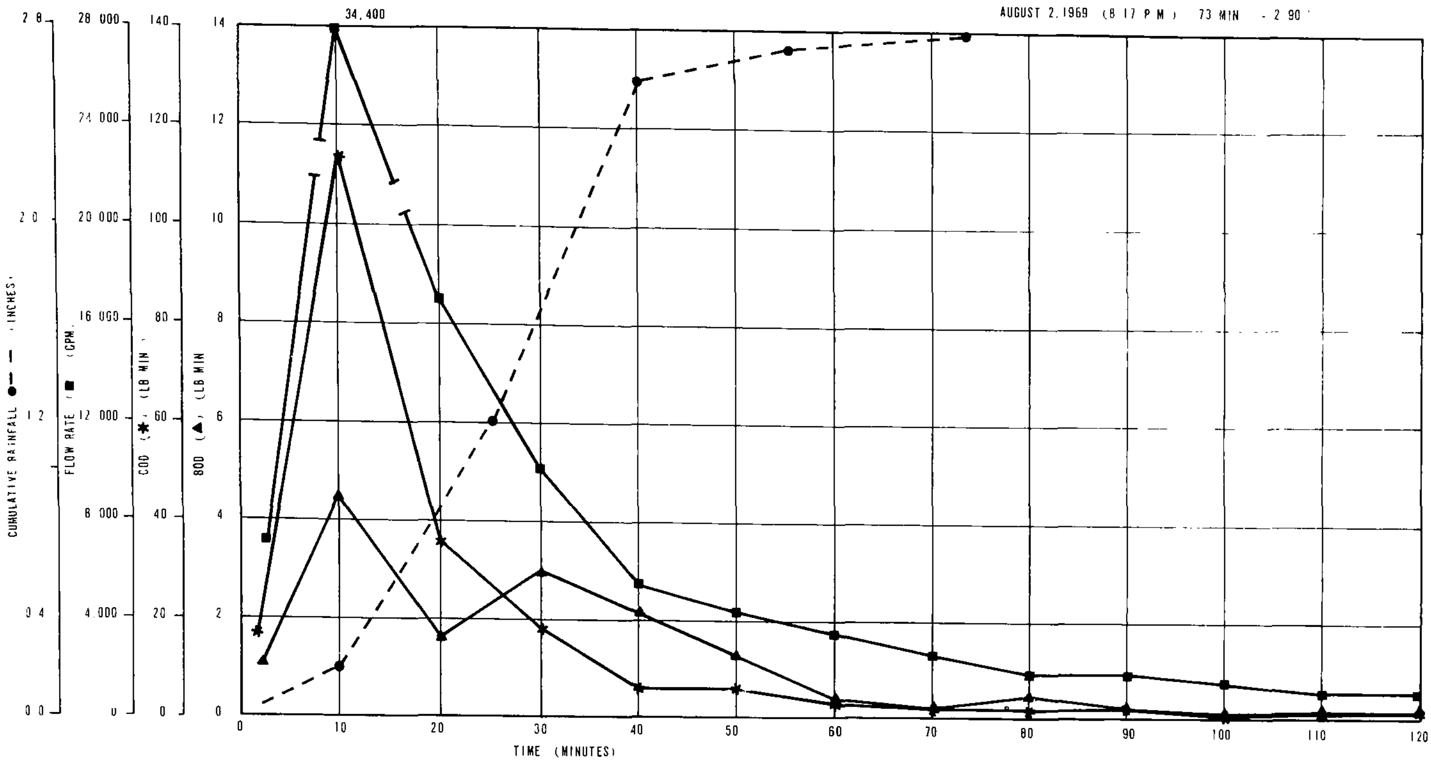
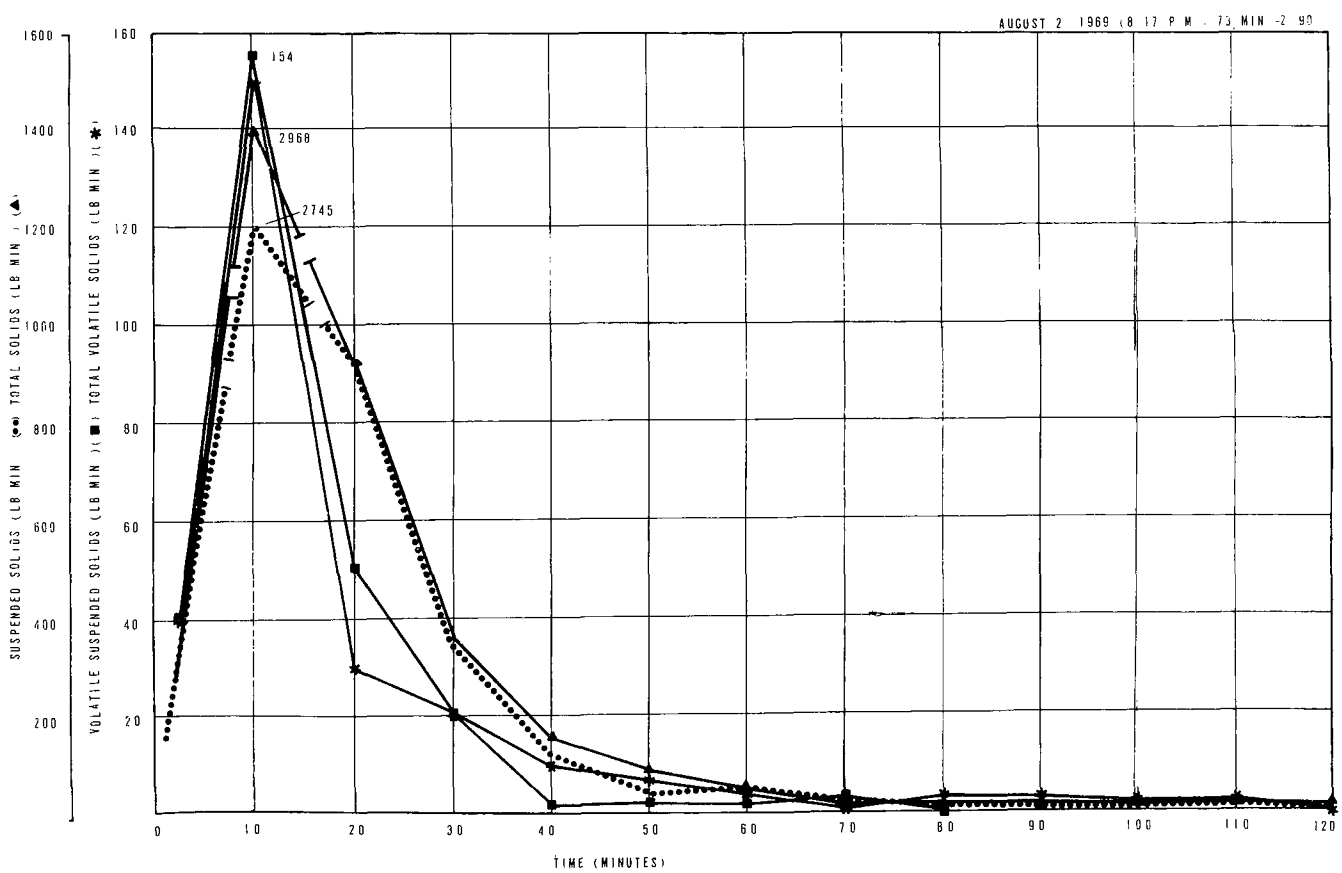


FIGURE E-20

WASTE LOADING ASSOCIATED WITH LONG INTENSE STORM IN SEPARATED SEWER DISTRICT





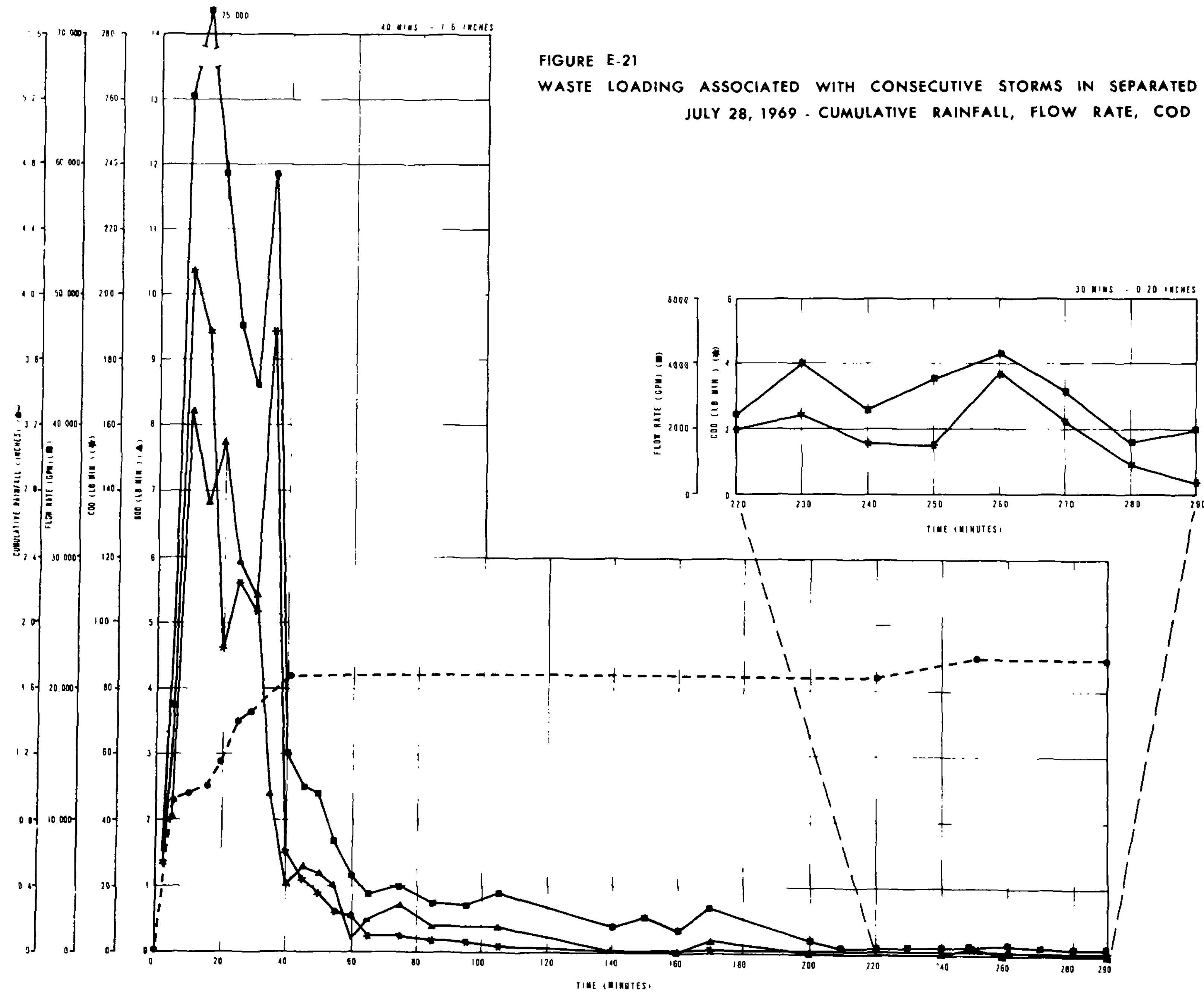


FIGURE E-21
WASTE LOADING ASSOCIATED WITH CONSECUTIVE STORMS IN SEPARATED SEWER DISTRICT
JULY 28, 1969 - CUMULATIVE RAINFALL, FLOW RATE, COD

FIGURE E-22

WASTE LOADING ASSOCIATED WITH CONSECUTIVE STORMS IN SEPARATED SEWER DISTRICT

JULY 28, 1969 TOTAL SOLIDS; SUSPENDED SOLIDS, TOTAL VOLATILE SOLIDS, VOLATILE SUSPENDED SOLIDS

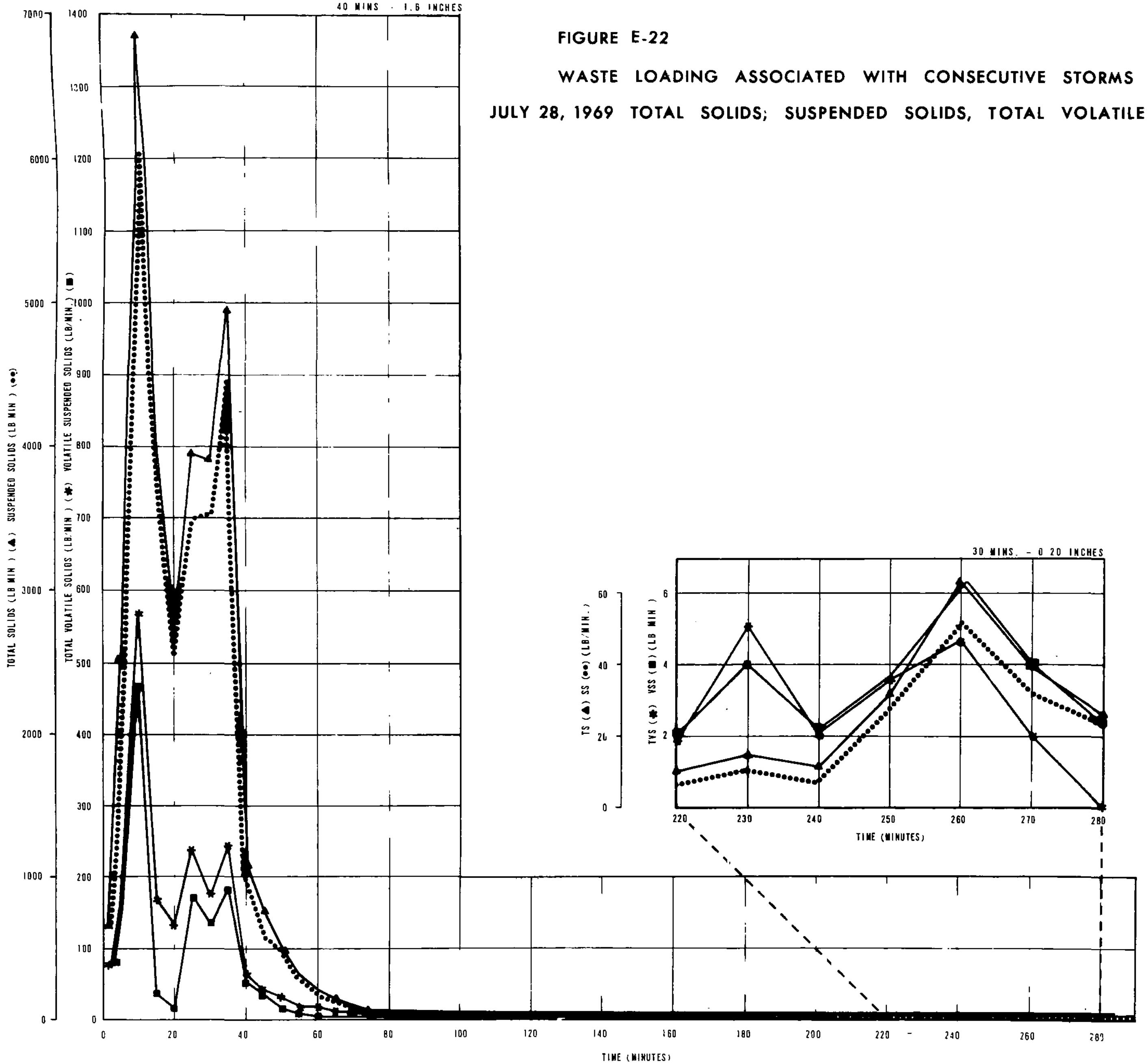


FIGURE E-23

CORRELATION BETWEEN COD WASTE LOADING AND TOTAL RAINFALL

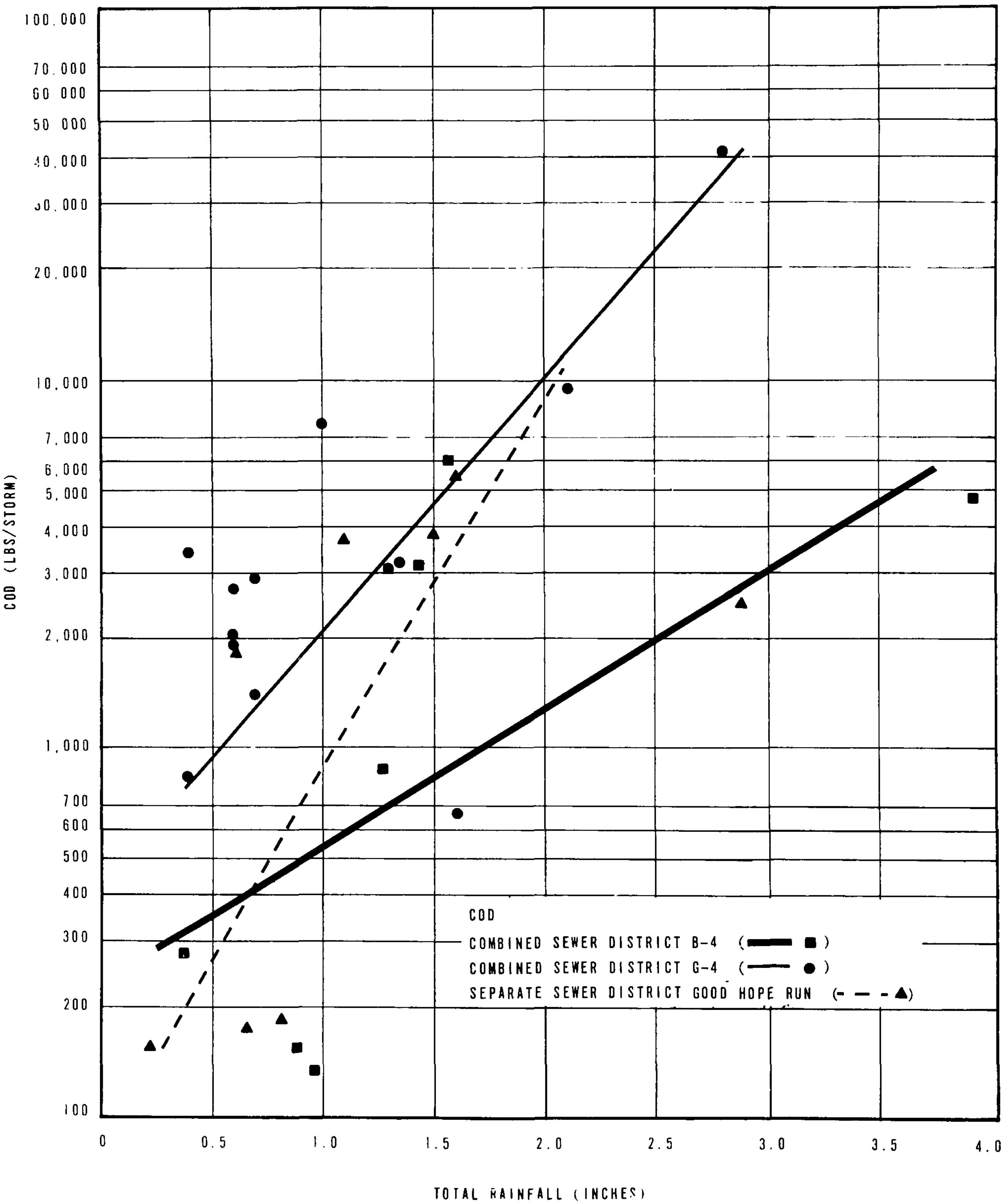
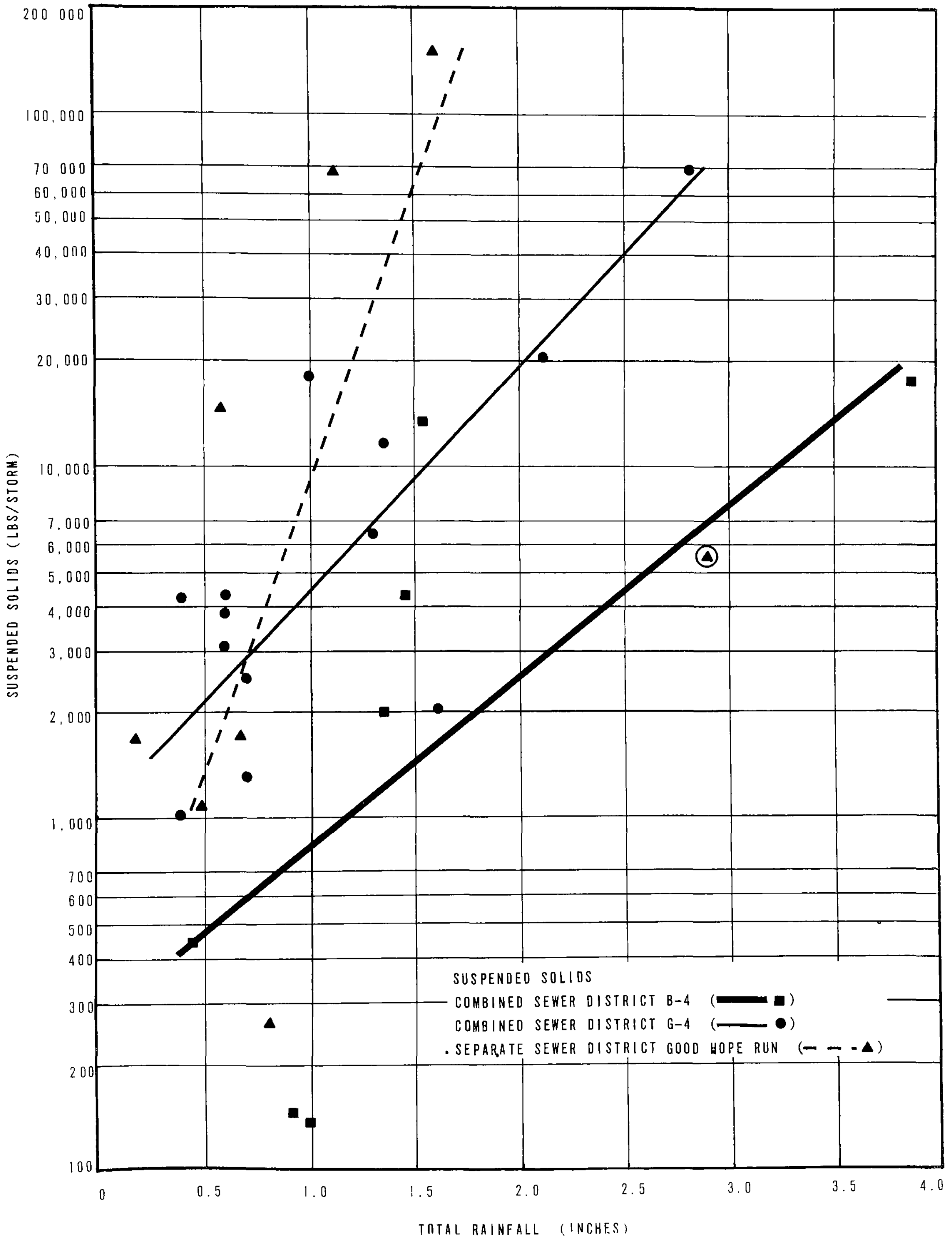
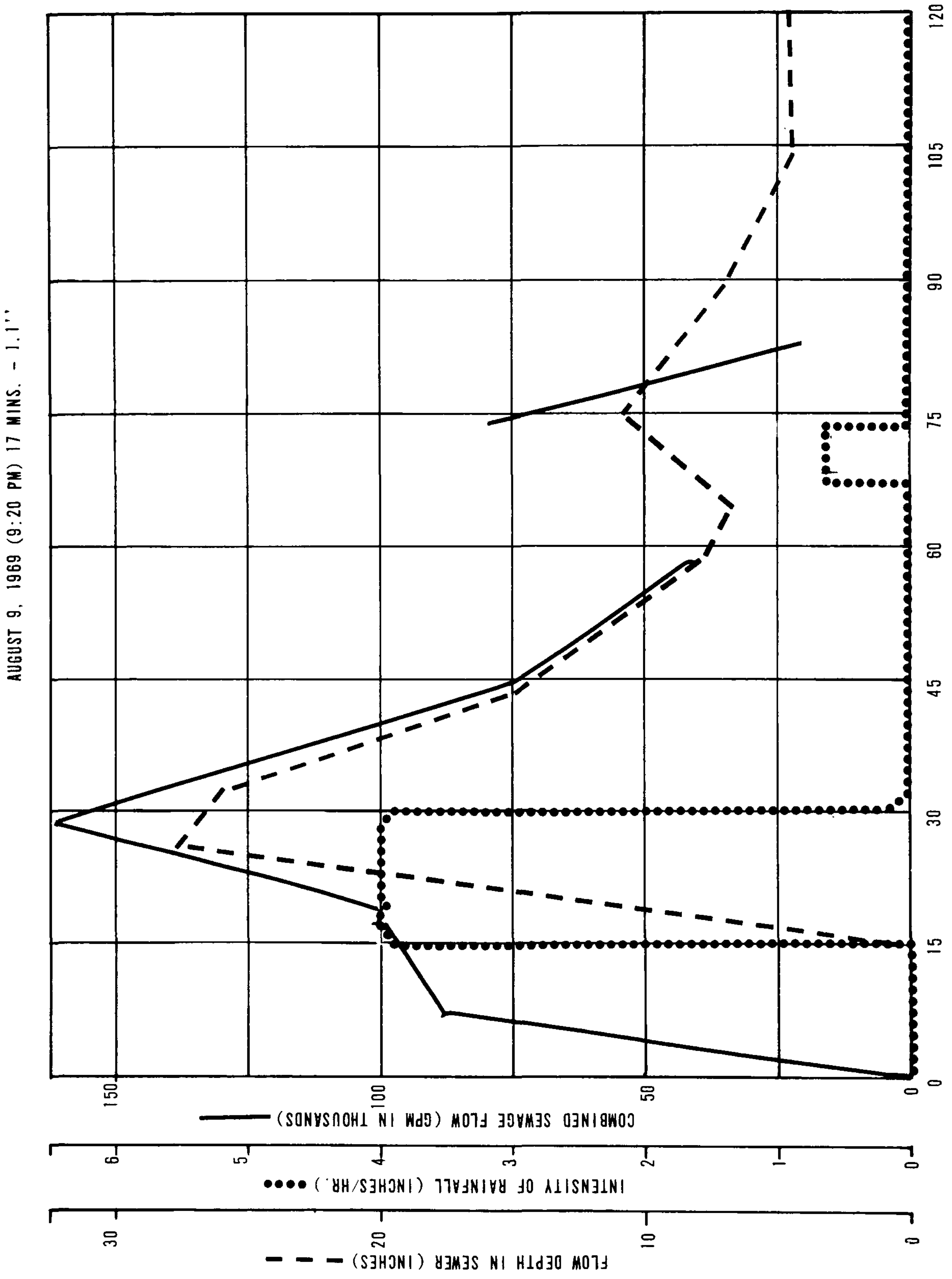


FIGURE E-24
CORRELATION BETWEEN SUSPENDED SOLIDS WASTE LOADING AND TOTAL RAINFALL



E-24

FIGURE E-25
REPRESENTATIVE RAINFALL AND RUNOFF MEASUREMENTS FOR SHORT INTENSE STORM IN COMBINED SEWER DISTRICT G-4



E-25

FIGURE E-26
REPRESENTATIVE RAINFALL AND RUNOFF MEASUREMENTS FOR LONG INTENSE STORM IN COMBINED SEWER DISTRICT G-4

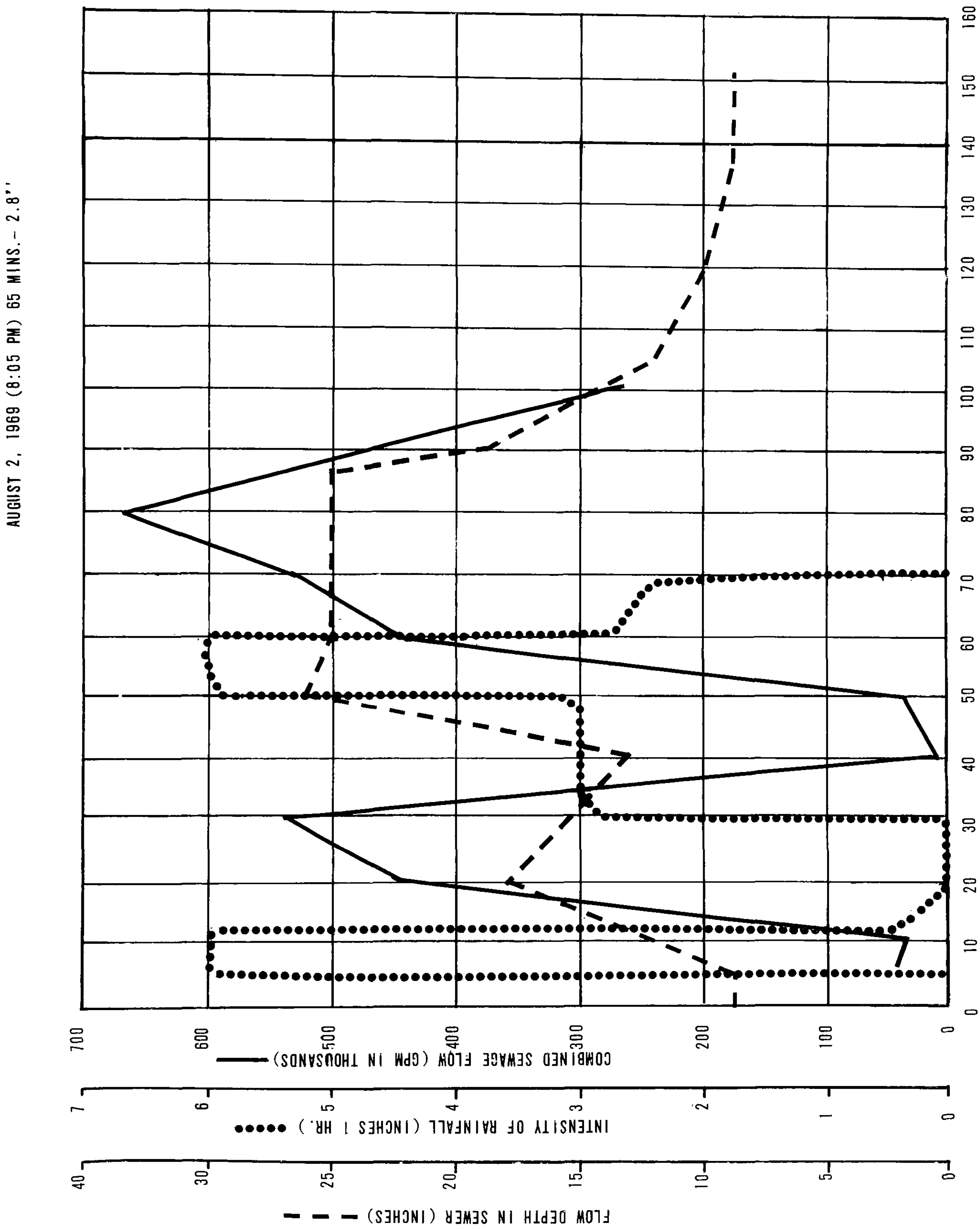
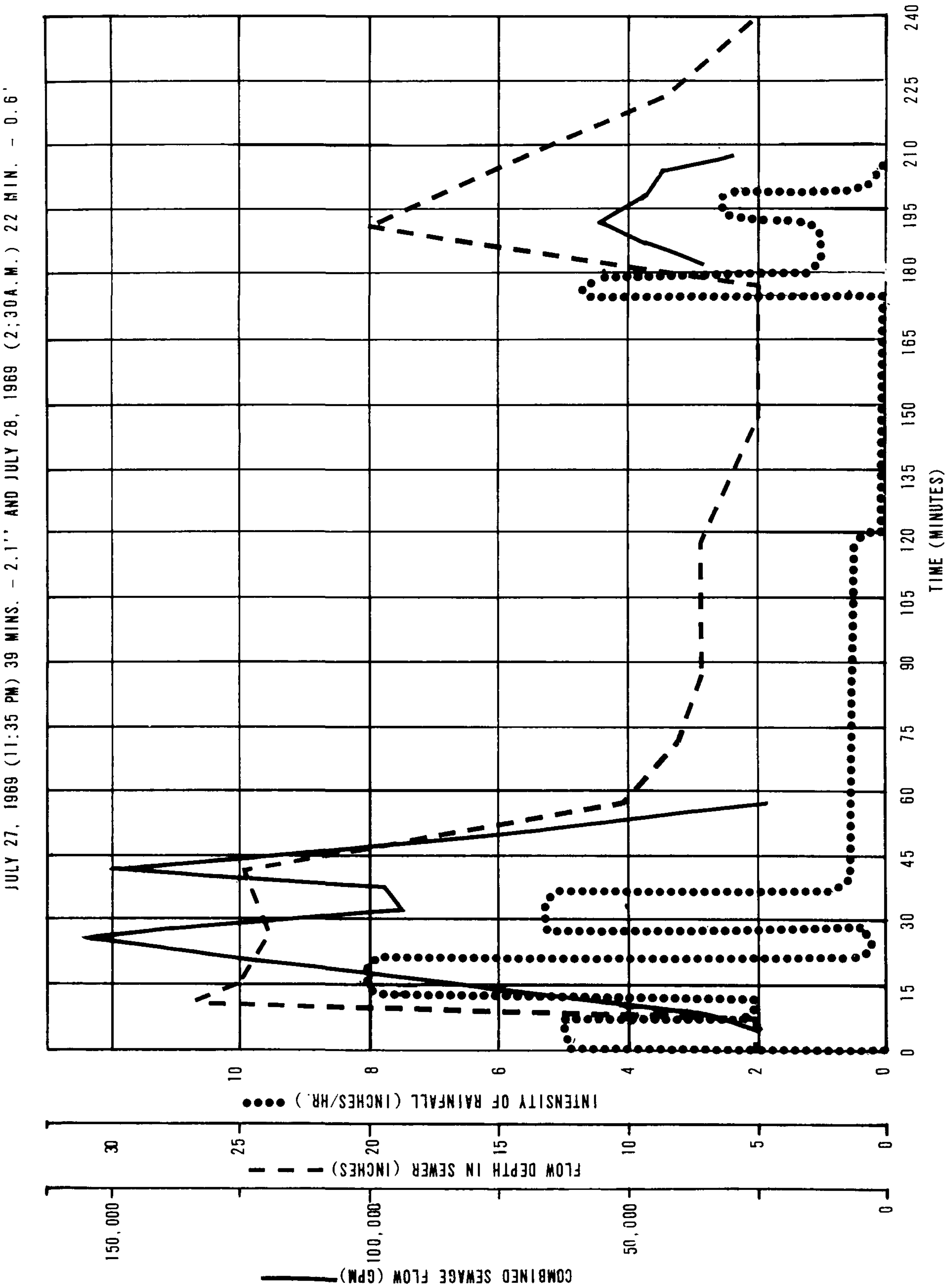


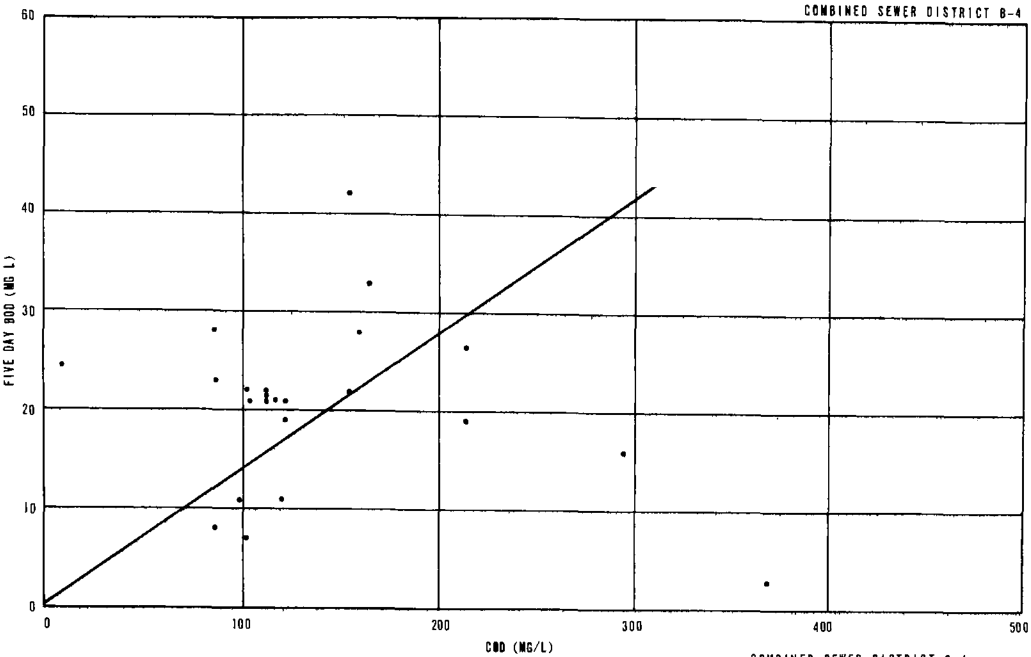
FIGURE E-27

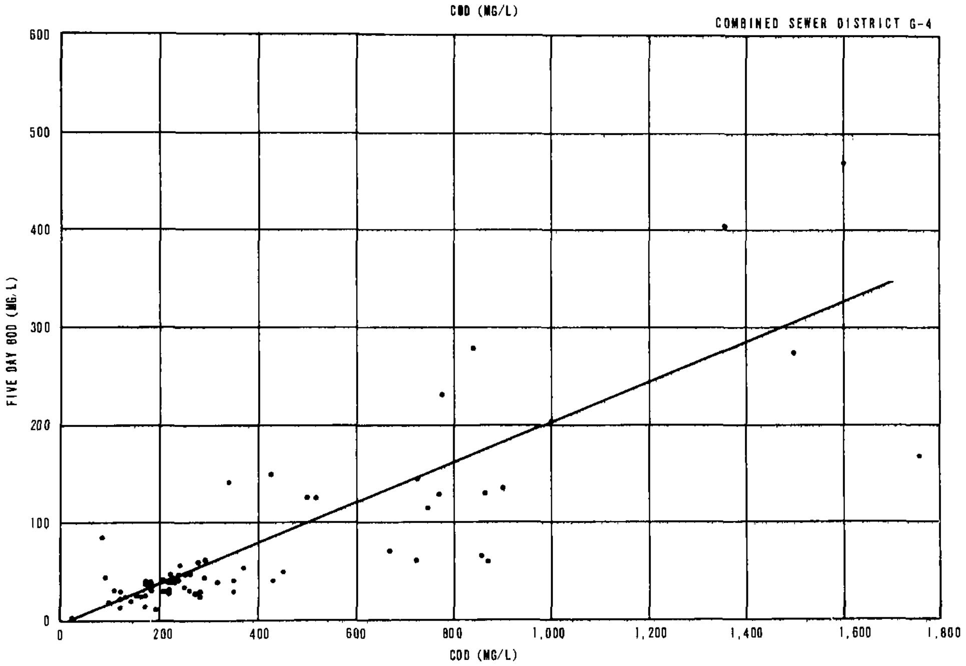
REPRESENTATIVE RAINFALL AND RUNOFF MEASUREMENTS
FOR CONSECUTIVE STORMS IN COMBINED SEWER DISTRICT G-4

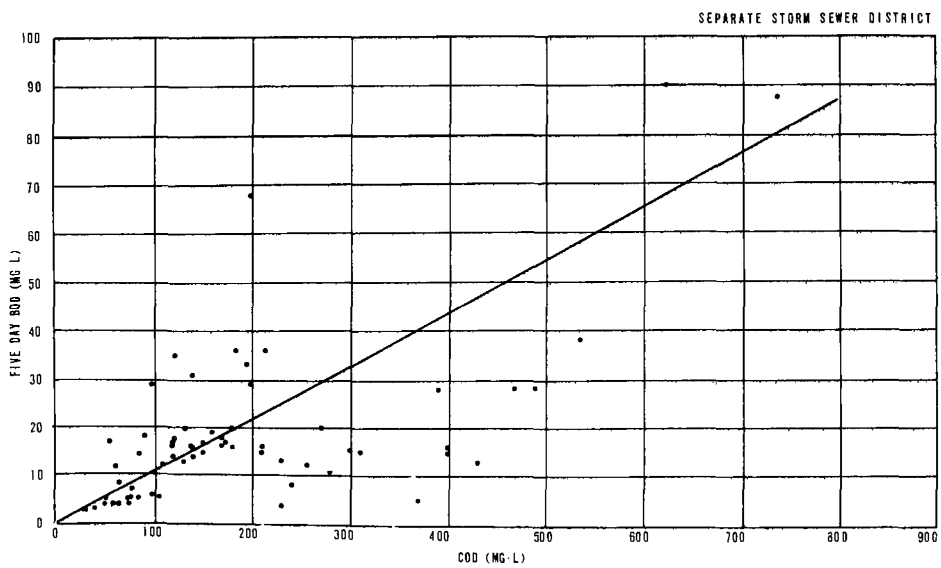


E-27

FIGURE E-28
CORRELATION BETWEEN BOD AND COD



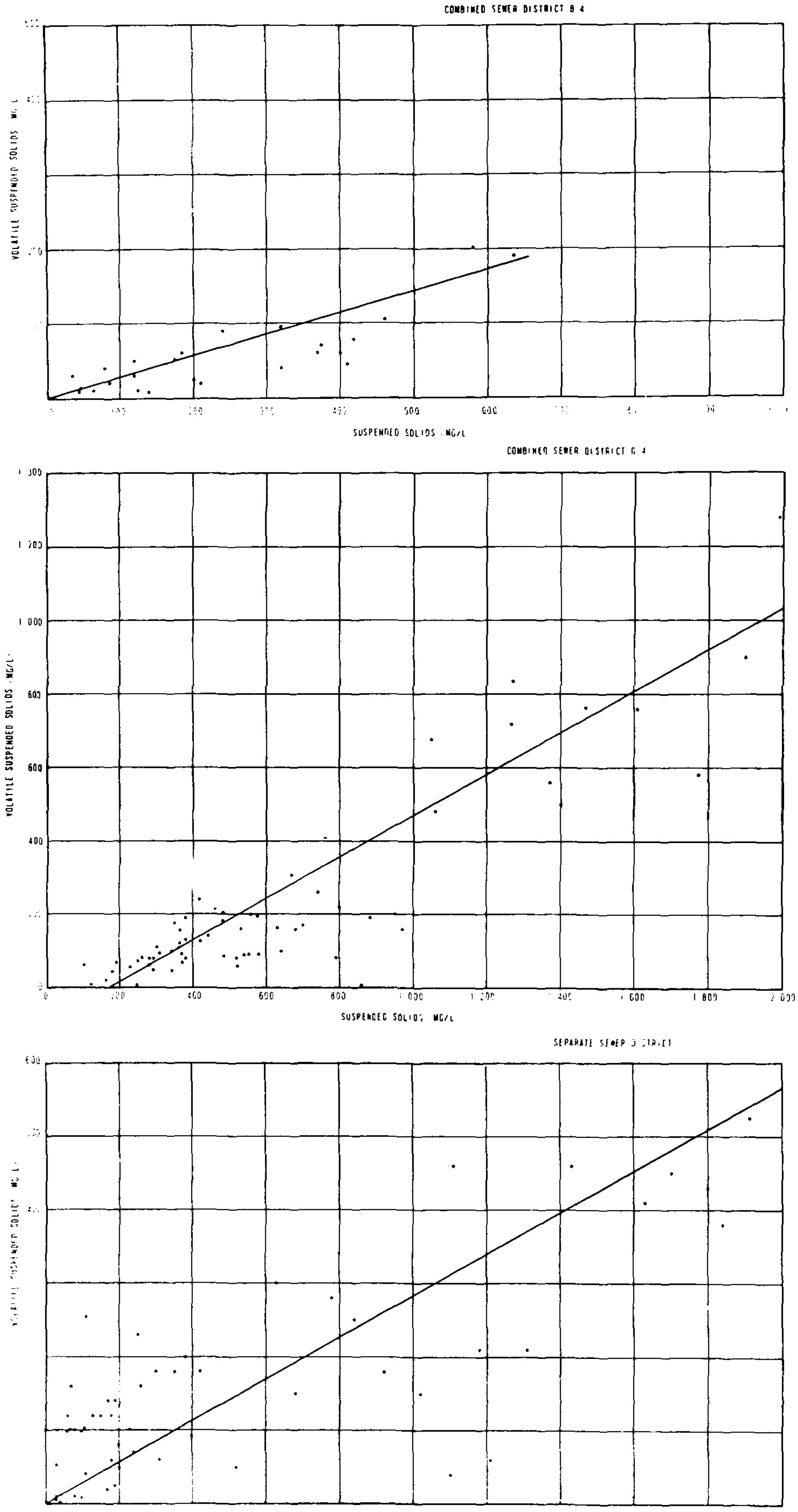




NOTE: LINES OF BEST FIT

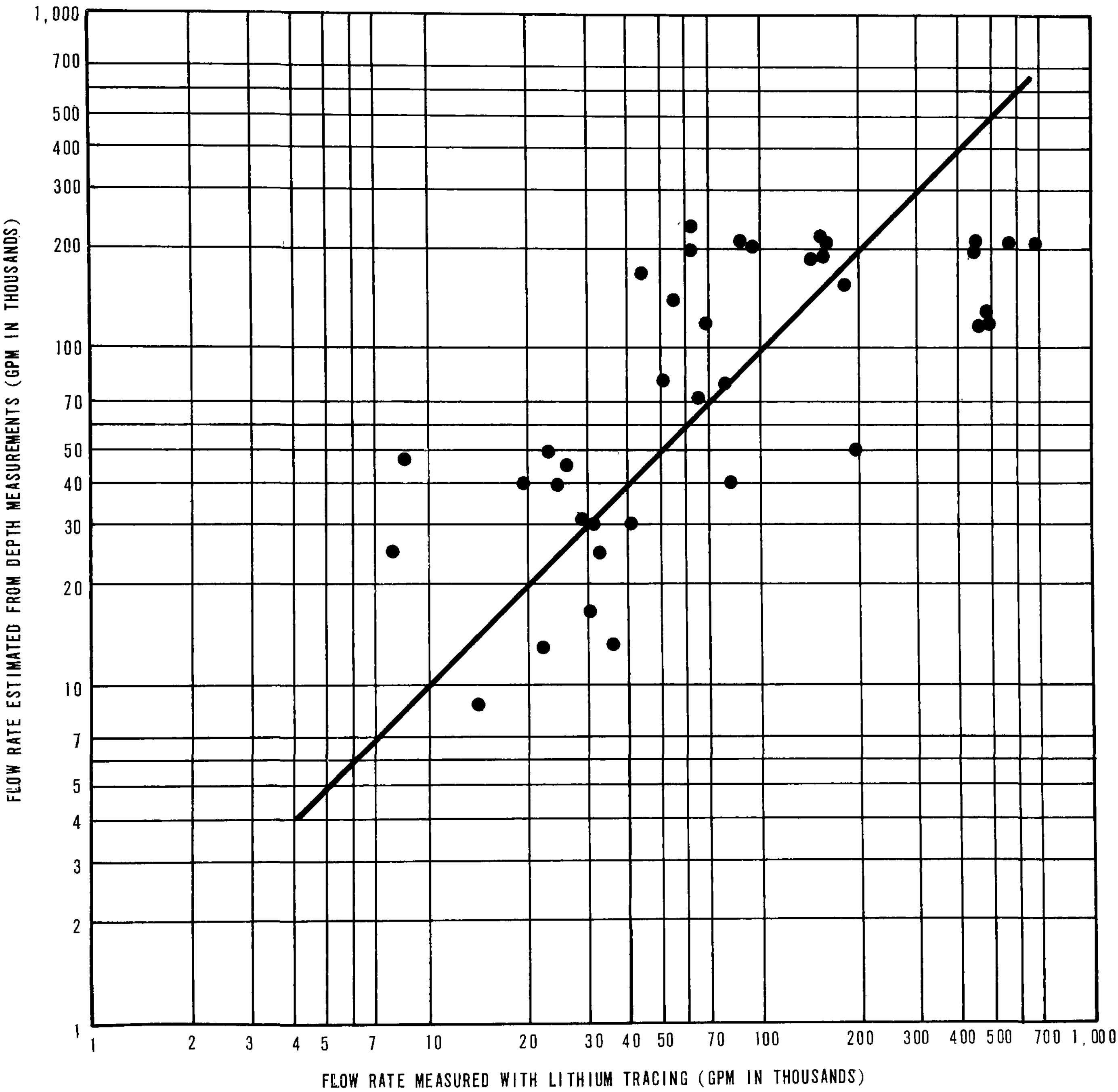
FIGURE E-29

CORRELATION BETWEEN SUSPENDED SOLIDS AND VOLATILE SUSPENDED SOLIDS



NOTE: LINES OF BEST FIT

FIGURE E-30
COMPARISON OF FLOW MEASUREMENT TECHNIQUES FOR COMBINED SEWER OVERFLOWS



E-30

APPENDIX F

ULTRA-HIGH-RATE FILTRATION

Introduction

High-rate filtration has been extensively applied for the removal of suspended impuritites from raw water or wastewater, especially when the impurities are primarily non-volatile discrete particulates, such as the wastewater from steel mills. In combined sewer overflow and separated sewer discharge, large fractions of the suspended waste constituents are recognized to be non-volatile discrete solids; thus, high-rate filtration may be an effective treatment method. Despite the many studies that have been undertaken, the status of filtration development is still in transition from an art to a science. The practical design parameters for the filtration process to treat a specific wastewater must still be determined from results of specific laboratory or pilot-scale investigations. Moreover, ultra-high-rates of filtration (greater than 15 gpm per square foot) must be applied in order to cope economically with the unique hydraulic characteristics of the combined sewer overflow or separated sewer discharge--high discharges within a short-time period. This adds another dimension of uncertainty to the development of a feasible filtration process to treat the excess urban wastewater derived from intense storms. Therefore, a filtration study was conducted with the following objectives:

1. To evaluate the applicability of ultra-high-rate filtration to the treatment of combined sewer overflows,

2. To determine the flocculation effects of chemical additives on the solids and organic material removal and

3. To provide a conceptual design basis for pilot-scale or full-scale treatment units.

The principal process variables evaluated in the laboratory program were:

1. Filter media, including type, depth, size and arrangement;
2. Filtration rates;
3. Effects of addition of flocculants and flocculant aids;
4. Variation of solids concentration in the wastewater;
5. Backwash rate and quantity;
6. Air-scouring rate, duration, and sequence in the backwash procedure;
7. Effluent quality characteristics, including suspended solids, COD, total five-day BOD, and soluble five-day BOD. (Soluble BOD_5 was measured by performing BOD_5 analyses upon the filtrate produced by vacuum filtration of the wastewater sample passed through a diatomaceous earth filter);
8. Length of filter run; and
9. Head loss requirements.

Theoretical Background

The basis for this discussion of technical background was a comprehensive review of the technical literature on removal mechanisms in the filtration process, analysis of filter performance, and considerations for practical filter applications.

Particle Removal Mechanisms

The possible mechanisms for the removal of particulate material from water by filtration through porous media may be categorized into two types: physical and chemical removal mechanisms.

Physical Removal

Physical removal mechanisms include: 1) straining, 2) sedimentation, 3) inertial impingement and centrifugal collection, 4) Brownian movement, 5) physical contact caused by the convergence of fluid streamlines, and 6) diffusion of particulate materials. They are all dependent in varying degrees on different physical and operation variables such as size of the media, size of the particulates, filtration rate, temperature, etc.

Straining: Straining (or direct sieving) has been considered as the primary mechanism to remove suspended solids in the traditional sand-filtration process. The probability of removal for suspended solids by straining, P_s, was defined as:

$$P_s \sim \frac{D}{d}^{\frac{3}{2}}$$

in which D and d are, respectively, the diameters of the suspended particle and the sand.

Sedimentation: The ability of the void spaces in a rapid sand filter to act as settling basins was first postulated by Hazen. Stanley found that the sand filter can be expected to remove particulates which are one-twentieth the diameter of the particles removed by a settling process at the same hydraulic loading. It has also been found that the removal increases with the square of the particulate size and the difference in density between the particle and the fluid. Increases in filtration rate and the fluid viscosity may impair the particle removal rate. Particle growth by flocculation within the pores of a filter increases the effectiveness of the straining and sedimentation removal mechanisms. The mean velocity gradient affects the rate of flocculation in the filter and the total amount of flocculation produced is proportional to the product of mean velocity gradient and average detention time.

Inertial Impingement: As the suspension flows around the filter media with continual changes of direction, the momentum associated with the inertial force may cause the particles to impinge on the surface of the filter media. Chen describes the probability of impingement for the air flow through a fibrous medium as:

$$P_i \sim \frac{\rho D^2 v}{fd}$$

where:

P_i is the probability of impingement,
D is the diameter of the particle,
ρ is the density of the particle,
v is the particle velocity,
f is the fluid viscosity, and
d is the diameter of the filter medium fibers.

Brownian Movement: This removal mechanism is not too significant in rapid sand filtration. It would provide the contact between the particulates and the surface of the filter media, but would have little effect for the particles larger than two microns, based on theoretical calculations.

Physical Contact: Physical contact between the particles and the surface of the filter media will be generated as a result of convergence of the fluid streamlines. Stein proposed the following relationship:

$$P_c \sim \frac{D^2}{d^3}$$

where:

P_c is the probability of removal of a suspended particle,
D is the diameter of the particle, and
d is the diameter of the filter medium grains.

Diffusion: The diffusion of suspended particles into the "dead spaces" of the filter medium was proposed as a removal mechanism by Hunter and Alexander. The dead spaces are those regions where the fluid flow is essentially zero. The colloidal particles diffuse across the stream-line driven by the particle concentration gradient. Because of the tendency to migrate to the regions of low shear, the colloidal particle concentration in the dead space may become considerably higher than that in the ambient fluid.

Chemical Removal

Review of filter performance in the treatment of uncoagulated and coagulated suspensions indicated that the physical removal mechanisms are inadequate to expain the entire filtration process. Thus, chemical removal mechanisms were postulated. Basically, these all stem from the electric charges on the surface of particles.

All the electrostatic charges on the surface of the particles in water are derived from one or more of the following sources: 1) ionization of molecules at the particle surface, 2) imperfections of the crystal lattice, 3) direct chemical reaction with specific ions in the water, and 4) physical adsorption of ions from the water solution. The principal chemical removal mechanisms are discussed in the following paragraphs.

Electrokinetic Effects: From observations of suspension penetration in filter beds, different investigators have concluded that the electrokinetic forces were the primary removal mechanism for charged particles. Additional support for this conclusion was found from filter bed examinations which showed the floc had no preference for horizontal surfaces or pore interstices. Using positively pre-charged media, Hunter and Alexander were able to improve the filter efficiency for negatively-charged clay particles. O'Melia and Crapps found that the floc suspensions with zeta potential ranging from low negative to high positive would allow little bed penetration in sand filters; they also suggested that rapid sand filtration could be chemically controlled through the influence of specific chemicals on the adsorption of particulates to the filter media.

Van der Waal Forces: These are molecular cohesive forces between particles. The intensity increases drastically as the particles approach each other. Between two atoms, these forces are proportional to r^{-7}, where r is the distance between the two atoms. For large multi-atom particles, the Van der Waal forces are proportional to r^{-3}. Mackrles contended that these attractive forces were not affected by the electrokinetic nature of the particles but depended almost completely on their density.

Mathematical Relationships

Many different mathematical models have been developed to describe the behavior of filtration through granular media for the removal of suspended solids from water. Iwasaki first formulated the filtration phenomenon in terms of a first-order equation as:

$$\frac{\partial c}{\partial l} = - \lambda c \qquad \text{(F-1)}$$

where:

c is concentration of suspended solids,
l is depth of filter layer from top surface, and
λ is impediment modulus, or filter coefficient.

Iwasaki suggested another relationship to express the removal of suspended solids as the increase of deposition onto each layer:

$$\frac{\partial c}{\partial l} = - \frac{1}{v} \frac{\partial \sigma}{\partial t} \qquad \text{(F-2)}$$

where:

σ is the specific deposit on the filter,
t is the filtration time, and
v is filtration rate.

These two equations have been widely used in filtration studies for many years. Although other researchers have applied these equations to evaluate filtration behavior, no basic modifications have been made.

The filter coefficient, λ, varies with the filtration time, t, due to progressive clogging of the filter pores, and may be defined by the specific deposit, σ, as suggested by Iwasaki:

$$\lambda = \lambda_o + k\sigma \qquad \text{(F-3)}$$

where:

λ is initial filter coefficient, and
k^o is constant.

Ives later suggested another equation to describe the variation of the filter coefficient, λ, with the specific deposit, σ, as

$$\lambda = \lambda_o + k\sigma - \frac{\phi\sigma^2}{p_o - \sigma} \tag{F-4}$$

where:

P_o is the porosity of clean bed, and
ϕ is a filtration parameter.

In Inves' formula, the increase of the interstitial velocity and the reduction of available grain surface area near the end of filter run were considered.

Sholji generalized the Ives equation based on his theoretical and experimental analyses and expressed the filter coefficient in terms of different operation variables as:

$$\lambda \sim \frac{1}{v^{a_1} d_o^{b_1} \mu^{c_1}} \tag{F-5}$$

where:

v	is	approach velocity of filtration,
d_o	is	geometric mean size of media,
μ	is	dynamic viscosity of influent fluid, and
a_1,b_1,c_1,	are	constants.

However, Fox and Cleasby found that Ives' equation could not adequately describe the behavior of filtration of hydrous ferric floc suspensions, but they did verify the initial linear relationship of λ versus t. The failure to fit the experimental data with Ives' equation could be attributed to the electric potential associated with the hydrous floc, which was not considered in the derivation of the equation. Recently, Deb suggested a new set of filtration equations which have incorporated the unsteady component of local variation of suspension concentrations with time into the non-dimensional forms. Specifically,

$$\frac{\partial \underline{c}}{\partial \underline{l}} + (p_o - \sigma)\frac{\partial \underline{c}}{\partial \underline{l}} + \frac{1 - p_s}{c_o}\frac{\partial \sigma}{\partial \underline{t}} = 0 \tag{F-6}$$

$$\frac{1 - p_s}{c_o(p_o - \sigma)}\frac{\partial \sigma}{\partial \underline{t}} = \lambda \underline{c} \tag{F-7}$$

where:

$$\underline{c} = \frac{c}{c_o}\text{, dimensionless concentration ratio} \tag{F-8}$$

$$\underline{t} = \frac{v}{L}\,t\text{, dimensionless time value} \tag{F-9}$$

$$\underline{l} = \frac{1}{L}\text{, dimensionless depth} \tag{F-10}$$

c_o is initial concentration of suspended solids,

L is total depth of filter bed, and

p_s is porosity of deposited material.

The filtration behavior of unsized Fuller's earth could be defined satisfactorily. However, the application of this model to the removal of flocculated material was not considered.

Considering the probabilistic nature of particle transport and attachment to a granular medium, Hsiung proposed a new filter performance prediction theory based on a "Random Walk" analogy. If P represents the probability of penetration of a unit of filter depth by a particle, the probability of a particle moving down to depth l at time t will be:

$$P\,(l\!: vt, p) = \binom{vt}{1}\, p^l\, q^{(vt\,-\,l)} \tag{F-11}$$

The total number of particles deposited between l and l+ 1 can be represented by:

$$\Delta m \sim P\,(l\!: vt, p)\; Q\, c_o\; \Delta t \tag{F-12}$$

Then, from equation (F-2), the following can be derived:

$$\frac{\partial c}{\partial l}\; v\,\Delta t \sim P\,(l\!: vt, p)\; v\, c_o\, \Delta t \tag{F-13}$$

$$\frac{1}{c_o}\;\frac{\partial c}{\partial l} \sim P(l\!: vt, p)$$

Equation (F-l3) indicates that the particle removal per unit depth of filter could be characterized by a probability function. Based on the Chi-square distribution, Hsiung proposed a deposit index, U, to translate filtration performance data for practical design. This probability theory application permits description of the random nature of the suspension, of the filter media, and of the transport and removal mechanisms (including both physical and chemical mechanisms) in a relatively practical and simple formula.

Considerations For Practical Application

Filtration efficiency is a function of many design variables: filtration rate, media size, filter depth, and the properties of the suspension. With the use of the deposit index, U, in the "random walk" analogy, Hsiumg also studied the relationship between different design varables for high-rate filtration, and found that any increase in filtration rate must be accompanied by a decrease of grain size in order to produce the same effluent quality. On the other hand, he also pointed out that the grain size of the meia should be increased in order to provide comparable filtration results for the same influent at a higher filtration rate at the same allowable head loss. Two types of performance curves were proposed for evaluation of various ranges of grain size, flow rate, and influent suspended solids concentration; one based on required effluent quality and other on terminal head loss.

For the multi-media filter, Conley and Hsiumg showed the application of the random walk analogy by using an equivalent grain size, d_e, as:

$$d_e = x_1\, d_1 + x_2\, d_2 + x_3\, d_3 \qquad \text{(F-14)}$$

where:

x_1, x_2, and x_3 are the percentages by volume of individual media, and

d_1, d_2, and d_3 are the mean grain size of individual media, respectively.

Hudson and Hsuing suggested the following relationship for head loss and filtration time:

$$H_t - H_o \sim v^{1.5}\, c_o^{\,1.4}\, \frac{L^{0.5} t}{d^{1.9}} \qquad \text{(F-15)}$$

where:

H_t is head loss at filtration time t, and
H_o is head loss at beginning of filtration through a clean bed.

Thus, the head requirement will increase with filtration rate and initial suspension concentration very rapidly. According to the relationship between the depth of filter, L, and the lumped data of G ($G = v^{0.29}\, d^{0.62} t$) it is suggested that, for the same filtration efficiency and same length of filter run, the filter bed depth requirement increases with the filtration rate.

Therefore, in order to facilitate ultra-high-rates of filtration, ($v \geqslant 15$ gpm/sq.ft.), the filter bed must be deep, the total head available must be high, and the size of the filter media must not be too small. However, for an influent containing a high solids concentration, such as combined sewage, the grain size of the media cannot afford to be too large either.

In various types of filter applications, the arrangement of media is still an art. Oeben et al found that the reverse-graded filter will permit decreasing the head requirement or increasing the filter run. Upflow with deep depth of sand has been applied to facilitate utilization of the entire filter bed. Mixed-media filters were reported as being able to provide longer filter runs or better effluent quality than regular sand filters. Filter media made of fiberglass were also reported to have potential application for ultra-high-rate filtration when the suspended solids concentration is high in the influent.

The backwash after each filter run has gradually attracted the attention of various researchers. In general, it was found that the air scouring would improve the backwash efficiency and reduce the backwash water requirements. The air may be introduced at the same time as the low-rate backwash water, but generally the air scouring is applied in alternation with water at 3-5 cfm for 5 minutes every cycle.

For the treatment of steel mill industrial wastewater, ultra-high-rate filtration has been applied. The filtration rate could be as high as 20 gpm/sq.ft. with influent solids concentration of 50 mg/L. Some deep-bed pressure filter applications for steel mill wastewater treatment reported similar results.

Description of Laboratory System

Three independent laboratory filtration systems, identical except for the filter media, were constructed for this program. As shown on the schematic diagram (Figure F-1), the principal components were the filter column, storage tank, transmission facilities, flocculant supply system, and various controls and safeguards. Photographs of these principal components are shown in Figure F-2.

Filter Column

A filter column was required with associated instrumentation to monitor and/or control filtration rate, operating pressure, total head loss, and fluid temperature. The column had to have adequate strength to withstand elevated pressures, adequate depth for deep bed filtration, and easy disassembly for the purpose of changing or modifying the filter bed.

Initially, each system included a glass column as described below (a second column was subsequently provided for the fiberglass system when a new form of fiberglass medium was introduced). The column is a 9-foot, jointed, glass pipe with a four-inch inside diameter (Figure F-1). Enclosed at the top by a one-inch thick PVC plate, the column will withstand internal pressures up to 35 psig. The underdrain also consists of a one-inch thick PVC plate, perforated with 30 to 40 evenly-distributed 1/2- and 1/4-inch holes. Mechanical joints consisting of cast iron flanges and Teflon gaskets are located 60, 74, and 86 inches above the underdrain, connecting the four sections of conical pipe. The underdrain is attached to the bottom section of the column so that one or more sections may be removed from the frame, retaining the media while making it accessible for purposes of replacement or modification. The maximum available depth for a fluidized granular bed during backwash is 92 inches.

The top section of the column is a cross tee, with 2-inch diameter branches. When the column is operated as a downflow filter, the wastewater enters through one of the branches and exits through the underdrain. The other branch of the cross accommodates the backwash effluent, as well as filter effluent when the apparatus is used for upflow operation.

Midway through the study, when a new form of fiberglass medium was introduced, a special filter column, constructed by Owens-Corning Fiberglas Corporation, was installed in the system to accommodate the new fiberglas-reinforced-plastic (FRP) cartridges. This new column is constructed of plexiglass in five sections. Each end section contains a splash plate, a perforated baffle and 1/2-inch pipe connections. The three intermediate sections each can accommodate cartridges of 12 to 24 inches in depth. The total available depth for the fiberglass medium is 60 inches, and no space is required for backwash expansion. Taps located at each end of each section of the column permit multi-depth sampling and measurement of head loss distribution with depth. Fabrication of the column is such that it may be inserted in place of the glass column without disruption or major alteration of the appurtenant piping. Consequently, the appurtenant piping and instrumentation are the same for both types of columns.

Filtration rate is controlled by a constant-pressure differential flow controller located on the effluent line and capable of maintaining a constant flow as long as there is a constant pressure on the downstream side of the controller. Flow can be regulated and automatically controlled between 0.4 and 4.4 gallons per minute, with a minimum pressure differential of 6 psi across the controller. Filtration rate and backwash rate are measured by a rotameter located in-line between the filter and the flow controller.

Pressure taps are located at the top of the filter and at the underdrain, and a precision pressure gage is connected to either end (depending upon whether the filter is operated downflow or upflow) to monitor influent pressure.

Head loss across the filter bed is measured directly with a mercury manometer having an upper limit of 66 inches of mercury. The temperature of the wastewater in the glass filter column is measured with a dial-type thermometer located 67 inches above the underdrain in a tee-section of the filter column.

The filter bed is backwashed with tap water supplied by a connection below the underdrain. Air scouring of the bed may be used in the backwash process. The air supply is also connected to a tap located below the underdrain. Distribution of the air bubbles is accomplished by the underdrain and by the supporting gravel layer of granular beds. Air pressure may be regulated at any level to 35 psig. An air relief valve is connected to the top of the filter to permit evacuation of accumulated air in the filter column prior to bringing the filter up to operating pressure. Filtered water samples are collected from the sampling valve downstream of the flow controller.

Storage Tank

Storage of sufficient capacity was required to provide for the maximum anticipated volume of wastewater which the system may process in a single filter run.

The storage tank has a gross capacity of 1,120 gallons and is capable of sustaining a filtration rate of I5 gpm per square foot for 12 hours, or a filtration rate of 50 gpm per square foot for 3-1/2 hours. The fiberglass-lined wooden tank is 5 feet square and 7 feet deep, and is enclosed on top by wooden hatches and insulated on the sides and bottom by 5 inches of fiberglass insulation. A thermostat-controlled 2,000-watt immersion heater compensates for heat loss to maintain the desired water temperature. The wastewater is continuously mixed by an electric, propeller-type mixer suspended over the center of the tank.

Transmission Facilities

Transmission facilities between the storage tank and the filter were required to deliver the wastewater over the desired ranges of flow and pressure without materially affecting its quality.

Wastewater is transmitted by a centrifugal pump from the storage tank through a 1/2-inch PVC pipe to the filter. At an operating pressure of 35 psig at the filter, the pump delivers more than 10 gpm. System pressure is controlled by a back-pressure relief valve located on the pump throttle line. All pump discharge in excess of the filtration rate is returned to the tank. The small (1/2-inch diameter) transmission line was chosen to minimize the settling of solids without excessive head loss due to friction.

Flocculant Supply

Separate systems were required for the simultaneous injection of the flocculant and flocculant aid in the transmission line so as to avoid contact between the chemicals before mixing with the wastewater.

Each injection system consists of a metering pump which feeds a PVC injection nozzle strategically located in the transmission line near the filter (Figure F-1). Mixing is achieved by three mechanisms: turbulence induced by the projection of the nozzle into the flow of the wastewater, turbulence occurring at three to six elbows between the injection point and the filter, and longitudinal mixing in non-laminar pipe flow.

System Controls and Safeguards

Coordination and protection of the filtration system is provided by the devices listed below.

Safeguards

Pressure regulator ahead of filter.
Low water-level auto-shutoff for tank heater and mixer, with alarm and warning light.

Monitors

Tank water-level indicator.
Pressure gauges on filter and backwash air and water lines.
Flow meters for air and water.
Thermometer on filter column.

Controls

Back-pressure relief valve on transmission line.
Flow controller.
Pressure Regulators for air and backwash water.
Theremostat on tank heater.
Purgemeter with needle valve for air scour.

Discussion of Laboratory Procedures and Data

Outline of Laboratory Investigation

The purpose of the laboratory investigation was to evaluate the technical feasibility of ultra-high-rate filtration for the treatment of combined sewer overflow. Minimum performance levels could not be prescribed, because the state of the art would not support such assumptions with any degree of confidence. Consequently, a more generalized approach was applied to the investigation.

Technical feasibility depends primarily on three broad conditions: 1) filtration capacity, 2) effluent quality, 3) operating efficiency. These conditions were evaluated in terms of filtration rate and duration of filter run; removal of BOD_5, COD, and suspended solids; and quantity of filtrate required for backwashing.

Three types of filters were operated at rates of 15 or more gallons per minute per square foot, utilizing a simulated wastewater and both inorganic and polyelectrolyte flocculants. Three filters, with different types of filter media and filter-bed arrangements, were evaluated in this study:

1. A newly-developed fiberglass filter medium previously untested in application to high-rate filtration of combined sewer overflow, provided by Owens-Corning Fiberglas Corporation.

2. A deep-bed tri-media filter specially designed for this application, provided by Neptune-Microfloc Corporation.

3. A garnet bed operated as an upflow filter.

Alum was used as the inorganic flocculant and Rohm & Haas Primafloc C5 as the cationic polyelectrolyte flocculant.

Filter runs were also conducted with excess activated sludge added both to use as a flocculant and to exploit its biosorptive capacity in an attempt to effect significant removal of soluble BOD.

The study was conducted within a two-phase framework. The first phase consisted of a series of runs to determine a suitable flocculant for each filter. Separate runs were made with:

1. Plain wastewater (no flocculant dosage).
2. Wastewater plus alum.
3. Wastewater plus alum and Rohm & Haas Primafloc C5.
4. Wastewater plus activated sludge.

These runs were performed at the lower end of the range of filtration rates to be covered, i.e., at 10 and 15 gallons per minute per square foot (gpm/sq.ft.).

The second phase of the study consisted of a series of runs designed to evaluate the performance of each filter over various ranges of filtration rate and suspended solids concentration. In each case, the flocculant used was the one found to be most suitable to each filter as determined in the first phase of the study. The use of air scouring and agitation was evaluated as an aid to backwashing of clogged filters in an attempt to economize on backwash requirements.

Laboratory System Design and Operation

Process and System Parameters

Wastewater characteristics which were controlled and/or observed were: size and concentration of suspended solids, BOD_5 and COD concentrations, and temperature. The operating variables were: filtration rate, pressure, backwash rate and quantity, air-scouring rate and duration, and flocculant dosage. The design variables included the type, depth, size, and arrangement of filter media.

Three identical independent filtration systems were constructed at laboratory scale, as shown in the schematic flow diagram, Figure F-1. The description in the preceding section of this appendix is typical of each system exclusive of the filter media.

Characteristics of Wastewater Feed

The combined sewer overflow used in this study was a simulated combined sewer wastewater, composed of diluted raw domestic sewage and silt. A wastewater with suspended solids and BOD_5 concentrations of 400 mg/L and 40 mg/L, respectively, was desired for the purpose of uniformity in the flocculant evaluation and for some other comparisons. The intended procedure for synthesis of the waste was to dilute the domestic sewage with fresh water to attain the desired BOD_5 concentration and then to add a slurry of silt to the diluted sewage to bring the suspended solids concentration to the desired level. However, the character of the domestic sewage was too variable to permit determination of an appropriate dilution ratio; therefore, the wastewater formulation became largely a matter of adjusting the suspended solids concentration.

Soil was oven-dried at 250°C and then pulverized. A slurry of the resulting silt was prepared at a concentration of 100 grams per liter. After thorough mixing (at least 10 minutes) the slurry was allowed to settle for one minute. The supernatnat was then used (at a suspended solids concentration of 30,000 mg/L) to adjust the suspended solids concentration of the simulated wastewater.

Sewage was delivered to the laboratory by truck from a local sewage treatment plant. Several loads were anaerobic upon delivery, and on at least one occasion, the pH was a high as 10 (pH affects flocculant action and the attachment of solids to the filter medium, the fiberglass being especially sensitive to pH variation).

Large particles in the sewage were removed during the formulation of the wastewater, to prevent subsequent clogging of pumps, transmission lines, regulators, and valves. Most of this solid matter consisted of gravel and grease which was retained in a residual sludge in the delivery truck. Attempts to flush this material from the tank truck before loading with sewage at the treatment plant were unsuccessful. The problem was resolved by pumping the first 50-100 gallons from the truck into 55-gallon drums. The sewage was passed through a wire cloth-lined basket to remove the large but less dense solids. The effect of this screening procedure was basically the same as that of a grit chamber except that large buoyant solids were also removed.

Substantially different suspended solids and BOD_5 concentrations were encountered when the three storage tanks were filled one at a time from the truck. This difference was attributed to the density stratification of solids in the tank truck during transport. It was therefore necessary to fill all three tanks simultaneously so as to obtain uniform wastewater characteristics. This was achieved by pumping from the truck into a small dosing tank which drained into all three storage tanks.

Filter Media

Fiberglass Filter--Two basic forms of fiberglass were utilized in this study: fiberglass plugs and fiberglass-reinforced-plastic (FRP) cartridges. Several configurations of each form, differing in depth, density, fiber diameter, fiber orientation, density stratification, and combination with granular media, were tested.

The fiberglass plugs were cut from laminated fiberglass boards, with the fibers sharing a common alignment (either parallel or perpendicular to the direction of fluid flow). Plugs with parallel fiber orientation were 3" long and slightly more than 4" in diameter so that they fit tightly inside the 4" glass column. Insertion of the plugs inside the column required approximately 5 percent compression of the medium Plugs with perpendicular fiber orientation were actually disc-shaped, having a 4" diameter and a 5/8" thickness. The disc were used for one run only and were found to be unsuitable because of high resistance to flow.

The first seven runs were performed on beds consisting of plugs in layers of two densitites. The upper layer in all cases had a density of 5 lbs. per cu.ft., although the layer thickness and fiber diameter varied among the runs. The lower layer, also varying in thickness and fiber orientation, was in all cases 10 lbs. per cu.ft. The compositon of the bed for each test run is presented in Table F-1.

Each of the first five runs was attended by a media-collapse phenomenon, wherein the individual plugs were observed to collapse inward and to separate from the glass wall of the filter column. At the termination of each run, the collapsed plugs were no longer suitable for use, because backwashing did not restore the original structure of the plug. Two subsequent runs were performed on beds 3" shallower than those of the earlier runs. The collapse phenomenon was not evident in these runs, but recycling on the same medium was not attempted. On several occasions (including Runs 6 and 7), the plugs were displaced by the backwash water.

Runs 8 through 16 were performed on FRP cartridges, fiberglass modules consisting of an inner core of fibers bonded to a rigid resinous casing. This form was introduced in an attempt to circumvent the collapsing phenomenon observed with the fiberglass plugs and to achieve more effective backwashing. Two densities of fiberglass medium were tested in the FRP cartridge form.

Each cartridge had a 4"-square cross section. The 5 lbs/cu.ft. cartridges were 24" deep, and the denser medium was 10" deep.

Five cartridge configurations were tested. Three cycles (Runs 8 through 10) were made on Configuration A, which consisted of 48" of 5 lbs/cu.ft. material with .00150 fiber diameter over 10" of 10 lbs/cu.ft., .00050 material. Two attempts were made at running on Configuration B, which consisted solely of a 10" depth of 10 lbs/cu.ft. cartridge; both attempts were unsuccessful because of excessive head loss, and no effluent samples were collected nor measurements taken. Cartridge Configuration C was the first dual-media filter utilizing fiberglass. It consisted of 10" of 10 lbs/cu.ft. material, 24" of 5 lbs/cu.ft. (.00150) material, and a cover layer of 6" of coarse and 6" of fine garnet. The garnet was chosen for its high specific gravity to permit high backwash rates without necessitating special measures for preventing loss of media. The filter performance was the same as Configuration B and is not reported in the results of the study. In Configuration D, the garnet was replaced by coarse anthracite, and the performance was more satisfactory (Run 11). Configuration E was used in five cycles (Runs 12 through 16); the only distinction between Configurations D and E was in the fiber diameter of the 5 lbs/cu.ft. material. The latter contained .00110 fibers, and the former contained .00150 diameter fibers. The anthracite was contained in a 4"-diameter PVC pipe covered with a perforated plate to prevent loss of media during backwash.

The collapse problem appears to have been solved by the use of the FRP cartridges, although internal splitting of the medium may be another manifestation of the same basic circumstance.

A summary of cartridge configurations for which runs are analyzed in this report appears in Table F-2.

Tri-Media Filter--The tri-media filter consisted of an anthracite-sand-garnet bed on a 12" gravel and coarse garnet base. The depth, grain sizes, and specific gravities of the various constituents are listed in Table F-3.

The gravel and garnet were placed in 3" layers. After placing 25" of the intermediate-sand, fine-garnet mixture, the bed was backwashed and 1" of media was syphoned from the surface. Similarly, after placement of about 40" of anthracite, the bed was again backwashed and 4" of media syphoned off the surface. After an initial trial run, which lasted only 15 minutes, the anthracite depth was decreased to 30" by syphoning off the top of the surface. After Run 5, the anthracite depth was increased to 36" in an attempt to improve effluent quality. The depth was again decreased to 30" after Run 10, when the bed was repacked after a failure of the glass column.

Up-Flow Filter--The up-flow filter contained a 48" garnet bed on a 12" gravel, coarse-garnet base identical to that of tri-media filter. The garnet bed consisted basically of two grain sizes (0.707 mm and 1.19 mm) in equal volumes, except for the first run, when the bed was composed of the finer grain size only.

Results of Filter Runs

Methods and Criteria of Evaluation--The effluent quality aspect of filter performance was evaluated in terms of the percentage removal of BOD_5, COD, and suspended solids from the raw wastewater; the effects of differences in concentrations of these materials in the raw wastewater were also considered. Soluble BOD_5 was monitored for the purposes of characterizing the wastewater and evaluating the usefulness of activated sludge as a soluble BOD_5 removal agent. Raw wastewater samples were collected from the pump throttle lines at the storage tanks. Filtrate samples were taken from the sampling valve located downstream from the flow controller. Streaming current and pH were measured on a majority of the samples.

The capacity evaluation was based upon filtration rate and length of filter run. The length of run as reported herein does not necessarily refer to the actual duration of filter operation. Time designated as "T_{15}" is the time to the point at which head loss equalled 15 psi. The length of run characterized by a breakthrough of suspended solids is designated by "T_b" and is defined as the time to the midpoint of a breakthrough. The breakthrough was taken as a significant upturn in the effluent concentration from a lower stable state. This definition was preferred to the more traditional definition based on a percentage increase in concentration, because it is more characteristic of the filter and is less of a reflection of an imposed effluent criterion. The length of run is assumed to be the lesser of T_{15} and T_b. Consequently, the average effluent concentration (and percent removal) is computed from the analytical results of samples taken prior to breakthrough or head loss in excess of 15 psi. Therefore, T_b tends to be a conservative estimate of run time; backwash requirements (based on actual backwash and filtrate volumes) also tend to be conservative.

The backwash requirements were evaluated independently of the other parameters as the search for an economic backwashing procedure was carried out. Backwash requirements are expressed as percent of filtrate volume and as gallons of backwash water required per pound of solids removed from the wastewater during the run.

Fiberglass Filter Flocculant Evaluation (Runs 1, 3, 5, 6)--The performance of the fiberglass filter was not enhanced by the use of flocculants. Four filter runs at 15 gpm per sq.ft. were compared in this evaluation (Figure F-3 and F-4); Run 1 was made with plain wastewater (no flocculant was added); Run 3 with a dosage of 150 mg/L of alum injected continuously in-line; Run 5 with Primafloc C5 (4 mg/L), a cationic polyelectrolyte, used in addition to the alum; and Run 6 with 50 mg/L of activated sludge added to the wastewater.

The effluent quality of the run with plain wastewater was superior to that of each of the comparable runs. With the apparent exception of BOD_5 removal in the alum-C5 run, the removal of COD, BOD_5, and suspended solids was unexcelled by any of the runs utilizing a flocculant. However, the influent BOD_5 concentration for the plain run was suspiciously low with respect to the influent COD concentration (Table F-4). While the ratio of COD to BOD_5 concentration for the plain 1.8 to 4.0, the ratio in Run 1 was about 38. (In all of the subsequent runs the COD/BOD_5 ratio was still below 5.5 except in Run 11, where the ratio was 42.) Apparently there was an erroneous influent BOD_5 measurement in the case of Run 1 and 11. If in fact the influent BOD_5 in the plain run were of the order of 100 mg/L (giving a COD to BOD_5 ratio of 7.5), then the degree of removal would amount to 95 percent. On the basis of effluent quality, the plain and the alum-C5 runs were significantly more effective. Of the two, the plain run was considered to be better because of greater COD removal and longer run time.

Breakthrough of suspended solids was observed in each of the four runs. The time to breakthrough (T_b) was nearly 85 percent longer in the case of the alum and the activated sludge runs as compared to the 3-hour plain run. However, the 35 percent higher solids and 140 percent higher COD of the influent of the plain run undoubtedly contributed significantly to the shorter run time.

The apparent deficiency in run time is not considered to be of sufficient importance to outweigh the substantially higher effluent quality obtained from the plain run. All subsequent runs on the fiberglass filter were performed without benefit of flocculant or flocculant aid.

Efflect of Solids Concentration (Runs 8 and 14)--Two runs at 25 gpm/sq.ft., with wastewaters having high and low suspended solids concentrations are depicted in Figure F-5 in terms of head loss (Runs 8 and 14). The rate of increase in head loss was approximately 7 times greater for the run with the higher solids. The non-soluble BOD_5 was the same for each wastewater (approximately 40 mg/L), but the COD concentration was higher for the low solids run. Therefore, the higher rate of head loss is attributable to the higher solids concentration rather than to higher organic content.

Filtration Rate (Runs 15, 13, and 12)--Three filter runs at 15, 35, and 50 gpm per square foot, show a sharply reduced run time at the two higher rates (see Table F-4 and Figure F-6). The run time at the lower of the three rates was 2 hours (T_{15} = 122 minutes), while at rates of 35 and 50 gpm per square foot, the run times dropped to less than one-half hour (T_{15} = 28 and 25 minutes, respectively).

Effluent quality was lower at the higher filtration rates also, but to a lesser extent. COD removal showed the greatest reduction of the three effluent parameters evaluated. The removal of COD at 15 gpm per square foot averaged 78 percent, whereas removals at the higher filtration rates were 52 percent and 64 percent. Suspended solids removal dropped from 95 percent to 87-91 percent, and BOD_5 removal dropped from 72 percent at 15 gpm/sq.ft. to 58-68 percent at 35 and 50 gpm/sq.ft.

A significant change in filter performance occurs between 15 and 35 gpm per square foot. The performance levels at the two higher rates were very nearly the same, and the runs were very short. It should be pointed out that all three runs were performed on the same fiberglass cartridges, but in order of decreasing filtration rate. Thus, the shorter run times for the higher rates cannot be attributed to solids retained in the bed from previous runs.

Fiber Backwash--A unique feature of the fiberglass medium undergoing backwash is that the medium remains unexpanded. Consequently, the solids, wherever they may be lodged in the bed, must be forced through the medium through which they have penetrated during the filtration cycle. Unlike a fluidized granular bed, the fiberglass retains the ability to trap solids during the backwash cycle and to impede their passage, thereby limiting backwash efficiency. The solids, wherever they may be lodged in the bed, must be forced back through the medium which they have penetrated during the filtration cycle. This limitation may be partially offset by providing some mechanism for releasing solid particles so entrapped. The mechanism chosen for study was air agitation. Air was discharged through the bed, both alternating with and concurrent with backwash water, and the process was repeated until the backwash effluent remained clear at high backwash rates. The alternating discharge method was the more effective technique.

Due to the high porosity and compressibility of the fiberglass medium fiber reorientation is possible. An extreme case of such distortion occurs when fiberglass plugs collapse during filtration. The media collapse phenomenon, in all cases, was observed to occur in a progression from the upstream plug to successive plugs in a downstream direction. The plugs collapsed anisotropically inward, normal to the planes of laminations.

The probable mechanism by which collapsing occurs is basically that as the filter run progresses, a layer of solids builds up near the upstream face of the plug or directly on the upstream face, thereby creating a sharp pressure differential within the plug. Simultaneously, as wastewater seeps between the plug and the glass wall of the column, the flow acquires a radial component, and another layer of solids builds up within the plug, concentric to the column. The pressure differential then increases until failure occurs in the direction of the weakest plane. This process recurs in each successive plug.

Backwash requirements of the plug-type medium ranged from 6.5 percent to 14.5 percent of the filtrate volume for first-run beds, or 13 to 41 gallons per pound of solids retained (Table F-4). Backwashing of a second-run bed required 41 percent of the filtrate, or nearly 90 gallons per pound of solids.

Backwashing of fiberglass plugs was not effective. Although the collapsed plugs partially recovered their original shape after the run terminated, the basic weakness of the plugs remained, probably due to broken fibers. Only one recycle was attempted on plugs. Runs 3 and 4 constituted the first and second runs on one set of plugs. However, the first run was terminated on the basis of head loss before all the plugs had collapsed. When the second cycle began, the weakened plugs again collapsed almost immediately, but the remaining plugs were effective for 1-1/4 hours, at which time breakthrough occurred.

In addition to the problem of media collapse, the plug-form of fiberglass was shown to be susceptible to dislodging during backwash. This usually occurred in the denser layer of the bed, and all the plugs above it were displaced upward.

The use of FRP cartridges, to avoid the media collapse phenomenon and to enhance the washability of the bed, was largely successful. The bonding of the outer fibers to the casing apparently prevented the build-up of the forces which are presumed to have caused the plugs to collapse, and the medium also was held firmly in place during backwashing. In a few of the cartridges, however, internal splits, which were visible at the surface, developed. These less extreme fiber reorientations were relatively small, the largest opening being less than 1/4 inch wide and approximately one inch long. The depth was not measured, but obviously did not extend through the entire depth of the cartridge. The split appeared to be the result of a weakness in the fabrication of the medium

The backwash requirements for the cartridge were not very different from those for the plugs, and they ranged from 3.6 percent to 22 percent of the filtrate volume, or 11 to 43 gallons per pound of solids retained (Table F-4). However, backwash requirements decreased with successive cycles. For example, only 3.6 percent of the filtrate was required to backwash the medium after its fifth filtration cycle. This indicates an increasing degree of fiber reorientation and that increased quantities of backwash water flows through channels or "corridors" of high permeability. The degree to which the agitating air bubbles contribute to such channeling effects was not estimated, but the effectiveness of air agitation in loosening bound solids was repeatedly demonstrated.

The effectiveness of air agitation is shown in Figures F-7 and F-8. Five cycles are depicted in terms of head loss and the removal of suspended solids, BOD_5, and COD. The initial head loss increased from 3.5 psi to 7.5 psi between Cycle 1 and Cycle 5, at a filtration rate of 50 gpm per square foot. (The effect of subsequent cycles on the initial head loss was lessened somewhat by the difference in influent suspended solids concentration, since a higher concentration was associated with the earlier cycle.) The removals of suspended solids, BOD_5, and COD were not greatly affected by repeated cycles, although a slight decline in percentage removal appears to be related to the difference in influent concentrations.

Tri-Media Filter Runs--Flocculant Evaluation (Runs 1, 3, 4, 5)--The combination of alum and Primafloc C5 was found to be the most suitable of the flocculants tested in terms of both effluent quality and length of filter run. The flocculant evaluation was based upon four runs at 10 pgm/sq.ft.: Run 1 with plain synthetic wastewater, Run 3 with an alum dosage of 150 mg/L, Run 4 with the same dosage of alum plus 4 mg/L of Primafloc C5, and Run 5 with activated sludge at a concentration of 55 mg/L.

The alum-C5 run did not demonstrate the highest degree of removal of either suspended solids, BOD_5, or COD, nor did any other flocculant exhibit consistent removal superiority. As may be seen in Figure F-9 and Table F-5, the alum C5 combination was generally the second best of the flocculants with respect to effluent quality. Suspended solids removal in excess of 90 percent was achieved in the alum C5 run, and was second only to the 98 percent removal in the laum run. The BOD_5 removal of nearly 60 percent for this run was exceeded only in Run 1 (plain wastewater), for which removal averaged slightly less than 70 percent; COD removal was 40 percent. The alum-C5 combination was considered to be the best avialable alternative, largely on the basis of greater length of filter run, which was 80 to 400 percent longer than the others. Head loss of 15 psi was reached after 3-1/4 hours.

Effect of Suspended Solids Concentration (Runs 4, 6, 10)--The effect of influent suspended solids concentration on filter performance was demonstrated by Runs 4 and 10 (Figure F-10 and Table F-5), with influent solids concentrations of 410 mg/L and 2,420 mg/L respectively, and COD and BOD_5 concentrations approximately the same in both runs. The run with the lower solids concentration (No. 4) was nearly 90 percent longer than Run 10.

Run 6 also appears to show the effect of the organic content of the wastewater (Figure F-10). The wastewater used in Run 6 had a lower suspended solids concentration than either of the other runs, but had a much higher organic content--200 mg/L BOD_5 and 560 mg/L COD in Run 6, versus 25 mg/L BOD_5 and 160 mg/L COD in Runs 4 and 10. The effect of the higher organic content was a 50 percent shorter run time than that in the high-solids run (Run 10).

Filtration Rate (Run 8)--Operation of the tri-media filter at rates in excess of 10 gpm/sq.ft. proved to be unsatisfactory with respect to effluent quality and length of run. Although performance was erratic at rates above 10 gpm/sq.ft., Figure F-11 illustrates a typical performance for Run 8 at 20 gpm/sq.ft. This run lasted less than one hour, and the removal of suspended solids, BOD_5, and COD declined typically as the run progressed. The average suspended solids concentration in the filter effluent was 240 mg/L (50 percent removal), while removal of COD and BOD_5 each averaged approximately 30 percent.

Backwash Requirements--The normal backwash requirements of the tri-media filter were 12 percent to 25 percent of the filtrate, or 40 to 75 gallons per pound of solids retained (Table F-5). However, frequently more backwash was required (up to 250 gallons per pound of solids) because of two factors: 1) fibrous solids clinging to the surface of the bed during backwash, and 2) difficulties in fluidizing the bed. The fibrous solids could be removed only by expanding the bed to the overflow level. Failure of the bed to break up readily resulted in the entire sand and anthracite layers being lifted without fluidizing. The anthracite layer was particularly difficult to break up, and the problem was usually caused by the accumulation of a layer of tiny air bubbles at the sand-anthracite interface.

Upflow Filter Runs--The upflow filter was limited to a maximum filtration rate of 15 gpm per square foot, because the bed (a garnet medium) would fluidize at higher rates. Nine runs were performed at 5, 10 and 15 gpm per square foot (Table F-6).

Flocculant Evaluation--The use of flocculants did not appear to have a beneficial effect upon effluent quality at either 15 or 10 gpm per square foot. Run 1, with no flocculant addition, was terminated because of excessive head loss (15 psi) after less than 1-1/4 hours; only 60 percent removal of suspended solids was achieved.

Since a more rapid head-loss buildup was anticipated with a flocculant added, the garnet bed was modified after the first run. Fifty percent of the fine garnet medium (by volume) was replaced by a garnet sand of slightly larger grain size. This was effective in increasing the length of run, and head loss did not again reach 15 psi in any run until much later in the study when the bed depth was increased.

The second run (at 15 gpm per square foot) was made with an alum dosage of 150 mg/L. Compared with the first run, the performance was only slightly better. The percent removal of suspended solids was about the same, with the influent solids concentration about 40 percent lower and the effluent concentration correspondingly low. The BOD_5 removal was about the same and the COD removal (75 percent) was double that of the first run.

Two subsequent runs at 15 gpm per square foot were made with different dosages of alum and Primafloc C5 polyelectrolyte. In both cases, the removals of suspended solids, BOD_5, and COD were all less than one-half the corresponding removal in Run 1.

The consistently poor performance of the upflow filter at 15 gpm per square foot and the ineffectiveness of flocculant addition seemed to indicate that a lower filtration rate was appropriate. This was reinforced by the knowledge that at 15 gpm per square foot the bed was on the verge of expansion. The flocculant evaluation was repeated at a filtration rate of 10 gpm per square foot; in this series, a run with activated sludge as a flocculant was also included.

In general, the filter performance did not improve at the lower filtration rate. Runs with plain wastewater and with the combination of alum and Primafloc C5 were slightly better at 10 gpm per square foot than at 15 gpm per square foot. However, the effluent suspended solids concentration never dropped below 100 mg/L, nor did the removal exceed 80 percent. The maximum BOD_5 and COD removals were 75 percent. A final run at 5 gpm per square foot with the addition of alum was not substantially better in terms of effluent quality.

Filter performance at 5, 10, and 15 gpm per square foot is shown in Figure F-12. The runs at 15 and 5 gpm per square foot (Runs 2 and 9) were both made with an alum dosage of 150 mg/L, and the run at 10 gpm per square foot was made with no flocculant added. Runs 2 and 9 were characterized by steadily decreasing removal efficiencies; thus, the average concentrations are strongly affected by the length of run. On the other hand, Run 6, with plain wastewater, exhibited relatively constant removal efficiencies.

The reduced fluid velocity in the filter column above the garnet bed was not sufficient to keep all of the solids in suspension in the filter effluent. Consequently, the solids concentration continuously increased in this portion of the column. Filter effluent samples taken from a valve downstream from the column were not truly representative of the filter effluent. Only when the solids concentration in the column reached such a level that the overflow concentration equalled the filtrate concentration would the data be representative. Thus, the effluent concentrations were actually higher than the data indicate.

Backwashing the upflow filter was a formidable task; almost always, the bed was so bound up with solids within the gravel layer that the entire bed was lifted as a solid plug when backwashing was attempted. The most successful technique was to backwash at a low rate (less than 15 gpm per square foot) to remove fine solids from the upper portion of the bed. Air was then introduced at a pressure of 25 psig, and a drain valve at the bottom of the filter was opened with the filter under pressure, resulting in a very rapid downflow. The high velocity downflow stripped accumulated solids from the lower portion of the bed. This procedure generally had to be repeated six or more times before the bed could be backwashed by the more normal procedure.

Backwash requirements generally amounted to 13 to 18 percent of the filtrate volume. However, these figures are of limited value in that the filter runs were not terminated due to head loss or turbidity breakthrough. Effluent quality was so consistently poor that the filter runs were terminated at the convenience of the operator.

Filter Runs With Activated Sludge--One run was performed on each filter with activated sludge as a flocculant and as a potential removal agent for soluble BOD_5. (Run 6 in the fiberglass filter series, Run 5 in the tri-media series, and Run 5 in the upflow filter series.) In no case was the activated sludge determined to be the most suitable flocculant, nor did the sludge have a noticeably detrimental effect on effluent quality or length of filter run (see Tables F-4, F-5, and F-6). No significant removal of soluble BOD_5 was observed. In fact, the soluble BOD_5 decreased by only 1 mg/L on the average for each run with activated sludge. (Average reductions of zero to 30 mg/L were observed for other runs, by comparison.

Summary of Findings

The laboratory test program indicated that ultra-high-rate filtration, at rates of 15 or more gallons per minute per square foot, is a technically feasible process for the removal of suspended solids and associated non-soluble BOD from combined sewer overflow. Of the three filter systems tested, the fiberglass filters performed best, achieving at least 90 percent removal of suspended solids and 70 percent removal of non-soluble BOD_5 at filtration rates of 15-30 gpm/sq.ft. and with filter runs of 1 to 3 hours duration. The addition of flocculants and flocculant aids was not effective in improving the performance of the fiberglass filters. Comparable effluent quality was not achieved in the tri-media filter runs at filter rates above 10 gpm/sq.ft. Upflow filtration through a garnet bed was unsatisfactory, largely on the basis of poor effluent quality.

Soluble BOD removal was negligible in all three filter systems, even with the addition of activated sludge to the influent wastewater. The non-soluble organic content of the influent wastewater appeared to have a greater impact on head-loss building than did the suspended solids content.

From the results of these observations, it can be concluded that an improved effluent quality (i.e. lower concentrations of suspended solids, BOD_5 and COD) could be obtained from the fiberglass filter by:

1. increasing total bed depth
2. increasing media density in the bottom layer
3. optimizing density gradation

A multi- or graded-density fiberglass bed is needed to retain large solids and to permit the passage of smaller solids so as to make the most efficient use of pore space and avoid premature clogging at the shallower depths of the bed. Turbidity of effluent from the fiberglass filter was due to very fine particulates, and it is believed that a bottom density in excess of 15 lbs/cu.ft. could reduce effluent concentrations of suspended solids to less than 40 mg/L.

The economic feasibility of fiberglass filter process for ultra-high-rate filtration may depend on extending the useful life of the fibergalss medium beyond the limits indicated by the laboratory tests. Improvement of the backwash operations through modification of underdrain design, staged removal of backwash effluent, the use of air scouring during backwash, and development of improved fiberglass bed designs and fiberglass filter regeneration techniques appeared to be promising approaches to extension of filter life.

Table F-1

Characteristics of Fiberglass Plug Filter Beds[1]

Run No.	Upper Layer, 5 lbs./cu.ft. Layer Thickness inches	Upper Layer, 5 lbs./cu.ft. Fiber Diameter inches	Lower Layer, 10 lbs./cu.ft. Layer Thickness inches	Lower Layer, 10 lbs./cu.ft. Fiber Diameter inches	Total Depth inches
1	48	.0011	12	.0005	60
2	54	.0015	6	.0005[2]	60
3-4	54	.0011	6	.0005	60
5	54	.0011	6	.0005	60
6	51	.0011	6	.0005	57
7	51	.0011	6	.0005	57

[1] All fibers aligned parallel to flow unless otherwise noted.
[2] Fibers aligned perpendicular to flow.

Table F-2

Summary of Filter Cartridge Configurations

Configuration	Medium	Depth	Fiber Diameter or Grain Size	Density
		inches	10^{-5} inches	lbs/cu.ft.
Configuration A (Run Nos. 8-10)				
Upper Layer	----	48	150	5
Lower Layer	----	48	50	10
Configuration D (Run No. 11)				
Upper Layer	Anthracite	18	coarse	----
Middle Layer	Fiberglass	24	150	5
Lower Layer	Fiberglass	10	50	10
Configuration E (Run Nos. 12-16)				
Upper Layer	Anthracite	18	coarse	----
Middle Layer	Fiberglass	24	110	5
Lower Layer	Fiberglass	10	50	10

Table F-3

Characteristics of Media Used in the Tri-Media Filters

Material	Layer Depth inches	Grain Size	Specific Gravity
Anthracite	30-36	2.00-2.83 mm	1.6
Mixed: Sand	15	0.50-1.00 mm	2.6
Fine Garnet	9	0.35-1.00 mm	4.2
Coarse Garnet	3	1.41-4.00 mm	4.2
Fine Gravel	3	4.00-8.00 mm	2.6
Medium Gravel	3	5/16-5/8 inch	2.6
Coarse Gravel	3	1-2 inches	2.6

Table F-4

Filtration Study Data Summary
Fiberglass Filter

Run No.	1	2	3	4	5	6	7	8
Nominal Flow per Unit Area, gpm/sq.ft.	15	15	15	15	15	15	25	25
Flocculant Dosage								
Alum, mg/L	0	0	150	150	150	0	0	0
Primafloc C5, mg/L	0	0	0	0	4	0	0	0
Activated Sludge, mg/L	0	0	0	0	0	50	0	0
Suspended Solids								
Influent, mg/L	670	465	500	632	580	506	232	176
Effluent, mg/L	10	51	60	32	6	73	46	26
Percent Removal	98	89	88	95	99	84	80	85
BOD_5								
Influent, mg/L	19	58	97	62	124	110	87	105
Effluent, mg/L	5	32	34	14	6	23	58	85
Percent Removal	72	45	65	78	95	77	53	19
Soluble BOD_5								
Influent, mg/L	....	....	....	....	18	12	32	67
Effluent, mg/L	....	....	....	....	6	11	30	54
Percent Removal	....	....	....	....	65	10	3	19
COD								
Influent, mg/L	750	214	288	250	220	320	296	350
Effluent, mg/L	68	118	112	58	44	144	148	178
Percent Removal	91	45	61	77	80	55	50	49
pH								
Influent	6.8	6.7	7.1	7.0	6.8	6.8	7.2	6.8
Effluent	7.0	6.8	6.8	6.9	6.9	6.8	7.2	6.8
Length of Filter Run, minutes	187$_{b}$	149$_{b}$	346$_{b,15}$	75$_{b}$	114$_{b}$	330$_{b}$	210$_{b}$	220
Head Loss								
Initial, psi	1.4	1.4	0.9	1.7	0.4	0.5	0.8	2.2
Final, psi	7.9	8.2	15.0	9.1	8.4	9.3	6.9	15.0
Filtrate Volume (actual), gallons	299.4	248.8	463.5	173.7	217.2	510.9	590.0	544.6
Backwash (actual)								
Volume, gallons	21.1	....	30.1	71.2	14.2	73.8	....	....
Volume as Percent of Filtrate,	7.0	....	6.5	41.0	6.6	14.5	....	....
Volume per Pound of Solids Retained, gal/lb.	13.0	....	17.9	88.4	15.2	40.8	....	....
Influent Pressure, psig	24	27	25	25	25	25	24	21
Actual Run Terminated By:	Media-Collapse	Media-Collapse and Head Loss	Media-Collapse and Head Loss	Media-Collapse and Head Loss	Media-Collapse, Head Loss, and Turbidity	Turbidity (no collapse)	Shortage of Synthetic Wastewater (no collapse)	Head Loss
Cycle	....	....	1st of 2	2nd of 2	....	....	....	1st of 3
Form	Plugs	Plugs	Plugs	Plugs	Plugs	Plugs	Plugs	FRP

Run No.	9	10	11	12	13	14	15	[illegible]
Nominal Flow per Unit Area, gpm/sq.ft.	35	25	30	50	35	25	15	[illegible]
Flocculant Dosage								
Alum, mg/L	0	0	0	0	0	0	0	
Primafloc C5, mg/L	0	0	0	0	0	0	0	
Activated Sludge, mg/L	0	0	0	0	0	0	0	
Suspended Solids								
Influent, mg/L	560	242	680	1,000	760	840	888	334
Effluent, mg/L	56	24	48	90	99	42	44	40
Percent Removal	90	90	93	91	87	95	95	88
BOD_5								
Influent, mg/L	98	95	4	86	48	45	96	22
Effluent, mg/L	61	58	1	36	15	10	27	9
Percent Removal	37	39	71	58	68	77	72	58
Soluble BOD_5								
Influent, mg/L	(45)	43	0	20	13	8	11	5
Effluent, mg/L	(45)	(43)	0	15	11	7	9	5
Percent Removal	0	0	....	25	8	11	18	6
COD								
Influent, mg/L	410	400	170	343	212	243	435	91
Effluent, mg/L	287	208	60	123	102	61	78	46
Percent Removal	30	48	65	64	52	75	82	49
pH								
Influent	6.9	6.8	7.4	7.0	7.2	6.9	7.1	7.0
Effluent	7.0	6.7	7.6	7.1	7.3	7.1	7.1	7.0
Length of Filter Run, minutes	28	62	75$_{b}$	25	28	52	122	52
Head Loss								
Initial, psi	5.3	5.4	2.1	3.4	3.5	1.9	0.4	7.4
Final, psi	15.0	15.0	9.3	15.0	15.0	15.0	15.0	15.0
Filtrate Volume (actual), gallons	160.3	234.2	234.9	126.1	102.3	183.8	201.2	350.6
Backwash (actual)								
Volume, gallons	....	....	51.2	15.1	9.2	13.3	15.9	12.7
Volume as Percent of Filtrate,	....	....	21.8	11.9	9.0	7.2	7.9	3.6
Volume per Pound of Solids Retained, gal/lb.	....	....	43.2	15.8	16.3	10.9	11.3	14.7
Influent Pressure, psig	25	24	23	18	21	22	28	19
Actual Run Terminated By:	Head Loss	Head Loss	Head Loss	Turbidity and Head Loss	Turbidity and Head Loss	Head Loss	Head Loss	Head Loss
Cycle	2nd of 3	3rd of 3	....	1st of 5	2nd of 5	3rd of 5	4th of 5	5th of 5
Form	FRP	FRP	FRP	FRP	FRP	FRP	FRP	FRP

b or b,15 = Breakthrough of Suspended Solids.

Table F-5

Filtration Study Data Summary
Tri-Media Filter

Run No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Nominal Flow per Unit Area, gpm/sq.ft.	10	15	10	10	10	10	15	20	6	10	10	20	15	10	20
Flocculant Dosage															
Alum, mg/L	0	75	150	150	0	150	150	150	150	150	0	150	150	150	0
Primafloc C5, mg/L	0	0	0	4	0	4	4	4	4	4	0	4	4	4	4
Activated Sludge, mg/L	0	0	0	0	55	0	0	0	0	0	0	0	0	0	0
Suspended Solids															
Influent, mg/L	390	440	418	410	475	218	158	528	188	2,420	620	540	640	720	334
Effluent, mg/L	23	42	8	29	101	87	96	243	113	436	112	173	90	161	37
Percent Removal	94	90	98	93	79	60	39	54	40	82	82	68	86	79	89
BOD_5															
Influent, mg/L	31	47	40	25	68	196	159	89	111	19	5	29	42	65	22
Effluent, mg/L	10	23	18	11	37	137	102	60	58	8	1	15	8	4	9
Percent Removal	68	52	54	57	44	30	36	33	48	59	83	48	80	94	57
Soluble BOD_5															
Influent, mg/L	--------	--------	--------	--------	19	103	110	69	50	1	1	11	9	15	5
Effluent, mg/L	--------	--------	--------	--------	18	(103)	(110)	46	20	3	0	6	5	4	4
Percent Removal	--------	--------	--------	--------	6	0	0	33	60	19	100	41	48	76	20
COD															
Influent, mg/L	--------	137	108	107	339	560	670	360	340	162	120	172	253	232	91
Effluent, mg/L	110	70	77	65	207	431	456	256	156	32	80	69	56	51	36
Percent Removal	--------	49	29	39	39	23	20	29	54	80	37	60	78	78	60
pH															
Influent	6.8	6.7	7.0	6.9	6.8	6.8	7.0	6.7	7.0	7.1	7.9	7.0	7.0	7.0	7.0
Effluent	6.8	6.9	7.0	6.9	6.9	6.9	7.0	6.7	6.7	7.2	7.8	6.5	5.4	6.0	7.0
Length of Filter Run, minutes	46	42	75	198	111	54	38	57	120	105_b	105_b	14	13	21	24
Head Loss															
Initial, psi	1.5	2.3	1.4	1.4	1.3	1.3	1.6	3.1	0.7	1.4	1.5	2.7	2.2	1.5	4.4
Final, psi	15.0	15.0	15.0	15.0	9.1	15.0	15.0	15.0	3.0	13.2	3.3	15.0	15.0	15.0	15.0
Filtrate Volume (actual), gallons	51.8	67.6	87.2	208.3	91.1	61.6	--------	123.2	61.2	102.0	102.0	33.1	17.2	22.0	51.7
Backwash (actual)															
Volume, gallons	13.8	12.0	21.9	26.0	20.8	17.5	--------	16.5	6.8	--------	15.3	8.2	6.6	7.4	9.1
Volume as Percent of Filtrate	26.6	17.8	25.2	12.5	22.8	28.4	--------	13.4	11.2	--------	15.0	24.9	32.0	33.5	17.6
Volume per Pound of Solids Retained, gal./lb.	87	54	73	40	86	248	--------	61	177	--------	37	81	71	71	71
Actual Run Terminated By:	Head Loss	Head Loss	Head Loss	Head Loss	Turbidity	Head Loss	Head Loss	Head Loss	Turbidity	Turbidity	Turbidity	Head Loss	Head Loss	Head Loss	Head Loss

b = Breakthrough of Suspended Solids.

FIGURE F-1
SCHEMATIC DIAGRAM OF FILTRATION SYSTEM

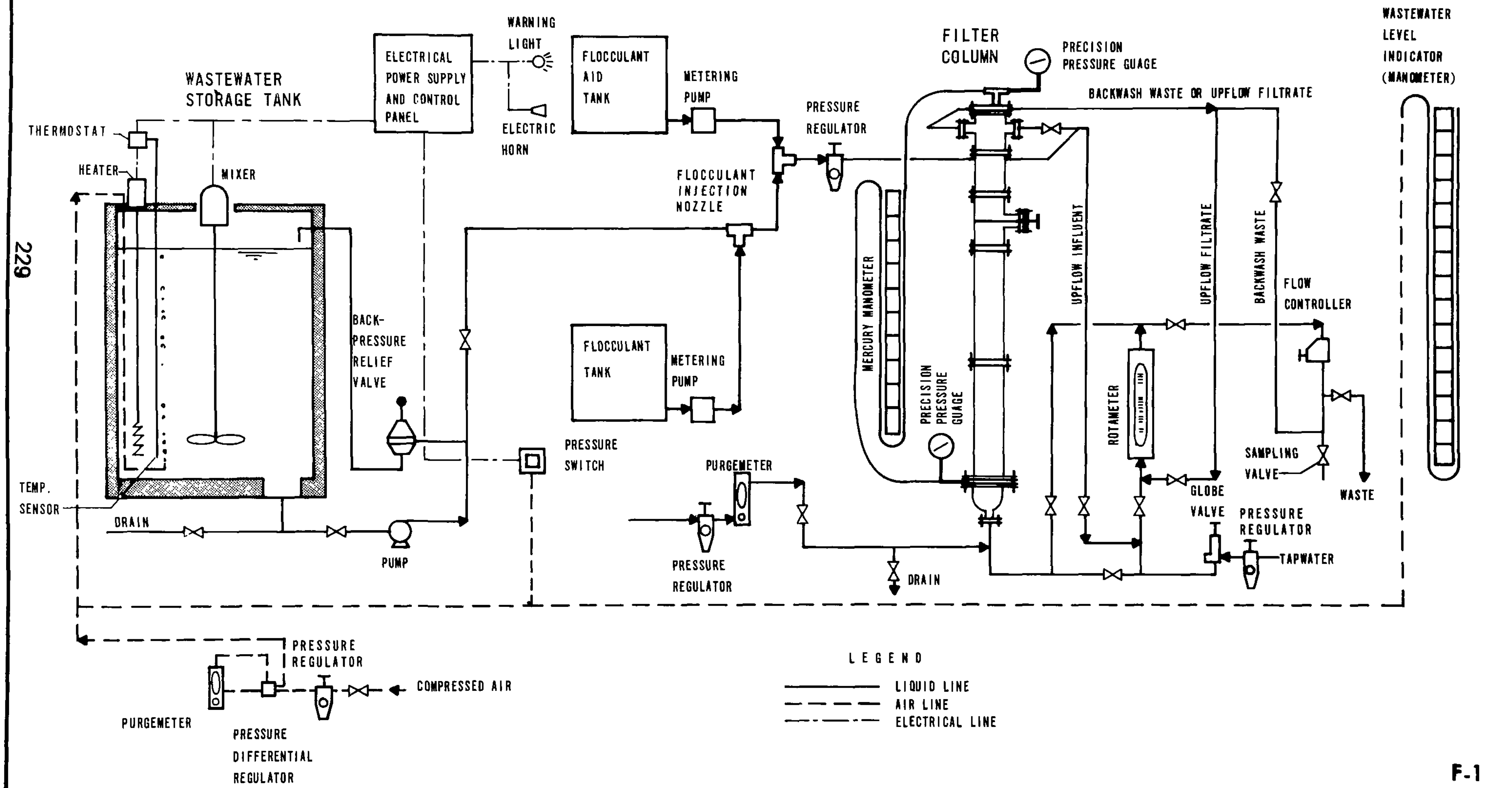
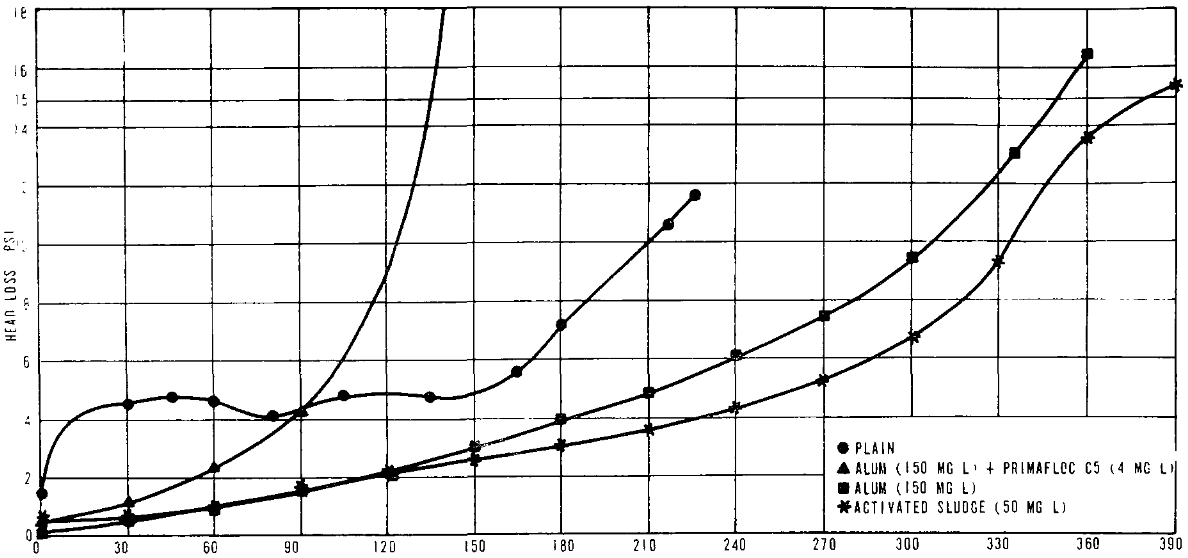


FIGURE F-2

FIBERGLASS FILTER: FLOCCULANT EVALUATION (RUNS 1,3,5,6)

-HEAD LOSS AND SUSPENDED SOLIDS REMOVAL

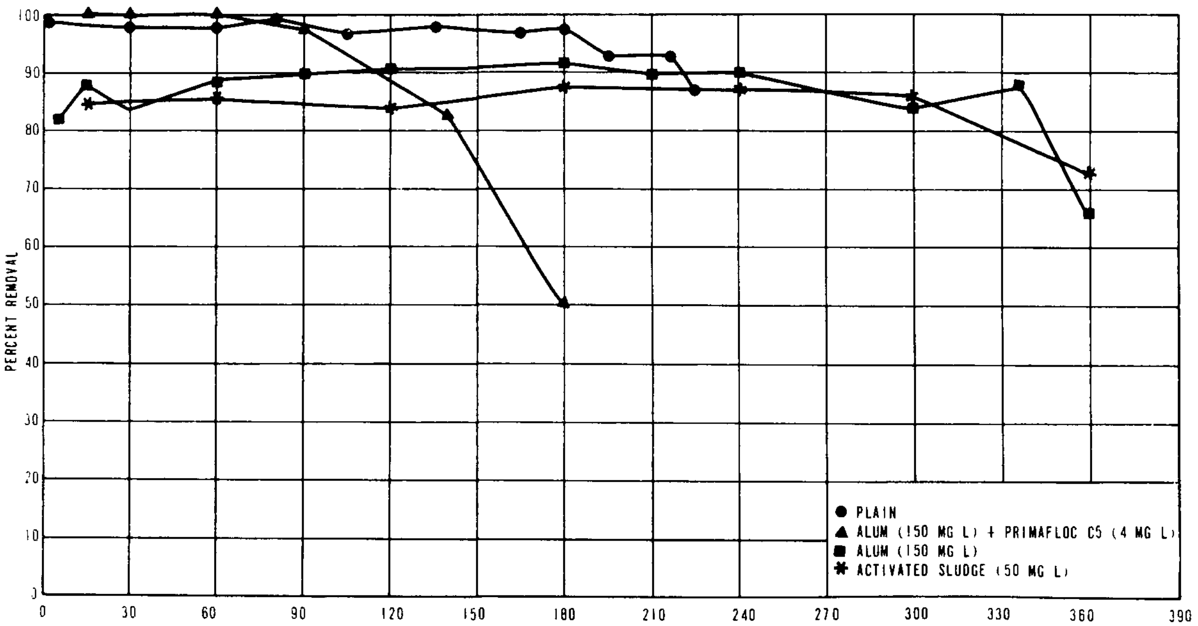
A HEAD LOSS



FILTRATION TIME, MINUTES

(FILTRATION RATE = 15 GPM SF)

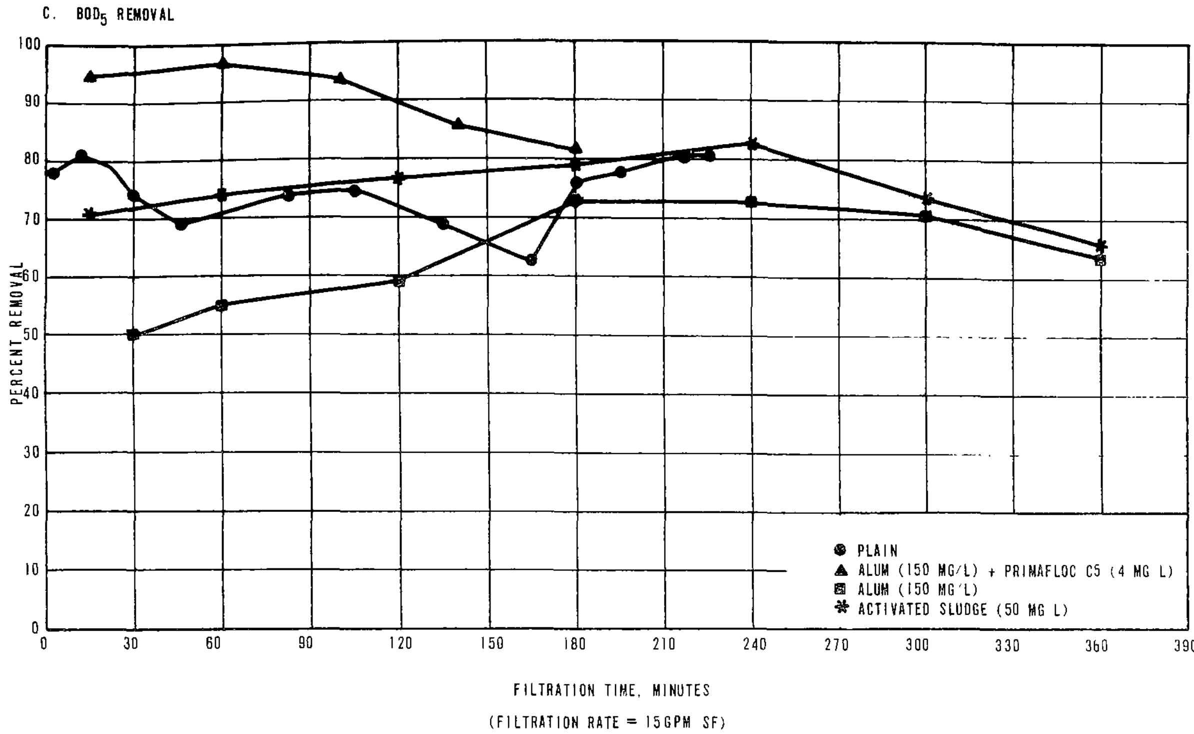
B SUSPENDED SOLIDS REMOVAL



FILTRATION TIME MINUTES

(FILTRATION RATE = 15 GPM SF)

FIGURE F-3
FIBERGLASS FILTER: FLOCCULANT EVALUATION (RUNS 1,3,5,6)
BOD AND COD REMOVALS



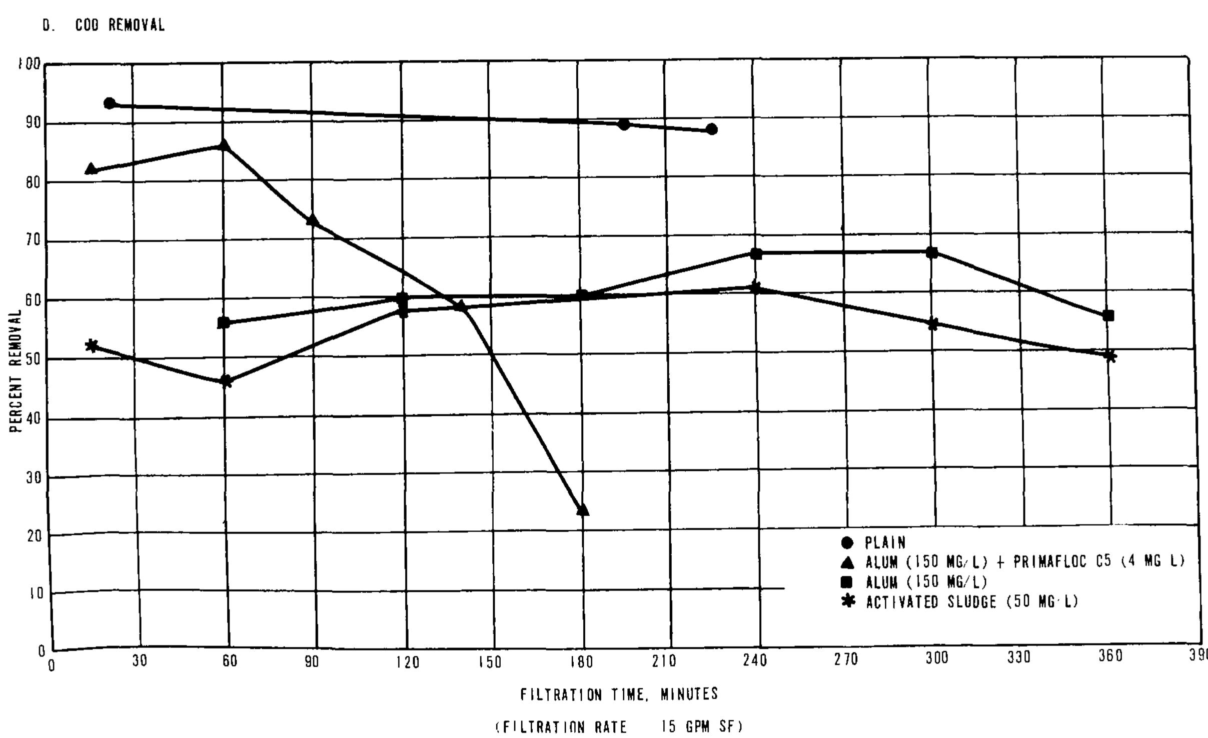
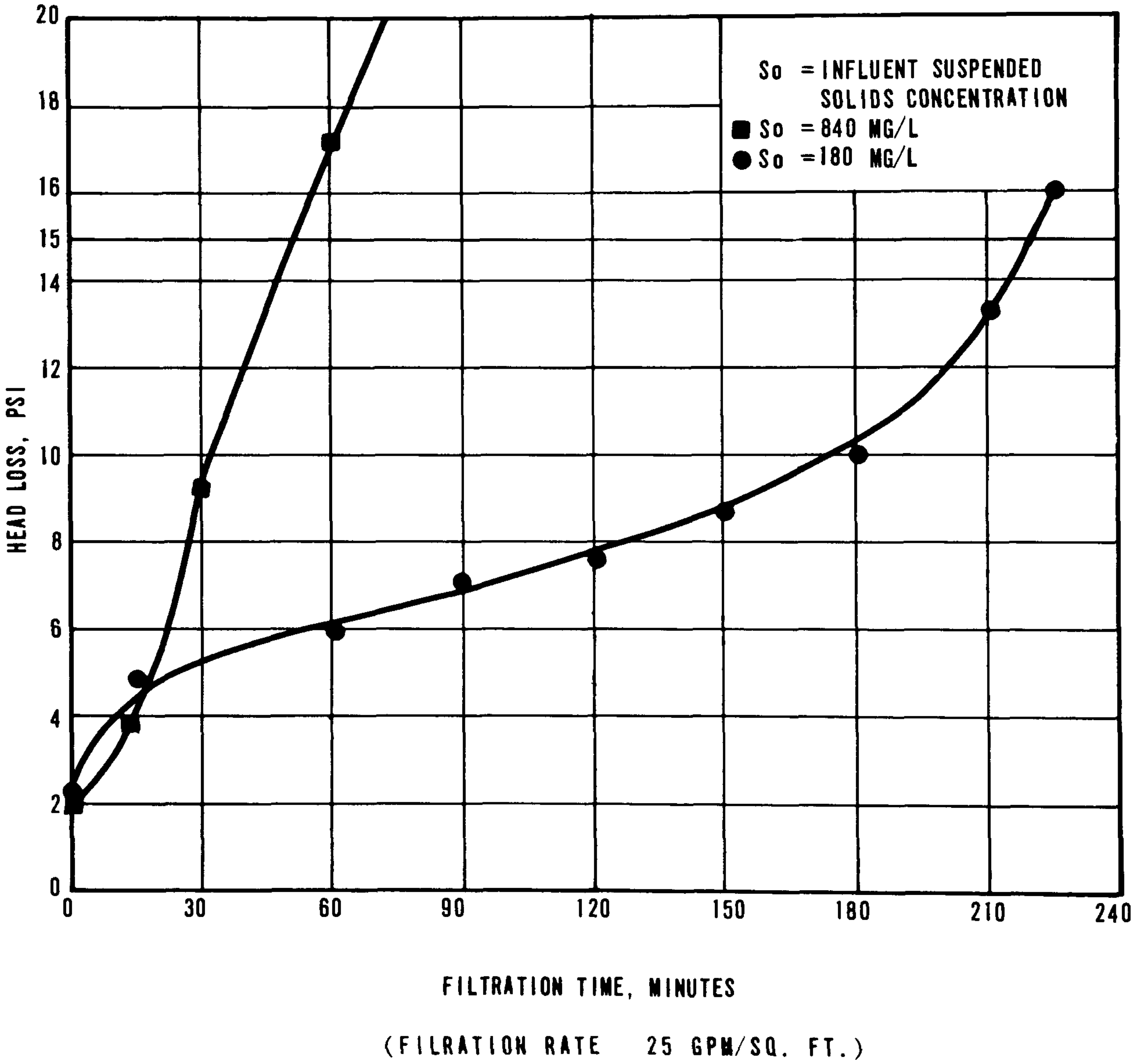


FIGURE F-4

FIBERGLASS FILTER: EFFECT OF INFLUENT SOLIDS CONCENTRATION ON HEAD REQUIREMENT (RUNS 8 , 14)

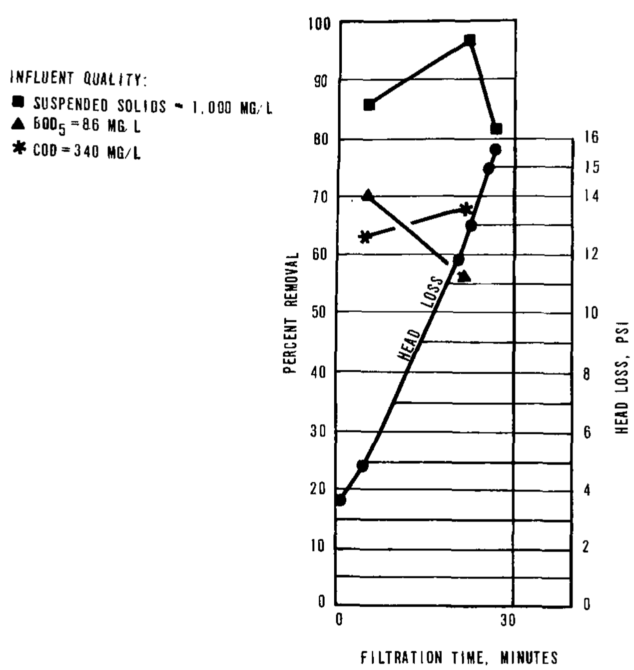


F-4

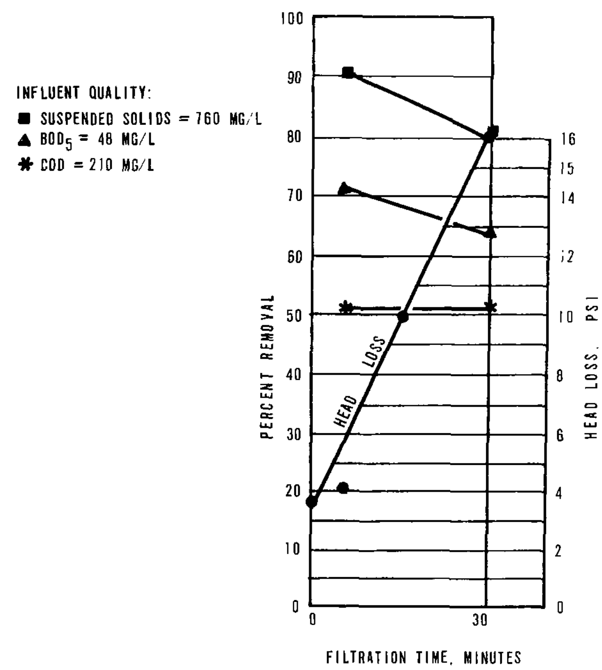
FIGURE F-5

FIBERGLASS FILTER: PERFORMANCE AT 15-50 GPM/SQ. FT.

PERFORMANCE AT 50 GPM/SQ. FT. (RUN NO. 12)



PERFORMANCE AT 35 GPM/SQ. FT. (RUN NO. 13)



PERFORMANCE AT 15 GPM/SQ. FT. (RUN NO. 15)

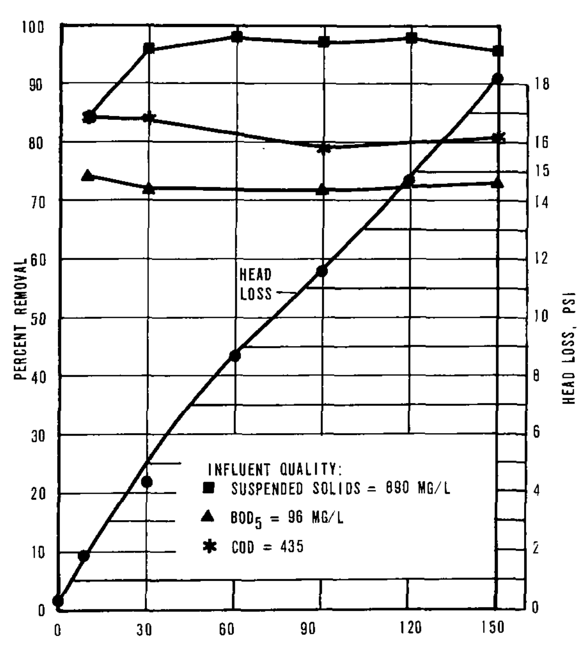
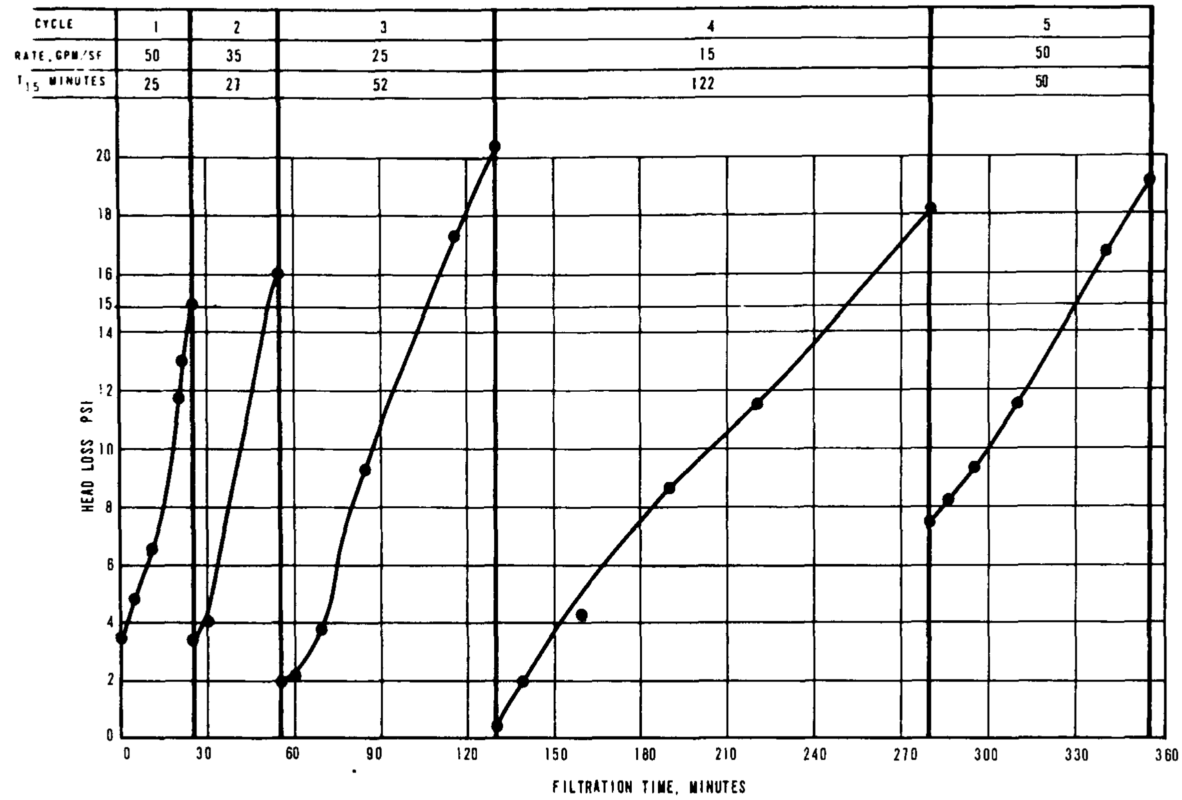


FIGURE F-6

FIBERGLASS FILTER: FIVE CYCLES ON FRP CARTRIDGES

HEAD LOSS AND SUSPENDED SOLIDS REMOVAL

A HEAD LOSS



B SUSPENDED SOLIDS REMOVAL

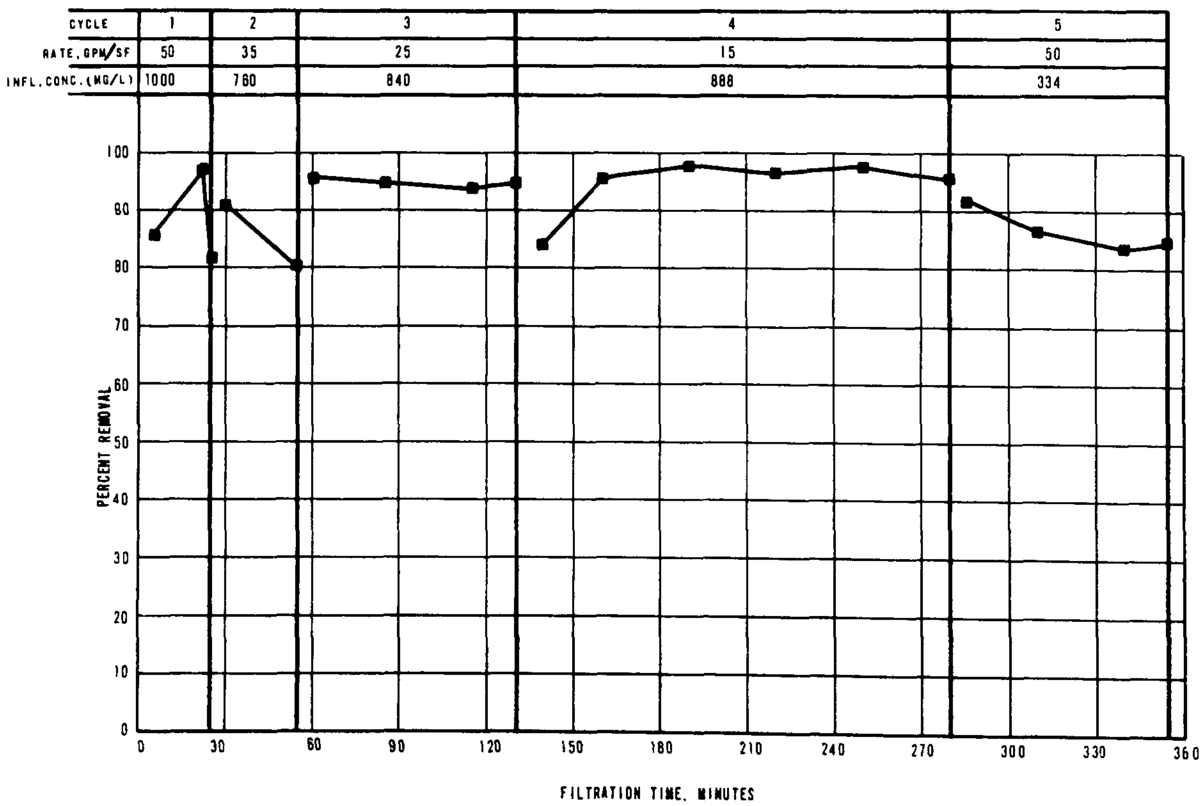
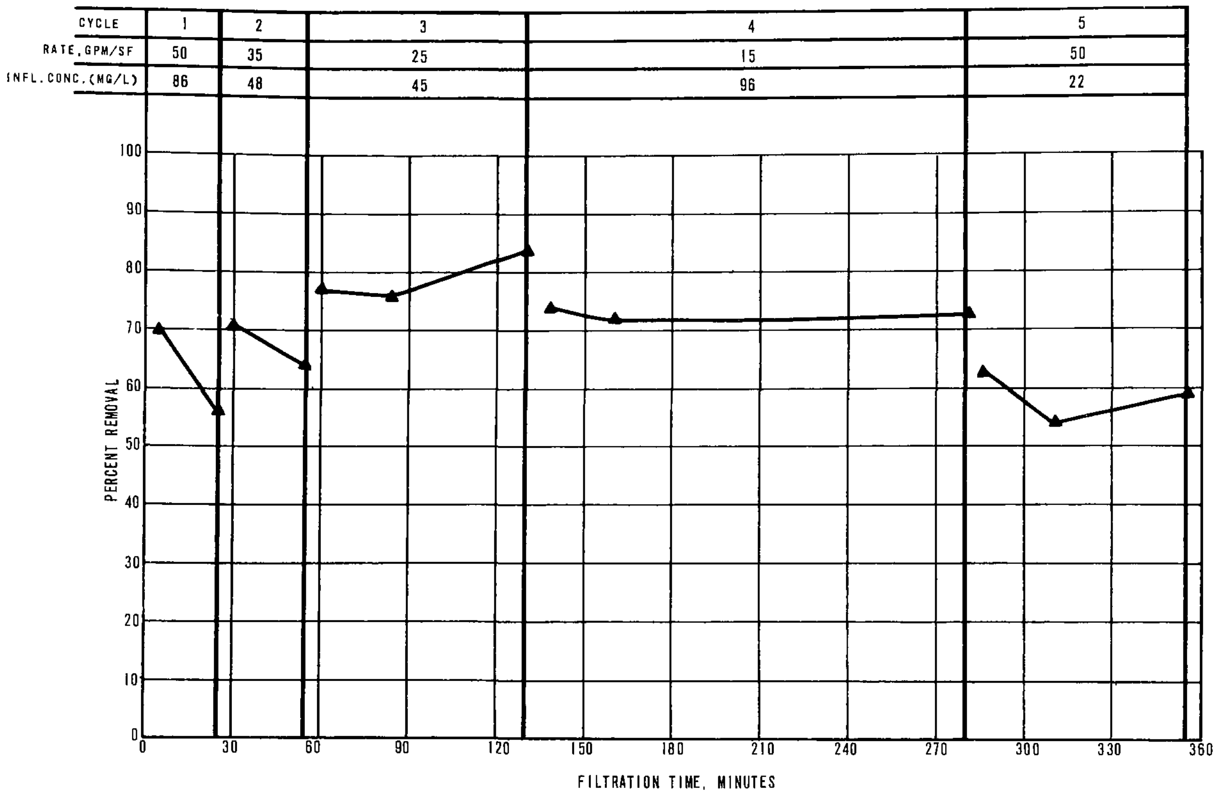


FIGURE F-7

FIBERGLASS FILTER: FIVE CYCLES ON FRP CARTRIDGES

BOD_5 AND COD REMOVALS

C. BOD_5 REMOVAL



D COD REMOVAL

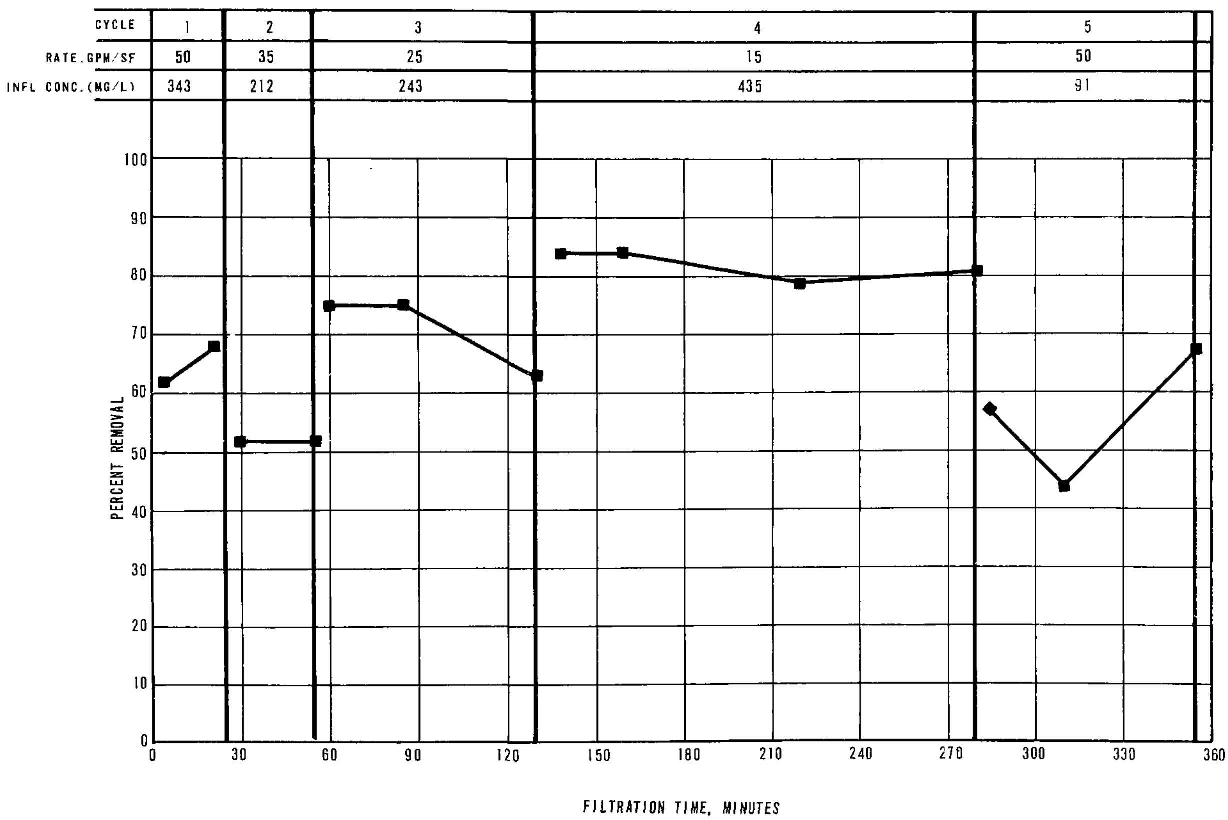


FIGURE F-8

TRI-MEDIA FILTER: FLOCCULANT EVALUATION (RUNS 1,3,4,5)

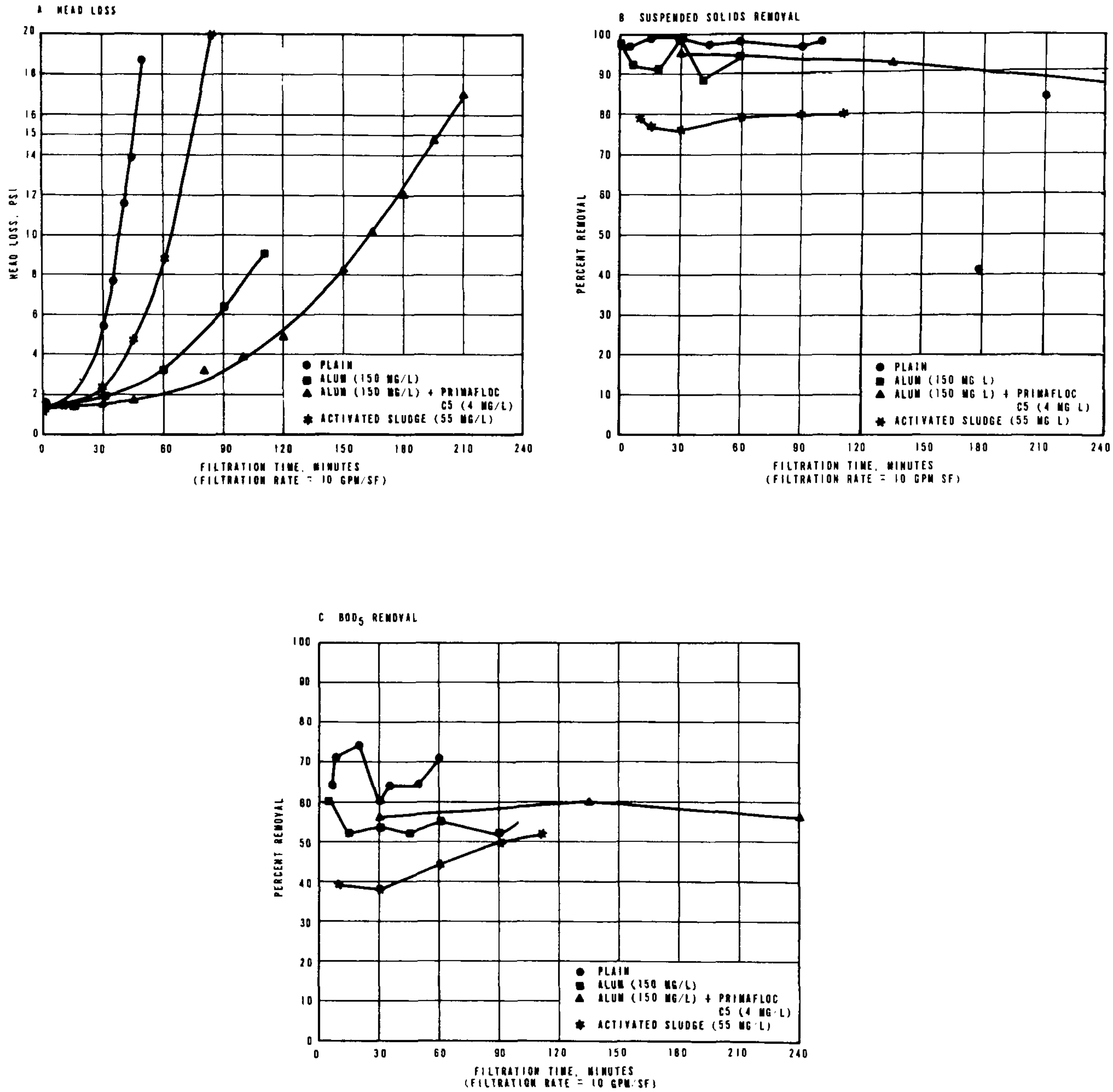
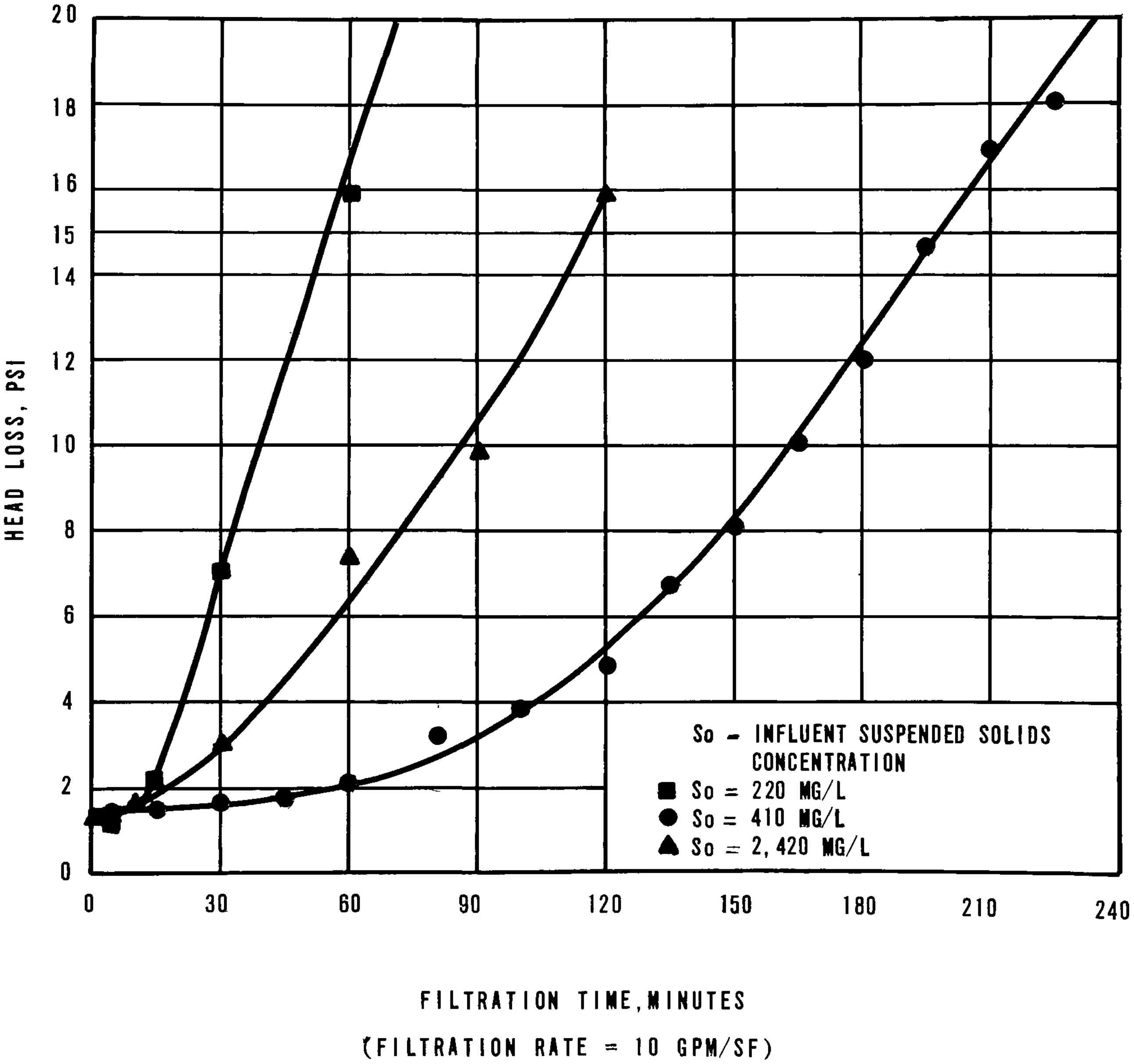


FIGURE F-9

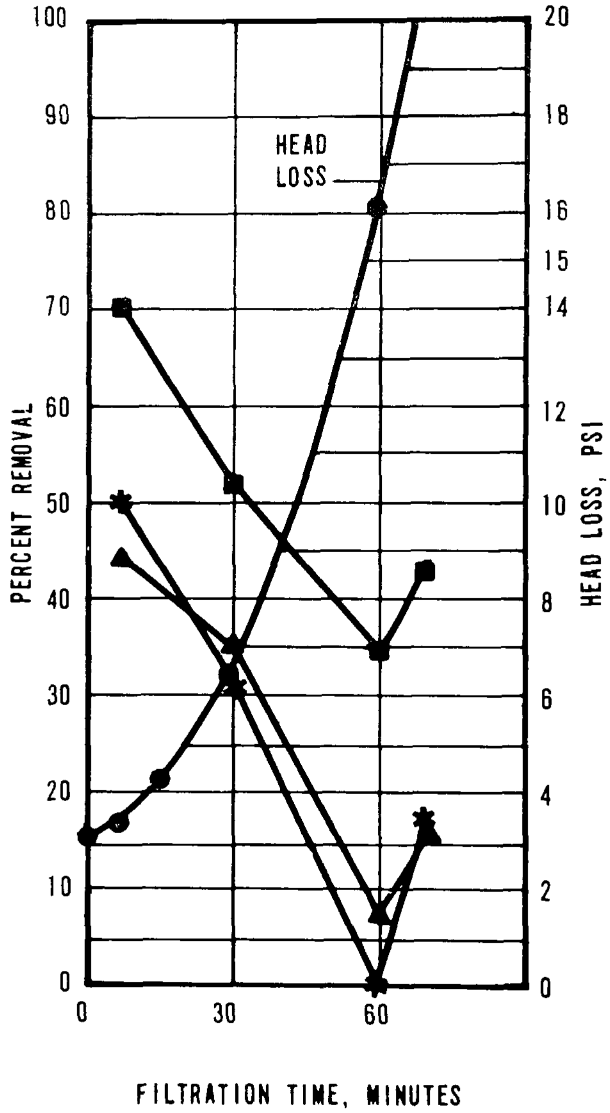
TRI-MEDIA FILTER: EFFECT OF INFLUENT SOLIDS CONCENTRATION ON HEAD REQUIREMENT (RUN 4, 6, 10)



F-9

FIGURE F-10

TRI-MEDIA FILTER: PERFORMANCE AT 20 GPM/SQ. FT. (RUN 8)



INFLUENT QUALITY:

■ SUSPENDED SOLIDS = 530 MG/L

▲ BOD_5 = 89 MG/L

✱ COD = 360 MG/L

F-10

FIGURE F-11

UPFLOW FILTER: PERFORMANCE AT 5,10,AND 15 GPM/SQ.FT.

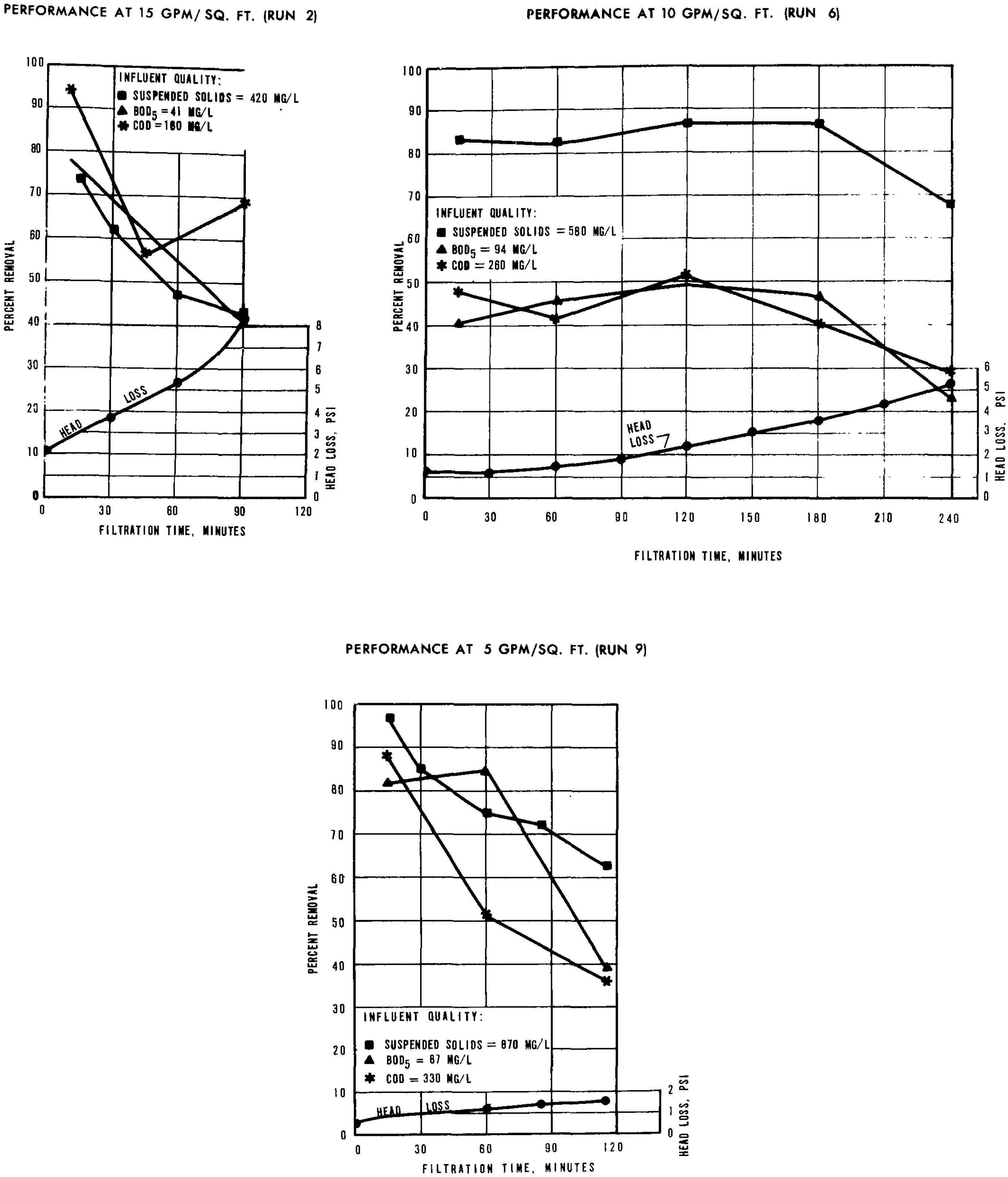
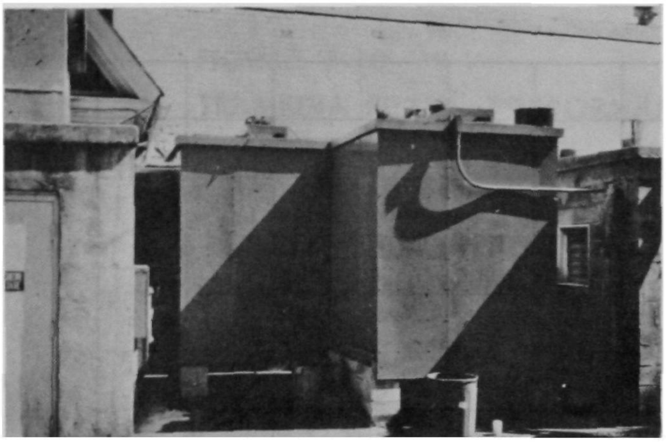
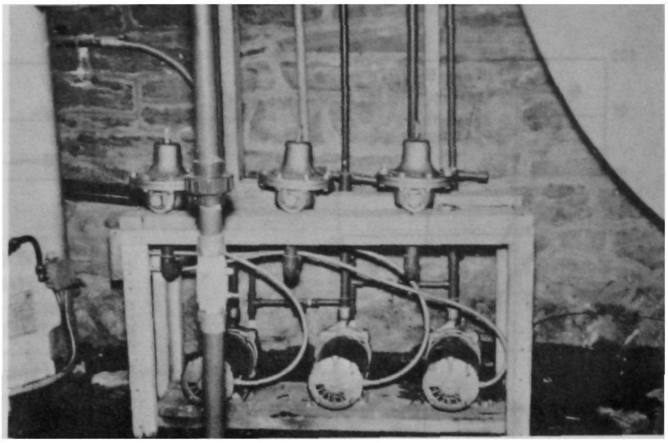


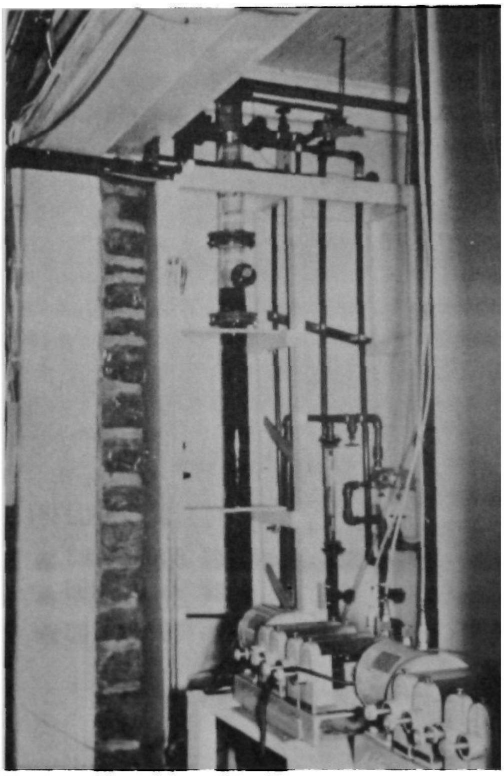
FIGURE F-12
FILTRATION SYSTEM COMPONENTS



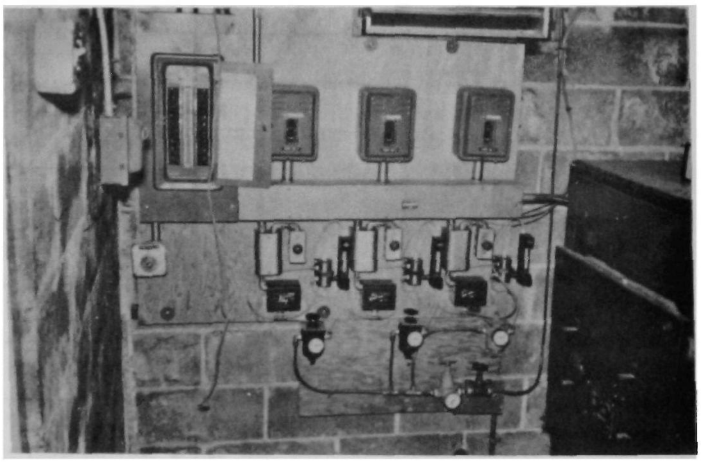
WASTE STORAGE TANK



WASTE FEED PUMPS



FILTER COLUMN



SAFEGUARD DEVICES

APPENDIX G

KINGMAN LAKE PROJECT

Summary

This conceptual engineering study concerns the reclamation of combined sewer overflows and utilization of the reclaimed waters in a major water-oriented recreational facility for the District of Columbia. The investigation encompasses a comprehensive solution of environmental problems by proposing multi-use objectives and facilities.

Principal objectives of the project included: 1) evaluation of rainfall runoff relationships for sizing of storage and treatment plant capacities; 2) confirmation of treatment feasibility using filtration and an activated carbon process; and 3) development of sufficient data for preliminary design purposes.

Laboratory studies not only demonstrated process feasibility, but showed the need for including flocculation and sedimentation for removal of minute particles, together with chlorine and iodine addition for maximum disinfection. The recommended storage/treatment plan provides for a 175 million gallon storage basin, a 50 million-gallon-per-day reclamation facility and two 46-acre swimming and boating lakes.

Cost effectiveness (Cost/Benefit Ratio) of the project, as envisioned, has been indicated to be 1.6 at an estimated total project cost of $45,200,000, and an estimated annual operating cost of $1,777,000. Implementation of the proposed plan would not only provide a least-cost alternative over single-purpose projects to attain identical objectives, but would also reduce the annual pollution now discharged by the Northeast Boundary Trunk Sewer by approximately 99 percent.

This report was submitted in fulfillment of Program No. 11023 FIX under Contract No. 14-12-829 between the Federal Water Quality Administration and Roy F. Weston, Inc.

Conclusions

It is the considered opinion of Roy F. Weston, Inc. that this project in its entirety is both technically and economically feasible. It is possible for the Federal Government to demonstrate that wastewater and waste land can be reclaimed for the use and advancement of society, and that a total environmental approach to the problem of pollution can be effective.

Combined Sewer Overflow

1. The project conceived as a result of the study would be the largest control and treatment works for combined sewer overflows in the United States and would elevate the effluent quality above all other existing or planned combined sewer overflow projects.

2. The Northeast Boundary Trunk Sewer serves approximately one third of the combined sewer area of the District of Columbia.

3. The methodology developed for this study provides a reasonably accurate definition of the quantity, quality, and variability of overflow from the Northeast Boundary Trunk Sewer.

4. The annual discharge of BOD_5 in the overflow from the Northeast Boundary Trunk Sewer accounts for over 25 percent of the recommended allowable waste loading to the Potomac River in the Washington metropolitan area. Even the storm that is expected to occur with a frequency of four times or more per year results in a BOD loading of over twice the recommended allowable daily loading for discharges from all waste treatment facilities in the metropolitan Washington area. The recent Potomac Enforcement Conference held 21 and 22 May 1970 required that these recommended allowable loadings be further reduced.

5. The impact of BOD loading in the overflow on dissolved oxygen levels is more serious than the 25 percent value reflects, due to the combined effect of long residence times of estuarine waters and the true form of overflows occurring at discrete impact loadings.

Storage Alternatives

6. Storage is an essential element of any plan for the abatement of pollution from the Northeast Boundary Trunk Sewer, because of the extremely high overflow rates.

7. Locating a surface storage basin in the vicinity of Kingman Lake is feasible and desirable since all overflow can be collected there without extensive modifications and without pumping, and also because much of the area is undeveloped, indicating the release of open space for a storage basin is possible. The estimated cost for surface storage was $14.3 million as compared to $33.4 million for comparable mined storage.

8. Preliminary soils investigation indicates that the construction of a storage basin in the lower section of Kingman Lake is technically feasible.

9. The tunnel storage concept is applicable to the Northeast Boundary Trunk Sewer and has the decided advantage of providing surcharge relief of the existing sewer system.

Treatment Alternatives

10. Different levels of benefits are associated with each capacity of a wastewater reclamation facility; these include allowable bather load, probability of not overflowing the storage basin, and the additional storage capacity provided by the plant's operating during the period of the storm.

11. A literature evaluation disclosed that there are some promising alternatives to those unit processes investigated in this study; however, confirmation work is necessary before any one can be applied totally to the Kingman Lake Project. Included in this listing are fiberglass filtration media and microstraining for suspended solids removal, U-tube aerators for aeration, and ozonation for odor control and disinfection.

12. The use of iodine in conjunction with chlorine will provide effective disinfection of viruses as well as bacteria in the swimming lake.

13. The use of chlorine in the swimming lake will prevent excessive algae growth, but there may be an algae problem in the fishing-boating lake, which does not otherwise require chlorination.

14. The recommended standards for the influent to the swimming lake are: pH 7.5 to 8.0; BOD 5.0 mg/L; Suspended Solids -15.0 mg/L; Total Phosphorus 0.05 to 1.0 mg/L; Free Chlorine 1.0 mg/L; Free Iodine 1.0 mg/L; Fecal Coliform 200 per 100 ml.

Laboratory Investigative Program

15. Coagulation-Sedimentation followed by multi-media filtration, activated carbon adsorption, and disinfection will produce an effluent which meets the water quality criteria objectives for swimming and fishing.

16. Filtration through fiberglass was shown to be an effective method for removal of suspended solids; however, significant development work is required prior to any major facility application.

Soils Investigation

17. Construction of a storage basin with vertical walls to a depth of minus 40 feet is feasible.

18. Construction of a storage basin with sloping side walls is undesirable because of the restrictions on side slopes, the period of construction, and the relatively small storage capacity available.

19. Construction cost estimates indicate that significant economics can be effected through use of the slurry wall construction method rather than the conventional approach using sheeting and bracing.

20. The foundations for all structures must extend to the sand and gravel layer or to the underlying stiff clay. For shallow structures, it will generally be most economic to provide pile foundations extending to these strata. The deeper structures can be founded directly on the above-mentioned strata or may be constructed on pile foundations.

21. Problems concerning general side grading, seepage, and other work associated with raising the fishing and swimming lakes to elevation +3.5 feet are not anticipated.

Selection of Alternative

22. The alternative scheme evaluated to have the highest cost effectiveness encompasses a storage basin capacity of 175 million gallons and a reclamation plant capacity of 50 million gallons per day.

23. The alternative scheme selected will provide:

    a. Ninety-nine percent reduction of annual pollution load from Northeast Boundary Trunk Sewer, eighty percent reduction of pollution from a storm with a two-year recurrence frequency, and sixty-one percent reduction of pollution from a storm with a five-year recurrence frequency.

b. Effective storage capacity to contain a storm with a recurrence frequency of 1.2 years.

c. A ninety-six percent probability that the reclamation facility will draw down the storage basin prior to the recurrence of a second storm overflow (the volume of which would exceed the remaining volume in the storage basin).

d. Sufficient treatment plant capacity to support a maximum of 30,000 bathers per day in the swimming lake.

24. The covering of the storage basin with a parking roof is justified on the basis of comparable land acquisition and development costs and estimated annual revenues.

25. The total project cost estimate of $45,200,000 compares extremely well with benefits estimated at $72,755,000.

26. Operating costs are estimated to be $1,777,000 per year. Included in the operating costs are: administration, labor, maintenance, utilities, chemicals, make-up carbon, fuel oil, and an operating contingency.

Recommendations

1. Demonstrate how the treatment of combined sewer overflows can serve additional beneficial uses.

2. Demonstrate how the FEDERAL GOVERNMENT can approach the total solution to environmental problems.

3. Design and construct a 175,000,000-gallon storage facility, a 50,000,000-gallon per day water-reclamation facility, and the associated swimming, boating, fishing, and parking facilities.

4. Continue to investigate and gather additional data to refine this report and estimate prior to and during the engineering design and construction phases of this project.

5. Provide sufficient space within the water-reclamation plant and establish a field test facility to demonstrate new and promising processes for improving the treatment of combined sewer overflows.

1 Accession Number	2 Subject Field & Group	SELECTED WATER RESOURCES ABSTRACTS INPUT TRANSACTION FORM

5 *Organization*

ROY F. WESTON, West Chester, Pennsylvania

6 *Title*

COMBINED SEWER OVERFLOW ABATEMENT ALTERNATIVES:
WASHINGTON, D.C.

10 *Author(s)*

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Lee, James A.
Kane, John K.

16 *Project Designation*

EPA, WQO Contract No. 14-12-403

21 *Note*

22 *Citation*

23 *Descriptors (Starred First)*

*Storm Runoff, *Overflow, *Flow Measurement, *Underground Storage, *Filtration, Design Storm, Depth-Area-Duration Analysis, Rainfall-Runoff Relationships, Organic Loading, Treatment Facilities, Tunnel Design, Reservoir Design, Sewers, Capital Costs, Annual Costs, Comparative Costs, Geology, Tracers, Analysis, Sludge, Hydrology.

25 *Identifiers (Starred First)*

*Combined Sewers, *Storm Water, Potomac River, District of Columbia

27 *Abstract*

Objectives of the project were: 1) define the characteristics of combined sewer overflow; 2) investigate the feasibility of high-rate filtration for treatment of combined sewer overflow; and 3) develop and evaluate alternative methods of solution.

Investigative activities included: review of pertinent reports and technical literature; field monitoring of combined sewer overflows and separated storm water discharges at three sites; laboratory studies of ultra-high-rate filtration of combined sewer overflow; hydrological analysis; and evaluation of feasible alternatives (based on conceptual designs, preliminary cost estimates, and other factors).

Reservoir Storage, Treatment at Overflow Points, Conveyance Tunnels and Mined Storage, and Sewer Separation were the approaches considered sufficiently promising for detailed evaluation. Tunnels and Mined Storage with treatment at the Blue Plains plant and at Kingman Lake after subsidence of the storm is recommended. Estimated capital costs (based on the 15-year storm) are $318,000,000 with annual operation and maintenance costs of $3,500,000. This approach also was preferable to the others on the basis of systematic evaluation of reliability, flexibility, public convenience and other non-quantifiable factors.

Abstractor John L. Simons | Institution ROY F. WESTON

WR:102 (REV. JULY 1969)
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11023 FIX 08/70	Conceptual Engineering Report - Kingman Lake Project
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