WATER POLLUTION CONTROL RESEARCH SERIES • 11024---06/70

Combined Sewer Overflow Abatement Technology

June 1970

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

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Combined Sewer Overflow Abatement Technology

A Compilation of Papers Presented

at the Federal Water Quality Administration

"Symposium on Storm and Combined Sewer Overflows"

June 22-23, 1970

Pick Congress Hotel

Chicago, Illinois 60605

for the

FEDERAL WATER QUALITY ADMINISTRATION

DEPARTMENT OF THE INTERIOR

June, 1970

For sale by the Superintendent of Documents, U. S. Government Printing Office
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FORWARD

This compilation of papers entitled "Combined Sewer Overflow Abatement Technology" has been prepared and made available to you so that you can benefit from the current demonstration grants and contracts that are being supported by the FWQA.

During this two day Storm and Combined Sewer Overflow Symposium we will discuss several demonstration projects. Material from these projects to be highlighted will include (1) alternatives to storm and combined sewer pollution in a small urban area; (2) screening and air floatation for solids removal; (3) underflow deep tunnel system concept; (4) urban erosion and sediment control; (5) sewer monitoring and remote control; (6) combined sewer overflow regulators; (7) use of fine mesh screens; and (8) land use and urban runoff pollution.

The concepts and information that this symposium will present, hopefully will help solve your community's problems or at least stimulate in you some new ideas as to how you might solve your storm and combined sewer overflow pollution problems.

Francis T. Mayo

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C O N T E N T S

SECTION I

STORM WATER POLLUTION FROM URBAN LAND ACTIVITY

for
Presentation at the
Storm and Combined Sewer Seminar
Federal Water Quality Administration
Great Lakes Region
Chicago, Illinois
June 22-23, 1970

by

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ABSTRACT

An investigation of the pollution concentrations and loads from storm water runoff in an urban area was conducted in Tulsa, Oklahoma during the period from October 1968 to September 1969. The scope of the project included a field assessment of the storm water pollution by obtaining samples of the water resulting from precipitation and surface runoff from selected test areas in the metropolitan area; development of an analytical procedure for correlation of storm water pollution with selectively defined variables of land uses, environmental conditions, drainage characteristics, and precipitation; and development of a plan for implementing remedial measures necessary to abate or control sources of pollution in an urban area.

Storm water runoff samples were collected from 15 "discrete" test areas in the Tulsa Metropolitan areas. These samples were analyzed in terms of quality standards for BOD, COD, TOC, organic Kjeldahl nitrogen, soluble orthophosphate, chloride, pH, solids, total coliform, fecal coliform, and fecal streptococcus pollutants.

The land usage and environmental conditions of the 15 test areas varied. The parameter averages that were determined for the test areas exhibited these differences. The range of values for the bacteriological densities varied from 5,000 to 400,000 counts/100 ml for total coliform, 10 to 18,000 counts/100 ml for fecal coliforms, and 700 to 30,000 counts/100 ml for fecal streptococcus. The average storm water loadings for other selected pollution parameters ranged from 12 to 48 pounds/acre/year for BOD, 60 to 470 pounds/acre/year for COD, 0.8 to 3.6 pounds/acre/year for organic nitrogen, 1.1 to 8.0 pounds/acre/year for soluble orthophosphate, and 490 to 5100 pounds/acre/year for total solids.

This investigation was performed for the Storm and Combined Sewer Pollution Control Branch, Federal Water Quality Administration by AVCO Economic Systems Corporation under Contract 14-12-187. A draft copy of the final report has been submitted to FWQA for review and comment.

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STORM WATER POLLUTION FROM URBAN LAND ACTIVITY

INTRODUCTION

This paper covers an investigation of urban area storm water pollution, or more precisely, an assessment of pollution in storm water as it relates to land activity. The central purpose of this effort was to design a method of analysis which would enable the city planner and engineer to assess the quality as well as quantity of storm water, and to do so by looking at land activity, selected environmental factors, and precipitation. In an engineering sense, the process was to relate land use, land condition, and hydrological input to a pollutional output for homogeneous areas. The predicted area load thus is aggregated to provide an estimate of pollution. The process is similar to the determination of runoff from urban areas.

Given the relationship of man's activities to storm water drainage, alteration in space and/or time through civic actions can be identified that would reduce pollutional loads. Certain environmental factors such as watershed characteristics and precipitation, alleviation of pollutant conditions through civic actions can be identified that would reduce pollutional loads in storm water. If urban planning and proper regulation of land activity can reduce the overall costs associated with the achievement of an acceptable quality of the environment in the urban area, such activities should be considered the first order of business and an adjunct to any construction of physical systems for collection, disposal, or treatment.

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DESCRIPTION OF THE URBAN AREA

The urban area selected for study was Tulsa, Oklahoma, a relatively young city. Incorporated in 1907, Tulsa is typical of many southwestern and western urban areas. From 1940 until today the Tulsa Urban Area has grown rapidly to a population of over 400,000.

Tulsa was selected because of (1) separate storm and sanitary sewer systems and (2) the land use data file maintained by the Tulsa Metropolitan Area Planning Commission.

Drainage of storm water runoff from urban Tulsa is into two main receiving streams. The northern part of the city of Tulsa and the north portion of Tulsa County drain into the Verdigris River, which in turn drains into the Arkansas River at Muskogee, Oklahoma. The original townsite and large portions of the western and southern parts of the city drain directly into the Arkansas River.

Precipitation is generally well distributed throughout the year. The season of maximum rainfall is the spring and much of this occurs through thunderstorm activity. The high levels of soil moisture and the high precipitation intensities produced by the thunderstorms help to increase the percentage of storm runoff during this season. The precipitation regimen of the Tulsa area was examined by a study of the number of events and the amount of rainfall in the events for a five year period (1964-1968). The mean annual precipitation was 37.25 inches for this period. This amount was produced by an average of 93 events. Of these events, 52 produced amounts in excess of 0.1 inch and were probable producers of runoff from subareas within the urban drainage basins of Tulsa.

As mentioned earlier, Tulsa was selected because of the amount of land use and planning data available to characterize homogeneous areas for testing. Subdrainage basins representative of specific classifications had to be selected, and appropriate sampling sites found. Selection of discrete areas of land activity, although the main criterion for selection, was limited by other factors that had to be considered. The most important of these were:

1. accessibility of the sampling site.
2. size of area large enough to represent certain type land use.
3. lack of known point sources of pollution in the drainage area.
4. security of the sampling instruments from vandalism.

The locations of the 15 test areas and sampling sites are indicated in Figure 1. A summary of the general description of the test areas is given in Table 1.

Land use activity within each of the 15 drainage sheds was determined by utilizing the TMAPC's Land Activity File. After the test areas had been defined by true ridge lines, the census tracts, and the planning blocks; a retrieval program was written to sum various land use activities within each basin. The results of this retrieval are summarized in Table 2 and Table 3.

The drainage characteristics (see Table 4 and Table 5) of each test area were determined from the appropriate USGS quadrangle maps and the City of Tulsa Storm Drain Atlas.

FIGURE 1
LOCATION OF TEST AREAS
TULSA, OKLAHOMA

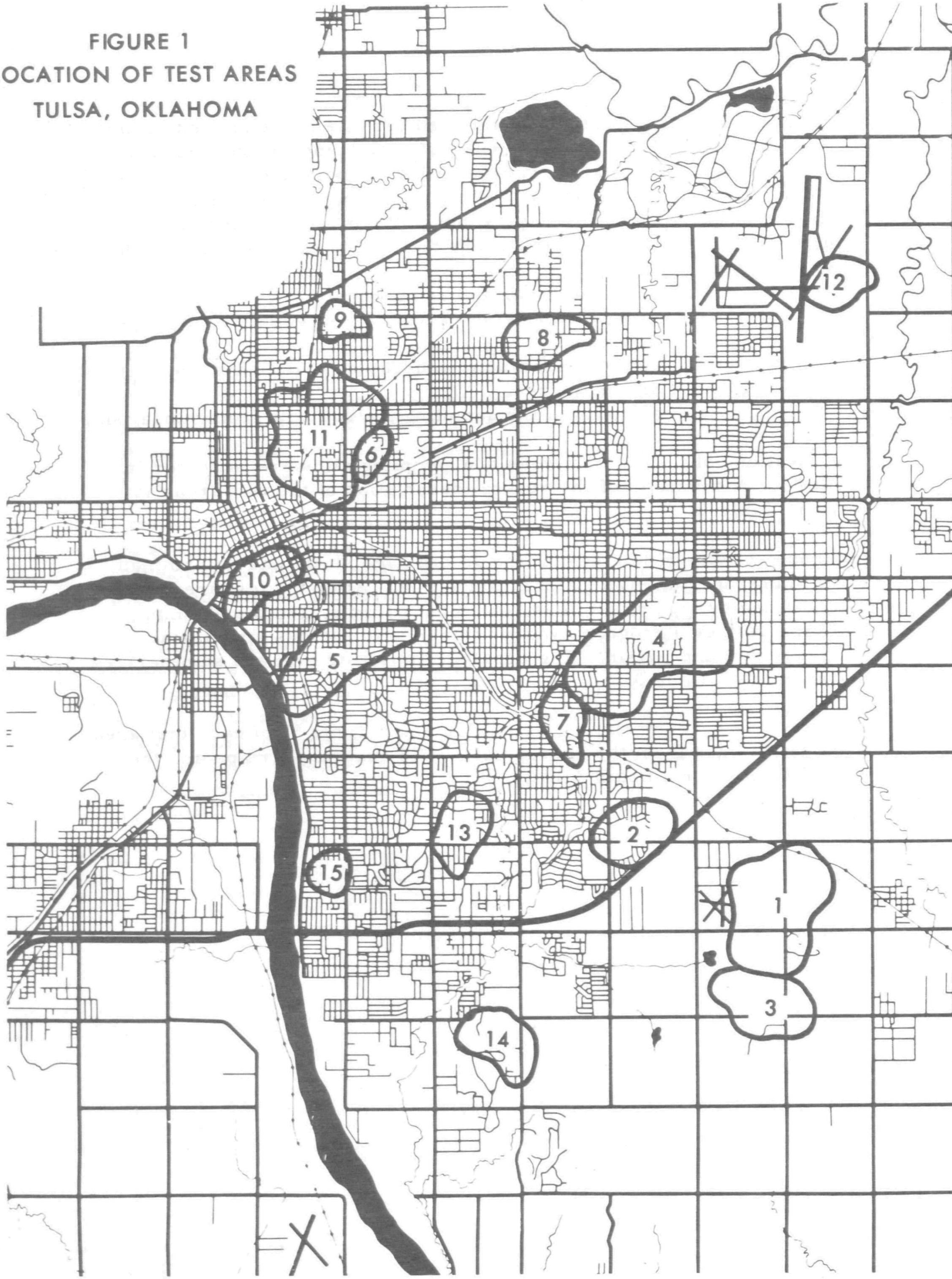

TABLE 1

GENERAL DESCRIPTION OF THE TEST AREAS

Test Area No.	General Landuse Classification	Specific Zoning Classification	Socioeconomic Class	Remarks
1	Industrial	Pred. U-4A Small amount U-3A		Light industrial, warehousing, industrial sales--new industrial development containing little outside storage--large portion still in construction stage--water quality should reflect cement company waste in lower reaches of watershed.
2	Commercial	Pred. U-3E some U-1C, U-2B, U-3A	Some upper-middle class residential	Shopping center with large paved parking areas--includes drainage from large grassy slope (portion of Pan American Research Laboratories property)
3	Residential	Pred. U-1C Small amount U-1B, U-3DH	Upper-middle class	Relatively new additions with little tree cover and well-kept lawns--area swimming pool probable drains into storm sewer--some commercial on major streets.
4	Industrial and Residential	U-4B U-1C	Residential portion-lower middle class	Light to moderate industrial with approximately 1/3 residential--far upper reaches drain portion of Tulsa State Fairgrounds--industrial is approximately 1/2 older development and 1/2 new development or open land zoned for industrial use--considerable amount of outside storage of industrial products--railway service to most of area for shipping.
5	Residential	Pred. U-1C	Upper-middle class some lower-upper class-some lower middle class in upper reaches	Large older homes--great amount of tree cover--some small older housing in upper reaches of watershed includes some commercial on major streets, drainage from Woodward Park, Tulsa Garden Center, and overflow from Swan Lake.

TABLE 1

GENERAL DESCRIPTION OF THE TEST AREAS (CONT'D)

Test Area No.	General Landuse Classification	Specific Zoning Classification	Socioeconomic Class	Remarks
6	Industrial	U-4B		Older industrial area with considerable amount of outside storage--water quality should reflect waste from trucking firm--lower middle class residences make up the upper and eastern reaches of the watershed.
7	Residential	U-1C	Upper-middle class	Postwar addition of mostly three bedroom frame and brick houses with medium-sized trees--well-kept area.
8	Residential	U-1C	Lower-middle class	Postwar addition of mostly two bedroom frame and brick houses with medium-sized tree cover.
9	Residential	Pred. U-1C traces of U-4B	Lower class	Older houses of various sizes, many nearing delapidation--ill-kept area residentially with some commercial on major thoroughfares.
10	Commercial-Office and Residential	3/4 U-3DH and remainder in U-2A and U-2B	Some lower-middle class	Upper portion of watershed is commercial-office including multi-story buildings-middle areas of watershed are largely open areas with considerable tree cover--these areas have been cleared by the Tulsa urban renewal authority for eventual redevelopment--some urban renewal work is still underway in the area--lower areas of the watershed are old residences of various size houses with great amount of tree cover.

TABLE 1

GENERAL DESCRIPTION OF THE TEST AREAS (CONT'D)

Test Area No.	General Landuse Classification	Specific Zoning Classification	Socioeconomic Class	Remarks
11	Residential and Commercial	U-1C	Lower-middle class	This drainage basin is in the heart of Tulsa's model city area--mostly small older frame houses with great amount of tree cover--some commercial on major streets.
12	Industrial and Commercial	U-4A		Runways and supporting buildings with some light industrial--great deal of open grassy areas.
13	Residential	U-1A	Lower-upper class	Non-sewered, newly laid concrete pipe into unimproved open channel, large lots with a number of swimming pools--well-kept lawns.
14	Recreational			Southern Hills Country Club--most of drainage basin includes golf course.
15	Residential	U-1C	Lower-middle class	Postwar addition of small 2-3 bedroom frame and brick houses with coverage of medium sized trees.

TABLE 2 PERCENTAGE OF LAND DEVOTED TO VARIOUS LAND USE ACTIVITIES IN THE FIFTEEN TEST AREAS, TULSA, OKLAHOMA

Test Area No.	PERCENT OF TOTAL AREA									
	Residential	Commercial	Industrial	Institutional	Transportation	Open Space	Unused Space	Arterial Streets	Other Streets	Total
1	4.23	7.28	47.35	0.15	1.46	0	24.77	6.99	7.72	99.95
2	30.32	22.38	.36	0	1.44	24.55	.72	5.42	14.80	99.90
3	56.54	.91	18.34	4.18	0	3.46	16.00	2.36	16.73	100.18
4	24.94	19.08	.20	5.65	2.98	.85	5.33	5.86	16.95	99.98
5	52.85	1.97	35.05	6.51	2.96	9.86	5.92	3.94	15.78	99.99
6	32.60	3.26	0	.54	2.45	0	2.99	2.17	20.92	99.98
7	64.97	1.52	4.74	9.14	0	0	.51	1.52	22.33	99.99
8	51.66	6.16	0	1.42	3.32	0	4.27	14.22	14.22	100.01
9	46.86	10.93	0	0	0	0	4.69	10.93	26.55	99.96
10	16.02	15.53	5.03	1.94	.49	0	15.53	18.93	31.55	99.99
11	44.99	1.96	1.41	1.84	.37	.61	3.44	5.88	35.80	99.92
12	0	0	0	0	48.36	50.23	0	0	0	100.00
13	75.46	0	0	2.83	0	0	2.36	5.19	14.62	100.46
14	26.99	0	0	0	0	65.39	0	4.56	3.04	99.98
15	70.25	0		1.35	0	0	6.76	0	21.62	99.98

TABLE 3   POPULATION CHARACTERISTICS OF THE FIFTEEN TEST AREAS

TEST AREA NO.	TOTAL LIVING UNITS	POPULATION	POPULATION ESTIMATOR PEO. / UNIT	RESIDENTIAL AREA ACRES	RESIDENTIAL DENSITY PEO. / RES. ACRE	TOTAL AREA ACRES	AVERAGE DENSITY PEO. / ACRE
1	100	350	3.50	29	12.07	686	0.51
2	369	1100	3.00	84	13.09	272	4.04
3	1147	3925	3.42	311	12.62	550	7.13
4	1122	3625	3.23	234	15.49	938	3.86
5	1765	4525	2.56	268	16.88	507	8.93
6	501	1200	2.37	120	10.00	368	3.26
7	616	2275	3.70	128	17.77	197	11.55
8	715	2400	3.35	109	22.02	211	11.37
9	267	875	3.26	30	29.17	64	13.67
10	425	885	2.08	33	26.82	206	4.30
11	3396	2800	2.30	367	21.25	815	9.57
12	0	0	0	0	0	223	0
13	168	500	3.01	160	3.13	212	2.36
14	77	250	3.01	71	3.52	263	0.95
15	282	830	2.95	52	15.96	74	11.22

TABLE 4. DRAINAGE CHARACTERISTICS OF TEST AREAS

TEST AREA NO.	(1) A	(2) L	(3) L_c	(4) H	(5) S_c	(6) SL	(7) C	(8) FF	(9) $Gx10^2$
1	686	9050	6000	113	0.011	3.19	30	0.83	1.07
2	272	4230	2040	92	0.011	3.48	55	2.85	0.95
3	550	6890	3000	186	0.009	3.82	27	2.66	1.41
4	938	9260	4800	126	0.010	2.89	51	1.77	1.00
5	507	11200	4800	140	0.013	3.29	30	0.96	2.16
6	368	2170	3600	91	0.009	2.19	24	1.24	0.55
7	197	4500	2100	85	0.013	2.89	32	1.94	1.52
8	211	4800	1800	95	0.013	1.67	37	2.84	2.99
9	64	2600	1380	60	0.011	1.55	31	1.47	3.61
10	206	6350	3300	140	0.032	2.26	74	0.82	4.69
11	815	9000	4200	162	0.007	1.83	41	2.01	2.24
12	223	5710	2400	58	0.007	0.75	46	1.68	4.53
13	212	7840	2600	140	0.015	4.60	23	1.37	2.54
14	263	6400	3480	171	0.014	4.25	11	0.95	2.24
15	74	2700	1600	30	0.012	0.78	38	1.26	3.21

Legend:

1. Test Area (A), acres
2. Length of the main stream (L), feet.
3. Length of the main stream from the sampling site to the point nearest area centroid (L_c) feet.
4. Fall of the watershed (H), feet.
5. Average main channel slope (S_c), feet per foot.
6. Average land slope (SL), percent.
7. Impervious cover (C), percent.
8. Form Factor (FF) = 43,560 A/ $(L_c)^2$, dimensionless.
9. Geometry Number (G)

$$= \frac{(L)\ (H)}{(43,560)\ (A)\ (SL)}, \text{ dimensionless}$$

TABLE 5. STREET AND DRAINAGE CHANNEL CHARACTERISTICS

TEST AREA NO.	TOTAL AREA ACRES	STREETS: ARTERIAL[1] ACRES	STREETS: ARTERIAL[1] MILES	STREETS: OTHER[2] ACRES	STREETS: OTHER[2] MILES	DRAINAGE CHANNEL: TOTAL[3] MILES	DRAINAGE CHANNEL: COVERED[4] MILES
1	686	48	3.44	53	13.83	1.71	1.05
2	272	15	1.21	41	10.70	0.80	1.07
3	550	13	1.07	92	24.01	1.30	1.25
4	938	55	4.52	159	41.50	1.75	3.30
5	507	20	1.63	80	20.88	2.11	2.14
6	368	8	0.62	77	20.10	0.41	1.55
7	197	3	0.26	44	11.48	0.85	1.02
8	211	30	2.48	30	7.83	0.91	1.08
9	64	7	0.60	17	4.44	0.49	0.78
10	206	39	3.21	65	16.97	1.20	1.13
11	815	48	4.00	292	76.21	1.70	3.75
12	223	0	0	0	0	1.08	0
13	212	11	0.88	31	8.09	1.48	0.82
14	203	12	1.14	8	2.09	1.21	0
15	74	0	0	16	4.18	0.51	0.69

[1] Arterial streets are major thoroughfares.

[2] Other streets are all streets less arterial streets.

[3] Total drainage channel as used here refers to the length of the main interceptor channel.

[4] Covered drainage channel as used here refers to all covered drainage conduit (interceptor and lateral) greater than 24 inches diameter.

ENVIRONMENTAL CONDITIONS

In 1968 the Tulsa City-County Health Department conducted a Community Block Survey in the City of Tulsa. The purpose of the survey was to delineate the general environmental condition that existed in the community. An analysis of the data resulting from this survey provides a method of locating environmental conditions which contribute to the origin, frequency, and distribution of communicable disease within a community. Also, with this data and additional census block data, a community can be stratified into socioeconomic areas.

The environmental factors included in the survey were land use, exterior housing quality, water supply, human waste disposal, refuse storage, rubble accumulations, junked cars, dilapidated sheds, vacant lot sanitation, poor drainage areas, vector harborage, and the presence of livestock, poultry, or dog pens.

Since the normal procedure (1) in stratifying a community into socioeconomic strata is not applicable to large areas and could not be applied to commercial or industrial areas, a method was devised by the author of this study for determining the general environmental condition of the fifteen test areas. An Environmental Index (EI) was calculated for each of the test areas, as follows:

Environmental Index (EI) = f (housing condition, vacant lot condition, parcel deficiencies)

Assuming that the parcel deficiencies should be weighted more heavily than the housing conditions and that the housing conditions should be weighted more heavily than the vacant lot conditions:

$$EI = \frac{2\,(A) + B + 3\,(C)}{6}$$

Where:

$$A = \frac{\text{Total Housing Structures}}{(1)\,(G) + (2)\,(F) + (3)\,(P)}$$

Note: G = no. of good vacant lots
F = no. of fair vacant lots
P = no. of poor vacant lots

$$B = \frac{\text{Total Vacant Lots}}{(1)\ (G)\ +\ (2)\ (F) + 3\ (P)}$$

Note: G = no. of good vacant lots
F = no. of fair vacant lots
P = no. of poor vacant lots

$$C = \frac{\text{Total Structures} - \text{Total Deficiencies}}{\text{Total Structures}}$$

Note: Total deficiencies include the sum total of refuse, burners, rubble, lumber, old autos, poor sheds, livestock, poultry, and privies.

The above three factors (A, B, and C) are a measure of the general housing condition, the vacant lot condition, and the parcel deficiencies, respectively. Factors A and B vary from a low of .33 to a high of 1.00. Factor C varies from a negative number to 1.00. The smaller numbers indicate poor environmental conditions while the larger numbers indicate good environmental conditions.

Applying the above formula will result in a number that varies from a negative number to a maximum of 1.00. A value of 1.00 will denote an area of all good houses, all good vacant lots, and no parcel deficiencies.

Not included in the above index are several other factors that, if used, would result in a better measure of the "general environmental condition of an area." Such items are: air pollution sources, population and structure density, point water pollution sources, parks, noise level, and traffic volume. If these data items were available and each could be expressed by a number and weighted, a better EI could be developed. Applying the above formula to the survey data, an EI for each of the Test Areas was calculated. Table 6 presents these calculations with the resulting EI.

TABLE 6

CALCULATED ENVIRONMENTAL INDEX (EI)
FOR THE FIFTEEN TEST AREAS
TULSA, OKLAHOMA

Test Area No.	Calculated Factor[1] A	B	C	Environmental Index (EI)
1	.99	1.00	1.00	1.00
2	1.00	1.00	.99	.99
3	1.00	.83	1.00	.97
4	1.00	1.00	.71	.86
5	1.00	1.00	.98	.99
6	.68	.62	.48	.57
7	1.00	1.00	.96	.98
8	1.00	1.00	.62	.81
9	.70	.53	-.19	.23
10	.84	.97	.93	.91
11	.46	.56	-.34	.08
12				1.00[2]
13	1.00	1.00	.94	.97
14	1.00	1.00	1.00	1.00
15	1.00	1.00	.71	.86

[1] Calculated factors from environmental survey data for use in equation 1.
[2] Test Area No. 12 was assumed to have an EI of 1.00.

SAMPLING INSTRUMENTS AND METHODS USED

The collection of storm water runoff samples required the use of several different types of instruments and methods. A stationary automatic sampling method was used when a time series of samples was desired. Standard manual sampling procedures (grab sampling) were used when baseline samples or additional storm water runoff samples were collected. The bacteriological samples were collected in sterile plastic bags.

A schematic diagram of the storm water sequential sampling equipment is shown in Figure 2. Views of the sampling equipment are shown in Figure 3 through 6.

After sample collection, all samples were stored and analyzed in accordance with "Standard Methods for the Examination of Water and Waste Water, Twelfth Edition." Bacteriological samples were examined for total coliform, fecal coliform, and fecal streptococcus by the membrane filter (MF) technique with the respective use of M-Endo, M-FC, and KF Streptococcus media. The organic pollution parameters measured were 5-day BOD, COD, and TOC. Analyses for TOC (Total Organic Carbon) were preformed by the Civil Engineering Department, University of Arkansas with the use of a Beckman TOC Analyzer (Model 915). The nutritional content of the samples were indicated by tests for organic Kjeldahl nitrogen and soluble orthophosphates. Measurements were also made for total solids, suspended solids, dissolved solids, volatile suspended solids, volatile dissolved solids, pH, chloride, and specific conductance.

During the period from October 1968 to September 1969, a total of 456 composite and grab samples were collected and analyzed from 30 separate precipitation events. A total of 37 baseline samples were collected on four days from the test areas that had dry weather flow.

SCHEMATIC DIAGRAM OF STORM WATER SEQUENTIAL SAMPLING EQUIPMENT

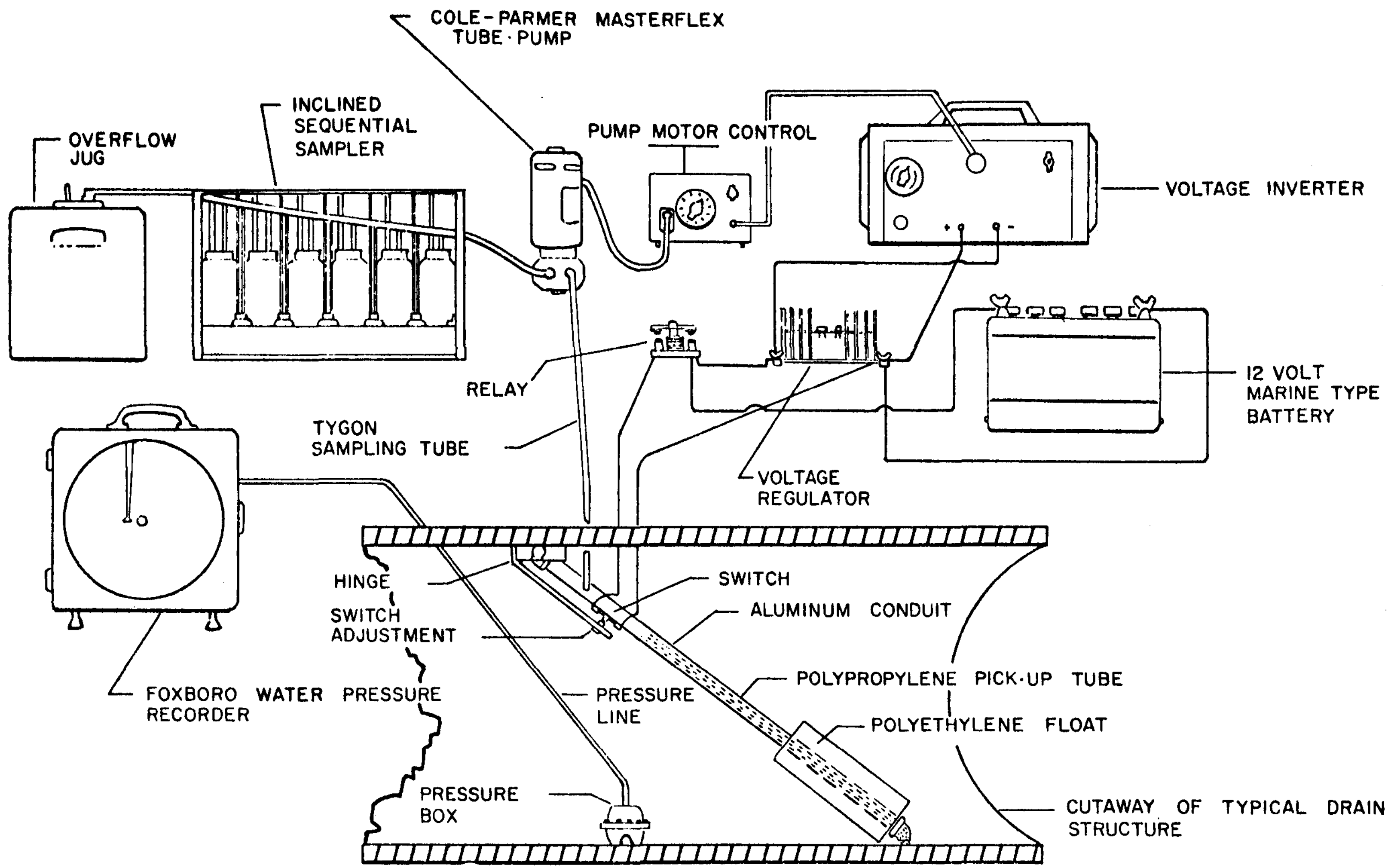

FIGURE 2.

Figure 3.--Instrument enclosure and sampling probe located at Test Area No. 3.

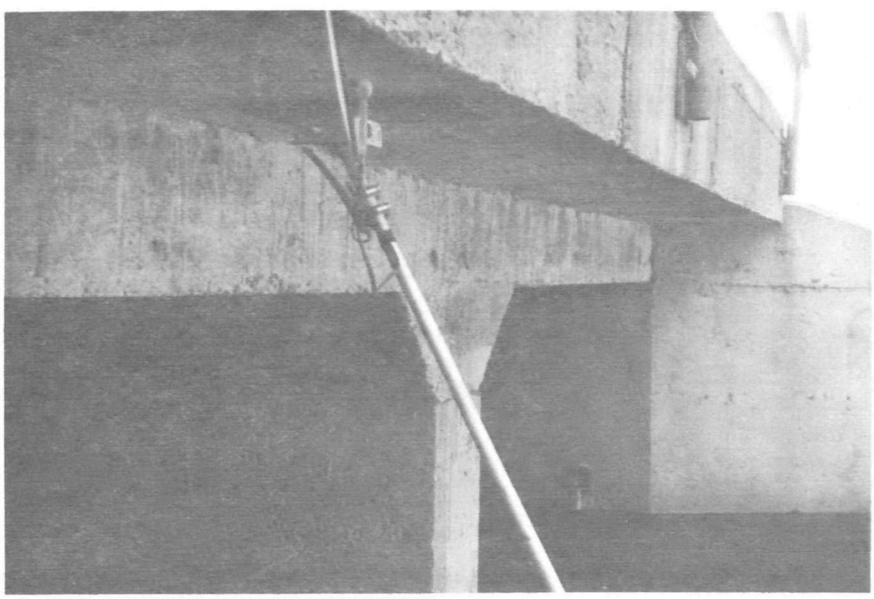

Figure 4.--Sampling probe hinge and switch.

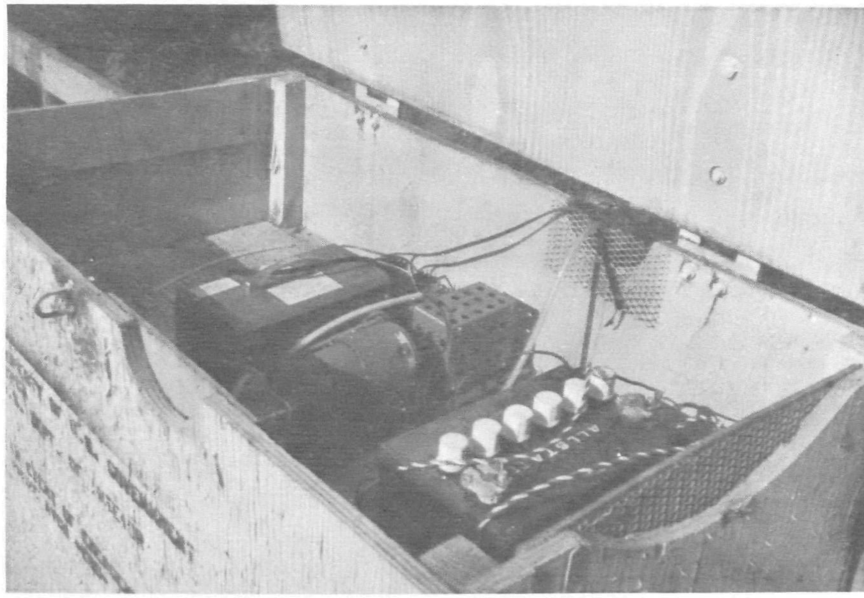

Figure 5.--Tube pump, control unit, inverter, and 12-volt battery located in top compartment of enclosure.

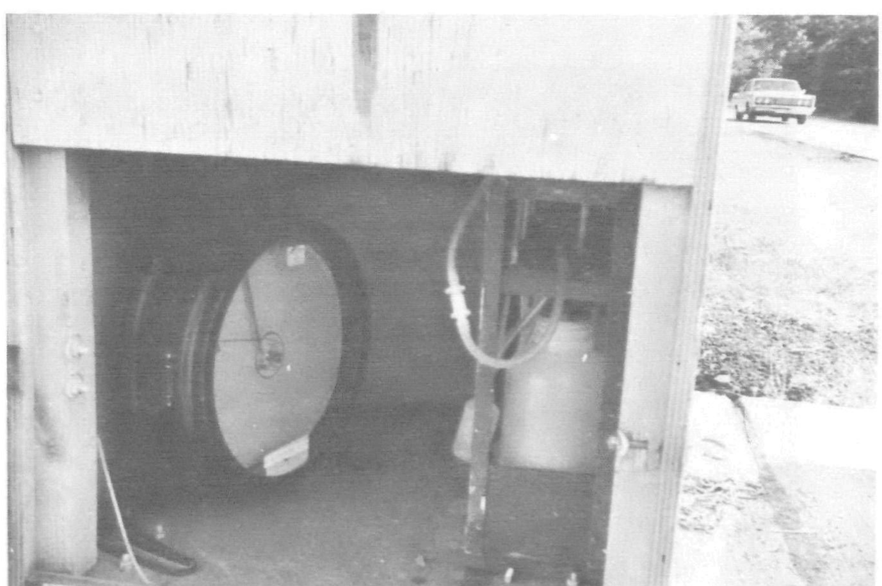

Figure 6.--Pressure recorder and inclined sequential sampler located in bottom compartment of enclosure.

ANALYTICAL RESULTS OF URBAN STORM WATER SAMPLES

This section presents the results of the analytical observations of the various pollution parameters measured throughout the testing period. These results are presented in tabular form in five pollution classifications: Bacterial, Organic, Nutrient, Solids, and Other Parameters. The results are presented as average values of the separate precipitation events and not from the averaging of the individual samples collected. This was done to more effectively compare the individual event characteristics. Since continuous sampling on each site for each event was not practicable, the averaging of the sequential samples for the sites which were continuously monitored was felt more representative for event comparison between these sites and those where only grab samples were obtained.

Bacterial

The three bacteriological parameters measured on this project were fecal coliform, total coliform, and fecal streptococcus. All samples were examined by the membrane filter (MF) technique.

The geometric means of the three bacteriological parameters measured from each test area are shown in Table 7. Below is a comparison of the arithmetic mean of the fifteen test areas with the arithmetic average of the four seasons from the Cincinnati Study (2)

Parameter	Number/100 ml.			
	Tulsa Study	Cincinnati Study		
	All Test Areas	Wooded Hillside	Street Gutters	Business District
Total Coliform	134,000	65,415	95,750	107,500
Fecal Coliform	1,940	630	13,420	14,950
Fecal Strep.	10,245	10,473	78,825	37,000

For the fifteen test areas, the fecal coliform value was, on the average, 3% of the total coliform value. The average fecal coliform to fecal streptococcus ratio varied from a low of 0.081 (Test Area No. 10) to a high of 0.893 (Test Area No. 9). These low ratios indicate the source of the bacterial pollution to be warm-blooded animals other than man (2). At the start of the project, it was suspected that Test Area No. 13 might record a high fecal coliform to fecal streptococcus ratio due to this drainage basin being unsewered and utilizing septic systems for liquid waste disposal. After checking with the authorities at the Tulsa

TABLE 7
GEOMETRIC MEAN FOR BACTERIAL DENSITY (NUMBER/100 ml) IN URBAN STORM WATER FROM 15 TEST AREAS IN TULSA, OKLAHOMA
DATES: SEPTEMBER 1968 TO SEPTEMBER 1969

Test Area No.	Land Use Classification	Total Coliform Geometric mean	Fecal Coliform Geometric mean	Fecal Streptococcus Geometric mean
1	Light Industrial	71,000	940	4,200
2	Commercial-Retail	43,000	1,900	780
3	Residential	100,000	3,300	15,000
4	Med. Ind.-Residential	25,000	770	12,000
5	Residential	150,000	1,500	3,800
6	Medium Industrial	140,000	18,000	24,000
7	Residential	32,000	120	2,300
8	Residential	240,000	450	5,800
9	Residential	400,000	290	7,600
10	Commercial (Office)	130,000	300	30,000
11	Residential-Com. Mix	370,000	620	6,800
12	Openland and Runways	56,000	10	700
13	Residential	28,000	180	5,700
14	Recreation-Golf	5,000	370	21,000
15	Residential	220,000	350	14,000

City-County Health Department, it was learned that the septic systems in this area function properly, and very few complaints have been reported from this area in regard to "pooling" of septic systems.

Test Area No. 9 a small drainage area with poor environmental conditions had the highest total coliform geometric mean (400,000 #/100 ml.). The lowest total coliform mean (5,000 #/100 ml.) was recorded from Test Area No. 14, which is a Country Club golf course. This low geometric mean may be due to the small number of sample analyses from this drainage shed. The one characteristic of this shed which distinguishes it from the other test areas is that the shed has two small recreation ponds on the drainage channel; these ponds capture almost all of the runoff water. The only time that the drainage channel flows is after or during a precipitation event of high intensity or large amount. Such an event normally occurs during the spring of the year. Due to this characteristic, the samples actually collected were from the overflow of impounded water rather than from actual runoff water.

No clear patterns were established by ranking the test areas by each separate bacteriological parameter. Patterns refer to groupings as to land activity--e.g., residential, commercial or industrial.

Organic

In general, the three organic pollution parameter concentrations cannot be considered to be high when compared to effluents from secondary sewage treatment plants. The highest average values, with the exception of Test Area No. 10, occurred from test areas with moderate to heavy tree cover. Also, all of these areas had one other common factor: the condition of the drainage channels offered many opportunities for the leaves and grass trimmings to become trapped in depressions, thus allowing an opportunity for decomposition. This condition could explain the higher average BOD values. Test Area No. 10 is a downtown commercial type drainage shed with a high percentage of impervious cover and traffic volume.

The BOD/COD ratio varied from 0.105 (Test Area No. 10) to 0.342 (Test Area No. 15). The average ratio from all fifteen sites was 0.171. The high ratio from Test Area No. 15 may be due to the small number of events sampled. Also, Test Area No. 14 is not typical, since the samples collected were not from runoff, but from overflow water from the ponds on the drainage shed.

The average BOD/TOC ratio from the fifteen test areas was 0.405. The range of values was from 0.289 (Test Area No. 1) to 0.577 (Test Area

No. 15). In general, these ratios are not useful for characterization of the test areas. The ratios show considerable variation between the test areas, and each test area has high standard deviations.

Total Organic Carbon (TOC) was measured in conjunction with BOD and COD to further characterize the test areas. It was hoped that a constant relationship could be found between samples. The TOC/COD ratio varied from 0.289 (Test Area No. 7) to 0.847 (Test Area No. 15). The average of all fifteen test areas was 0.468.

The average values of the fifteen test areas show no positive groupings. The test areas with the three highest values are each classified differently. In several instances the TOC concentrations were higher than the COD concentrations, indicating that the standard COD test does not detect some organic compounds. At present, this finding cannot be readily explained.

Table 8 summarizes the analytical results from the fifteen test areas by averages and ranges which are based on the average of the separate rainfall events.

Nutrients

Organic Kjeldahl nitrogen and soluble orthophosphate were the nutrients measured in the study. The average and range of values of these two components are shown in Table 9.

Several possibilities as to the sources of nutritional pollution can be advanced with knowledge of the present land use on some of the sites. Other sites exhibit such variation as to season, level, etc., that logical deductions as to cause cannot be made unless more complete land use information is available.

The organic Kjeldahl nitrogen measured in the runoff could have been obtained from several sources. The entrainment of organic matter by surface flows and the eluviation of decay products from organic matter are probably responsible for a large portion of the nitrogen load. Derivatives from commercial fertilizers are potential high pollution sources in the event that precipitation events occur at high intensities after these fertilizers have been applied on the land surface. Ammonia and organic nitrogen are also washed from the air at rates of 2 to 6 pounds per year (3).

A valid apportionment of the measured nutrients to these sources is not possible, and only inferences can be made. In the spring, Test Areas

TABLE 8

AVERAGE AND RANGE FOR ORGANIC CONCENTRATIONS IN URBAN STORM WATER RUNOFF FROM 15 TEST AREAS IN TULSA, OKLAHOMA

DATES: SEPTEMBER 1968 TO SEPTEMBER 1969

Test Area No.	Land Use Classification	BOD (mg/l)			COD (mg/l)			TOC (mg/l)		
		Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.
1	Light Industrial	13	23	3	110	215	54	43	71	17
2	Commercial-Retail	8	16	2	45	94	21	22	36	12
3	Residential	8	21	2	65	162	20	22	31	14
4	Med. Ind.-Res.	14	29	4	103	232	14	42	74	22
5	Residential	18	38	3	138	261	37	48	85	11
6	Med. Industrial	12	18	6	90	133	39	34	42	12
7	Residential	8	17	2	48	69	12	15	20	0
8	Residential	15	25	3	115	405	50	37	82	5
9	Residential	10	15	4	117	263	40	35	61	13
10	Commercial (Office)	11	27	4	107	240	36	28	80	0
11	Res.-Com. Mix	14	23	4	116	167	80	33	49	17
12	Openland-Runways	8	16	6	45	69	21	20	40	6
13	Residential	15	39	4	88	220	13	35	66	17
14	Recreation (Golf)	11	23	6	53	74	22	29	36	18
15	Residential	12	24	1	42	62	18	34	75	11

TABLE 9
AVERAGE AND RANGE FOR NUTRIENT CONCENTRATIONS IN URBAN STORM WATER RUNOFF FROM 15 TEST AREAS IN TULSA, OKLAHOMA
DATES: SEPTEMBER 1968 TO SEPTEMBER 1969

Test Area No.	Land Use Classification	Organic Kjeldahl Nitrogen (mg/l)			Soluble Orthophosphate (mg/l)		
		Avg.	Max.	Min.	Avg.	Max.	Min.
1	Light Industrial	1.11	2.95	0.06	3.49	15.10	1.20
2	Commercial-Retail	0.95	3.61	0.17	0.86	1.50	0.24
3	Residential	1.48	3.28	0.24	1.92	3.70	0.10
4	Med. Ind. -Res. Mix	0.97	3.03	0.00	1.05	3.00	0.36
5	Residential	0.72	1.80	0.00	0.87	1.53	0.53
6	Med. Industrial	0.65	1.50	0.16	0.86	1.40	0.58
7	Residential	0.80	1.60	0.01	0.67	1.43	0.28
8	Residential	0.69	2.52	0.00	1.15	2.60	0.00
9	Residential	0.67	1.30	0.14	1.02	1.92	0.48
10	Commercial (Office)	0.83	2.40	0.06	0.70	1.50	0.30
11	Res. -Com. Mix	0.66	1.82	0.13	1.11	1.88	0.60
12	Openland-Runways	0.39	1.26	0.01	0.54	1.68	0.20
13	Residential	1.46	5.32	0.15	1.18	1.97	0.10
14	Recreation (Golf)	0.96	2.40	0.13	0.99	2.25	0.09
15	Residential	0.36	0.98	0.15	0.81	1.17	0.35

TABLE 10
AVERAGE VALUES FOR SOLIDS FROM 15 TEST AREAS IN TULSA, OKLAHOMA
DATES: SEPTEMBER 1968 TO SEPTEMBER 1969

Test Area No.	Land Use Classification	Solids (mg/l)				
			Suspended		Dissolved	
		Total	Total	Volatile	Total	Volatile
1	Light Industrial	2242	2052	296	190	111
2	Com. -Retail	275	169	48	106	70
3	Residential	680	280	53	400	317
4	Med. Ind. -Res.	616	340	83	276	87
5	Residential	271	136	54	135	76
6	Med. Industiral	346	195	55	151	66
7	Residential	413	84	28	328	124
8	Residential	382	240	96	141	75
9	Residential	417	260	70	157	98
10	Commercial-Office	431	300	61	132	71
11	Res. -Com. Mix	575	401	95	174	83
12	Openland-Runways	199	89	24	110	59
13	Residential	469	332	85	137	73
14	Recreation (Golf)	592	445	206	147	53
15	Residential	273	183	122	89	56

2, 3, and 13 exhibit increased levels of organic nitrogen which can be attributed to fertilization of lawns within these high-income, residential areas. Other sites have high values during the fall, winter, and spring which could be assigned to products of organic decay. A decrease of this form is seen during the growing season due to the rapid assimilation of any free nitrogen by growing vegetation.

The varying amounts of orthophosphates found in the analysis of the test areas can likewise be assigned to various sources. The frequency of street sweepings; the amounts, types, and location of organic material and its decay products; the application of commercial fertilizer; the season; the number of sampled events; and the drainage characteristics can either singularly or in combination influence the washout of orthophosphates from the test site.

The presence of a concrete plant upstream from the sampling point was the prime cause of high level of orthophosphates in Test Area No. 1. Test Areas No. 3 and 13 exhibited high average orthophosphate levels which resulted from the heavy lawn fertilizations in the spring. The high maximum levels which are shown for 8 and 14 are caused by organic decay products. Test Area 12 had low orthophosphate levels due to low runoff volumes and to the lack of decidous vegetation.

If the amounts of orthophosphate washed from the test area are apportioned just to the impermeable portions of the site as shown on Table 4, Test Area 10 which is in the central business district has 4.34 pounds--the lowest annual amount per impermeable acre. This finding appears reasonable in that most of the runoff-producing portion of the streets is swept each night, and there is relatively little organic matter from vegetal sources in the drainage ways of the area. Test Area No. 2 was also low in pounds per impermeable area, but since it contained a higher percentage of residential area with its characteristic vegetation the yield was greater than from the pervious areas. The remaining areas had larger yields of orthophosphates per impervious area; this finding was attributed to the larger amounts of tree cover in these older developed areas.

Solids

The five solids constituents measured on this project were total solids (TS), suspended solids (SS), volatile suspended solids (VSS), dissolved solids (DS) and volatile dissolved solids (VDS). The arithmetic averages of these constituents are summarized in Table 10.

Total solids is the sum total of the suspended solids and dissolved solids fractions and is closely related to the topography and soil conditions of the various test areas. It should be noted that, due to the sampling techniques, total solids is not a measure of all solids found in urban storm runoff. "All solids" would be sum of total solids and the floating and large particles not picked up by the sampler used on this project. These "other solids" include such materials as tree limbs, leaves, paper, plastics, etc. These materials are not only objectionable as to the aesthetics, but indirectly add to the bacterial, organic, and nutrient storm water loads. For example, during late fall a large portion of the leaves reach the storm drainage system and become trapped in depressions within the system. Between the event that carries the leaves to the system and the next rainfall event, the leaves have time to decay and disintegrate, thus adding additional organic and nutrient contaminants to the runoff water.

The average values for the solids show considerable variation. The lowest average value (199 mg/l) was found from Test Area No. 12. The highest average value (2242 mg/l) was found from Test Area No. 1. This extremely high concentration can be explained by exposed open land. Shortly after the start of the project, construction began on a large apartment house complex. The land was stripped of its ground cover, cuts were made for streets, and water and sewer line trenches were dug. Construction continued throughout the project. Therefore, this test area is representative of a drainage basin that is under development.

The second highest average value (680 mg/l) recorded was from Test Area No. 3. This test area is a new fully developed middle-class subdivision. A large portion of the main drainage channel is open and unimproved, with unstable banks.

The percent of suspended solids varied from a low of 38% (Test Area No. 12) to a high of 82% (Test Area No. 1). The remaining test areas had percentages from 40% to 60%. The low value from Test Area No. 12 is due to the fact that the runoff comes from airport runways and is channeled to the main drainage channel by well-kept drainage ditches alongside the runways. Also, the main sources of suspended solids in fully developed residential and commercial areas are the streets, in that they collect the dust, dirt, and clay droppings from automobiles. It is interesting to note that Test Area No. 12 also had one of the four highest volatile suspended solids to total suspended solids ratio.

Generally, the volatile suspended solids followed the same pattern as suspended solids, and formed 20-50 percent of the total suspended solids. It should be remembered that high values of volatile matter in storm water may not necessarily be decomposable organic material. The relatively low BOD values found on this project support this idea, as does the fact that clay will lose considerable weight on ignition.

The average total dissolved solids ranged from a low of 89 mg/l (Test Area No. 15) to a high of 400 mg/l (Test Area 3). The overall mean of the test areas was 178 mg/l. The volatile portion of the dissolved solids averaged 49% for the 15 test areas. The range of values was from 33% (Test Area No. 4) to 62% (Test Area No. 9).

Other Parameters

In addition to the bacterial, organic, nutrient, and solids pollution parameters measured on this project, the pH, chloride, and specific conductance were measured.

The range of the average pH from the fifteen test areas varied from a high of 8.4 (Test Area No. 1) to a low of 6.8 (Test Area No. 15). All of these average values are within the State of Oklahoma's Water Quality Criteria for the Arkansas River and Verdigris River. The Criteria call for the pH to be between 6.5 and 8.5, and all values below 6.5 and above 8.5 must not be due to a waste discharge. The only observations of pH values that were higher than these limits were found from Test Area No. 1, which can be classified as a Light Industrial Area. The test area recorded a maximum pH of 12.2 on October 16, 1968. This particular sample was the third in a series of seven 30-minute composite samples, and was collected approximately 5.4 hours after the rainfall event started. All the samples collected from this test area had consistently high pH values. The only sources of land contaminants that could be found within this drainage shed were piles of cement, waste concrete, and other waste associated with a concrete batch plant operation. The batch plant is located on the bank of the unimproved open channel that drains the lower portion of this shed.

The only test area that approached the lower limit of the State of Oklahoma's pH criteria was Test Area No. 15. The average pH value was 6.8 and the lowest observed value was 6.4. The pH value of the runoff from Site 15 can be attributed to contributions from several factors. The soils of the watershed were developed under forest-like conditions found along the terraces adjoining the Arkansas River bottoms before Tulsa developed. These conditions produced soilds which were slightly acid. This area is located in a fairly old residential area, and tree cover and other vegetation levels are approaching those levels once found in the primitive state. The decomposition of vegetation both on the ground surface and in covered storm sewers of the area contributes to lower pH values in the runoff water.

Average concentrations of chloride (Cl) from the fifteen test areas varied from 2 mg/l (Test Area No. 15) to 46 mg/l (Test Area No. 7). None of these values are excessive considering the average concentrations found in the two receiving streams in Urban Tulsa. The 50% value for chloride measured in the Arkansas River at Sand Springs, Oklahoma is 970 mg/l. The average concentration found in Bird Creek is 126 mg/l.

The only samples collected which were expected to show a possible increase in concentrations were those of February 20, 1969. These samples were collected from runoff originating from melting snow. The runoff samples were from the street source areas only, since the snow had not started melting on the roofs and yard areas. The runoff can be attributed to the heavy traffic volumes on the streets. The results of these observations were very low (less than 15 mg/l).

Due to the very few snow and ice events in the Tulsa Urban Area, very limited amounts of salt are applied to the streets for snow and ice control. The main material used in the City of Tulsa for snow and ice control is sand. Due to the very limited use, the natural concentrations found in the receiving streams, and the concentration found from the fifteen test areas, the chloride (Cl) load reaching the receiving streams does not present a problem in the Tulsa Area.

The average specific conductance from the fifteen test areas varied from a low of 36 micromhos/cm to a high of 220 micromhos/cm. The mean ratios of dissolved solids to specific conductance varied from 1.19 (Test Area No. 14) to 2.54 (Test Area No. 15). The overall average of the means of the test areas was 1.579. None of the average values of the fifteen test areas deviated significantly from this mean, with the exception of Test Area No. 15. This fact tends to indicate that the dissolved substances in the runoff water from this test area are higher in organic compounds than in inorganic ions. This finding is also supported by the relatively high volatile dissolved solids to total dissolved solids ratio of 0.594. This ratio, as compared to the other fifteen test areas, was second highest.

Phenols determinations were made on samples collected on June 17, 1969 from Test Areas No. 2, 5, 6, 10 and 11. The results of these determinations are shown below:

Test Area No.	μg/l
2	14
5	18
6	10
10	35
11	18

The above five values are within the range (1-30 μg/l) as reported in the Detroit-Ann Arbor study (4). It should be noted, however, that

Test Area No. 10 recorded the highest concentrations (35 μg/l). This test area is a downtown central business district having a high percentage of streets and traffic volumes.

Since phenols are subject to rapid biochemical and chemical oxidation, they must be preserved and stored at cold temperatures if not analyzed within 4 hours after collection. Due to this requirement and to the sampling procedures used on this project, additional determinations were not made.

ESTIMATES OF STORM WATER POLLUTION LOADS FROM THE STUDY SITES

In the preceding section, the data presented was based on factual analytical observations whereas in this section the calculated pollution loadings presented are estimates. These calculations were based on valid assumptions and current data. Also, for comparison, the combined effluent loads for the four treatment plants in Tulsa are presented.

The amounts of the various pollution parameters from each site were obtained by multiplying the average values of the parameter by the estimated monthly flows. A more representative figure would have been obtained by basing the figure on the acres of imperviousness within each site. Further differentiaion was not attempted since the samples taken at each site were not from source points within the sites. Table 11 and Table 12 give the estimted average yearly loads per acre and the estimated average daily loads per mile of street from each test area, respectively. Table 13 presents the comparison between the average daily load from storm water runoff and the effluent from Tulsa's sewage treatment plants. The characteristics of these plants are:

Flat Rock (4 mgd)--primary and secondary treatment processes--secondary treatment accomplished by contact stabilization--discharge to Bird Creek.

Coal Creek (4 mgd)--primary and secondary treatment processes--secondary treatment processes accomplished with trickling filters processes--discharge to Bird Creek.

Northside (11 mgd)--primary and secondary treatment processes--secondary treatment accomplished with trickling filter processes--discharge to Bird Creek.

Southside (21 mgd)--primary treatment processes--discharge to the Arkansas River.

Considering the loading estimates presented in the tables, it is reasonable to speculate that with the continued urbanization of the Tulsa area in conjunction with the demands for increased efficiencies in domestic and industrial waste treatments facilities, storm water runoff in the Tulsa area may well become the prime source of stream pollution within the next decade.

Of greater importance is not the estimated average daily loads, but the "shock" loads of urban storm water runoff. There are an average of 52 rainfall events over 0.1 inch in Tulsa each year. Assuming each event to be equal and the yearly load to be 365 times the average daily load, each rainfall event will carry approximately seven times the

TABLE 11
CALCULATED AVERAGE YEARLY LOADS FROM
THE FIFTEEN TEST AREAS, TULSA, OKLAHOMA

Test Area No.	Acres	Pollution Load: lbs./acre/year BOD	COD	Organic Nitrogen	Soluble Orthophosphate	Total Solids
1	686	30	250	2.5	8.0	5100
2	272	27	150	3.3	2.9	920
3	550	14	110	2.6	3.3	1200
4	938	44	320	3.0	3.3	1900
5	507	33	250	1.3	1.6	490
6	368	21	160	1.1	1.5	600
7	197	15	90	1.5	1.3	790
8	211	33	250	1.5	2.5	840
9	64	20	230	1.3	2.0	830
10	206	48	470	3.6	3.1	1900
11	815	35	290	1.7	2.1	1400
12	223	25	140	1.2	1.7	630
13	212	25	150	2.4	2.0	780
14	263	12	60	1.1	1.1	660
15	74	25	90	0.8	1.7	570

TABLE 12
AVERAGE DAILY LOADS PER MILE OF STREET
FROM THE 15 TEST AREAS, TULSA, OKLAHOMA

Test Area No.	Total Street Miles	Average Load: lbs./day/mile of street BOD	COD	Total Solids	Organic Kjeldahl Nitrogen	Soluble Orthophosphate
1	11.46	4.85	41.10	838	0.41	1.30
2	7.41	2.54	15.12	92	0.32	0.29
3	14.87	1.41	11.46	120	0.26	0.34
4	28.40	3.98	29.29	175	0.28	0.30
5	16.32	2.80	21.43	43	0.11	0.13
6	12.24	1.70	12.73	49	0.09	0.13
7	6.84	1.20	7.20	63	0.12	0.10
8	6.97	2.72	20.89	69	0.12	0.21
9	3.11	1.12	13.09	47	0.07	0.11
10	12.99	2.10	20.44	82	0.16	0.13
11	49.05	1.60	13.29	66	0.08	0.15
12	3.39*	4.53	25.47	113	0.22	0.30
13	5.58	2.58	15.16	81	0.25	0.20
14	2.07	4.26	20.54	23	0.37	0.38
15	2.06	2.47	8.67	56	0.07	0.17

*Miles and Acres of Airport Runways

Table 13
Comparison Between Average Daily Load from Storm Water Runoff and Effluent from City of Tulsa's Sewage Treatment Plants

	Estimated Average Daily Load (lb/day)		
Pollution Parameter	Storm Water Runoff	Effluent from Sewage Treatment Plants[1]	Percent of Storm Water Load of Total Load
BOD	4,500	19,000	19%
COD	31,000	67,000	32%
Suspended Solids	107,000[2]	18,000	84%
Organic Kjeldahl Nitrogen	350	760	32%
Soluble Ortho-phosphate	470	11,000	4%

[1]Estimate based on 1968 flows and concentrations
[2]Estimate based on a 50% suspended solids fraction of Total solids.

average daily storm water load, which is 160% of the average daily BOD load from the treatment plants in the City of Tulsa. This load generally would reach the receiving stream in less than twenty-four hours. Such a loading of seven times the average daily load will occur on the average 52 times per year. This consideration points out the fact that any treatment facility being utilized for storm water pollution control in the City of Tulsa will be in operation only approximately 52 day per year, and the effluent from such a facility on these days will be 160% of the effluent from the sanitary sewage treatment plants.

When considered in the true context, the values of the pollution multipliers used in this section were based on a limited amount of information. The limitations emerge since the analysis was performed on a minute fraction of the flow volume taken over an infinitesimal portion of the time span in which the flow was occurring. Whether the samples were a representative mix of the multitudinous factors which contributed to the flow and pollution is unknown. It is speculation also as to whether the combined effects of these factors are reporduced over time. What is needed now is either a detailed and concentrated study on an individual urban site to thoroughly delineate the occurrence, nature, and concentration of pollutants in the storm flow so that a sound rational exists for current sampling procedures or new, versatile sampling techniques and procedures which better quantify the amounts of runoff and entrained pollutants encountered in urban situations.

At present, when compared to the ranges of concentration in the pollution parameters found in the effluents of the municipal treatment plants, the levels of pollution from storm water runoff found in the study samples are in themselves no cause for alarm except with the possible exception of the suspended solids concentrations. In newly developing areas the magnitude of the sediment loads may cause concern. In developed areas, however, the urban sediment load may be less than that found in rural watercourses.

The problem which emerges is the magnitude of the total pollutional loads which issue from an urban area. The estimates of pollution presented in this section are therefore presented as valid indicators of the pollutional loads which are generated annually on each of the study sites. The continued development of a metropolitan area such as Tulsa, and the unceasing aggregations of the pollutional loads into the drainage ways of the area point up the continued decline of a portion of the regional environment and the emergency of a problem which at present defies solution in a reasonable manner.

FINDINGS

1. By a study of 15 test areas of representative land use and environmental conditions, the average total coliform, fecal coliform, and fecal streptococcus densities were determined to vary respectively from 5,000 to 400,000 numbers/100 ml, from 10 to 18,000 numbers/100 ml, and from 700 to 30,000 numbers/100 ml.

2. The ranges of the average BOD, COD, and TOC concentrations from the 15 test areas were, respectively: 8 to 18 mg/l, 42 to 138 mg/l, and 15 to 48 mg/l. The organic pollution parameter ratios (BOD/COD and TOC/COD) and certain individual observations indicate that some organic material of storm water runoff does not show up in the standard COD test. The organic material may, therefore, include straight-chain aliphatic components, aromatic hydrocarbons, and pyridine. These components are not oxidized to any appreciable extent in the COD test.

3. The organic Kjeldahl nitrogen averages from the 15 test areas varied from 0.39 mg/l to 1.48 mg/l. The two highest averages were from residential areas of low population densities and good environmental conditions.

4. The soluble orthophosphate averages varied from 0.67 mg/l to 3.49 mg/l. The highest average value was found from a developing light industrial area containing large amounts of disturbed land. Located in the test area was a concrete batch plant which contributed to the source of phosphates.

5. The average total solids concentration for each of the fifteen test areas ranged from 199 mg/l to 2242 mg/l. The highest average value was eight to nine times greater than the average of the other 14 test areas and was a result of exposed loose subsoil from a portion of the test area that was being developed. The suspended solids concentrations averaged approximately 50% of the total solids and were ten to twenty times higher than the concentrations reported for the City of Tulsa's sewage treatment plants.

6. The average pH (8.4) from Test Area No. 1 approached the State of Oklahoma's Water Quality Criteria, and several samples exceeded the standard. The maximum recorded value from Test Area No. 1 was 12.2.

7. The average chloride (Cl) concentrations from the 15 test areas in Tulsa, Oklahoma were extremely low (2-46 mg/l) and can be considered to be of no consequence.

8. The calculated average yearly storm water pollution loads from the fifteen areas varied as follows:

Pollution Parameter	Range in pounds/acre/year	
	Low	High
BOD	12	48
COD	60	470
Organic Kjeldahl Nitrogen	0.8	3.6
Soluble Orthophosphate	1.1	8
Total Solids	470	5100

9. The calculated average daily loads per mile of street by land use were found to be:

Pollution Parameter	Range in pounds/day/mile of street		
	Residential	Commercial	Industrial
BOD	2.0	2.3	3.5
COD	14	18	28
Organic Kjeldahl Nitrogen	0.14	0.24	0.26
Soluble Orthophosphate	0.18	0.21	0.21
Total Solids	54	87	112

10. From the foregoing, it is evidently possible to estimate and predict for planning purposes storm water pollution to be expected in surface runoff from an urban area by assessment of land activity, meteorological and hydrological conditions. This will provide a very useful procedure for planning urban storm water systems and water quality management.

RECOMMENDATIONS

The recommendations presented below are based on the findings of the study and are applicable to all urban areas with separate storm drainage systems. Remedial measures and research of the nature proposed herein would reduce storm water pollution from urban areas.

Three approaches to abatement and control of the dispersed pollution load appear to be the most promising. These are: a reduction in total runoff, a reduction in the rates of runoff, and environmental policy.

1. It is recommended that structural measures be implemented to affect control within the first two areas. Examples of this type of control would be (1) devices or schemes that would eliminate or deplete runoff from rooftops, parking areas, and streets and (2) implementation of upstream retention programs for blue-green open space areas within the urban complex.

2. It is recommended that environmental controls be invoked through the enactment of:
    a. Regulations and enforcement procedures to control urban litter and general sanitary conditions of public and private areas.

    b. Performance standards in subdivision regulations for builders and contractors in reference to (1) exposing soil, (2) parcel "housekeeping" measures during and after construction, and (3) drainage practices during construction periods.

    c. Open storage regulations for commercial and industrial areas.

    d. Improved street cleaning and drainage channel maintenance practices with the primary intent of storm water pollution control rather than aesthetics or flood control.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to the Tulsa Metropolitan Area Planning Commission for assistance in locating the discrete test areas, furnishing Land Use Maps, and programming the land use computer retrieval; the City of Tulsa Engineering Department for furnishing official City Maps, use of precipitation records, and use of City property for locating the sampling equipment; and the Tulsa City-County Health Department for use of their environmental survey data.

The authors wish to acknowledge with grateful appreciation the very important original efforts of Professor George W. Reid and his experimentation at the University of Oklahoma for the helpful conceptual inputs to this research effort.

The authors also wish to thank Mr. Charles Johnston for his efforts throughout the sampling and laboratory analyses period of the study. Appreciation and a special thanks is due Mr. Gary Miessler for his assistance in the preparation of this paper.

The typing of the final manuscript by Mrs. Norma Whitworth is gratefully acknowledged.

This paper, for the most part, was based on two previous papers which were presented at the ASCE Annual and Environmental Meeting in Chicago on October 14, 1979 and the 52nd Texas Water Utilities Associations' Short School, College Station, Texas, March 2, 1970. The authors and title of these papers respectively are:

> "Storm Water Pollution from Urban Land Activity,"
> by Jerry G. Cleveland, George W. Reid and Paul R. Walters.
>
> "Storm Water Pollution from Urban Land Activity",
> by Jerry G. Cleveland and Ralph H. Ramsey.

Additional information was taken from the draft copy of final report which was prepared by AVCO Economic Systems Corporation under Contract No. 14-12-187 with the Federal Water Pollution Control Administration, Department of Interior.

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APPENDIX

From the analytical observations and the tabulated independent variables, multiple regression equations were developed to provide predictor models for estimating urban pollutant concentrations. In general, the models are not statistically significant, but they do provide a technique of estimating a possible range of values. It must be remembered that these equations were developed from data collected from drainage basins located in Tulsa, Oklahoma. In all likelihood they will not apply to all urban areas, especially in metropolitan drainage sheds which have a high degree of different land use types, environmental conditions, and drainage characteristics.

Table 14 presents a selection of the best equations by three catagories. These Catagories are: residential, commercial and industrial, and mixed. The residential models were based on the seven residential test areas. The commercial and industrial equations were based on the test areas which had a high percentage of this kind of activity. The test areas included in this analysis were Nos. 2, 4, 6, 10, and 11. The "mixed" regression equations were developed using all test areas except 12 and 14 which were considered as being non-typical urban uses. These equations can only be used successfully within the frame of reference of their development and with logical judgment of their accuracy.

Calculations with these equations using the minimum and maximum values of the observed independent variables are presented after Table 14. This is done to show the predictable range of pollutant concentrations obtained with use of the developed regression equation within the bounds of the test data. The minimum and maximum values obtained during the test are shown for comparison. Included also are examples showing the use of the equations with data from individual test areas.

TABLE 14--Continued

MIXED

Pollution Category	Inputs	Outputs	Equation	R^{2}*
Bacterial	Environmental Index (X_1) Dimensionless Covered Sewer/Total Length (X_{20}) Ratio Form Factor (D_9) Dimensionless	Total Coliform (1000/100 ml)	$M_1=565-420\ (X_1)-49.3\ (X_{20})$ $-6.70\ (D_9)$	.78
Organic	Environmental Index (X_1) Dimensionless % Arterial Streets (X_{21}) % Length of Main Stream (D_2) Feet	COD (mg/l)	$M_5=70.8-45.4\ (X_1)$ $+2.61\ (X_{21})\ +0.0062\ (D_2)$	.70
Nutrient	Residential Density (X_{17}) People/Res. Acre Covered Sewer/Total Length (X_{20}) Ratio Average Land Slope (D_6) %	Organic Kjeldahl Nitrogen (mg/l)	$M_7=0.23-0.\ (X_{17})-0.029\ (X_{20})$ $+0.256\ (D_6)$	.79
Solids	Covered Sewer/Total Length (X_{20}) Ratio % Other Streets (X_{22}) % Fall of Drainage Area (D_4) Feet	Total Solids (mg/l)	$M_9=130+8.99\ (X_{20})\ +2.59\ (X_{22})$ $+2.06\ (D_4)$	.48

*Coefficient of determination

TABLE 14

SELECTION OF BEST MULTIPLE REGRESSION EQUATIONS

	Pollution Category	Inputs	Outputs	Equation	R^2*
RESIDENTIAL	Bacterial	Environmental Index (X_1) Dimensionless Covered Sewer/Total Length (X_{20}) Ratio Form Factor (D_9) Dimensionless	Total Coliform (1000/100 ml.)	$M_1=269-309\ (X_1)-137\ (X_{20})$ $+0.580\ (D_9)$	.84
	Organic	Environmental Index (X_1) Dimensionless % Arterial Streets (X_{21}) % Length of Main Stream (D_2) Feet	COD (mg/l.)	$M_5=69-74.7\ (X_1)+3.68\ (X_{21})$ $+0.0105\ (D_2)$	.94
	Nutrient	Residential Density (X_{17}) People/Res. Acre Covered Sewer/Total Length (X_{20}) Ratio Average Land Slope (D_6) %	Organic Kjeldahl Nitrogen (mg/l)	$M_7=0.02-0.0072\ (X_{17})$ $+0.200\ (X_{20})+0.286\ (D_6)$	.79
	Solids	Covered Sewer/Total Length (X_{20}) Ratio % Other Streets (X_{22}) % Fall of Drainage Area (D_4) Feet	Total Solids (mg/l)	$M_9=-139-15.37\ (X_{20})$ $+15.98\ (X_{22})+2.57\ (D_4)$	.59

*Coefficient of determination

TABLE 14--Continued

COMMERCIAL AND INDUSTRIAL

Pollution Category	Inputs	Outputs	Equation	R^2*
Bacterial	Environmental Index (X_1) Dimensionless Main Covered Storm Sewer (X_{19}) Miles Form Factor (D_9) Dimensionless	Total Coliform (1000/100 ml.)	M_1=119-384 (X_1)-19.5 (X_{19}) -13.4 (D_9)	.90
Organic	Environmental Index (X_1) Dimensionless Main Covered Storm Sewer (X_{19}) Miles Length to Center of Area (D_3) Feet	TOC (mg/l)	M_6=3.8+4.76 (X_1) +2.10 (X_{19}) +0.0055 (D_3)	.94
Nutrient	Environmental Index (X_1) Dimensionless Covered Sewer/Total Length (X_{20}) Ratio Average Land Slope (D_6) %	Organic Kjeldahl Nitrogen (mg/l)	M_7=0.31-0.0810 (X_1) -0.0507 (X_{20}) +0.265 (D_6)	.92
Solids	Environmental Index (X_1) Dimensionless % Unused Space (X_{29}) % Fall of Drainage Area (D_4) Feet	Total Solids (mg/l)	M_9=1426-715 (X_1) +83.0 (X_{29}) -7.43 (D_4)	.78

*Coefficient of determination

Example Problems

1. Total Coliform

The multiple regression equation for Total Coliform (mixed use) is:

$M_1 = 565 - 420\ (X_1) - 49.3\ (X_{20}) - 6.70\ (D_9)$ Std. Error of Est. $= 70.2$

For an area with good environment ($X_1 = EI = 1.00$), this equation reduces to:

$M_1 = 145 - 49.3\ (X_{20}) - 6.70\ (D_9)$

The ranges of values for X_{20} and D_9 are:

Symbol	Min.		Max.	Item
X_{20}:	0.61	-	3.78	Covered Sewer/Total Length
D_9 :	0.82	-	2.85	Form Factor

At maximum values for X_{20} and D_9, M_1 becomes negative:

$M_1 = 145 - 49.3\ (3.78) - 6.70\ (2.85) = -60$

Since most values for X_{20} are somewhat smaller than the maximum, however, a negative calculated value for M_1 would probably be quite unusual.

For a bad environment (EI=0), the regression equation would be:

$M_1 = 565 - 49.3\ (X_{20}) - 6.70\ (D_9)$

Using minimum values for X_{20} and D_9, the maximum concentration would be:

$M_1 = 529$ (This compares with the highest value from the 15 test areas of 400.)

For Test Area 9, for example:

M_1=565-420 (0.23) -49.3 (1.59) -6.70 (1.47) = 380, which compares favorably with the actual value of 400.

2. COD

The COD equation (mixed use) is:

M_5=70.8-45.4 (X_1) + 2.61 (X_{21}) + 0.0062 (D_2) Std. Error of Est. = 20.7

For EI=1.00 (good environment):

M_5=25.4 + 2.61 (X_{21}) + 0.0062 (D_2)

For EI=0 (bad environment):

M_5=70.8 + 2.61 (X_{21}) + 0.0062 (D_2)

The ranges of values for X_{21} and D_2 are:

Symbol	Min.	Max.	Item
X_{21}	0	18.93	% Arterial Streets
D_2	2170	11,200	Length of Main Stream

For EI=1.00:

The minimum COD would be:

M_5=25.4 + 0.0062 (2170) = 38.9 (minimum from test sites studied: 42)

For EI=0:

The maximum COD would be:

M_5=70.8 + 2.61 (18.93)+ 0.0062 (11200) = 189.6 (maximum from test sites studied: 138)

For residential areas, the multiple regression equation is:

$M_5 = 69-74.7\ (X_1) + 3.68\ (X_{21}) + 0.0105\ (D_2)$ Std. Error of Est. =12.6

Minimum possible from data describing test areas studied:

$M_5=69-74.7\ (1.00) + 0.0105\ (2170) = 17$

Maximum possible from same data (and with EI=0):

$M_5=69 + 3.68\ (18.93) +0.0105\ (11200) = 256$

For Site 12:

$M_5=70.8 -45.4\ (1) + 2.61\ (3.94) + 0.0062\ (5710) = 60.8$ (actual value: 45)

For Site 5, a residential test area:

Mixed Use Equation:
$M_5=70.8-45.4\ (0.99) +2.61\ (3.94) + 0.0062\ (11200) = 106$

Residential Use Equation:
$M_5=69 - 74.7\ (0.99) +3.68\ (3.94) + 0.0105\ (11200) = 127$

Actual value: 138

One can conclude that this equation can be a useful predictor, even near the limits of some of the independent variables.

3. <u>Organic Kjeldahl Nitrogen</u>

The regression equation (mixed use) is:

$M_7=0.23-0\ (X_{17}) -0.029\ (X_{20}) + 0.256\ (D_6)$ (Independent of X_{17})
Std. Error of Est. =0.178

The ranges of values for X_{20} and D_6 are:

Symbol	Min.	Max.	Item
X_{20}	0.61	3.78	Covered Sewer/Total Length
D_6	0.75	4.60	% Land Slope (At D_6=0, the land slope would be at a minimum)

For D_6 =0, the equation would be:

$$M_7=0.23-0.029\ (X_{20})$$

The minimum value from this equation (at X_{20}=3.78) would be:

$$M_7=0.23-0.029\ (3.78) = 0.12$$

If there were no covered sewers (X_{20}=0), on the other hand, the nitrogen concentration would depend only upon the land slope:

$$M_7=0.23 + 0.256\ (D_6)$$

For a 4.6% land slope (maximum of test areas studied):

$$M_7=0.23 + 0.256\ (4.6) = 1.41$$

For Test Area 6:

$M_7=0.23-0.029\ (3.78) + 0.256\ (2.19) = 0.68$ actual value: 0.65

For Test Area 13:

$M_7=0.23-0.029\ (0.55) +0.256\ (4.60) = 1.39$ actual value: 1.46

For mixed land use, this regresssion equation was one of the most accurate ones obtained.

4. Suspended Solids

For commercial and industrial areas:

$$M_{12} = 1392 - 746\ (X_1) + 83.1\ (X_{29}) - 8.37\ (D_4)$$

The ranges of values for the independent variables are:

Symbol	Min.	Max.	Item
X_1	0	1.00	EI (could be <0)
X_{29}	0	24.77	% Unused Space
D_4	30	186	Fall of drainage Area

Using these limits, the minimum value for suspended solids would be:

$$M_{12} = 1392 - 746\ (1.00) + 83.1\ (0) - 8.37\ (186) = -911$$

The maximum would be:

$M_{12} = 1392 - 746\ (0) + 83.1\ (24.77) - 8.37\ (30) = 3199$ (maximum of test areas studied: 2052)

For Site 1:

$M_{12} = 1392 - 746\ (1.00) + 83.1\ (24.77) - 8.37\ (113) = 1758$ (actual value: 2052)

For Site 12:

$M_{12} = 1392 - 746\ (1.00) + 83.1\ (0) - 8.37\ (58) = 161$ (actual value: 89)

This general equation does not appear to be as useful in extreme cases as some of the other equations for different parameters.

SECTION 2

ROTARY VIBRATORY FINE SCREENING OF COMBINED SEWER OVERFLOWS

Primary Treatment of Storm Water Overflow
from Combined Sewers by High-Rate,
Fine-Mesh Screens

FEDERAL WATER POLLUTION CONTROL ADMINISTRATION
DEPARTMENT OF THE INTERIOR
CONTRACT 14-12-128

by
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Research and Development Program No. 11023 FDD

March 1970

ABSTRACT

The objective of this study was to determine the feasibility, effectiveness, and economics of employing high-rate, fine-mesh screening for primary treatment of storm water overflow from combined sewer systems.

The final form of the screening unit stands 63 inches high and has an outside diameter of 80 inches. The unit is fed by an 8-inch pipe carrying 1700 gpm (122 gal/min/ft^2) which is distributed to a 60-inch diameter rotating (60 rpm) stainless steel collar screen having 14 square feet of available screen area and a 165 mesh (105 micron opening, 47.1 percent open area). The screen is backwashed at the rate of 0.235 gallons of backwash water per 1000 gallons of applied sewage.

Based on final performance tests run on dry-weather sewage, the unit is capable of 99 percent removal of floatable and settleable solids, 34 percent removal of total suspended solids and 27 percent removal of COD. The screened effluent is typically 92 percent of the influent flow.

On the basis of a scale-up design of a 25 mgd screening facility, the estimated cost of treatment is 22 cents/1000 gallons. No finite cost comparisons were made with other treatment methods; however, when compared to conventional primary sedimentation, the selection of a screening facility as a treatment method is dependent on the value and availability of land, the design capacity of the treatment facility, the character of rainfall and runoff, and the available means of disinfection. It was observed that the proposed screening facility required 1/10 to 1/20 the land required by a conventional primary treatment plant.

This report was submitted in fulfillment of Contract No. 14-12-128 between the Federal Water Pollution Control Administration and Cornell, Howland, Hayes and Merryfield.

INTRODUCTION

NATIONAL IMPORTANCE OF STORM WATER OVERFLOWS

The majority of the existing combined sewers throughout the nation do not have adequate capacity during heavy storm periods to transport all waste and storm-caused combined flows to a treatment facility. The overflow is bypassed to a receiving stream, thus causing pollution in the nation's watercourses.

Combined sewers are designed to receive all types of waste flows, including storm water. In determining the size of the combined sewer, it has been common engineering practice to provide capacity for 3 to 5 times the dry-weather flow. During intensive storm periods, however, the storm-caused combined flow may be 2 to 100 times the dry-weather flow, making overflow conditions unavoidable. To compound the problem, most treatment facilities are not designed to handle the hydraulic load of the combined sewer and, therefore, are required to bypass a portion of the storm-caused combined flow to protect the treatment facility and treatment process from damage. The nation's treatment facilities bypass flows an estimated 350 hours during the year, or about 4 percent of the total operation time. The pollutional impact of the storm-caused combined overflow on the waters of the nation has been estimated as equivalent to as much as 160 percent the strength of domestic sewage biochemical oxygen demand (BOD). This amount creates a major source of pollution for the nation's watercourses.

The cost to physically separate the storm water from the sanitary wastes through the use of separate conduits has been estimated to be $48 billion. The development of an alternative means of treatment could conceivably reduce this cost to one-third.(1)

OBJECTIVE

The objective of this study is to determine the feasibility, effectiveness, and economics of employing high-rate, fine-mesh screens for primary treatment of storm water overflow from combined sewer systems. Prior to actual testing of the screening unit, several specific work goals were established to meet the objective. During the course of the investigation, it became apparent that some of these could not be fully met. As a result, these goals were ammended to fit the limitations of the testing facility. The specific work goals which were not met, and the changes made, are discussed in the text.

DEMONSTRATION PROCEDURE

SITE DESCRIPTION

The screening facility is located adjacent to the Sullivan Gulch pump station in Portland, Oregon. The Sullivan station serves a drainage basin of about 25,000 acres of Portland's metropolitan area, from which it pumps up to 53 million gallons a day (mgd). The drainage basin is a residential area, with about 30,000 single-family residences within its boundaries. A broad spectrum of services are available within the basin to support the population. However, the automobile related services are the most heavily represented in the drainage basin. This became visually apparent when periodic dumps of waste oil appeared at the screening facility.

PILOT PLANT OPERATION

GENERAL LAYOUT–Figure 1 illustrates the general layout of the screening facility and its relation to the Sullivan pump station. The combined sewage flow comes to the station in a 72-inch horseshoe trunk sewer. Before reaching the pump station, a portion of the flow is diverted to a bypass channel where it passes through a coarse bar screen prior to reaching the screening facility's feed pump sump. This diverted flow, which is now defined as combined sewage overflow, is lifted to the screening units by two 2100 gallon per minute (gpm) vertical turbine pumps. After passing through the screening units, the treated effluent and solids concentrate, or untreated effluent, are both returned to the trunk sewer. In an actual installation, the treated effluent will be bypassed to the receiving stream, and only the solids concentrate will be returned to the interceptor.

DESCRIPTION OF SCREENING EQUIPMENT–A perspective view of a single screening unit, as it existed in its original form, is shown on Figure 2. The unit is fed through the influent line with the feed changing direction from vertical to horizontal over the stationary distribution dome. The flow over the dome is ideally laminar. Upon leaving the dome, the flow strikes the rotating collar screen at a velocity of 5 to 15 feet per second, depending on the diameter of the influent line and the flow. The speed of the collar screen can be varied between 30 and 60 rpm by adjusting a variable drive unit at the 1/2 horsepower drive motor. Depending on the velocity of the feed, and the fineness, condition, and speed of the collar screen, approximately 70 to 90 percent of the feed will penetrate the screen. The remaining 10 to 30 percent, with the retained solids, drops onto the vibrating horizontal screen for further dewatering. The dewatered solids, through the vibrating action of the horizontal screen, migrate toward the center of the screen where they drop through an opening in the screen to a solids discharge pipe. This solids flow is returned to the interceptor sewer and subsequently to a sewage treatment plant. The screened flow is discharged to a receiving water body as treated effluent.

FIGURE 1

EXPERIMENTAL PILOT PLANT
SULLIVAN GULCH PUMP STATION
PORTLAND, OREGON

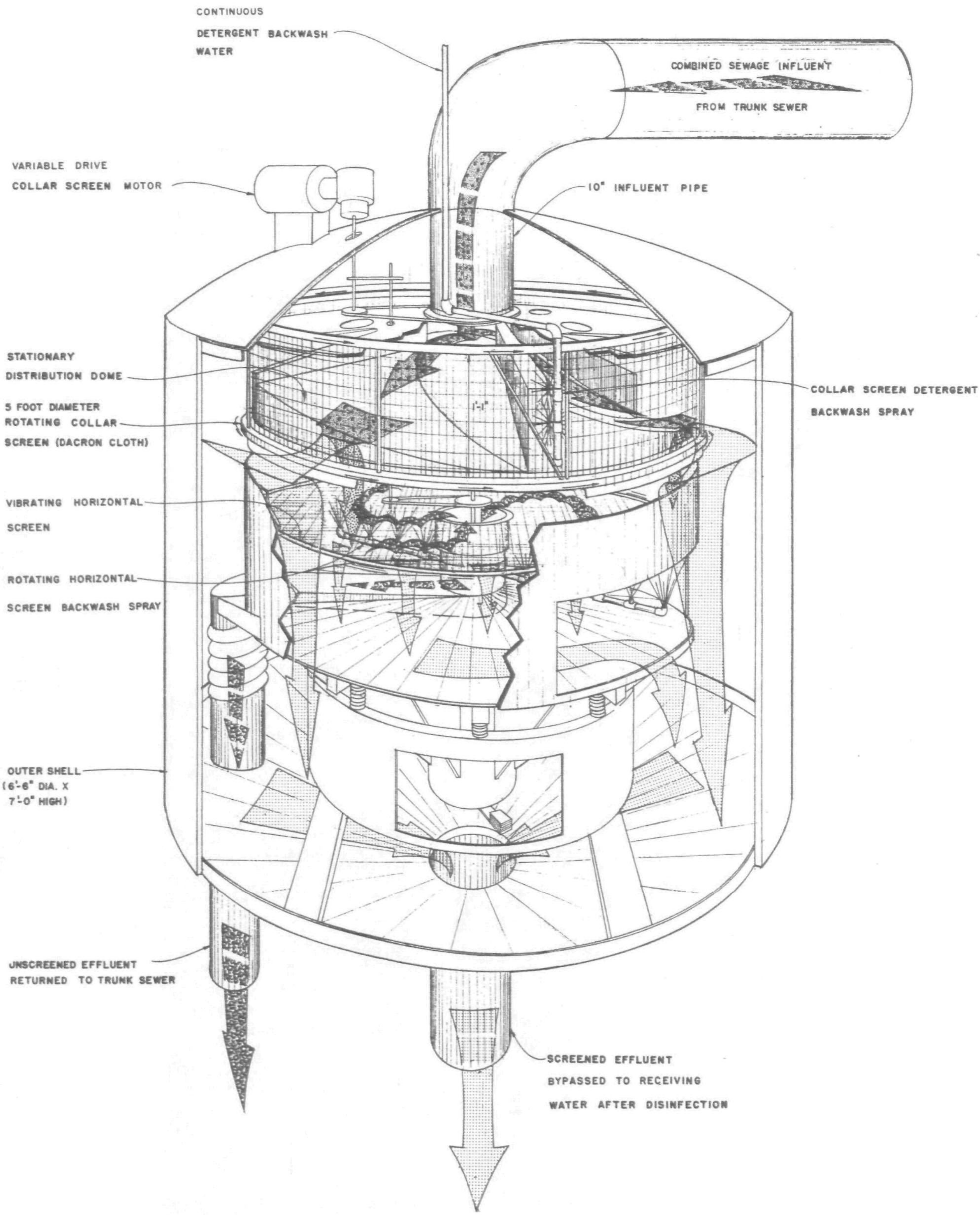


FIGURE 2

ORIGINAL SCREENING UNIT

ASSOCIATED EQUIPMENT–The screens are continuously cleaned with a solution of hot water and concentrated household detergent. The wash water is heated to approximately 170 degrees F. with a gas-fired, commercial water heater. The detergent is injected into the hot water piping by a 10 gpm positive displacement pump. The detergent is diluted about 800:1 at the spray nozzles, and is discharged at a rate of 1.8 gpm per nozzle at a pressure of 50 pounds per square inch (psi). The collar screen has two stationary nozzles directed at the outside of the screen, and the horizontal screen has four nozzles mounted on a rotating bar directed at the underside of the screen.

OPERATION OF SCREENING FACILITY–A specific goal of this study was to perform all test runs during storm-caused combined sewage conditions. However, after approximately one-third of the testing was accomplished, the rainy season came to an end and the project was faced with a possible delay. To avoid this possible one-year delay, it was decided to complete the study using dry-weather flow. In making this decision, it was assumed that the differences between dry-weather flow and storm-caused flow were not great enough to affect the objective of this study.

SAMPLING TECHNIQUE AND FREQUENCY–When the screening operation began, it was observed that the character of the waste frequently changed in concentration and color over very short periods of time. This was expected, and it was a specific goal to detect and characterize these changes with a grab sampling technique. During the course of the investigation, however, it became desirable to minimize the very short-term interferences associated with the variability of the sewage so that the long-term performance of the unit could be evaluated. To do this required composite sampling.

During the testing program, the duration of any one test ranged from a minimum of one hour to a maximum of twelve hours. In most tests, composite samples were collected every hour, with each composite consisting of three grab samples of equal volume collected in the middle of each one-third of that hour. The flow rate to the unit during any one test was constant. It was this type of composite sampling that was used to evaluate the long-term effectiveness of the screening unit, and also to obtain a general and representative description of the sewage being applied to the unit.

Grab sampling was used to describe the more unusual constituents of the sewage that affected the short-term performance of the screening unit. These unusual constituents and their affect on performance, were noted and are discussed in the text.

OBSERVATIONS–A schematic diagram of the screening facility, the process streams sampled, and the observations made on each stream are shown on Figure 3.

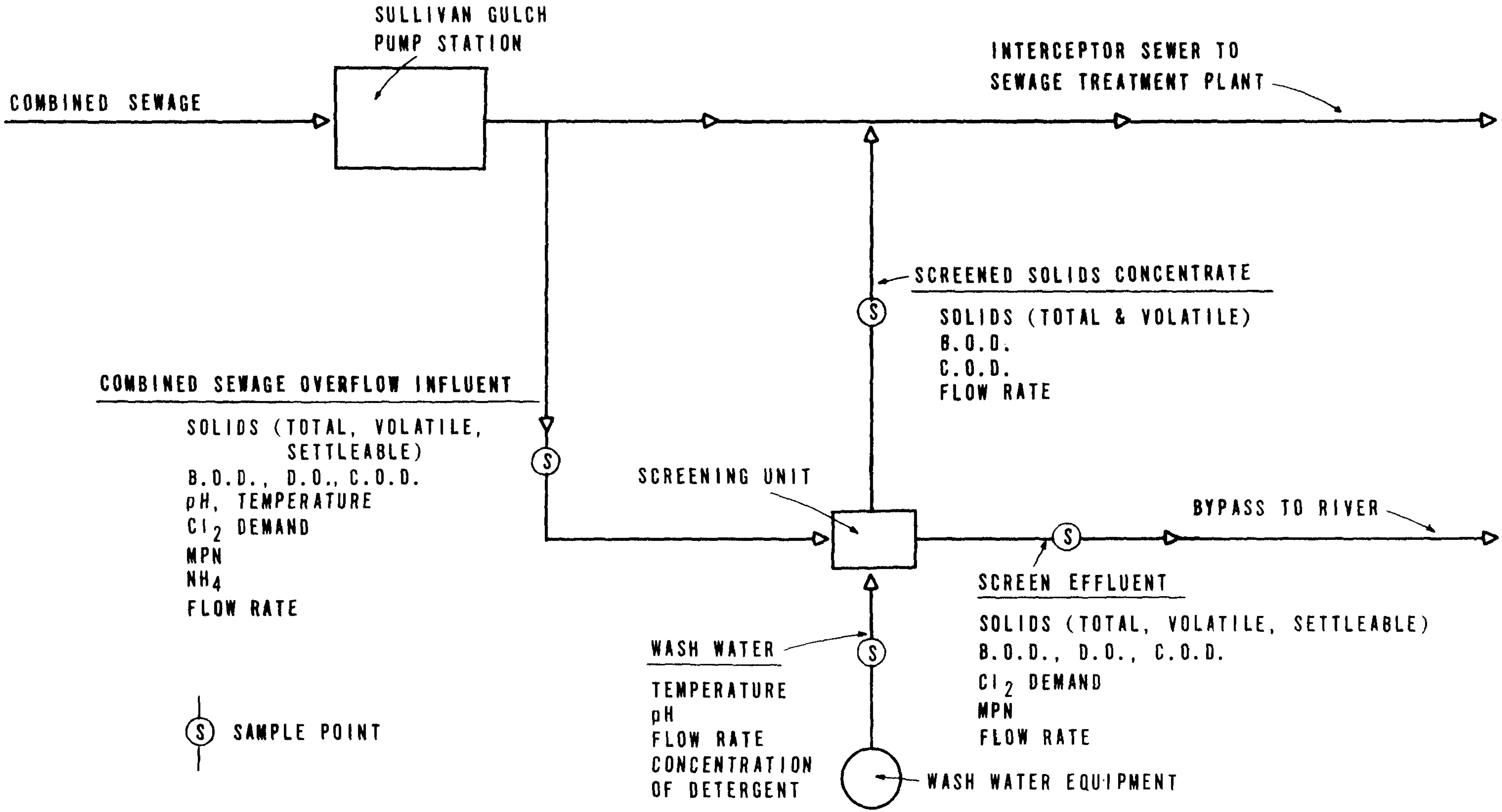


FIGURE 3

COMBINED SEWAGE OVERFLOW SCREENING
SAMPLING PROGRAM

All laboratory tests were performed according to *Standard Methods*[2] with the exception of COD. All samples, except settleable solids, were blended in a Waring blender prior to analysis to improve the precision of the results. Settleable solids determinations were made by the Imhoff cone procedure.

The COD test was performed according to the "rapid method" as described by Dr. John S. Jeris in the May 1967 issue of "Water and Wastes Engineering." The rapid method COD test made routine collection of organic strength data very reliable because it minimized the possibility of loss of data, which may have been experienced if only the 5-day BOD test was performed.

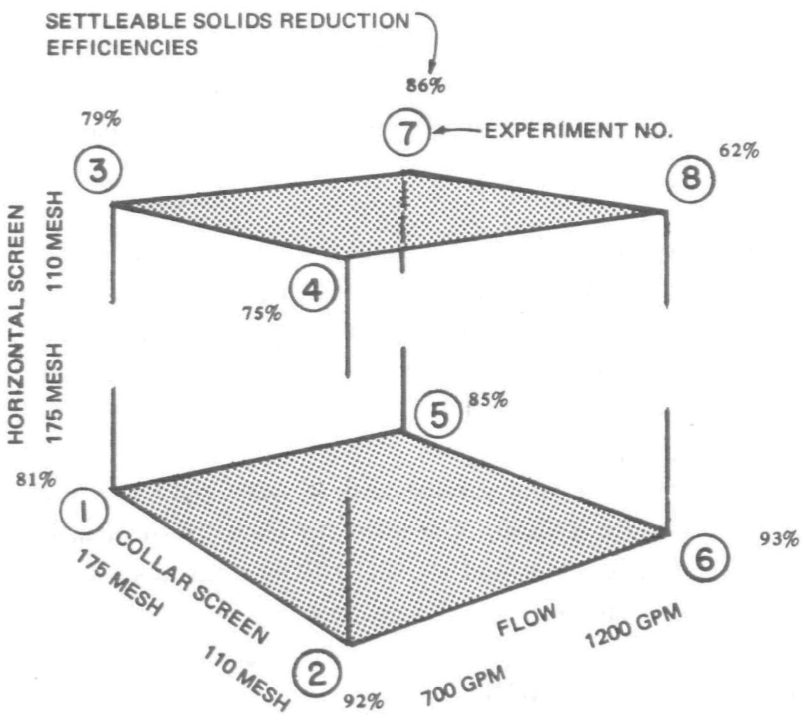
During the initial stages of the testing program, parallel tests of BOD and COD were performed on all process streams to establish a BOD/COD ratio for each stream. During subsequent tests, only the rapid COD test was run and the BOD/COD ratio was used to provide a BOD value when this appeared desirable.

EXPERIMENTAL DESIGN AND DATA REDUCTION

EXPERIMENTAL DESIGN–During startup of the screening unit, several variables were noted in its construction and operation that would affect its performance. These included influent flow rate, the velocity at which the feed strikes the collar screen; rotational speed of the collar screen; mesh size and material of the collar and horizontal screen; duration and frequency of the backwash; and type of detergent used in the backwash. With this many variables, a means of experimentation was required that would efficiently evaluate the relative influence each variable had on the overall performance of the unit. This required an experimental procedure which could investigate several variables simultaneously, and reveal what the exact effect of each variable was on the performance of the unit.

To accomplish this, a form of factorial experimental design was used for each investigation of the testing program. Figure 4 illustrates the initial experiment, which was designed to investigate the three variables that, at the time, were believed to have the most effect on performance. This experiment design is statistically termed a 2^3 Factorial Design, Multiple Response Experiment, which means that two levels of three variables are simultaneously investigated. If all combinations are tested, the experiment requires eight test runs. Under these particular set of conditions, the experiment can be visualized as a cube in which each corner of the cube represents a unique combination of the variables to be tested.

At the completion of the experiment, a cursory evaluation can be made by plotting any one, or all, of the responses observed at their respective positions on the cube. In most cases, the observer can immediately determine, by visual inspection, which of the three variables is contributing the most and/or least to the particular response observed.



RESPONSES

1. SETTLEABLE SOLIDS REMOVAL
2. TSS REMOVAL
3. VSS REMOVAL
4. B.O.D. REMOVAL
5. DURATION OF TEST RUN
6. CONDITION OF SCREEN
7. SOLIDS CONTENT OF SCREENINGS.
8. HYDRAULIC CAPACITY

EXP. NO.	RUN* NO.	COLLAR SCREEN	HORIZONTAL SCREEN	FLOW (GPM)
1	5	175	175	700
2	3	110	175	700
3	7	175	110	700
4	8	110	110	700
5	2	175	175	1200
6	4	110	175	1200
7	1	175	110	1200
8	6	110	110	1200

*TEST RUNS ARE RANDOMIZED TO MINIMIZE EFFECT OF A TIME TREND WHICH MAY EXIST DURING TESTING PERIOD.

FIGURE 4

EXPERIMENTAL DESIGN AND DATA REDUCTION

DATA REDUCTION—While in most cases a visual interpretation of the data is sufficient during the early stages of an investigation, the limitations of the eye are soon realized. A mathematical method is used to further inspect the data.

In reference to Figure 4, the effect that any one variable has on a particular response is calculated by subtracting the average of the four observations at the lower level of the variable **from** the average of the four observations at the higher level of the variable. For example, the observed reductions in settleable solids of the first experiment are plotted at their respective positions on the experimental diagram of Figure 4. The following calculation was made to determine the effect that changing the horizontal screen from 175 (105 micron opening, 52 percent open area) to 110 mesh (150 micron opening, 42 percent open area) had on the efficiency of settleable solids reduction.

$$\text{Average of higher level (110 mesh)} = \frac{79 + 75 + 86 + 62}{4} = 76$$

$$\text{- Average of lower level (175 mesh)} = \frac{81 + 92 + 85 + 93}{4} = 88$$

$$\text{Effect} = -12 \text{ percent}$$

From this calculation, one can conclude that: "When the horizontal screen was changed from 175 mesh (105 microns) to the coarser 110 mesh (150 microns), the settleable solids reduction efficiency was decreased by 12 percent, from 88 percent to 76 percent."

Using the same calculation for the collar screen variable and influent flow rate variable, the results of the first experiment for settleable solids reduction efficiencies can be summarized as follows:

Variable	Effect On Settleable Solids Reduction
Changing horizontal screen from 175 to 110	Decreased 12 percent
Changing collar screen from 175 to 110	Decreased 2 percent
Changing flow rate from 700 gpm (50 gal/min/ft^2) to 1200 gpm (86 gal/min/ft^2)	None

From this summary, one can conclude that the size of the horizontal screen most affects settleable solids removal, and the flow rate applied to the unit least affects settleable solids removal. If the next experimental design was based on only these results, a finer horizontal screen would be selected to obtain better results. Likewise, since increasing the flow rate to 1200 gpm (86 gal/min/ft^2) had little effect on the performance, it would also be natural to try a higher flow rate, since this would increase the hydraulic capacity of the unit. This type of analysis and reasoning was applied throughout the testing program; however, for any one experiment, several responses were evaluated before a change was made in the variables. A review of all the evaluations, collectively, provided most of the information necessary to evaluate the overall performance of the unit and to modify the unit to improve its performance.

INVESTIGATIONS

The chronology of the investigations, and the clarifying data, will be discussed in this section. Information of a more analytical nature will be found in Appendix C.

CHARACTERIZATION OF COMBINED SEWAGE OVERFLOW–Several composite samples were taken from the trunk sewer during storm periods for the purpose of characterizing storm-caused combined sewage. A summary of results is presented in Table 1.

TREATMENT CAPABILITIES OF SCREENING UNIT–Several levels of the known variables were tested. The results of these tests led to several equipment modifications in the course of developing the screening unit as it now exists. A list of the known variables, the range at which each was tested, and the level at which the best results occurred are presented in Table 2. The evolution of the screening unit from its original form to its present form is illustrated in Figure 5.

The major modifications included removing the vibrating horizontal screen, improving the backwash procedures, selecting an effective detergent, changing the screen materials and reducing the size of the influent pipe to increase the velocity of the feed striking the screen.

TABLE 1

SUMMARY OF CHARACTERIZATION OF COMBINED SEWAGE

SULLIVAN GULCH PUMP STATION

PORTLAND, OREGON

FEBRUARY -- APRIL, 1969

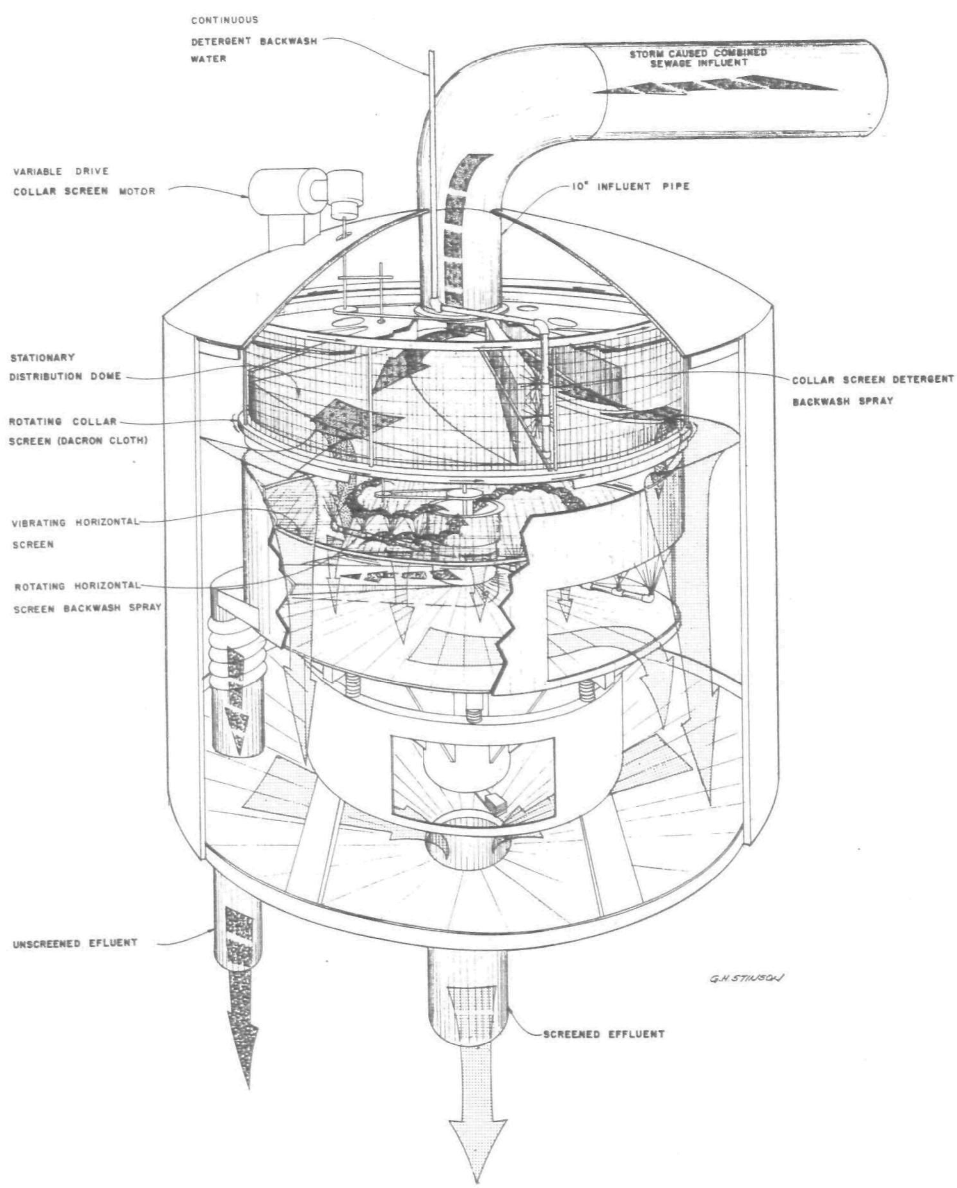
CHARACTERISTIC	NUMBER OF OBSERVATIONS	MEAN	STANDARD DEVIATION	MINIMUM	MAXIMUM
PH	26	5.0	+ .4	4.5	6.0
TEMPERATURE, °F	25	48.7	+ 6.5	34.0	56.0
DISSOLVED OXYGEN, MG/L	16	8.0	+ 2.2	3.7	10.4
SETTLEABLE SOLIDS, ML/L	25	3.1	+ 1.0	1.5	5.0
TOTAL SUSPENDED SOLIDS, MG/L	28	146	+ 59	70	325
VOLATILE SUSPENDED SOLIDS, MG/L	28	90	+ 25	57	166
% VOLATILE SUSPENDED SOLIDS	28	67	+ 17	36	93
B.O.D., MG/L	14	105	+ 36	57	155
C.O.D., MG/L	24	199	+ 50	138	324
B.O.D./C.O.D.	14	.51	+ .08	.35	.64
AMMONIA NITROGEN, MG/L	7	5.1	+ 1.4	3.7	7.0
ORGANIC NITROGEN, MG/L	7	8.2	+ 3.1	5.10	14.0
TOTAL NITROGEN, MG/L	7	13.3	+ 4.3	9.5	21.0

TABLE 2

RANGE AND LEVEL OF VARIABLES TESTED

VARIABLE	RANGE INVESTIGATED	LEVEL OF BEST PERFORMANCE
HORIZONTAL SCREEN MESH SIZE	110 (150 MICRON OPENING) TO 175 (105 MICRON OPENING)	REMOVAL OF HORIZONTAL SCREEN
COLLAR SCREEN MESH SIZE	105 (167 MICRON OPENING) TO 230 (74 MICRON OPENING)	165 (105 MICRON OPENING, 47.1% OPEN AREA)
COLLAR SCREEN MATERIAL	DACRON CLOTH, MARKET GRADE STAINLESS STEEL FABRIC, TENSILE BOLTING CLOTH.(1)	TENSILE BOLTING CLOTH
COLLAR SCREEN ROTATIONAL SPEED	30 RPM TO 60 RPM	60 RPM
INFLUENT FLOW RATE	700 TO 2000 GPM	1700 GPM
COLLAR SCREEN HYDRAULIC LOADING	50 GAL/FT^2/MIN. TO 143 GAL/FT^2/MIN.	122 GAL/FT^2/MIN.
VELOCITY OF FEED WATER STRIKING COLLAR SCREEN	3 TO 12 FT/SEC.	11 FT/SEC.
TYPE OF OPERATION	INTERMITTENT TO CONTINUOUS	4½ MIN. ON, ½ MIN. OFF FOR BACKWASH
BACKWASH RATIO (GAL. BACKWASH WATER/1000 GAL. APPLIED WASTE)	.200 GAL/1000 GAL. TO 25.6 GAL/1000 GAL.	.235 GAL/1000 GAL.

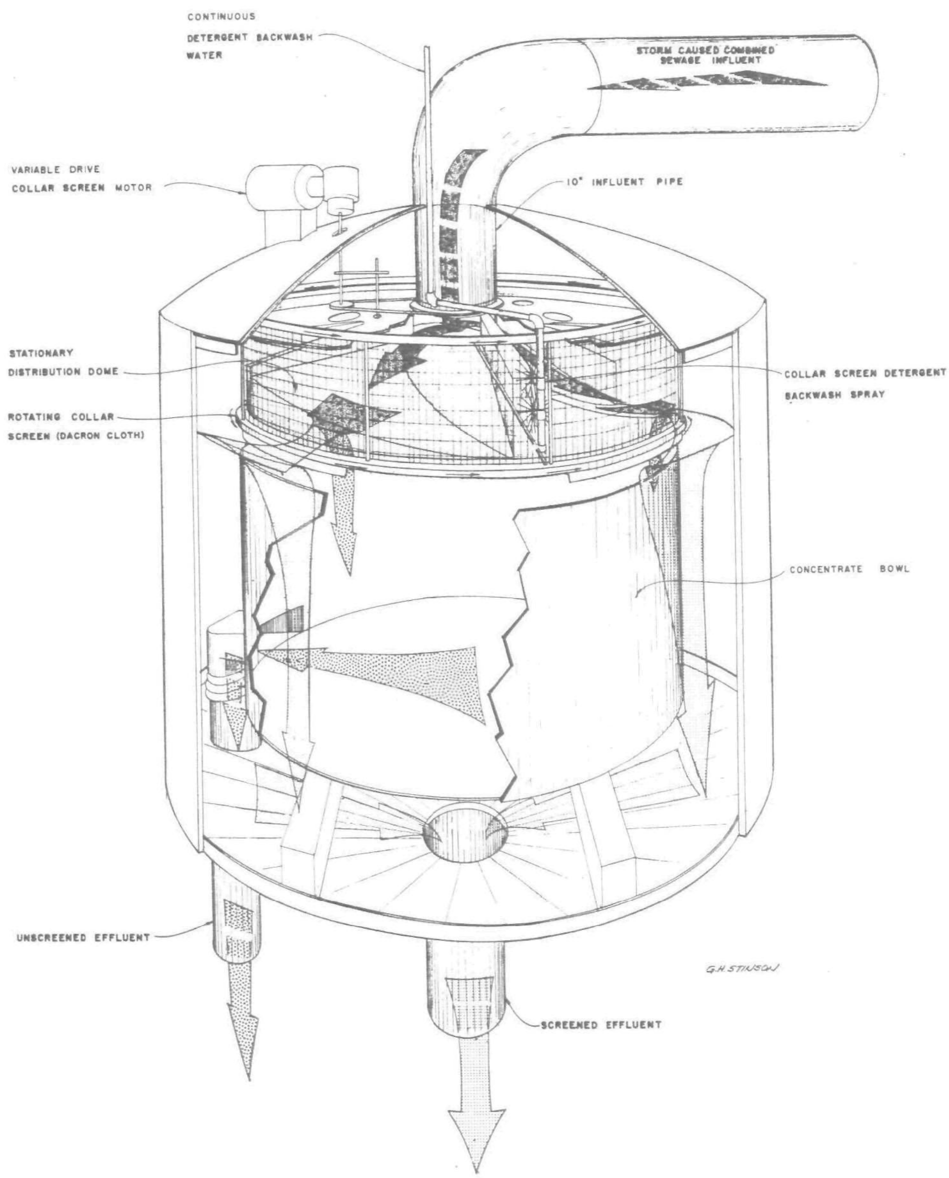
(1) SEE APPENDIX A FOR SCREEN SPECIFICATIONS.



ORIGINAL FORM

OPERATING CONDITIONS:		
INFLUENT FLOW RATE	–	50 TO 86 GPM/FT^2
COLLAR SCREEN SPEED	–	30 RPM
COLLAR SCREEN	–	105 TO 150 MICRON OPENING DACRON CLOTH
HORIZONTAL SCREEN	–	105 TO 150 MICRON OPENING DACRON CLOTH
BACKWASH RATIO	–	12.0 TO 20.6 GAL./1000 GAL.
PERFORMANCE:		
SETTLEABLE SOLIDS REMOVAL	–	62% TO 93%
T.S.S. REMOVAL	–	10% TO 26%
C.O.D. REMOVAL	–	5% TO 13%
SCREENED EFFLUENT AS % OF INFLUENT	–	99.99%

Figure 4

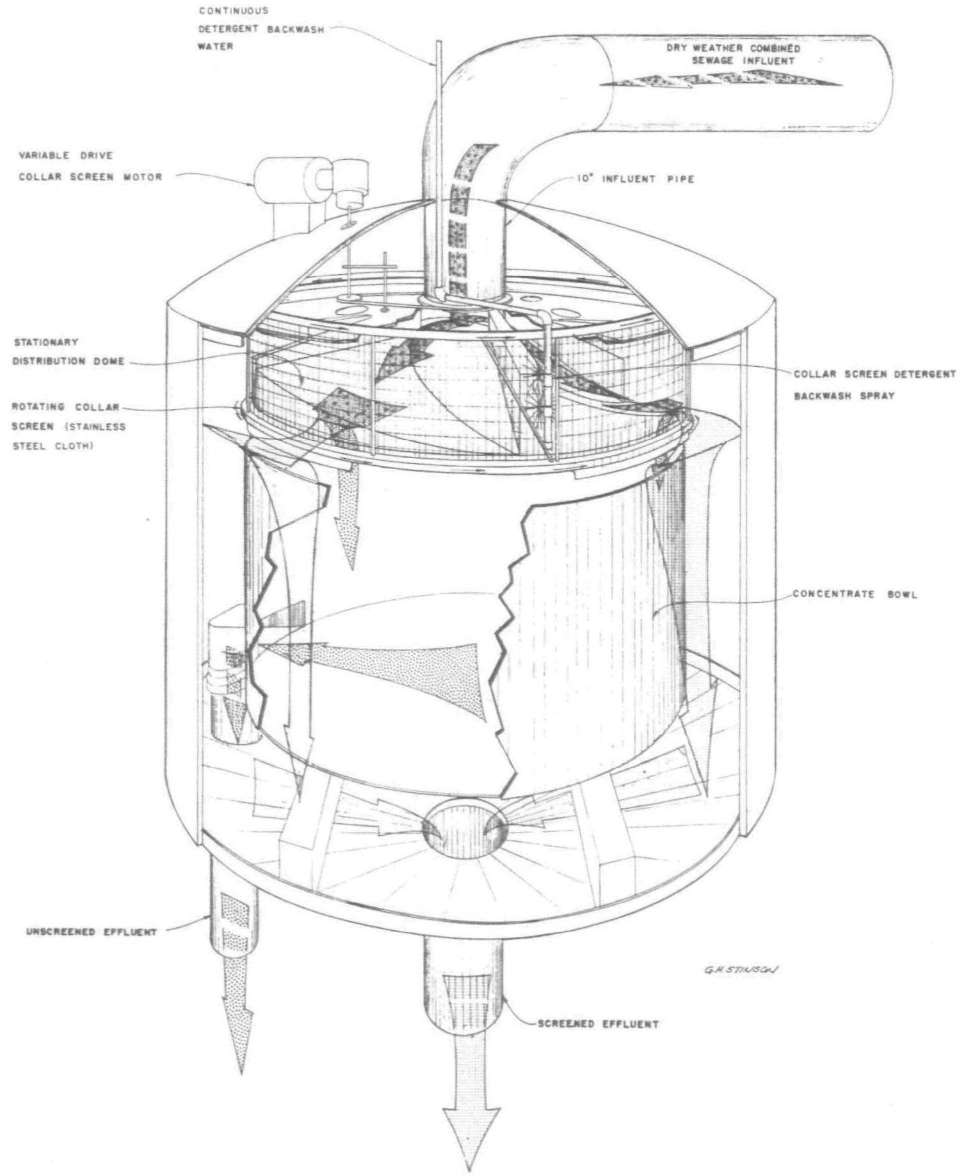


MODIFICATION 1

REMOVE HORIZONTAL SCREEN

OPERATING CONDITIONS:		
INFLUENT FLOW RATE	–	50 TO 86 GPM/FT^2
COLLAR SCREEN SPEED	–	30 TO 45 RPM
COLLAR SCREEN	–	105 TO 150 MICRON OPENING DACRON CLOTH
BACKWASH RATIO	–	3.0 TO 5.1 GAL/1000 GAL.
PERFORMANCE:		
SETTLEABLE SOLIDS REMOVAL	–	48% TO 90%
T.S.S. REMOVAL	–	18% TO 25%
C.O.D. REMOVAL	–	10% TO 18%
SCREENED EFFLUENT AS % OF INFLUENT	–	65% TO 81%

Figure 4 (cont.)

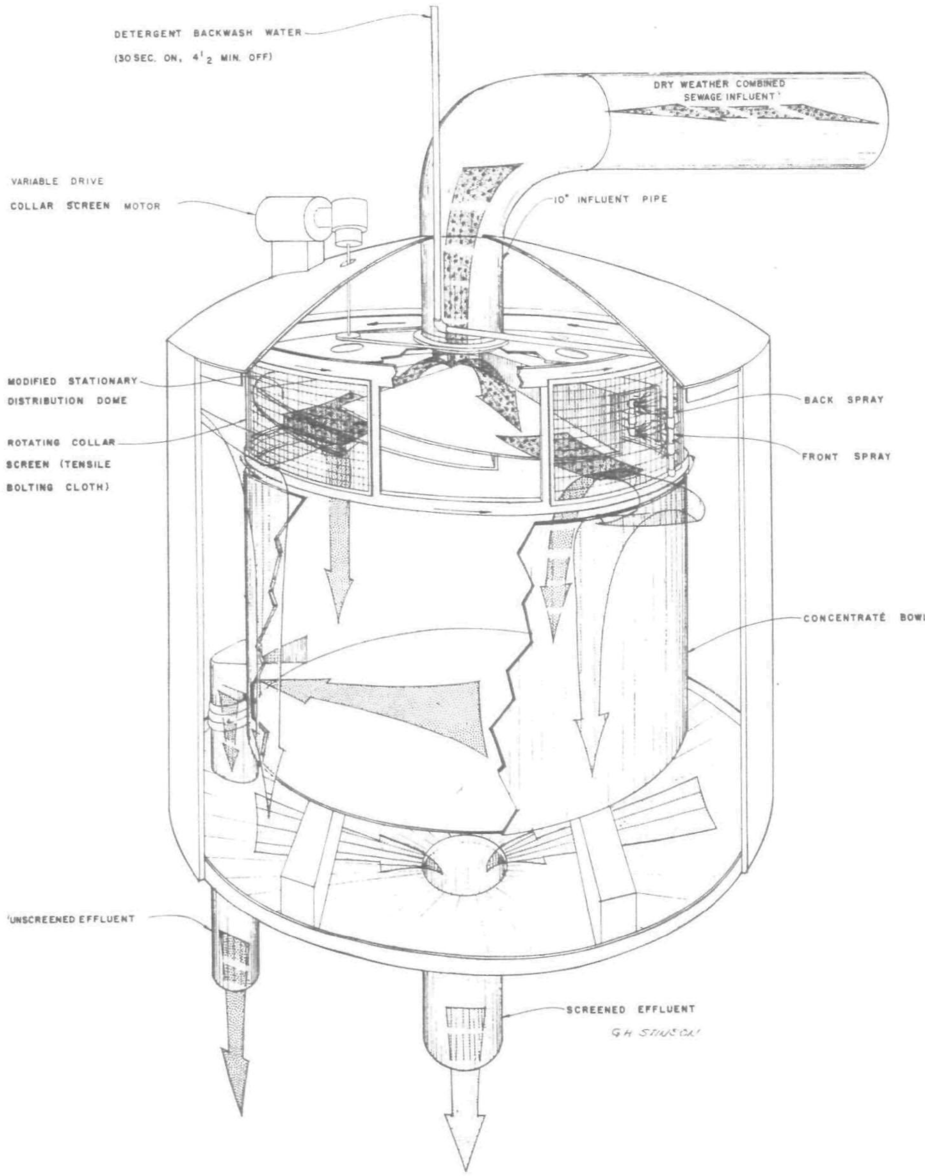


MODIFICATION 2

DRY–WEATHER COMBINED SEWAGE FEED
STAINLESS STEEL COLLAR SCREEN

OPERATING CONDITIONS:		
INFLUENT FLOW RATE	–	50 TO 86 GPM/FT^2
COLLAR SCREEN SPEED	–	30 TO 60 RPM
COLLAR SCREEN	–	74 TO 105 MICRON OPENING MARKET GRADE STAINLESS STEEL FABRIC
BACKWASH RATIO	–	3.0 TO 5.1 GAL/1000 GAL.
PERFORMANCE:		
SETTLEABLE SOLIDS REMOVAL	–	92% TO 100%
T.S.S. REMOVAL	–	11% TO 34%
C.O.D. REMOVAL	–	6% TO 13%
SCREENED EFFLUENT AS % OF INFLUENT	–	46% TO 74%

Figure 4 (cont.)

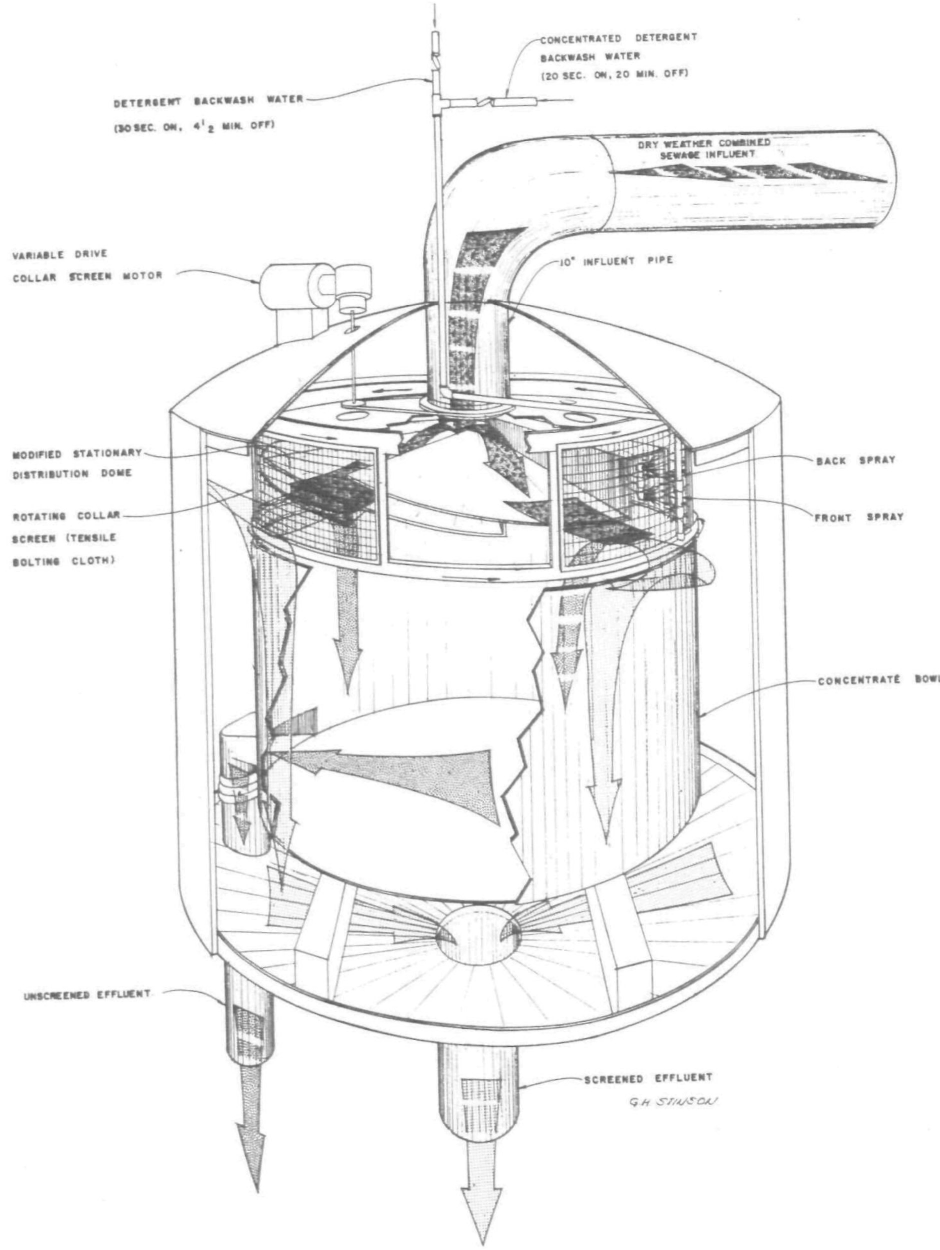


MODIFICATION 3

MODIFIED DISTRIBUTION DOME
ADDITION OF BACK SPRAY
MODIFIED BACKWASH PROCEDURE
IMPROVED COLLAR SCREEN MATERIAL

OPERATING CONDITIONS:		
INFLUENT FLOW RATE	–	100 TO 114 GPM/FT^2
COLLAR SCREEN SPEED	–	30 TO 60 RPM
COLLAR SCREEN	–	167 MICRON OPENING TENSILE BOLTING CLOTH
BACKWASH RATIO	–	.50 TO .57 GAL/1000 GAL.
PERFORMANCE:		
SETTLEABLE SOLIDS REMOVAL	–	70%
T.S.S. REMOVAL	–	7%
SCREENED EFFLUENT AS % OF INFLUENT	–	74% TO 80%

Figure 5



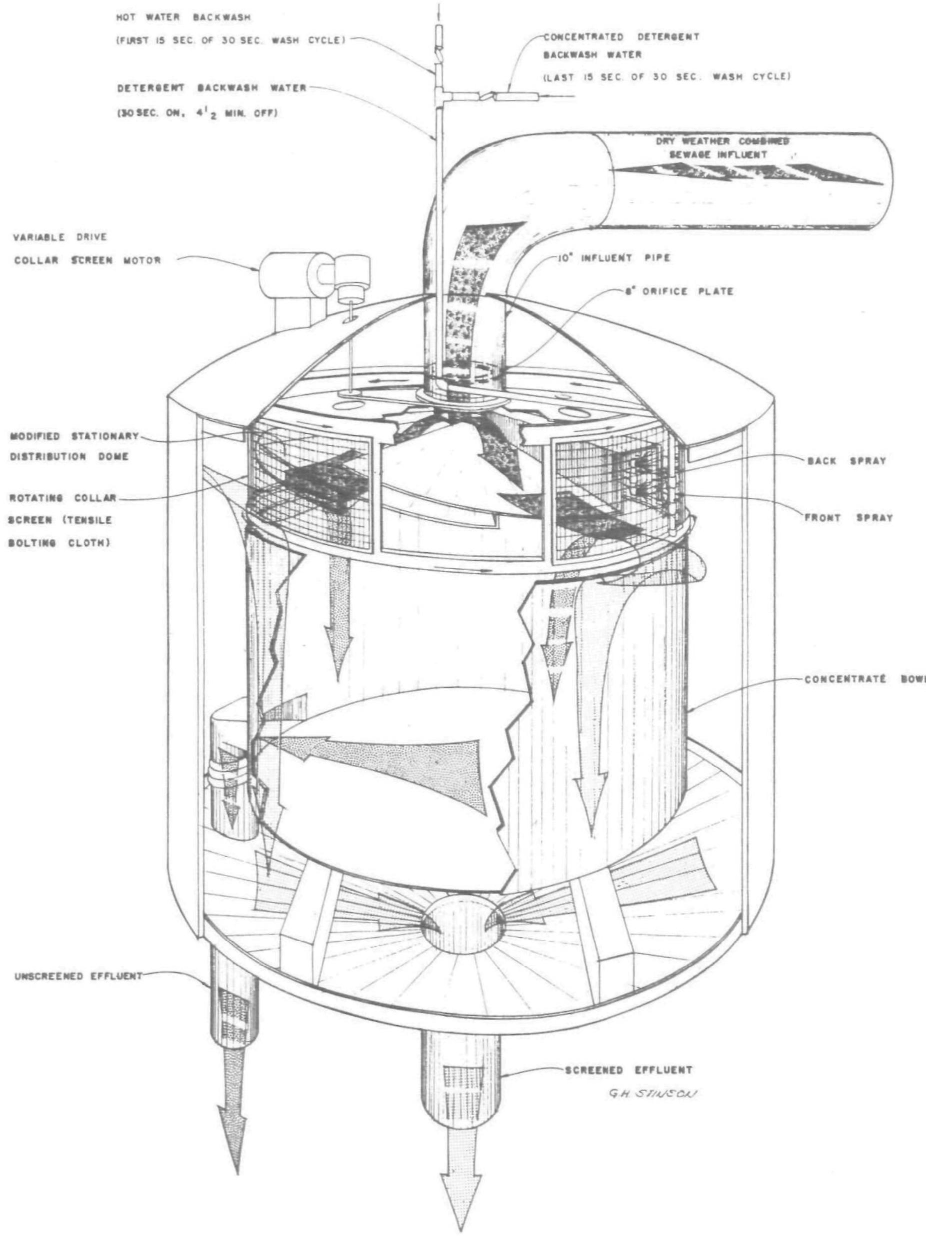
MODIFICATION 4

ADDITION OF CONCENTRATED DETERGENT
BACKWASH WATER

OPERATING CONDITIONS:		
INFLUENT FLOW RATE	–	100 GPM/FT^2
COLLAR SCREEN SPEED	–	60 RPM
COLLAR SCREEN	–	167 MICRON OPENING TENSILE BOLTING CLOTH
BACKWASH RATIO	–	.25 GAL/1000 GAL.
PERFORMANCE:		
SETTLEABLE SOLIDS REMOVAL	–	90%
SCREENED EFFLUENT AS % OF INFLUENT	–	90%

Figure 5 (cont.)

DEVELOPMENT OF A HIGH–RATE
FINE–MESH SCREENING UNIT



FINAL FORM

ADDITION OF ORIFICE PLATE
MODIFIED BACKWASH PROCEDURE

OPERATING CONDITIONS:		
INFLUENT FLOW RATE	–	122 GPM/FT^2
COLLAR SCREEN SPEED	–	60 RPM
COLLAR SCREEN	–	105 MICRON OPENING TENSILE BOLTING CLOTH
BACKWASH RATIO	–	.235 GAL/1000 GAL.
PERFORMANCE:		
FLOATABLE SOLIDS REMOVAL	–	100%
SETTLEABLE SOLIDS REMOVAL	–	98%
T.S.S. REMOVAL	–	34%
C.O.D. REMOVAL	–	27%
SCREENED EFFLUENT AS % OF INFLUENT	–	92%

Figure 5 (cont.)

DISCUSSION OF RESULTS

CHARACTERIZATION OF COMBINED SEWAGE

A summary of the characterization of storm-caused combined sewage was presented in Table 1. This characterization was based on the average of several composite samples collected during the early stages of the test program. The composite samples consisted of three grab samples collected over a one-hour period during a test run. Composite sampling was used in lieu of discreet sampling to obtain a more representative description of the sewage being applied to the screens over an extended period of operation. A review of the characterization did not reveal any unusual constituents in the sewage that could affect the long-term operation of the screening unit.

During this period of characterization, however, it was observed that there were several unusual constituents in the sewage which markedly reduced the short-term effectiveness of the screening unit. These include waste oil dumps, waste paint dumps, and the cleanup wastes associated with a fish packing plant. All of these waste dumps were of high concentration, low frequency and short duration, and significantly reduced the hydraulic capacity of the screening unit by their presence. When these constituents were encountered, grab samples were collected and analyzed.

The waste oil dump appeared about 3:00 p.m. every day and lasted for a period of approximately five to ten minutes. The oil was present in sufficient concentration to turn the sewage to a black color. The waste paint dumps were less frequent occurring only once or twice a week about the same time of day. The duration of the paint's presence was about the same as the oil and was also of sufficient concentration to change the color of the sewage. In the case of the paint, it was either a brilliant red or green. Both of these waste dumps also had a strong volatile odor associated with them.

The dump from the fish packing plant was observed a total of five times and each time for a period of approximately 15 minutes. No color change was noticeable by its presence. However, a strong odor of decayed fish made its presence known. The pH of the sewage during this period was 8.5, considerably above the normal of 5.0.

In each of these waste dumps, the hydraulic capacity of the screening unit was significantly reduced through grease-blinding of the collar screen. If the screens were not backwashed during this period, the hydraulic capacity was reduced to a point where only 40 percent of the feed would pass through the screen, down from the normal 80 to 90 percent passing the screen. After the waste dump would pass, the screens would not recover until they were backwashed. When the screens were backwashed during the waste dump flows, the reduction in hydraulic capacity was minor.

As previously discussed, it became necessary to complete a major portion of the testing with dry-weather sewage for the lack of storm-caused combined sewage. The dry-weather sewage was characterized in the same manner as the storm-caused combined sewage. A comparison of the two sets of data are included in Table 3. For all practical purposes, the two wastes are similar in character with regard to the affect they have on the long-term performance of the screening unit. The short-term reductions in hydraulic capacity, however, were more severe under dry-weather sewage conditions than under wet-weather sewage conditions.

TREATMENT CAPABILITIES OF SCREENING UNIT

The performance of the screening unit is ultimately evaluated by its ability to remove organic material from a wastewater stream, and by the volume of wastewater that it can process. These performance parameters are directly dependent on variables within the screening unit. The mesh size of the screen, the strength of the screen, the velocity at which the feed strikes the screen, and the backwash operation are among the most important variables. The final experiment, which was designed with these variables in mind, clearly defined the capabilities and limitations of the screening unit.

The final experiment consisted of six 3-hour tests. Each was performed on a different day. Four of the six tests investigated two levels of influent flow rate and screen-mesh size. The remaining two tests were duplicated at the intermediate levels to obtain an estimate of the day-to-day variances in operating the unit and in the character of the feed water. The tests at the intermediate levels also helped to interpret the final results. The design of the final experiment and the observations during the experiment are presented on Figure 6.

An examination of all the observations reveals that each response is dependent on both the flow rate and the mesh size of the screen. No response is completely independent of either flow rate or mesh size; however, the unit's efficiency in removing organic material is more dependent on the screen-mesh size than on the flow rate. The dependency of removal efficiency on screen-mesh size was expected. If a finer screen is installed on the unit, one could expect higher removal efficiencies. Other variables, however, tend to bias this dependency. In most instances, as the flow rate was increased, slightly poorer removal efficiencies were observed. It is believed the higher flow rates are fracturing the more friable solids at the surface of the screen and forcing them through the screen. The slight reduction in removal efficiency observed at the higher flow rate, however, is more than offset by the increase in hydraulic efficiency.

The hydraulic efficiency, as measured by the percentage of screened effluent and the condition of the screen, also shows a very strong interdependence on flow rate and screen-mesh size. As seen on Figure 6, the best hydraulic efficiency and most stable performance occurs at the higher flow and coarser screen condition. The hydraulic

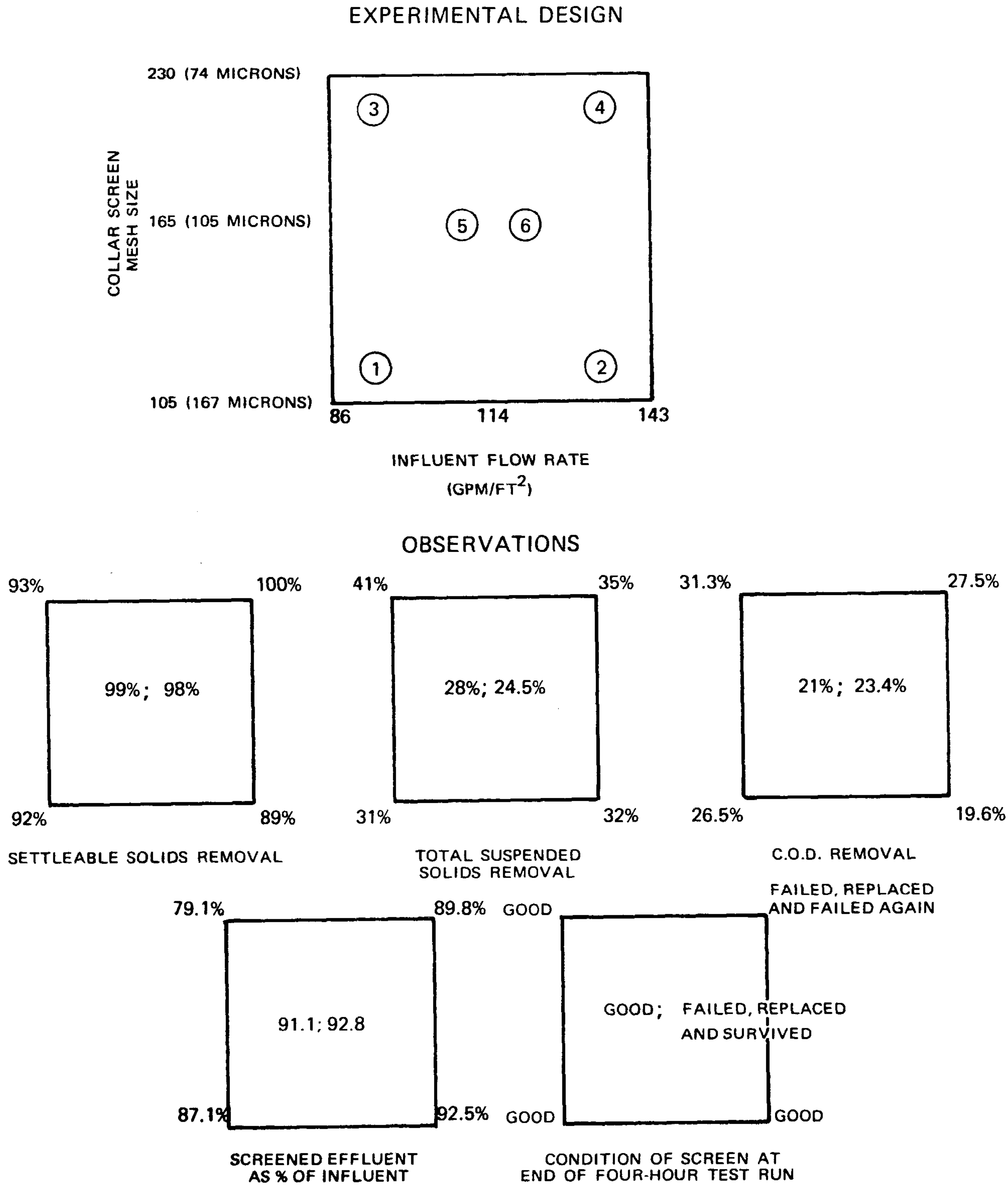


FIGURE 6

EXPERIMENTAL DESIGN AND OBSERVATIONS OF FINAL EXPERIMENT

TABLE 3

COMPARISON OF STORM – CAUSED COMBINED FLOW AND DRY–WEATHER FLOW

CHARACTERISTIC	STORM–CAUSED COMBINED FLOW					DRY–WEATHER FLOW				
	NUMBER OF OBSERVATIONS	MEAN	STANDARD DEVIATION	MIN.	MAX.	NUMBER OF OBSERVATIONS	MEAN	STANDARD DEVIATION	MIN.	MAX.
SETTLEABLE SOLIDS, ML/L	25	3.1	± 1.0	1.5	5.0	35	4.8	± 1.1	2.5	7.0
TOTAL SUSPENDED SOLIDS, MG/L	28	146	± 59	70	325	35	129	± 44	50	244
C.O.D., MG/L	24	199	± 50	138	324	25	345	± 138	144	696

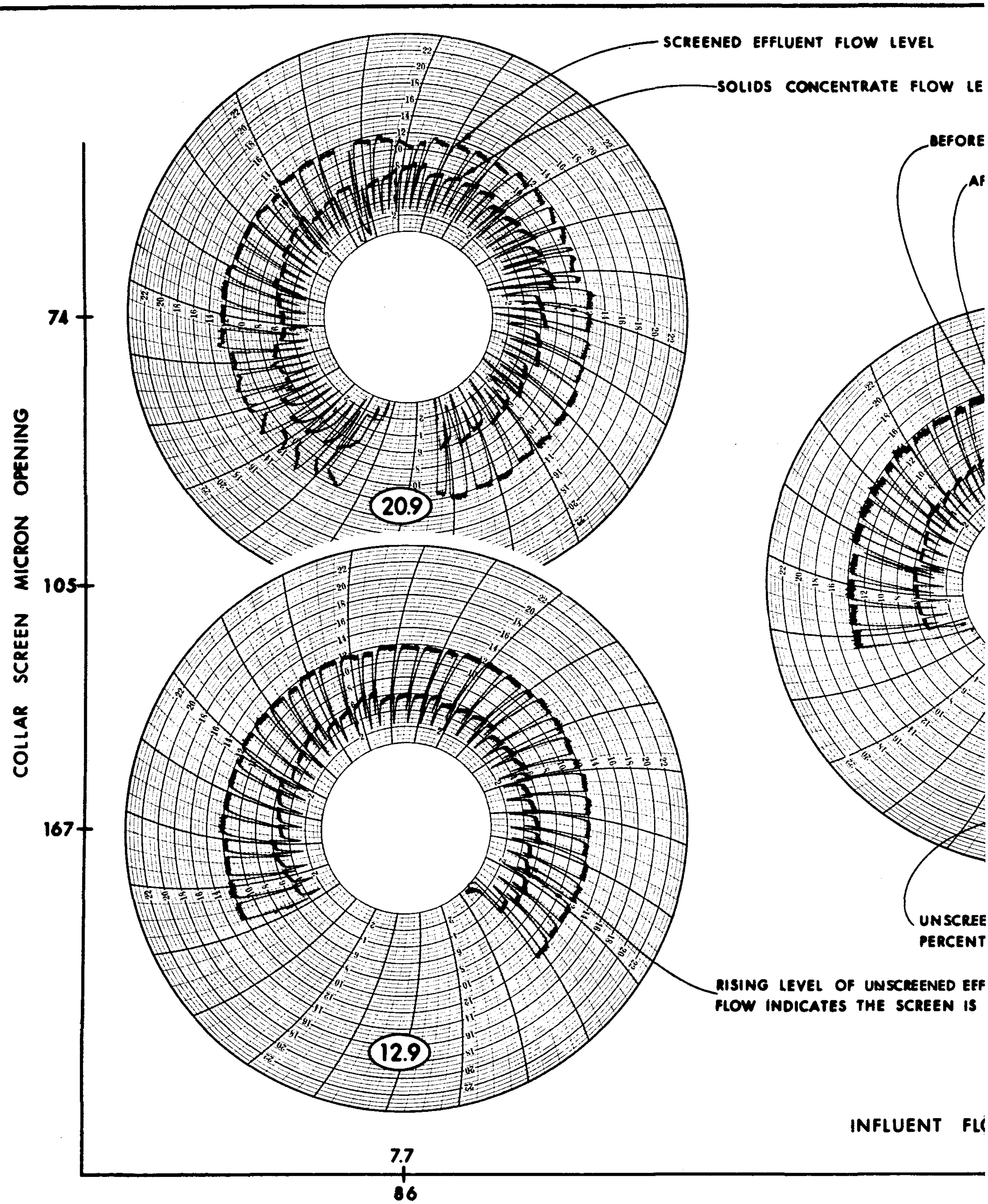
efficiency declines as both the flow rate decreases and the screen becomes finer. This is illustrated more vividly on Figure 7, where the actual flow recorder charts are displayed at their respective positions on the experimental design. The graphs were generated continuously by a four-hour flow recorder that pneumatically sensed the head over a 90-degree V-notch weir. The screened effluent flow and the unscreened flow were recorded simultaneously. The total influent flow was found by summation. The graphs are discontinuous because the screening unit was shut off for the backwash cycle.

For this final series of tests, the screening unit was operating 4-1/2 minutes on and 1/2 minute off. During the 1/2 minute, the flow was shut off and the screens were backwashed with an 800:1 dilution of hot water and liquid detergent. At the end of a 20-minute cycle, the flow was shut off, and the screens were backwashed with a 10:1 dilution of water and liquid detergent. The distinction between the two backwash cycles is easily seen on the flow charts. Frequent backwashing is necessary, as seen on the flow charts, at the 1200 gpm (86 gal/min/ft^2) flow level by the rapidly rising level of the unscreened flow graph. This need for backwashing diminishes at the higher flow level, and therefore the frequency of backwashing could have been reduced. Further examination of the flow charts shows that the flow rate, or velocity of flow, to the various screen-mesh sizes has a significant effect on hydraulic efficiency and performance stability.

High velocities and flow rates are limited, however, by the strength of the screen. Figure 6 shows that the 165 mesh screens (105 microns, 47.1 percent open area) started failing at 1600 gpm (114 gal/min/ft^2). Failure of the 230 mesh screen (74 microns, 46.0 open area) was persistent at 2000 gpm (143 gal/min/ft^2). Screen life is also approximated on Figure 7 by the relative length of chart run. The photographs on Figure 8 illustrate typical screen failures.

The failure of the steel screens was attributed to the tremendous live load applied to the screens during high-flow conditions. The forces contributing to the failure include the velocity head of the flow striking the screen, the centrifugal forces associated with the rotation of the screen, and the mass of water carried along on the inside of the screen. By calculating the velocity head and G-force at 2000 gpm and 60 rpm, and assuming a thickness of water on the inside of the collar screen, the equipment supplier found that the steel wires of the screens were stressed beyond their yield point soon after the 2000 gpm (143 gal/min/ft^2) was applied.

A failure of this kind was termed a mechanical failure, and the situation was corrected to a degree in reducing the effective live load on the screen by reducing the unsupported span of the screen. Recent developments in extending screen life by the equipment supplier have produced a 165 mesh screen (105 micron opening) that now has a probable life of 500 hours when operated at 1750 gpm (128 gal/min/ft^2). If operated at 2500 gpm (178 gal/min/ft^2), the probable life will drop to 300 hours.



OPERATING CONDITIONS: 4½ MINUTES ON, ½ MINUTE OFF FOR NORMAL BACKWASH, AND ½ MINUTE CONCENTRATED DETERGENT BACKWASH EVERY 20 MINUTES.

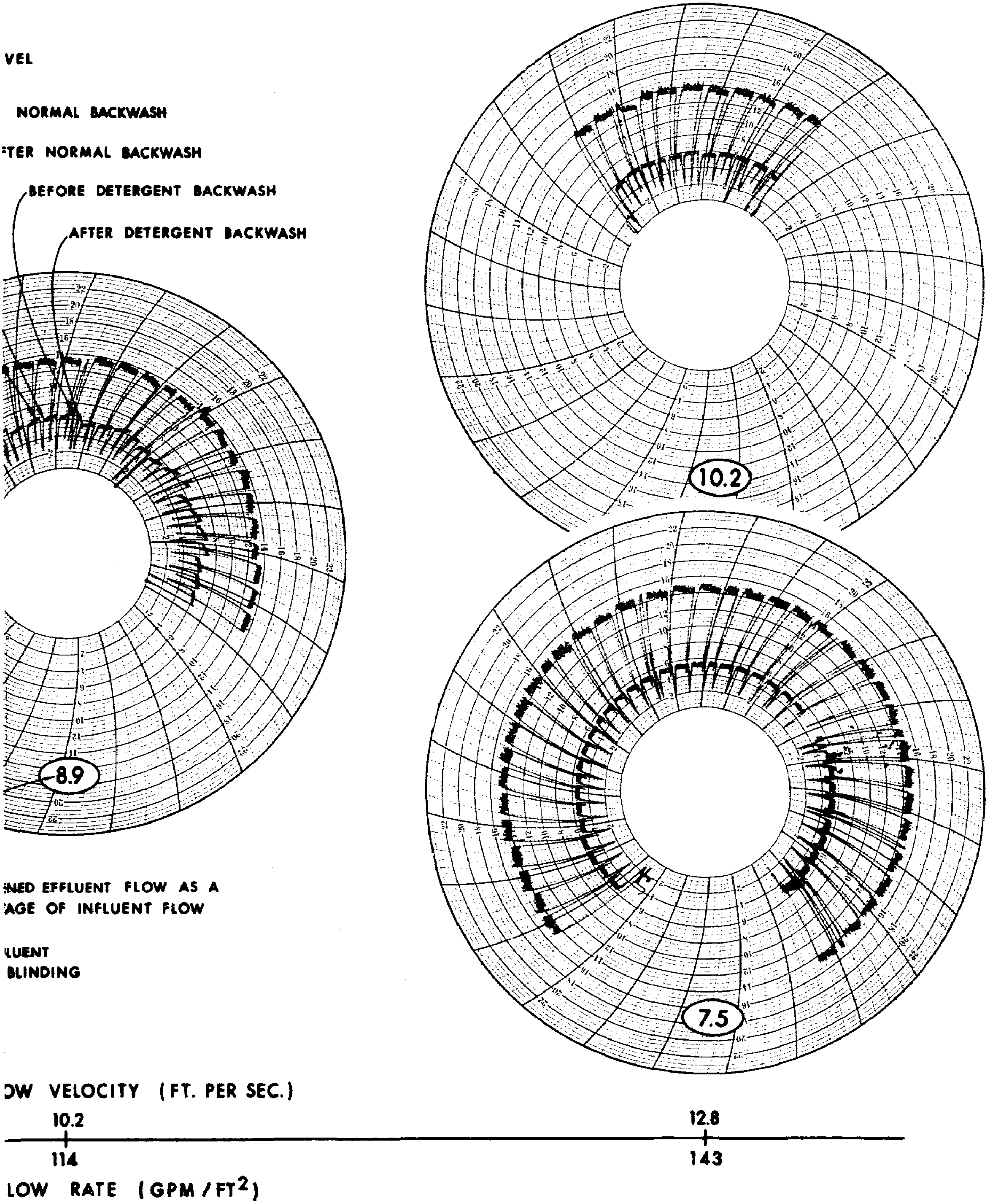
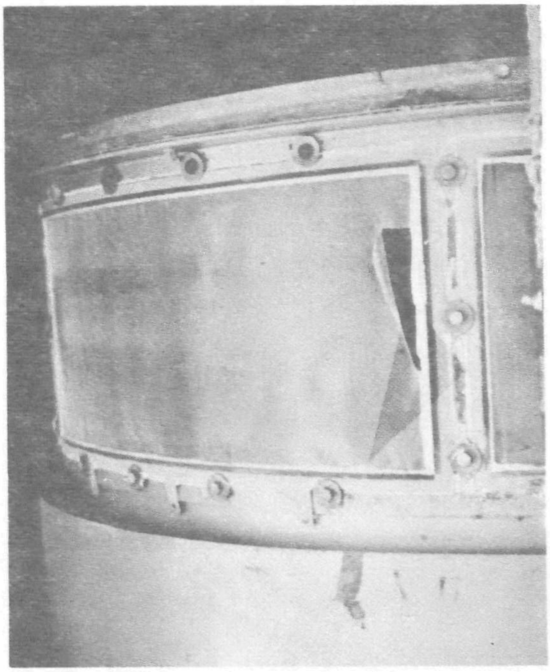
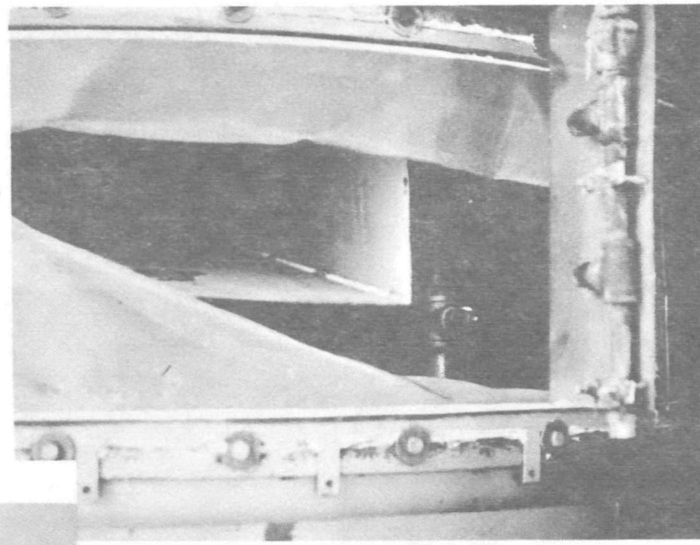


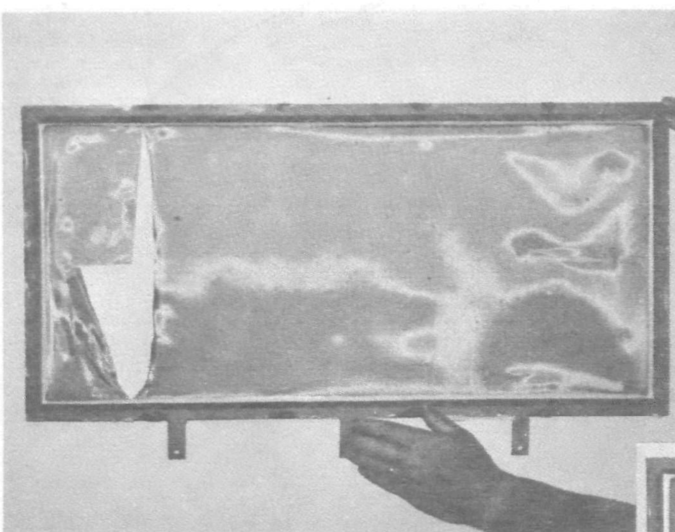
FIGURE 7

HYDRAULIC CAPACITY OF SCREENING UNIT

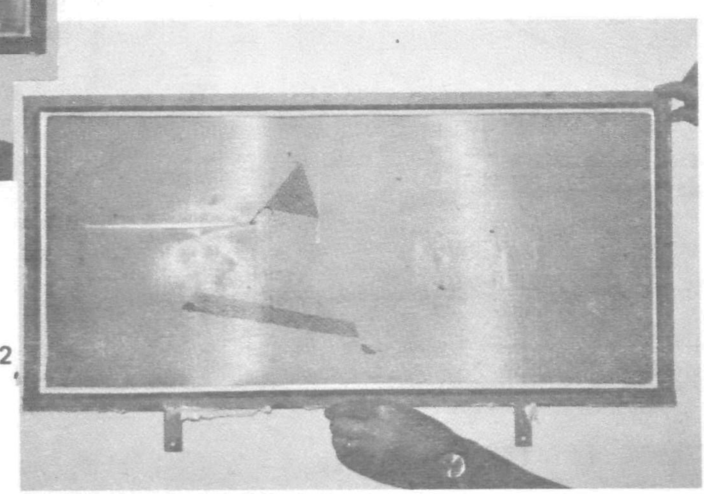


AT LEFT AND BELOW:
165 MESH TBC AT 114 GPM/FT^2,
FAILURE AFTER 6 HOURS





165 MESH TBC AT 122 GPM/FT^2,
FAILURE AFTER 12 HOURS
SHOWN AT LEFT



165 MESH TBC AT 122 GPM/FT^2,
FAILURE AFTER 6 HOURS
SHOWN AT RIGHT.

FIGURE 8

TYPICAL SCREEN FAILURES

Based on the results of the last experiment, a final test was performed to gather data on extended operation of the unit. The previous tests indicated that the unit operated best at 1700 gpm (122 gal/min/ft^2) on a 165 mesh screen (105 microns, 47.1 percent open area). To further stabilize the performance, backwash operation was changed to a 30-second wash to 40:1 solution of water and liquid detergent at the end of 4-1/2 minutes of operation. The test lasted for six hours and ended with the failure of three screens. A summary of the operating conditions, performance data, and character of flow streams are presented in Table 4. An Imhoff cone comparison of the flow streams is presented on Figure 9.

The results of the final test show that the unit's ability to remove organic material from the wastewater stream is good, and is comparable to the efficiency of a primary clarifier. The hydraulic efficiency of the unit is excellent; however, failure of the three screens shows that the unit is operating beyond its capacity. The screen is the limiting component of the entire unit.

TABLE 4

SUMMARY OF EXTENDED TEST

OPERATING CONDITIONS

INFLUENT FLOW RATE – 1700 GPM
(122 GAL./MIN./FT.^{2})
COLLAR SCREEN SPEED – 60 RPM
COLLAR SCREEN – 165 MESH TBC
(105 MICRON OPENING, 47.1% OPEN AREA)
BACKWASH RATIO – .235 GAL/1000 GAL.

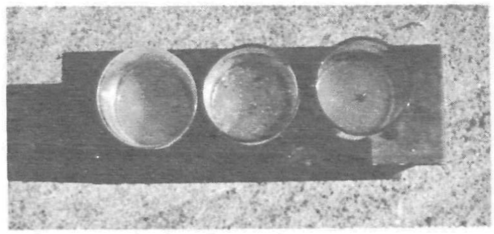
PERFORMANCE DATA

100% REMOVAL FLOATABLE SOLIDS
98% REMOVAL SETTLEABLE SOLIDS
34% REMOVAL TOTAL SUSPENDED SOLIDS
27% REMOVAL C.O.D.
8% OF INFLUENT AS A SOLIDS CONCENTRATE
RUN TERMINATED AT 6 HOURS DUE TO SCREEN FAILURES.

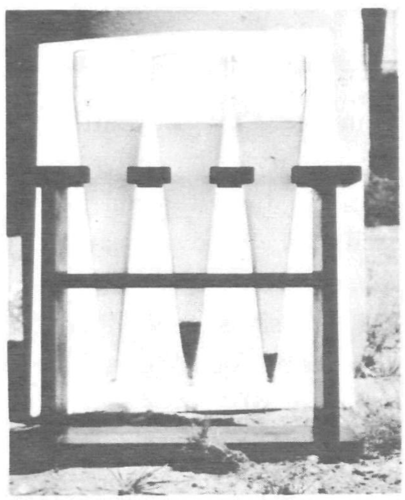
AVERAGE CHARACTER OF FLOW STREAMS

	INFLUENT →	SCREENED EFFLUENT	+ UNSCREENED EFFLUENT
FLOW (GAL./MIN.)	1700	1570	130
(% OF INFLUENT)	100%	92%	8%
SETTLEABLE SOLIDS			
(ML/L)	5.7	<0.1	73
TOTAL SUSPENDED SOLIDS			
(MG/L)	122	87	542
(POUNDS/MIN.)	1.73	1.14	0.59
C.O.D.*			
(MG/L)	302	240	1060
(POUNDS/MIN.)	4.30	3.15	1.15

* B.O.D./C.O.D. ≅ 0.5



100% REMOVAL
OF FLOATABLE SOLIDS
SHOWN AT LEFT



SCREENED EFFLUENT AT LEFT;
AT CENTER SOLIDS CONCENTRATE
AND INFLUENT COMBINED SEWAGE
SHOWN AT RIGHT.

FIGURE 9

IMHOFF CONE COMPARISON OF FLOW STREAMS

PRELIMINARY DESIGN OF A SCREENING FACILITY

The final performance data of the screening unit shows that fine-mesh screening could be used for treating combined sewage overflow; therefore, a preliminary design of a full-scale facility was warranted.

DESIGN CRITERIA

The proposed project site is located in Seattle, Washington. The site is in the heart of the business district of Seattle and is located within a valuable parking lot at the intersection of King Street and the Alaskan Way viaduct. The entire area surrounding the site is constructed on fill material, and almost every structure is supported on piling. The site is also close to the waterfront of Elliott Bay; therefore, high tide comes to within a few feet of the ground surface. Construction in this area is difficult and expensive.

The drainage basin above the site includes about 190 acres and is served by a 48-inch, pile-supported, concrete sewer. Since the drainage basin is almost entirely pavement or building roofs, a runoff coefficient of .95 was assumed to determine storm water volumes.

The rainfall pattern in the City of Seattle was studied to determine the design capacity of the screening facility. The intensity and duration of rainfall in the area received particular attention so that it could be estimated how long the facility would operate at a certain capacity. The study revealed that measurable precipitation occurred approximately 1,000 hours each year. While the rainfall occurrences were relatively frequent, they were of low intensities. Rainfall intensities up to .055 inches/hour produce a runoff of 10 mgd, and account for 75 percent of the rainfall occurrences. A summary of this study is shown on Figure 10.

Runoff in excess of 2.75 mgd will produce combined sewage overflow. This flow is based on the capacity that the dry-weather flow of the drainage basin requires in the interceptor sewer which carries this flow to the Seattle treatment plant. With the base flow of 2.75 mgd and the runoff pattern shown on Figure 10, the total volume of combined sewage overflow would be 282 million gallons a year. Based on the runoff pattern of this particular drainage basin, a design capacity of 25 mgd was chosen for the screening facility. With this capability, 96 percent of the total volume of overflow would receive treatment before being discharged to Elliott Bay. The added cost to achieve 100 percent treatment capabilities cannot be justified, as this would require a 40-mgd facility. Approximately 40 percent of the 40-mgd facility's capacity would be idle 95 percent of the time rainfall occurred.

AMOUNT AND RATE OF STORM–CAUSED COMBINED OVERFLOW AT KING STREET REGULATOR DRAINAGE BASIN SEATTLE, WASHINGTON

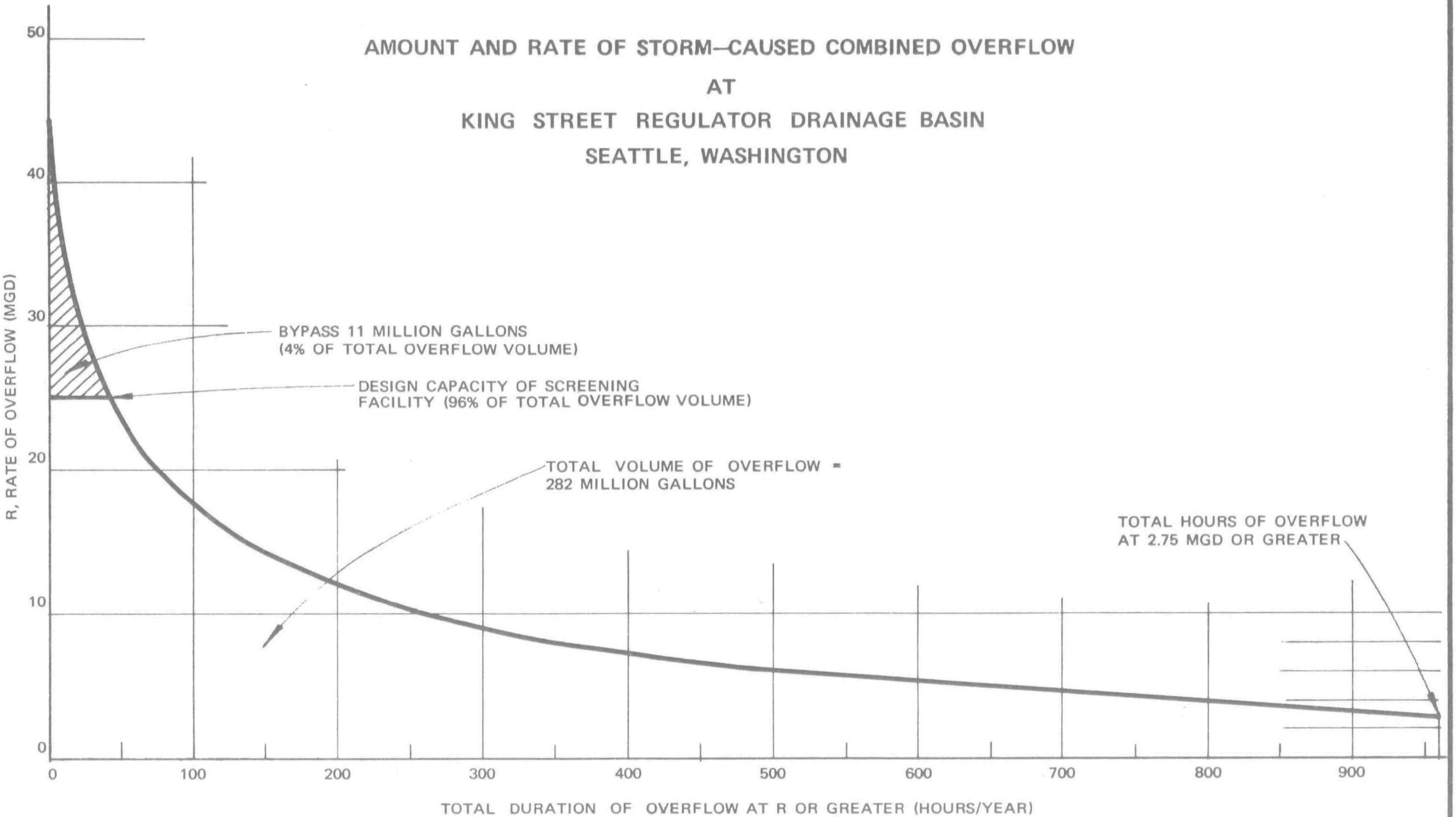


FIGURE 10

PRESENTATION OF PROPOSED SCREENING FACILITY

A 25-mgd screening facility requires a structure approximately 30 feet wide and 75 feet long. A perspective of the proposed facility is shown on Figure 11. The elevated facility is an attempt to illustrate what may be done to conserve the valuable parking area and still provide an attractive and functional treatment facility. The configuration of the elevated facility also offers the possibility of its becoming an integral part of an elevated parking facility. This would provide more parking than is now available, which is an asset to be considered. The facility does not provide disinfection.

Underground construction of the facility was investigated; however, this presented several hydraulic problems, and would be more costly than the elevated structure. A ground level structure for the Seattle facility was not investigated because conservation of the parking was a major design consideration.

A site plan of the proposed facility is shown on Figure 12. The combined overflow comes to the facility in the 48-inch sewer and would pass through a Parshall flume prior to reaching the facility. The flume would provide the primary control for the operation of the facility. After passing the Parshall flume, the flow would drop into a pump sump where it would be lifted to the screening units by a single 250 hp, vertical turbine, mixed-flow pump. The pump speed would be automatically controlled so that the pump discharge matches the flow in the incoming sewer. After the flow has passed the screening units, the screened effluent would be returned to the 48-inch interceptor downstream of the pump sump, and would be discharged to Elliott Bay. The unscreened flow would be returned to the trunk sewer where it would continue on to the treatment plant. It is assumed that the influent flow will be adequately disinfected upstream of the screening facility.

A design capacity of 25 mgd requires the use of ten 2.5 mgd screening units. The floor plan and sections on Figure 13 illustrate a proposed layout of such a facility.

The screening facility will be designed to operate automatically. The primary control for the facility would be located in the interceptor at the Parshall flume. The flume would monitor the depth of flow in the sewer, and screening units would be turned on and off in increments of 2.5 mgd as the depth of flow in the sewer rises and falls. Because occasional backwashing is necessary, a secondary control system is required to sense this need and to initiate the process. This would be accomplished by installing a flow meter on the screened effluent line. The flow signal from the effluent meter would then be compared to the flow signal from the flume to detect a decrease in hydraulic efficiency and, therefore, a need to backwash. It is anticipated that when the ratio of screened effluent flow/influent flow falls below .80, the backwash cycle will be initiated.

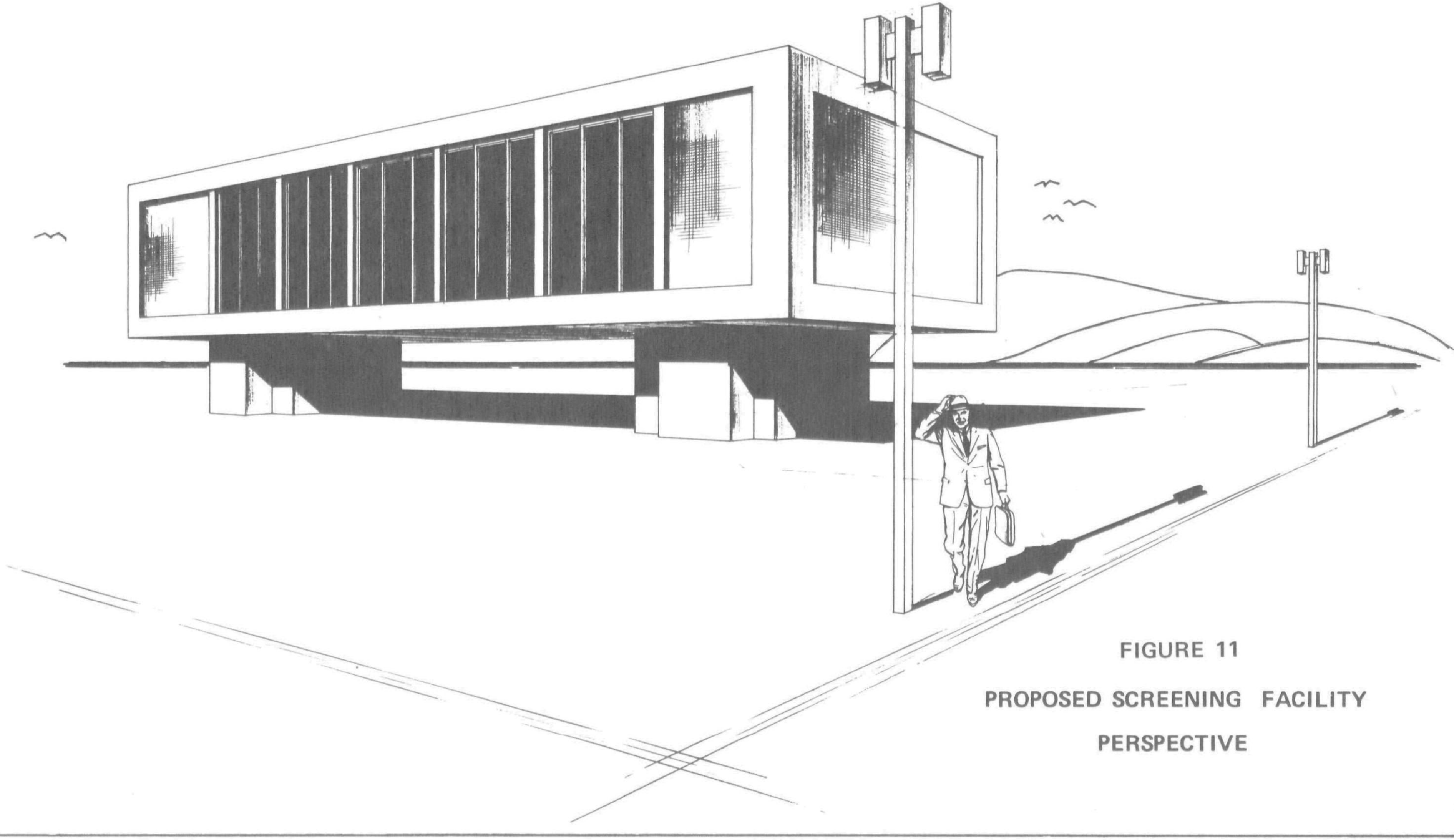
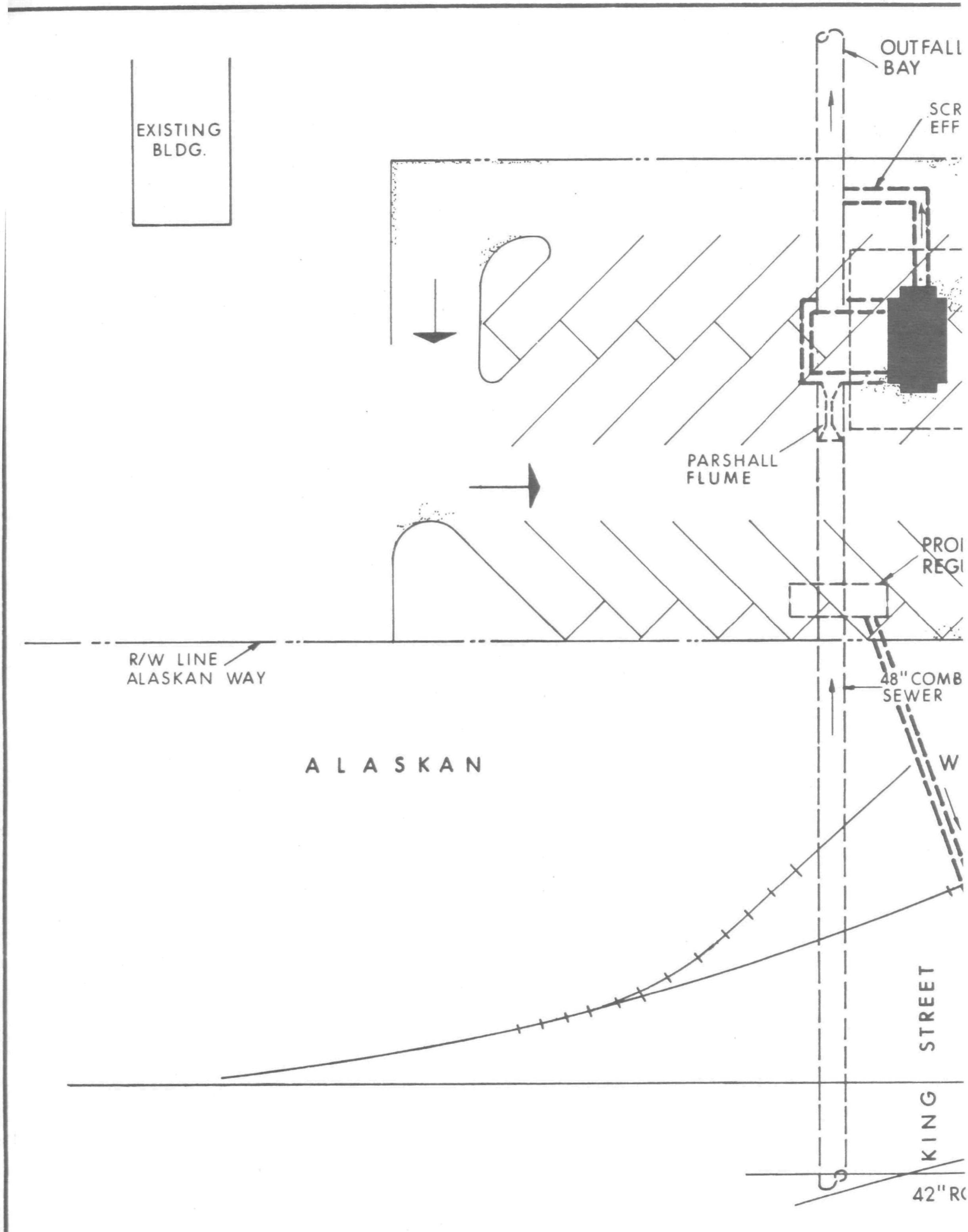


FIGURE 11

PROPOSED SCREENING FACILITY
PERSPECTIVE



OUTFALL
BAY
SCR
EFF
EXISTING
BLDG.
PARSHALL
FLUME
PRO
REG
R/W LINE
ALASKAN WAY
48" COMB
SEWER
ALASKAN
W
KING STREET
42" R

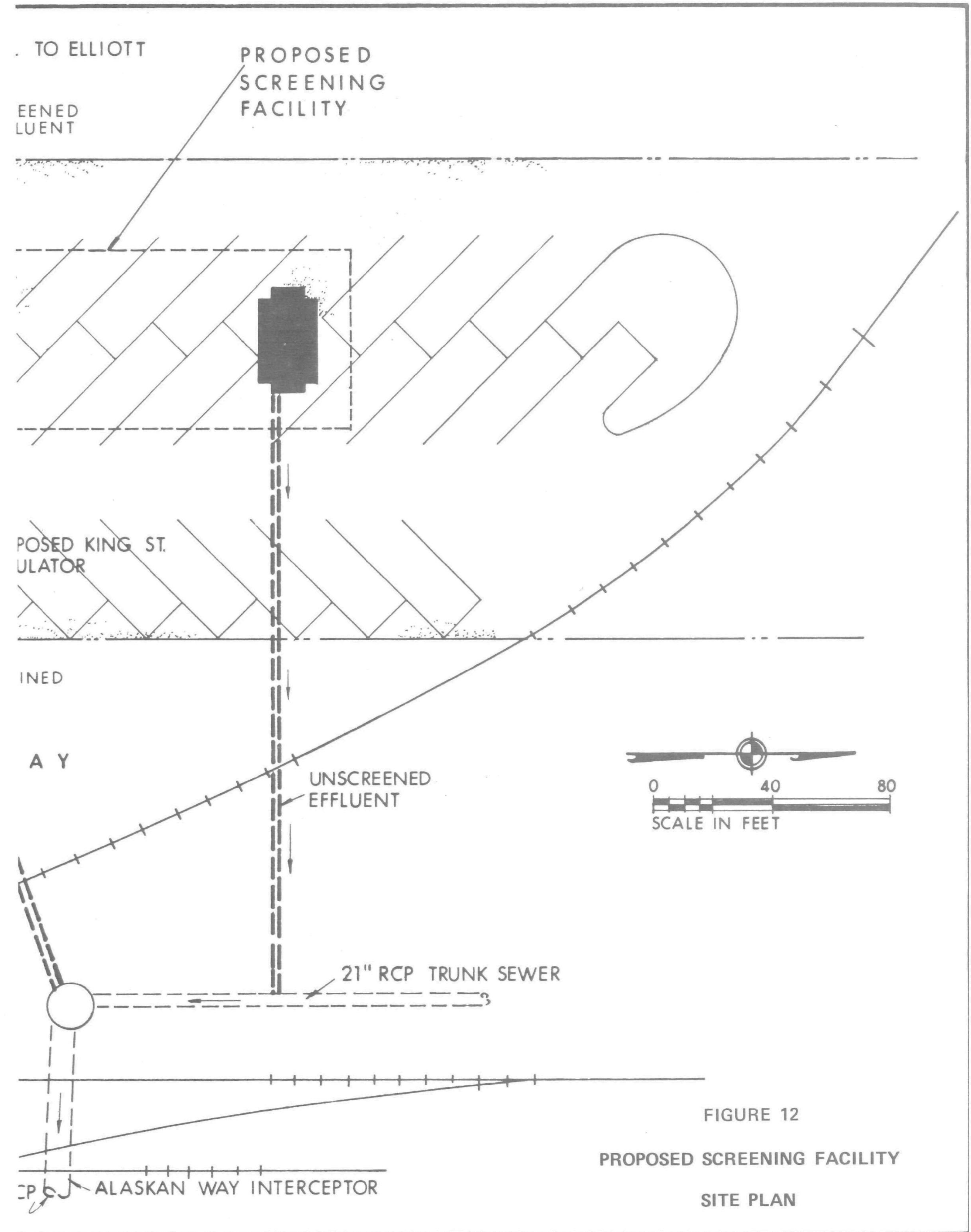


FIGURE 12

PROPOSED SCREENING FACILITY

SITE PLAN

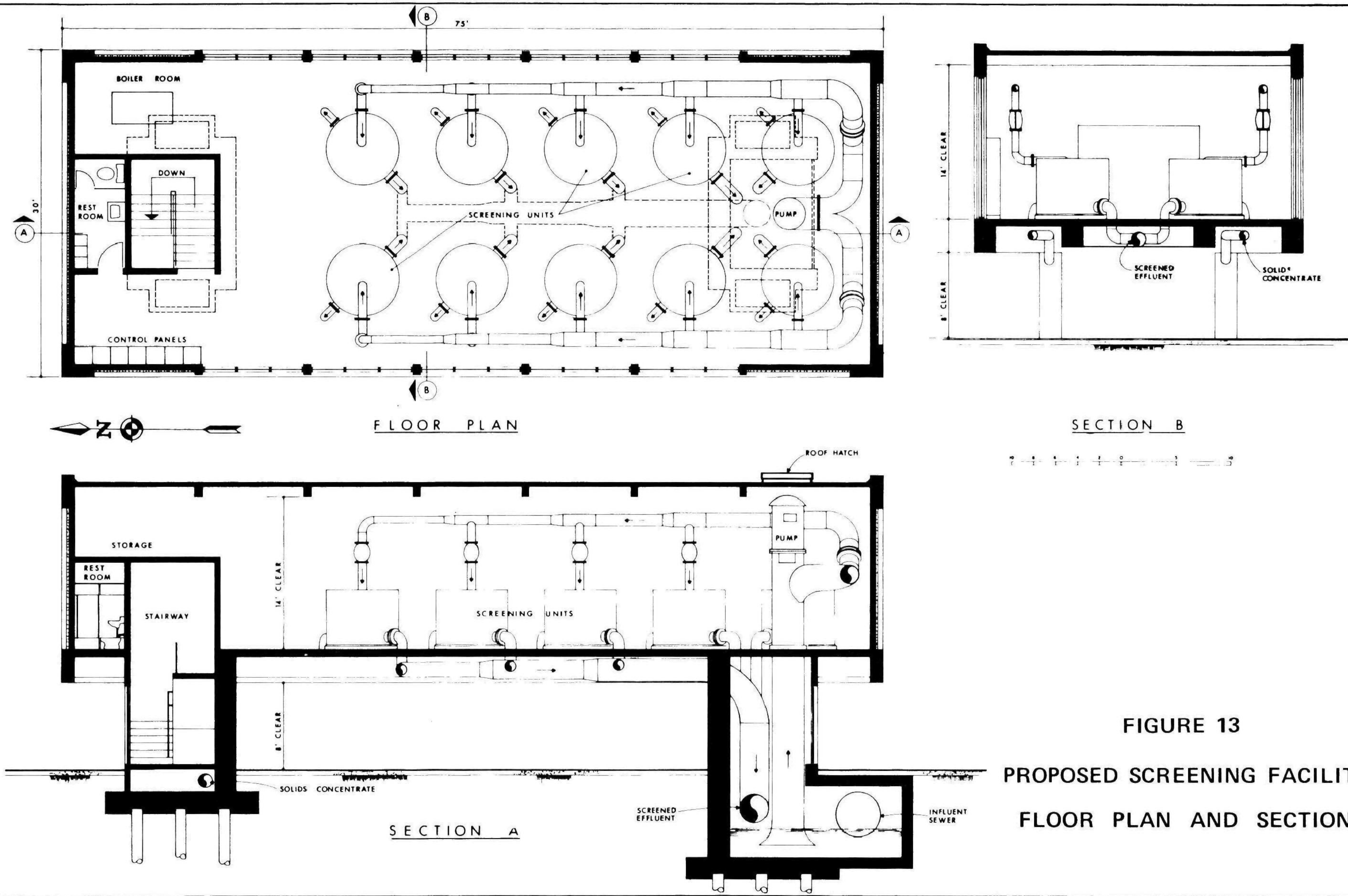


FIGURE 13

PROPOSED SCREENING FACILITY

FLOOR PLAN AND SECTION

ESTIMATED CONSTRUCTION COST OF SEATTLE FACILITY

The cost of constructing the Seattle screening facility is estimated to be $538,000. The construction cost estimate is based on estimated 1970 prices and assumes that all work will be performed by private contracting firms. The estimate also includes an allowance for design engineering, field surveying, soil exploration, construction supervision and inspection, legal fees and contingencies. The estimate does not include the cost of land or the cost of disinfection.

ESTIMATED ANNUAL OPERATION AND MAINTENANCE COSTS

A summary of the annual operation and maintenance costs is shown in Table 5. Annual labor costs are based on one man-hour for each hour of operation. This is based on the experience with the pilot unit, and is only an estimate of what may be experienced in a full-scale facility. Annual maintenance costs are based on 3 percent of the major equipment costs. Power and utility costs are based on present rates. Screen replacement costs are based on a predicted life of 500 hours. Costs for cleaning agent are based on the use of concentrated sodium hydroxide, purchased in bulk lots. The total annual operation and maintenance cost is estimated to be $18,500.

Table 5

Estimated Annual
Operation and Maintenance Costs

Item	Cost
Labor	$ 5,600
Equipment Maintenance	3,000
Screen Replacement	3,500
Power	3,000
Gas	1,200
Cleaning Agent	700
Vehicle Operation and Maintenance	1,500
Total Annual Operation and Maintenance	$18,500

ESTIMATED TOTAL ANNUAL COST

A total annual cost figure provides the best basis on which an economic comparison can be made, provided the items to be compared are relatively equal in basic design considerations. The construction cost estimate for the Seattle facility violates this premise in that the total cost includes provision for special foundation consideration and special architectural treatment.

In order to compensate for this in the total annual cost figure, another cost estimate was prepared for a more conventional type screening facility. In effect, the Seattle facility was moved to an assumed site that did not have any special foundation problems or did not require any special aesthetic considerations. It was assumed that this new structure would be of concrete block walls supported by a concrete wall footing. The floor would be a concrete slab on grade, and the roof would be of a timber joist system. All other mechanical and electrical items would be the same as the Seattle facility. These changes reduced the estimated total construction cost to $496,000 and it is this figure which is used in the total annual cost figure to represent a more typical screening facility.

The total annual cost summary is presented in Table 6. All costs shown in Table 6 have been adjusted to assumed 1970 prices and include an allowance for design engineering, legal fees, administrative costs and contingencies. The cost of land and disinfection is not included. The construction costs are amortized over a period of 25 years assuming an interest rate of 6-1/2 percent. The cost per 1000 gallons is based on treating a total of 271 million gallons per year.

Table 6

Estimated Total Annual Cost

Estimated Total Construction Cost	$496,000
Annual Debt Service	41,000
Annual Operation and Maintenance	18,500
Estimated Total Annual Cost	$ 59,500
Estimated Cost Per 1,000 Gallons = 22 Cents	

DISCUSSION OF FEASIBILITY

In order to get a feel for the economic position of this type screening facility relative to other possible methods of treatment, a brief economic comparison was made. Particular attention was paid to conventional primary sedimentation; however, since a detailed cost comparison was beyond the scope of this study, no cost figures will be presented. The brief comparison did reveal that screening can be a feasible treatment method depending on particular conditions present at the site.

A major advantage of conventional primary treatment is that disinfection, by means of conventional chlorination, can be accomplished within the primary clarifier. This eliminates the need for a separate chlorine contact chamber which, at the present state of the art of chlorination, would be required at a screening facility. This, of course, represents a considerable added cost when disinfection is found to be either desirable or mandatory.

This advantage, however, could be offset with a new method of disinfection that could be as efficient as chlorination and at the same time eliminate the long contact time that is presently required.

Another advantage of conventional primary clarification is that the volume of the primary clarifier would be large enough to completely hold the storm-caused combined sewage of the short-duration, low-intensity storm events. After the storm has passed and the peak flow in the sewer has subsided, the impounded sewage could be returned to the sewer at a reduced flow rate. This advantage is enhanced when there is a high percentage of short-duration, low-intensity storms such as in the Seattle area.

The most important disadvantage of conventional primary clarification is the large amount of land required. It has been estimated, by preliminary layouts, that conventional primary clarification requires 10 to 20 times more land area than a screening facility. The actual difference is dependent on the design capacity chosen for a primary treatment plant, and how much reserve capacity of a primary clarifier is actually used to meet the flow requirements of a particular drainage basin. This disadvantage becomes more severe as the size of the drainage basin increases, and as the value of the land increases. The Seattle site is an example of a site where conventional primary clarification would most likely not be feasible.

In summary, the screening unit can be an economically feasible method of treating combined sewage overflows when compared to conventional primary clarification. The selection of the screening unit as a method of treatment at a particular site, however, will require the review of at least four factors. These are:

1. The value and availability of land.
2. The size of the drainage basin, and therefore, the design capacity of the treatment facility.
3. The character of rainfall and the pattern of runoff.
4. Available means of disinfection.

Other factors that would require review also would include other methods of treatment, aesthetic considerations, and ancillary use of the facility, such as surrounding the Seattle facility with a parking structure. In all, it must be emphasized that each point of overflow is unique, and all these factors must be reviewed before the most economical and efficient method of treating combined sewage overflow is selected.

CONCLUSIONS

1. High-rate, fine-mesh screening is an economically feasible method of treating combined sewage overflows. When compared to conventional primary sedimentation, however, the selection of a screening facility as a treatment method is dependent on the value and availability of land, the design capacity of the treatment facility, the character of rainfall and runoff, and the available means of disinfection.

2. The characterization of storm-caused combined sewage and dry-weather combined sewage did not reveal any unusual constituents which could affect the long-term effectiveness of the screening unit. These characterizations were compiled on the basis of several composite samples.

3. The short-term effectiveness of the screening unit is significantly reduced by the presence of oil and grease in the combined sewage. Oil slugs were observed at least once a day for a duration of approximately 5 minutes, and were of a concentration substantial enough to make the sewage appear black in color. The presence of an oil slug reduces the hydraulic capacity of the screening unit by as much as 50 percent. Frequent backwashing during the presence of an oil slug will minimize this problem.

4. The vibratory horizontal screen is not required in screening combined sewage overflow. The presence of the vibratory horizontal screen reduces the hydraulic capacity of the unit and, in some cases, results in lower removal efficiencies (see Appendix C).

5. The overall performance of the screening unit is a function of the mesh size of the collar screen, the rotational speed of the collar screen, the strength and durability of the collar screen material, and the backwash operation.

6. The removal efficiencies of the screening unit increases as the mesh of the collar screen becomes finer, and as the volume of the feed applied to the screen increases. For example, 31 percent removal of total suspended solids was observed at an influent flow rate of 1200 gpm (86 gal/min/ft^2) with a 105 mesh screen (167 micron opening), while 35 percent removal was observed at a flow rate of 2000 gpm (143 gal/min/ft^2) with a 230 mesh screen (74 micron opening).

7. The removal efficiencies of the screening unit are independent of the rotational speed of the collar screen.

8. The hydraulic efficiency of the screening unit increases as the rotational speed of the collar screen increases, as the mesh of the collar screen becomes coarser, and as the velocity of the feed approaching the screen increases.

9. The life of the collar screen decreases as the velocity of the feed approaching the screen increases and as the mesh of the screen becomes finer. For example, the screen life observed at an influent flow rate of 1200 gpm (86 $gal/min/ft^2$) with a 105 mesh screen (167 micron opening) was more than four hours, while the screen life at a flow rate of 2000 gpm (143 $gal/min/ft^2$) with a 230 mesh screen (74 micron opening) was less than four hours.

10. Approximately 90 percent of the screen failures were mechanical failures caused by hydraulic overloading of the screen. The remaining 10 percent of the failures were caused by punctures from objects present in the feed.

11. It is possible to produce a 165 mesh screen (105 micron opening, 45 percent open area) with a probable life of 500 hours while operating at a flow rate of 1750 gpm (2.5 mgd or 128 $gal/min/ft^2$).

12. The use of a solution of hot water and liquid solvent in lieu of steam, was found necessary to obtain effective cleaning of the screens.

13. Of the solvents tested, a caustic solution was the most efficient solvent for backwashing the collar screen.

14. Screen blinding decreases as the velocity of the feed approaching the screen increases, as the mesh of the screen becomes coarser, as the frequency of backwash increases, and as the rotational speed of the collar screen increases.

15. A minimum of approximately 4.5 feet of fluid head above the downstream water surface of the screening unit is required for gravity flow operation.

16. Based on the intensity and duration of rainfall in the Seattle area, a screening facility in the Pacific Northwest can be expected to be in operation approximately 1000 hours a year.

17. The collar screen material is the limiting component of the screening unit. When a stronger and more durable screen material is developed, it will be possible to increase the hydraulic and removal efficiency of the screening unit.

18. The estimated construction cost for a 25 mgd screening facility is \$496,000. The estimated annual cost of operation and maintenance is \$18,500. Based on a 25-year bond issue, with an interest rate of 6-1/2 percent, the total annual cost is estimated to be \$59,500, which puts the cost of treatment at 22 cents/1000 gallons assuming 271 million gallons of overflow a year are treated. These cost figures are based on a preliminary design of a screening unit for Seattle, Washington, which is presented in this report.

19. Based on the scale-up design of the Seattle facility, a screening facility will require 1/10 to 1/20 the land that a conventional primary sedimentation plant.

RECOMMENDATIONS

1. It is recommended that a full-scale screening facility be designed and constructed to demonstrate the feasibility of utilizing high-rate, fine-mesh screens in the treatment of combined sewer overflows.

   The results of this study have established the feasibility of the high-rate, fine-mesh screens. The performance of the screens should now be demonstrated through the design and operation of a full-scale facility. Based, in part, on the results of this study, the equipment supplier has developed and tested a second generation unit. The new unit is operated at 3 mgd (2100 gpm, 150 gal/min/ft^2) with a 165 mesh (105 micron opening) stainless steel screen with little or no deterioration in the performance observed at the 2.5 mgd level. The equipment supplier has also developed a new screen that has a probable life of about 500 hours. This represents a hundredfold increase in life over that observed in this study.

   During a period of demonstration, these new units could be tested and further optimized with regard to inlet conditions, hydraulic capacity, screen life, backwashing technique, and control systems. The period of demonstration would also yield firm cost and operational data.

2. As part of a final design effort for a full-scale facility, it is recommended that a systems analysis be performed to investigate the compatibility of the electrical and hydraulic machinery.

   In the preliminary design of the full-scale facility presented in this study, it was a relatively simple matter to design a control system to operate the facility. Likewise, it was also relatively simple to design the hydraulic machinery required of the facility. The compatibility of the two systems, however, is very difficult to predict. It is therefore recommended that an analog simulator be employed to simulate the operation of a screening facility. The results of this study may reveal some basic problems in control that can be resolved prior to the completion of a final design.

3. It is recommended that flow measurement and sampling facilities be installed at all combined sewage outfalls where installation of treatment facilities is anticipated.

   Based on the experience of this study, continuous flow recording at an overflow point prior to the design of a treatment facility would be of significant value in determining both the design capacity of the facility and the total use of the facility. In addition, sampling facilities would be helpful in determining the character of the waste to be treated. Composite samples would yield a general description of the waste, and grab samples could be collected to determine the quality and frequency of any unusual constituents that may be present in the waste. If the installation of a screening facility was anticipated, this information would be required for sizing of screen materials and estimating the frequency and quality of backwashing.

4. It is recommended that a comprehensive study be conducted to determine an efficient method of contacting a disinfectant with a treated effluent.

   A major advantage in developing high-rate treatment equipment, like the proposed screening facility, is the ability of the equipment to treat large volumes of waste in a small area. This advantage would be negated, however, if conventional chlorine contact times are required to provide disinfection. Based on the findings of this study, the land required to provide conventional chlorination is 3 to 4 times that required of the screening facility. In some cases, this requirement can be reduced or eliminated by utilizing an existing outfall downstream of the facility for the contact channel; however, this is normally the exception rather than the rule. Therefore, in order to maintain the space advantage of high-rate treatment equipment, a high-rate method of disinfection must be developed.

   Currently, there is considerable research available describing the bactericidal mechanism of several different disinfectants. Several of these studies indicate that acceptable bacterial kills can be obtained with conventional disinfectants at contact times of 10 minutes or less. Based on these observations, it is recommended that additional research be performed to develop a contact chamber that will reproduce these laboratory results in the field. It is believed this research will lead to a contact chamber with two compartments. The first compartment would be a mechanically-mixed rapid-mix tank with a detention time of less than 3 minutes. This complete-mix tank would provide rapid and intimate contact between disinfectant and effluent. The rapid-mix tank effluent would then enter a period of quiescent contact provided by a plug-flow type tank with a detention time of less than 15 minutes. It is this combination of two distinct flow regimes that approaches many of the laboratory procedures used in bactericidal studies, and it is a type of flow regime that may provide a more efficient and economical method of disinfection.

REFERENCES

1. "Problems of Combined Sewer Facilities and Overflows 1967," Water Pollution Control Research Series WP-20-11, U. S. Dept. of Interior, FWPCA, December 1, 1967.

2. "Standard Methods for the Examination of Water and Wastewater." 12th Ed., Amer. Pub. Health Assn. New York (1965).

ACKNOWLEDGMENTS

The author wishes to thank the City of Portland, Oregon, and SWECO, Inc., of Los Angeles, California, for their cooperation and assistance in conducting this study for the Federal Water Pollution Control Administration.

SECTION 3

ASSESSMENT OF COMBINED SEWER PROBLEMS

by

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for the

FEDERAL WATER QUALITY ADMINISTRATION
DEPARTMENT OF THE INTERIOR

June, 1970

ASSESSMENT OF COMBINED SEWER PROBLEMS*

by Richard H. Sullivan
Assistant Executive Director
for Technical Services
American Public Works Association

The water pollution problems which have become the target of public opinion and public official concern are the sins of the past being imposed on the present. We are today racing headlong to catch up with yesterday's custom of using rivers to rid man's environment of his undesirable waste into the waters most convenient to his urban habitat. Now that there is a national desire to clean up the discharge of sewage and industrial waste by construction of treatment plants of adequate processing effectiveness, attention is turned to another sin of the past that is being imposed on the present--the discharge of excess flows from combined sewers everytime it rains.

The problem stems from the early use of storm drains to handle domestic sewage by admitted sanitary flows to these conduits. When sewage treatment was not practiced, the fact that combined sewers spilled their waste water into receiving streams was not a matter of concern, but when treatment was provided for sanitary sewage it becomes necessary to install in combined sewer interceptors, regulator devices which would divert dry weather flow to the treatment plant and during storm run-off period to excessive flows to receiving waters.

In urban areas where adequate sewage treatment is provided, these periodic overflows stand as a negative effect which minimizes investment in pollution control. A water course that is polluted periodically is only little more usable for most purposes than one that is continuously polluted. As more and more sewage treatment facilities are provided, meeting Federal and State Standards for high degrees of treatment, the anomaly of combined sewer overflows becomes more and more obvious.

COMBINED SEWER FACILITY INVENTORY

In 1967, at the request of the Federal Water Pollution Control Administration, the American Public Works Association undertook to make an inventory of combined sewer facilities in the United States. Every local jurisdiction with combined sewers whose population exceeded 25,000 was personally interviewed, as well as a large sampling of other jurisdictions--including communities with a population of less than 500. In all, 641 jurisdictions were interviewed. We estimated that 46 per cent of the communities with 94 per cent of the population and 84 per cent of the area served by combined sewers were directly interviewed.

The results of the survey indicated that 36,236,000 people, living on 3,029,000 acres were served by combined sewers. This total indicates that approximately 29 per cent of the nations total sewered population is served by combined sewers.

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*Prepared for seminar on Storm and Combined Sewer Problems, Chicago, Illinois, June 22-23, 1970

Mere numbers do not in themselves make a problem. In the past ten to fifteen years, there has been a substantial effort to construct waste water treatment facilities. Overflows from combined sewers are gradually being identified as one of the continuing sources of pollution. The early rationale that held that since the overflow was 99 plus per cent storm water it was "clean" has been disproved. Overflows are polluted.

The small flows of sanitary sewage in large combined sewers results in low velocities. Solids are therefore settled out along the sewer line. Storm flows tend to scour out this material and carry it to the overflow. A large proportion of the sanitary sewerage also escapes in the overflow. It has been estimated that from three to five per cent of the total organic load reaching the sewer leaves by the overflow.

A part of the problem of combined sewer overflows is the location of the overflow facilities and the nature of the receiving waters. Nationally, most overflows are adjacent to residential or industrially zoned land. The major receiving waters are dry water courses or waters used for limited body contact recreation or fishing.

These land and water uses are not suitable places for the discharge of sewage. Presence of the combined sewer overflows may have a serious impact upon land development and land values. For a hundred acre tract in one Michigan city, influenced by one combined sewer overflow, our appraiser estimated that a value loss of $600,000 in the immediate area and to the adjacent area of 1,333 acres, $4,476,000. This loss of value results in a tax loss to the city alone of $70,000 per year.

The American Public Works Association, as a part of its 1967 study, was asked to estimate the cost of separating combined sewers nationwide. We analyzed figures for weeks, adjusted for prices, inflation and about everything else, and ended up with $48 billion in 1967 dollars as the answer. Of this, $30 billion was for work in the public right-of-way and $18 billion for changing the plumbing on private property. The complete incapability of many of our major urban areas to bear the disruption of their major commercial areas and major streets makes complete separation an unlikely goal. Therefore we also investigated alternatives and from the information available we estimated that the cost of alternate methods of treatment or control would amount to about $15 billion. Such methods include in-system and off-system holding and drainage area control.

The States, in particular, and many other agencies have enacted regulations which prohibit the construction of new combined sewer systems or the additions to existing systems. Unhappily some of the progress which is being made in metropolitan areas is in new suburban developments where separate sanitary sewers in a great many cases discharge into combined sewers and add higher concentration of sanitary sewage to the overflows.

Another major finding from our interviews was the determination that less than 20 per cent of the combined sewer overflow regulators were of a true dynamic type, that is they could be adjusted to meet various flow criteria. Of the 10,025 regulators found in the jurisdictions interviewed, 40 per cent were nothing nothing more than simple weirs, many with design features which are not responsive to overflow regulation. In fact many were merely a hole in a manhole to relieve the system.

The use of improper types of regulators for the existing conditions as well as poor maintenance practices appeared to be one of the major reasons for unnecessary and prolonged overflows.

Another finding was that infiltration was recognized as being excessive in a great many systems. Although few jurisdictions had apparently surveyed their systems, treatment plant records indicate the excessive wet weather flows.

Sewer personnel across the country told us of their efforts to discontinue the connection of roof gutters, area drains and foundation drains to the combined sewer system. The flow from these sources is generally credited with overloading the sewer system, causing both basement flooding, innundation of mid-city areas and more frequent and prolonged overflows.

Questions were also asked of each jurisdiction as to the number of personnel and the level of training of employees associated with the operation and maintenance of the sewer system. In jurisdictions of less than 25,000, on the average less than one-half have a full time registered engineer or an engineer in training. For the 52 jurisdictions from 10,000 to 50,000, the average was only 3.3 registered engineers in training per jurisdiction. This group also averaged 5.4 certified plant operations per jurisdiction. Thus, it appears that generally there may be an inadequate number of trained personnel available to make maximum utilization of today's technology.

The full report is available from FWQA as publication WP-20-11 for $1.00.

STUDY OF URBAN STORM WATER POLLUTION

With sewage and industrial waste treatment a reality and the water resources of the nation--or at least of major watersheds--protected; and with the overflows of combined sewers effectively regulated and minimized, in terms of the "two Q's" of quantity and quality of the spilled waste water to receiving waters, still another "sin" of the past will stand as a challenge to the present and the future.

This will involve the evolution of a new concept of the pollutional impact of separate storm water discharges on water courses, lakes and coastal waters. Since everything is relative, it is understandable that storm water has in the past been considered harmless as compared with the pollutional nature of untreated or inadequately treated sewage and industrial wastes and the nature of combined sewer overflows of admixtures of sewage and storm water runoff.

But with the elimination or minimization of these two obvious sources of pollution, it will not be surprising that attention will eventually come to bear on storm water spills. Are they a source of pollution? What are these sources? What could be done about urban runoff waste waters? What is the role of agricultural land runoff in the total water pollution control picture and the problem of protecting the nation's water resources for use and reuse purposes?

Some of the answers of these basic question are found in the study of Water Pollution Aspects of Urban Runoff which was carried out from 1966 to 1968 by the APWA under a contract with the FWQA. The report on the study is published as WP-20-15 for $1.50.

"Clean" storm water is polluted. Rain scavenges air pollution out of the atmosphere; flows across roofs, across grass sprayed with insecticides and fertilized with nitrogen and phosphorous, pets and birds; along street gutters which may average a daily accumulation of more than a pound of debris each day per 100 ft. of curb; and finally through catch-basins where the flow displaces perhaps two cubic yards of stagnate water and carries with it some of the digested solids from the bottom of the catch-basin. By the time the storm water reaches the sewer, it may exceed the strength of sanitary sewage. When salts from snow and ice control, phenols and lead from automobile exhausts and other contaminates are added, the storm water may have a wide range of undesirable characteristics.

TYPES OF PROBLEMS

The pollution problems which have been generally identified with combined sewers include the following:

1. Pollution of receiving waters
    a. too frequent overflows
    b. dry weather overflows
    c. prolonged overflows
    d. carryover of solids
    e. by-passing to protect waste water treatment plant facilities

2. Disruption of waste water treatment plants
    a. concentration of solids and debris in primary treatment
    b. wash-out of secondary treatment process due to low strength flows
    c. salt water intrusion

At the heart of most of these problems appears to be the combined sewer regulator and the capacity of the treatment plant.

Most jurisdictions have not attempted to assess the extent of the pollution of receiving waters. In many areas the effects of combined sewer overflows are masked by other major sources of pollution, such as untreated or poorly treated sanitary sewage, industrial waste, agricultural land run-off, feed lot runoff, and urban storm water runoff.

The disruptive aspects of combined sewer flow at the waste treatment plant are readily determined by plant operators. In many instances this has led to even further diversion or by-passing to minimize treatment plant problems.

The APWA Research Foundation, under contract with the Federal Water Pollution Control Administration and some 25 local governmental agencies, has completed a cooperative study of combined sewer system overflow regulator facilities and practices. This study covered design, application, construction, control and operation and maintenance procedures. The specific purpose of this project was to analyze and evaluate the effectiveness of practices and to establish long-needed guidelines for more efficient and dependable control of overflows and for reduction in the frequency and duration of combined sewer flows and the resultant pollution in waters receiving such spills.

THE FUNCTION OF REGULATOR DEVICES

The volume of liquid flowing in a combined sanitary and storm water sewer is greater than the carrying capacity of the interceptor sewer system, the pumping capacity of a pumping station or the capacity of a sewage treatment plant, during periods of storm and runoff. It is the function of a regulating device and the chamber in which it is installed to regulate or control the amount of the flow which is allowed to enter the interceptor system and to divert the balance to holding or treatment facilities, or to discharge this balance to a point of disposal in nearby receiving waters. The regulator, thus, has the function to transmit all dry weather flow to the interceptor and hence to sewage treatment works, and to "split" the total combined storm and sanitary flow during periods of runoff so that a portion of the flow enters the interceptor and the balance is diverted to the other points listed above.

Regulators may be of various kinds--such as stationary, movable, mechanical, hydraulic, electrical, fluidic, variable, non-variable, etc.--but their function is as described. The 1967 study of overflow problems indicated the need for improvement in regulator devices and in their operation and maintenance. Over and above today's regulator facilities, the field of combined sewer service would be benefited by the availability of other types of devices and modifications of existing equipment. Among the challenges are greater sophistication in control and actuating facilities, including on-site remote sensing and control of intercepted flows, paced by conditions in interceptor and treatment works, and desired diversion of flows into holding and treatment processes for the effective reduction in storm water overflow pollution.

Figure 1, Static Regulator, Side outlet connection is a photograph of a typical static regulator with a low weir. This device, while inexpensive to construct, may be a source of dry-weather overflows due to clogging and cannot be adjusted to variations in dry-weather flow.

Figure 2, Typical Manually Operated Gate Regulator, although a static device, can be adjusted to various flow conditions. Such a facility can be modified with a motor operated gate and proper controls to be a dynamic regulator, responding to flow conditions in the collector or interceptor sewer.

Figure 3, Cylinder Operated Gate, indicate the layout of a hydraulic cylinder gate in Philadelphia. This is a dynamic regulator in as much as the position of the gate is controlled by a float-off of the collector sewer.

Figure 4, Float Operated Gate, is an isometric view of a dynamic gate, the position of which may be regulated by flow in either the collector or interceptor sewer.

(Insert Fig. 1-4)

The problem of design, manufacture, application and handling of regulators is made difficult by the conditions under which these devices and regulator chambers must function. These include complex and often unpredictable hydraulic conditions imposed by dramatic changes in runoff due to storms; the heterogeneous nature of the sewage-storm water which is handled, including grit, coarse debris and other clogging producing wastes; the corrosive nature of the liquids; and the humid and corrosive-gaseous conditions in the regulator chambers. Further complications are created by tide water backflows and other hard-to-predict hydraulic conditions in interceptor-treatment plant networks.

Our study of combined sewer regulators involved the interviewing of a group of jurisdictions and then in cooperation with a panel of consulting engineers preparing both a report and manual of practice. Representatives of financially participating jurisdictions as well as the WPCF and the ASCE served on the steering committees for the study.

The study report, "Combined Sewer Regulator Overflow Facilities" and the "Manual of Practice for Combined Sewer Regulation and Management" will soon be available from the FWQA.

Our detailed, extensive interviews of some seventeen jurisdictions have found only three where the operation of the regulators has been designed to minimize pollutions by assuring that the interceptor sewer is fully charged. These three projects have received FWQA demonstration grants. In Seattle, this is accomplished by a hydraulically operated gate controlled by a bubbler unit downstream in the interceptor. In Minneapolis-St. Paul Sanitary District, control is achieved through the use of an inflatable dam, increasing the head of the orifice discharge to the interceptor sewer. Detroit is also using a form of "traffic" control to maximize flow in the interceptor.

An additional principle of operation to minimize pollution is to maximize in-system storage. The Seattle system in particular insures that all of the collector storage capability is utilized prior to an overflow event. This capability does much to eliminate dry-weather overflows and minimize pollution in their system.

Engineering investigations are being made in Seattle to determine where there is justification for upgrading the facilities. The study is conducted by monitoring a facility for the length of time and quantity of flow during overflow events. From the characteristics of the contributory sewer system, a mass hydrograph is constructed to analyze the quantity and time of flow should a controlled facility be installed. One recent study indicated that for one small drainage area, for a short period of time when eight (8) events occurred which overflowed 6.4 million gallons, that had a dynamic regulator been installed only one event of 2.7 million gallons would have occurred, a reduction of 85 per cent in frequency and 42 per cent in volume.

When information of this type is available, the value of upgrading facilities can be made. There are no magic numbers or rules of thumb--an engineering study is needed in each case.

Some of the major findings and recommendations of the study call for a reduction in the number of small overflows, total systems management of the combined sewer system, use of dynamic type regulators to allow response to hydraulic conditions in the sewers, use of regulators that improve the quality of the overflow as well as the control of its quantity, and the need for improved maintenance and design of regulator facilities.

Figure 5, Spiral Flow Regulator, is a drawing of an experimental device which has been tested in England to induce helical motion in the flow. Such a secondary motion tends to concentrate the solids and they may be drawn off with the flow to the interceptor, improving the quality of the overflow.

Figure 6, Vortex Regulators, is a drawing of two regulators which have been installed in Bristol, England. The induction of the vortical motion acts to concentrate the solids in the flow to the interceptor. Both of these regulators are compact and appear feasible for installation in many existing situations. We have recommended that additional research be carried out to define design relationships and the efficiency of the units to remove solids from the overflow.

(Insert Fig. 5 & 6)

Maintenance was found to be an important factor in the successful use of various types of regulators. The amount of money allocated gives an indication as to how effective local officials judge their regulators to be. Where dynamic regulators have been used and low levels of maintenance have been provided, the dynamic regulators have often been taken out of service.

The survey found that many regulators have, by necessity, been constructed where maintenance is difficult and access almost impossible. Figure 7 is a picture of the boat and barge used by the Allegany County Sanitary Authority (ALCOSAN) for the maintenance of overflow facilities which can only be reached from the river.

Figure 8 is a photograph showing the variety of equipment carried on the barge.

(Insert Fig. 7 & 8)

There must be a commitment upon the local jurisdiction to properly maintain any new or improved regulators which may be installed in a pollution control program. Although out of sight, regulator facilities must be kept in mind.

ROLE OF INFILTRATION CONTROL

Expenditures for sanitary and combined sewers and treatment facilities amount to many millions of dollars annually and form a major part of the total amount budgeted for operations and capital improvement programs in every urban community.

Unfortunately, in most urban areas, little attention has been given to making sure that costly sanitary and combined sewers and sewage treatment facilities function properly, if at all, under wet ground conditions. So-called "separate" sanitary sewer systems often collect such large infiltration flows that they are ineffective in performing their primary function. Infiltration in sanitary sewers usually causes flows which exceed treatment plant capacity and, as a result, biological processes are either upset or raw sewage is by-passed into waterways which were intended to be protected from such contamination.

Infiltration was revealed as a contributing factor in combined sewer overflows in a report prepared in 1967 by the APWA Research Foundation which was previously described. Thirty-four per cent of the cities interviewed indicated that infiltration exceeded their specification. The increased flow in combined sewers due to infiltration decreases its in-system storage capability and results in more frequent and longer duration of overflows.

Most engineering consultants, scientists, and administrators in the field of design, operation and management of sanitary sewage collection systems have little quantitative data available to use in estimating the extent of infiltration and in making value judgements for the most effective means of prevention and control.

The APWA Research Foundation in cooperation with 35 local jurisdictions and the FWQA has undertaken a study of economics of infiltration control, design and construction practices for new construction and remedies for existing systems where the cost benefit ratio of control indicates that such action is desirable. This study will be completed in the next few months.

In this study, the factors contributing to storm and ground water infiltration are evaluated and analyzed to produce guidelines which will be of tangible value to designers, administrators and operators of combined and sanitary sewage collection systems and treatment plants.

The study is designed to aid in the formulation of an effective research and development program to reduce pollution resulting from combined sewer overflows and treatment plant by-passing attributable to infiltration.

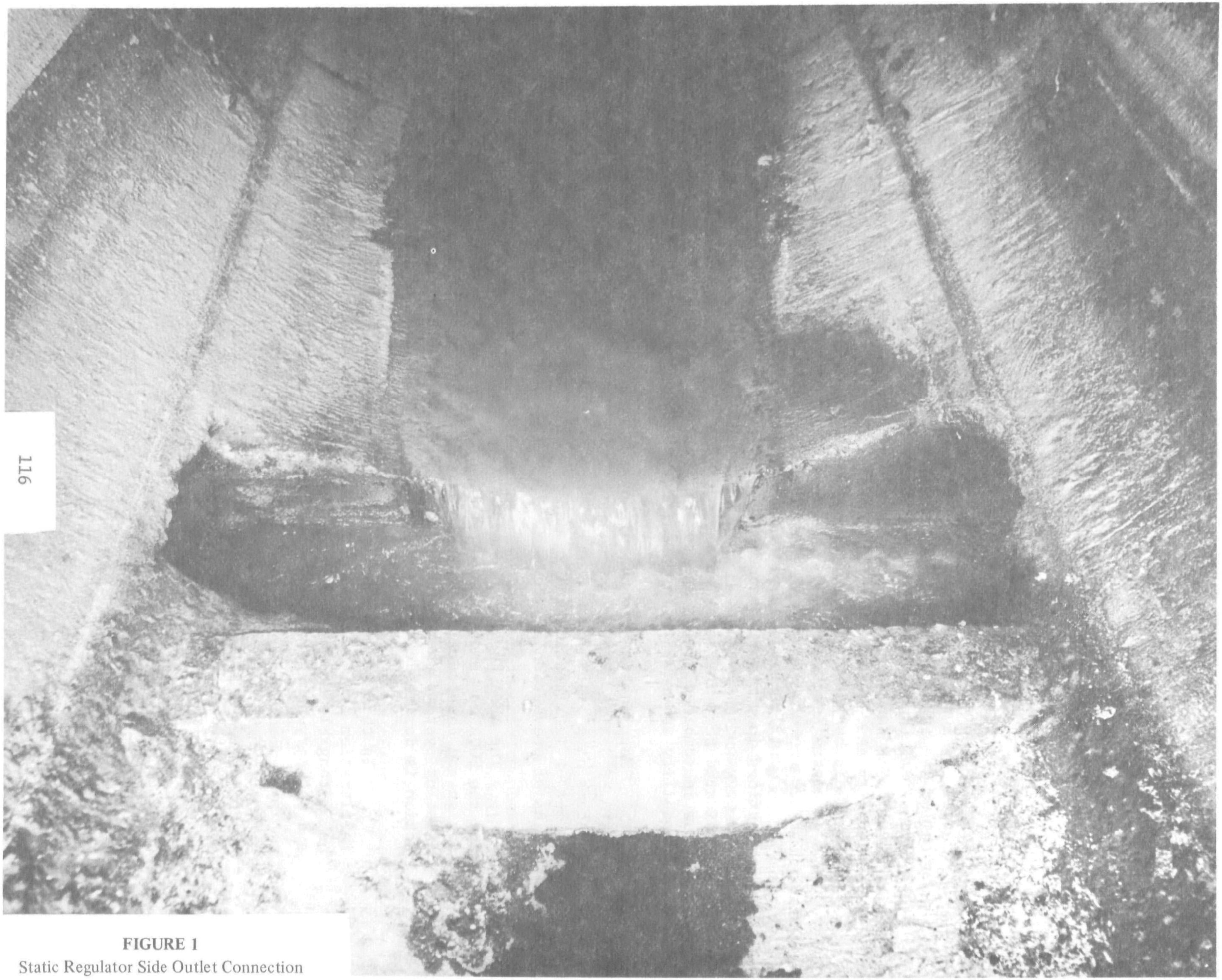
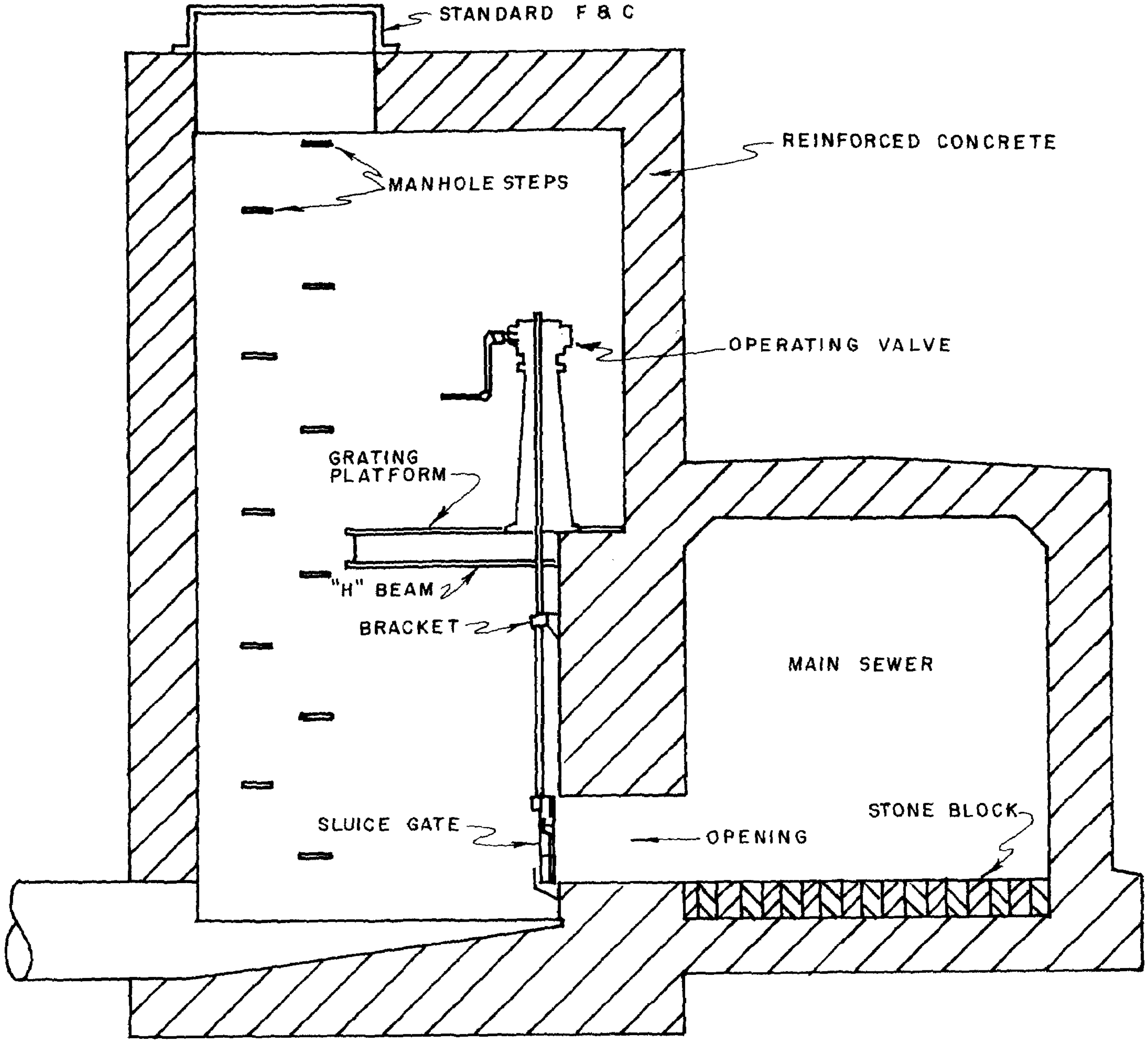


FIGURE 1
Static Regulator Side Outlet Connection



TYPICAL MANUALLY OPERATED GATE REGULATOR PHILADELPHIA

FIGURE 2

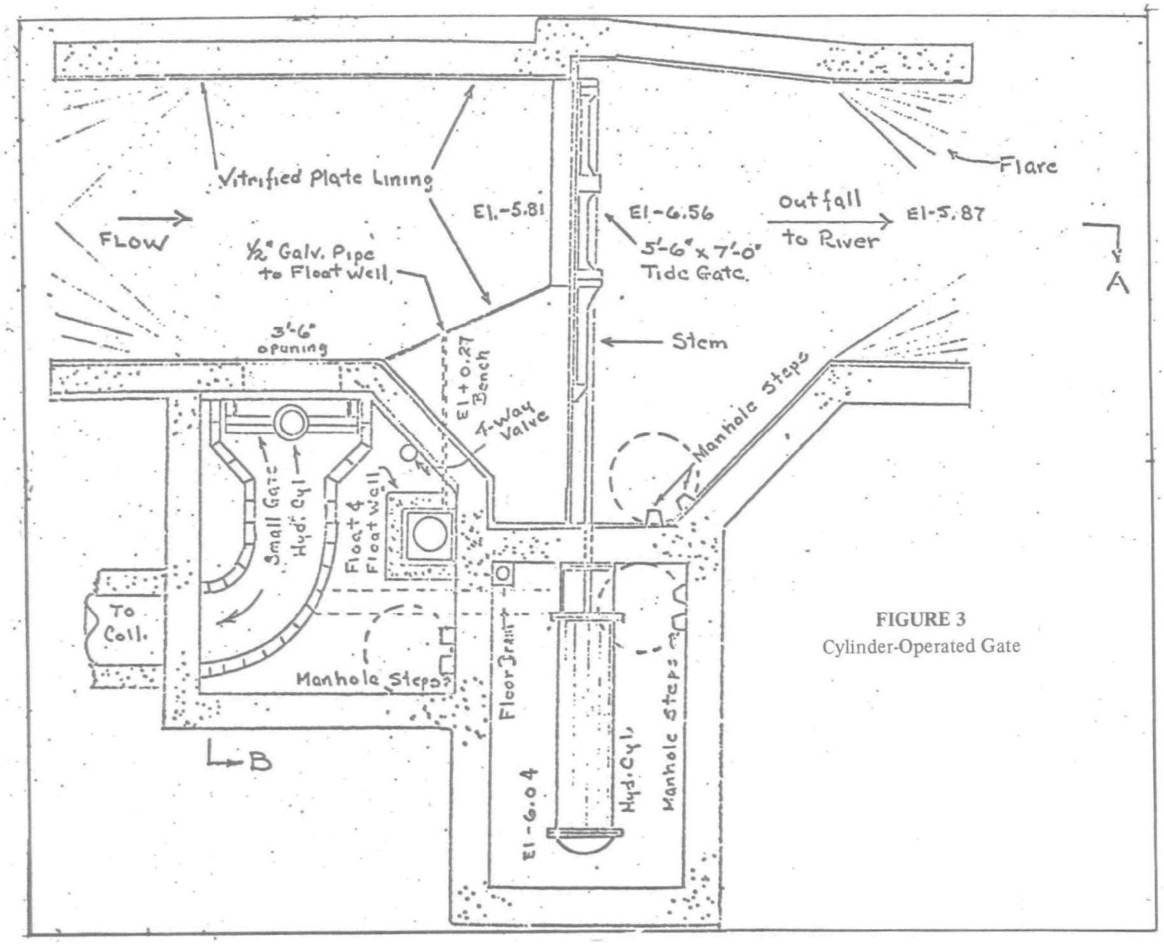


FIGURE 3
Cylinder-Operated Gate

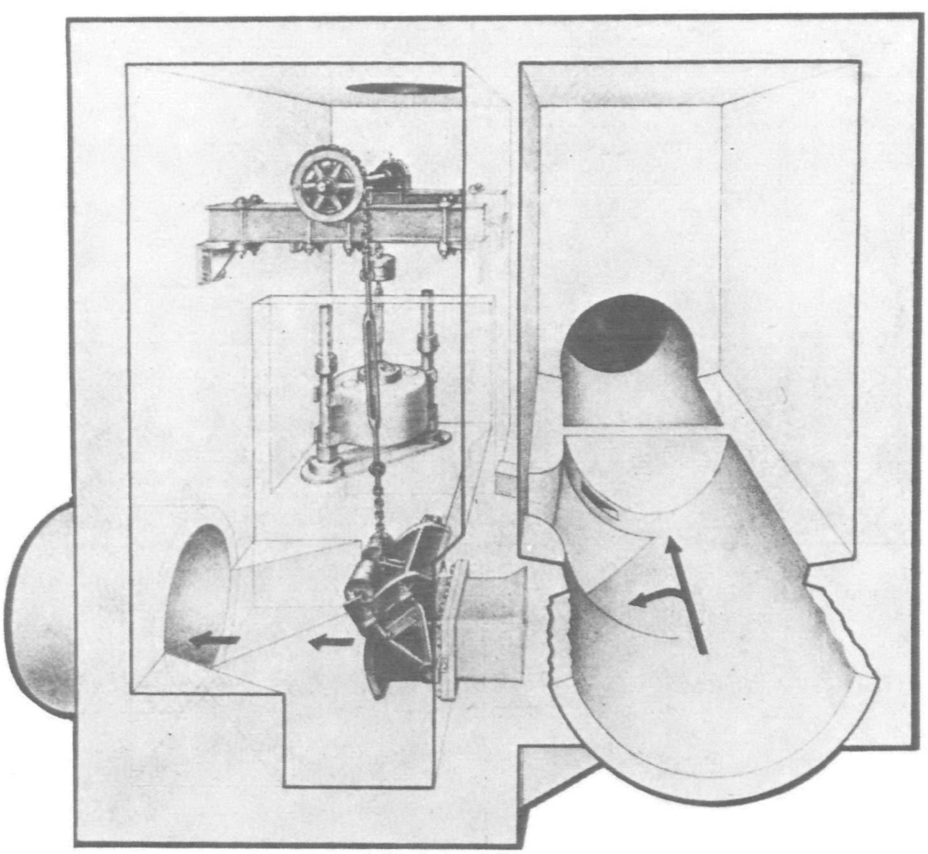
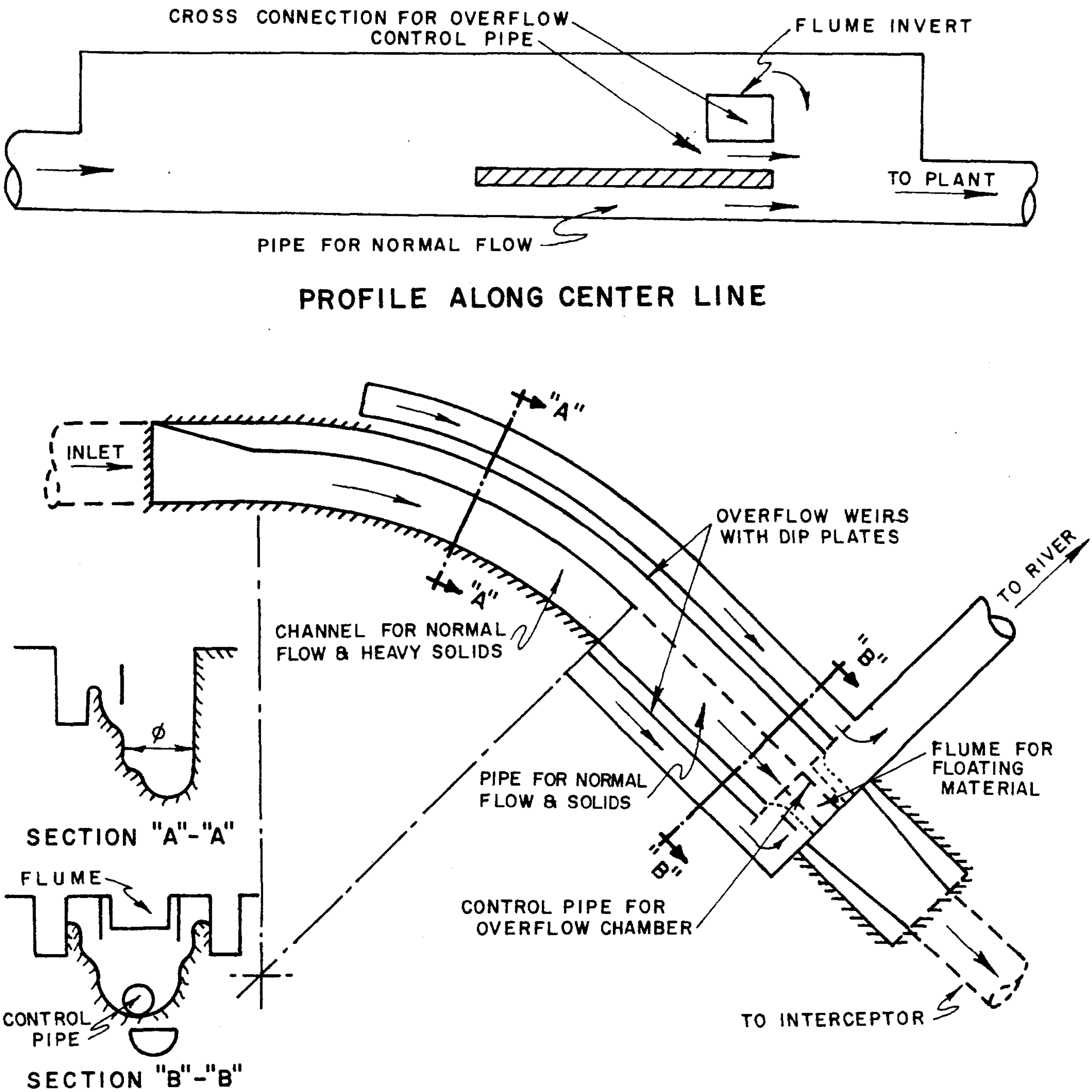


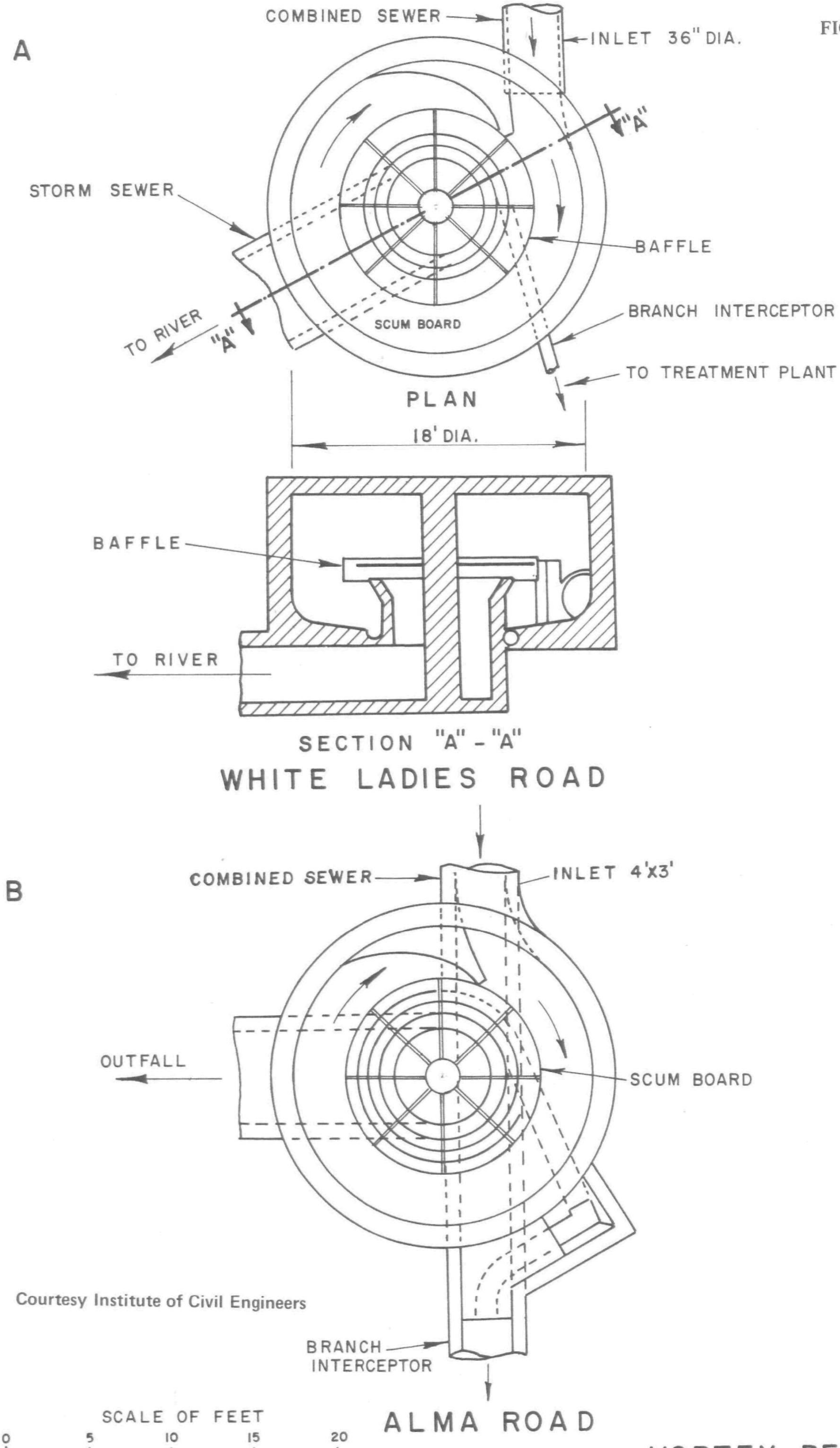
FIGURE 4
Float Operated Gate – Courtesy Brown & Brown, Manufacturing



SPIRAL FLOW (HELICAL)
REGULATOR

Courtesy Institute of Civil Engineers

FIGURE 6



Courtesy Institute of Civil Engineers

VORTEX REGULATORS

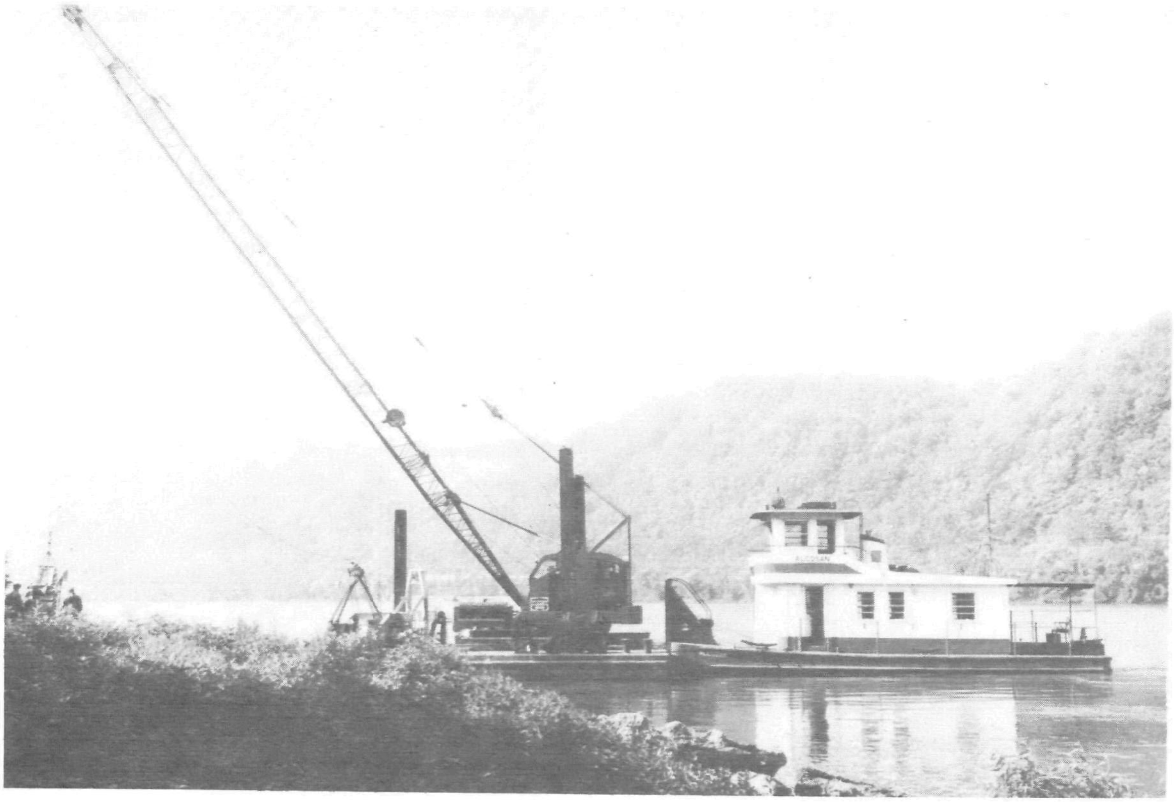


FIGURE 7
Boat and Barge Used by Allegheny County Sanitary Authority for Maintenance of Some Overflow Facilities

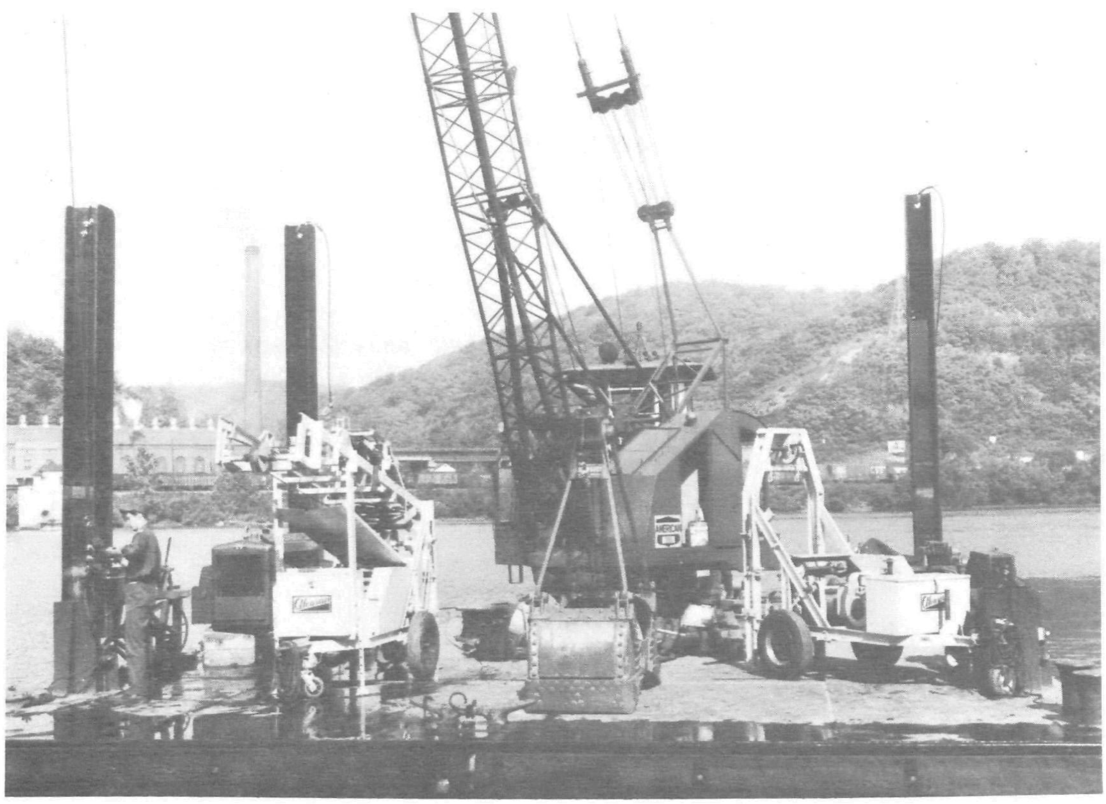


FIGURE 8
Derrick Barge with Equipment, Allegheny County Sanitary Authority.

SECTION 4

THE USE OF SCREENING/DISSOLVED-AIR FLOTATION FOR TREATING COMBINED SEWER OVERFLOWS

by

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for the
United States Department of the Interior
Federal Water Quality Administration

Contract #14-12-40

ABSTRACT

Results from the many projects now being sponsored by the Federal Water Quality Administration indicate that the majority of the pollutants present in combined sewer overflow are in the form of particulate matter. This indicates a high degree of treatment could be obtained by utilizing an efficient solids/liquid separation process. This report documents a study on the treatment of combined sewer overflow by screening and dissolved-air flotation. The objectives of the project are to determine the effectiveness and cost of a screening/flotation system.

A combined sewer (Hawley Road) in Milwaukee, Wisconsin was monitored and laboratory testing which included screening, chemical oxidation, and dissolved-air flotation was performed. The results of the laboratory tests indicated a combination of screening/flotation provided a feasible system and a prototype demonstration unit with a 5 MGD capacity was designed and installed.

The system has been operated on 30 overflows. Removals of BOD, COD, SS, and VSS have been in the range of 50 to 75%. The waste solids stream has averaged only about 1 percent by volume of the raw feed water. Operation has been very satisfactory with a minimum of maintenance required. Chemical flocculants have been utilized to increase the removal efficiencies to the upper values of the above range.

Cost estimates have been made and these indicate a total installed cost of $12,000 per MGD capacity. These costs do not include land or interceptor costs to combine a series of overflows. Operating costs are estimated at 1.0¢/1000 gallons without chemical flocculant addition. Chemical costs should be in the range of 2.0-2.5¢/1000 gallons.

Based on the results of this study screening/flotation offers an alternate to sewer separation in some application areas. The project reported herein is still underway and completion is expected in early summer of 1970.

INTRODUCTION

The pollutional characteristics of combined sewer overflow are being documented through the many federally sponsored projects which are now underway. Preliminary results indicate that the majority of the pollutional substances present in combined sewer overflow are in the form of particulate matter. This indicates that a high degree of treatment could be obtained by utilizing an efficient solids/liquid separation process. The objectives of this project (FWQA Contract #14-12-40) are to determine the design criteria, effectiveness, and economic feasibility of using screening and dissolved air flotation to treat combined sewer overflows.

The project is currently underway. Completion is expected by late spring or early summer of 1970. Discussed herein is a review of the results obtained to date, tentative design criteria, and expected removal efficiencies.

SUMMARY AND CONCLUSIONS

Based on the data collected during the study and reported herein, it appears that screening/dissolved-air flotation can be utilized as a successful alternate to sewer separation in some areas. Removals of BOD, COD, SS, and VSS in the range of 50-75% were recorded for the 30 overflows monitored to date. The solids removed from the overflows represented only about 1 percent (by volume) of the raw wastewater flow and had a concentration of 2 to 4%. The entire system is completely automated and requires a minimum of maintenance.

Cost estimates indicate the complete installed system capital cost will be $12,000 per MGD capacity. This cost does not include land or sewer interconnection costs. Operating costs were estimated at 3.0 to 3.5¢/1000 gallons based on the use of flocculating chemicals to obtain the maximum removal efficiency. Operating costs without chemicals is estimated at less than 1.0¢/1000 gallons.

DESIGN OF TEST FACILITY

During the fall and spring of 1967, the Hawley Road Combined Sewer in Milwaukee, Wisconsin was monitored. A total of 12 overflows were sampled. Laboratory scale testing on these samples included screening with various size media, chemical oxidation, flotation, and disinfection. Laboratory analyses on the untreated overflow as well as the effluents from the laboratory bench tests were analyzed for BOD, COD, SS, VSS, and disinfection requirements. It was determined from this testing that chemical oxidation did not appear technically feasible (1). However, encouraging results were obtained from the screening and flotation tests. These tests served as input data in the design of a test facility utilizing screening and dissolved-air flotation. A process flow sheet for the system is shown in Figure 1.

The system basically consists of a screen chamber and a flotation

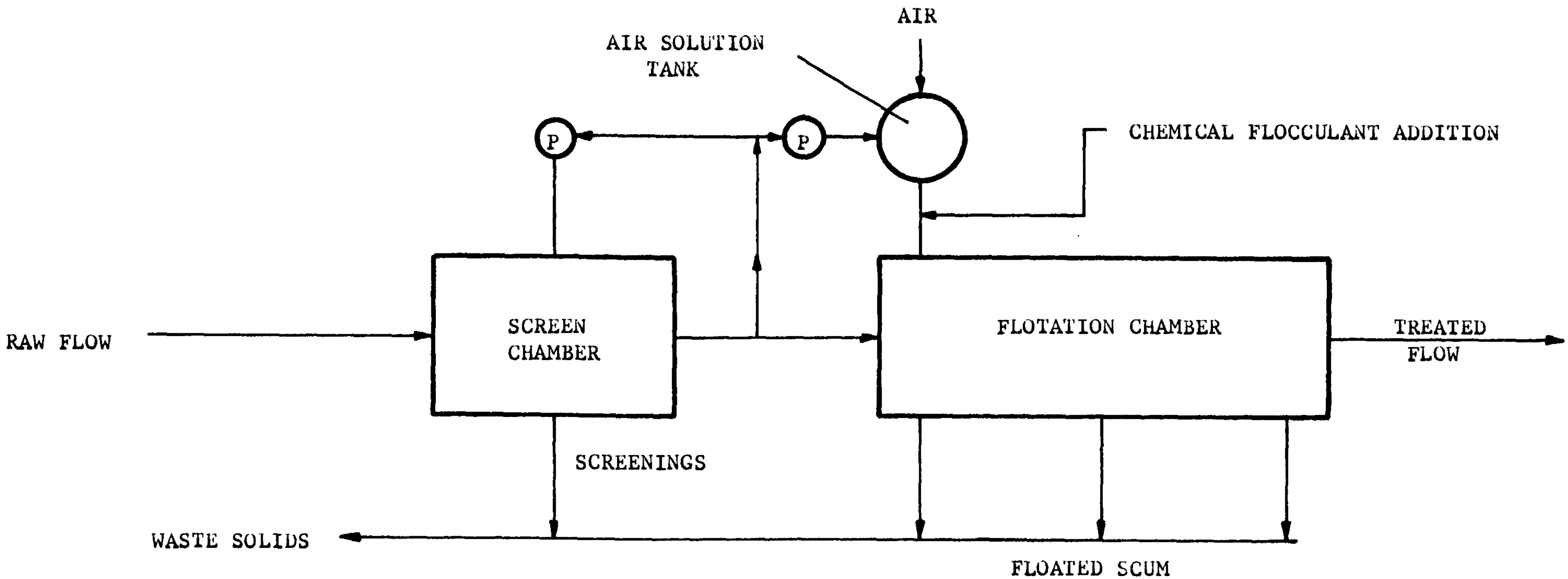


FIGURE 1

PROCESS FLOW SHEET

chamber. The screen is an open ended drum into which the raw waste flows after passing a ½" bar rack. The water passes through the screen media and into a screened water chamber directly below the drum. The drum rotates and carries the removed solids to the spray water cleaning system where they are flushed from the screen. Screened water is used for flushing. The spray water and drum rotation are controlled by liquid level switches set to operate at 6 inches of head loss through the screen. The flotation chamber is a rectangular basin with a surface skimming system to remove floated scum. Screened water is pressurized and mixed along with air in an air solution tank. The liquid becomes saturated with air and when the pressure is reduced minute air bubbles (less than 100 micron diameter) are formed. This air charged stream is then mixed with the remaining screened water flow. The bubbles attach to particulate matter and float it to the surface for subsequent removal by the skimmers. Chemical flocculants may be added to enhance the removal efficiency of finely divided particulate matter.

The design criteria utilized in the design of the test facility are shown in Figure 2. These criteria provide the wide flexibility necessary in a test facility. More precise design criteria will be given later. The system was designed to treat 5 MGD of combined overflow.

FIGURE 2

GENERAL DESIGN CRITERIA FOR DEMONSTRATION SYSTEM

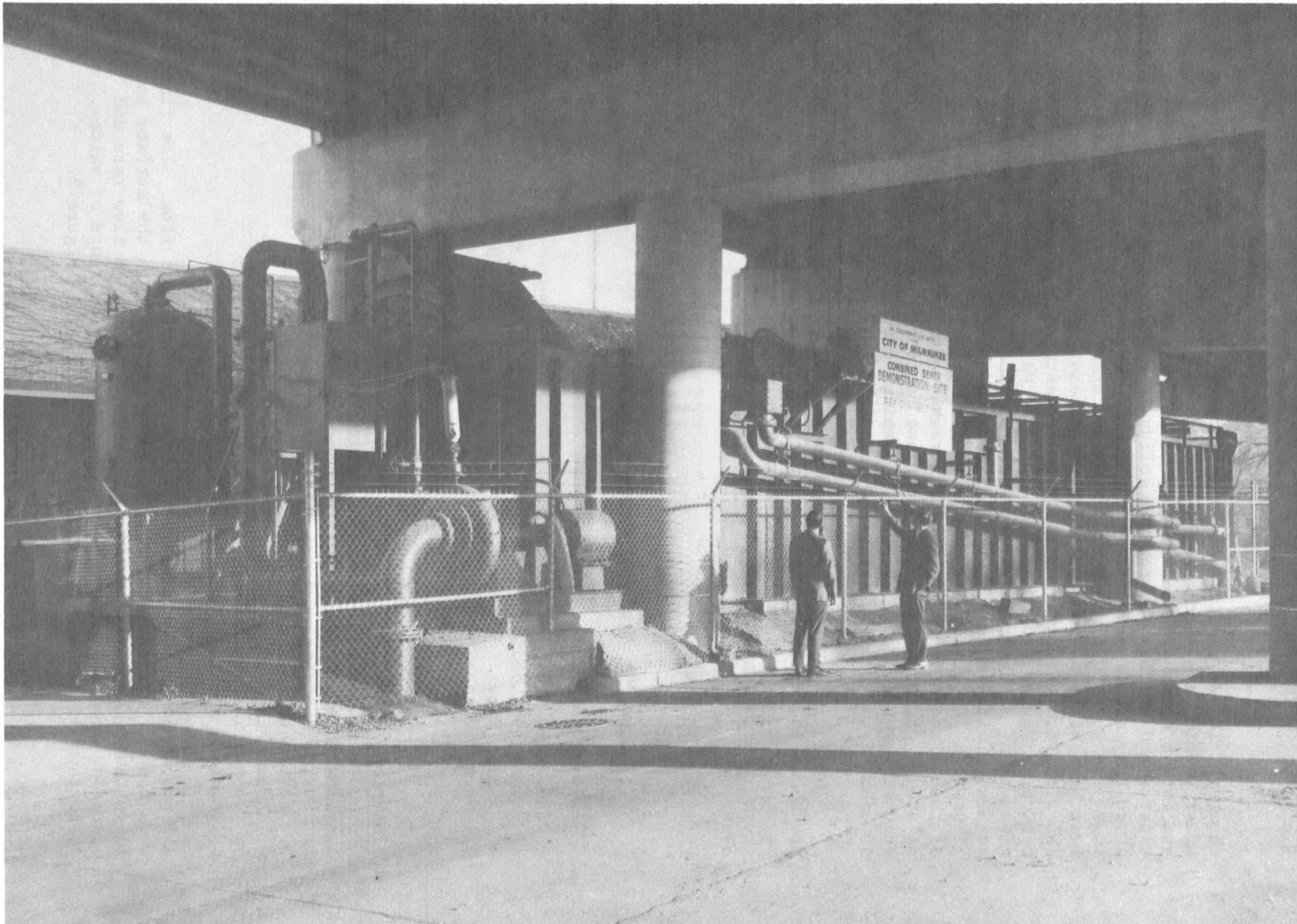
Screen

1.	Raw Flow Rate	3500 GPM
2.	Hydraulic Loading	50 GPM/sq ft
3.	Screen Size	50 x 50 297 micron openings
4.	Screen Wash	150 GPM maximum

Flotation Tank

1.	Flow Rate	3500 GPM
2.	Surface Loading	3-9 GPM/sq ft
3.	Horizontal Velocity	3 FPM
4.	Pressurized Flow Rate	400-1100 GPM
5.	Operating Pressure	40-70 PSIG
6.	Minimum Particle Rise Rate	0.5-1.5 FPM

All pumps and auxiliary equipment were sized on this flow. The flotation tank is compartmentalized to allow variation in the surface loading without changing the raw flow rate. Pressurized flow rate and operating pressures can be maintained over a wide range of values. A photograph of the demonstration system is shown in Figure 3.


CITY OF MILWAUKEE
COMBINED SEWER
DEMONSTRATION SITE

RESULTS OF OPERATION

The test facility was completed and put on stream in May of 1969. Since that time 30 overflows have been monitored. It has been observed that about 25% of these overflows have high pollutional load during the first portion of the overflow. This period of first flushes has never lasted longer than one hour and has been as short as 10-15 minutes. After these flushes pass the characteristics of the overflow become relatively constant. This period has been called the extended overflow period. The range of pollution parameters measured for these 30 storms at the 95% confidence level is shown in Figure 4.

FIGURE 4

CHARACTERISTICS OF COMBINED SEWER OVERFLOW FROM HAWLEY ROAD SEWER

	First Flushes
COD	500-765
BOD	170-182
SS	330-848
VSS	221-495
Total N	17-24
	Extended Overflows
COD	113-166
BOD	26-53
SS	113-174
VSS	58-87
Total N	3-6

All values in mg/l at 95% confidence level.

Coliform 310 x 10^3 to 1.5 x 10^3 per ml.

It may be observed that the first flushes data has quite a wide range of values, while the extended overflow data has a relatively narrow range. All laboratory analysis were performed according to Standard Methods (2). The data presented correlates well with combined overflow data from the Detroit Milk River Study (3) and other published data (4).

The operation of the previously described test facility during the spring, summer and fall of 1969 has provided valuable data on operational characteristics and removal rates. Figure 5 shows the data associated with operational variables. The range of values for screen wash and floated scum volume are shown at the 95% confidence level.

FIGURE 5

OPERATION DATA FROM HAWLEY ROAD DEMONSTRATION SYSTEM

Length of Run Hours	Raw Flow Rate GPM	Screen Wash As % of Flow %	Floated Scum As % of Flow %	Pressurized Flow Rate GPM	Operating Pressure PSIG
1-4	3500	0.29-0.64	0.43-0.85	400-900	40-60

The average run had a length of 1-4 hours. The flow rate for these runs was held constant at 3500 gpm. Pressurized flow was varied over the range of 400-900 gpm and the operating pressure from 40-60 psig. Of considerable importance in the design of this type of system, is the volume of residual solids produced during operation. As shown in Figure 5, the volume of water required to backwash and clean the screen ranges from 0.29 to 0.64 percent of the raw flow rate, while the volume of floated scum ranges from 0.43-0.85 percent at the 95% confidence level. Solids concentrations in these streams generally is in the range of 1 to 2%, and at this concentration they easily flow by gravity. Disposal methods utilized for these solids streams should be sufficient to handle the upper limit of the expected sludge volumes. Under the current contract, the solids are disposed via an interceptor sewer which directs them to the sewage treatment plant. Other alternatives for solids disposal include trucking in tanker trucks or providing a portable vacuum filter to visit the various treatment sites and produce a dry cake for hauling to ultimate disposal.

The efficiency of contaminant removal experienced for the overflows monitored to date, is shown in Figure 6. All runs were started with the tank full of water from the previous run. For this reason collection of effluent composite samples was not started until 15-20 minutes into the run to avoid collecting unrepresentative samples. All other samples were taken immediately. The tank can also be operated in a near empty mode for start up. Only a small amount of water is required to allow immediate start up of the pressurized flow system. Clarification will then start immediately as raw waste begins to enter the tank.

FIGURE 6

CONTAMINANT REMOVALS IN PERCENT BY SCREENING AND FLOTATION

	Screening		Screening and Flotation	
	Spring	Summer-Fall	W/O Chemical Flocculants (Spring)	W/Chemicals Flocculants (Summer-Fall)
BOD	23.4 ± 9.3	20.3 ± 6.5	48.4 ± 15.7	50.8 ± 12.5
COD	33.9 ± 10.7	22.4 ± 5.0	52.9 ± 8.7	53.4 ± 8.6
SS	28.8 ± 10.5	24.9 ± 9.8	53.7 ± 11.7	68.3 ± 8.4
VSS	28.2 ± 13.6	24.4 ± 13.2	51.0 ± 15.9	64.8 ± 10.0

NOTES: Removals as % @ 95% confidence level.
Screen openings 297 microns.
Surface loading 3 GPM/sq ft.

Two time periods are shown, spring storms and summer/fall storms. By observing the screen data in Figure 6, it may be seen that during the spring storms removals ranged from 23-33 percent for all listed parameters. This was consistent with the preliminary data collected the previous year. During the summer/fall storms, however, COD removals decreased indicating a change in the characteristics of the overflow. It was determined that an increase in soluble organics had occurred which was the probable cause for the noted decrease in COD removal across the screen. The mechanical operation of the screen has been very satisfactory. The media utilized was type 304SS. No permanent media blinding has been experienced. No build-up of greases or fats has occurred. Some clogging problems have been experienced with the spray nozzles, but this was caused by a sealing problem around the screen, which allowed unscreened water to pass into the screened water chamber.

The overall removals, i.e. screening plus flotation are also shown in Figure 6. Removals are shown with and without the addition of chemical flocculants. The chemical flocculants when utilized were a cationic polyelectrolyte (Dow C-31) and a flocculant aid (Calgon A25). The polyelectrolyte dosage was 4 mg/l and the coagulant aid dosage was 8 mg/l. Contaminant removal without chemical addition was about 50% for all parameters as shown in Figure 6. Adding chemicals caused an increase in SS and VSS removals to around 70%. COD and BOD removals, however, did not increase significantly. This was probably a result of the increase in soluble organics associated with the summer/fall overflows. Chemical addition also provided a strengthening effect on the floated sludge blanket which is very desirable from the solids handling aspect. Mechanical operation of the flotation tank has been excellent. No mechanical problems have been experienced. Maintenance on the entire system is limited to periodic lubrication and requires less than 6 man hours per month.

Another important aspect in the treatment of combined overflow is disinfection. Figure 7 shows the effect of chlorination on total coliform density from various overflows.

FIGURE 7

DISINFECTION DATA FOR COMBINED OVERFLOWS AT HAWLEY ROAD

Storm #	Raw Coliform Density per ml	Chlorine Dosage mg/l	Contact Time min.	Effluent Coliform Density per 100 ml
5	36,000	10	15	0
6	5,700	10	15	0
7	1,300	10	15	0
8	7,800	10	15	0
9	6,200	10	15	2
11	20,000	10	15	10
19	310,000	10	10	600
20	160,000	10	10	400
21	55,000	10	10	0
22	82,000	10	10	1500

In storms 5 through 11 chlorine was added in the pressurized flow line prior to blending with the remainder of the flow in the flotation tank. The dosage was 10 mg/l. The dosage may have actually been lower in some of the runs, since sodium hypochlorite was utilized as the source of chlorine and this solution decreases in strength over a relatively short period of time. Introduction of the chlorine in the pressurized flow allowed approximately 15 minute contact time before discharge from the unit. In storms 19 through 22 chlorine was added to the effluent from the flotation basin and allowed to react for a ten minute period. The chlorine was then deactivated with sodium sulfite and coliform analyses were performed. It may be observed in Figure 7, that coliform reduction was related to initial coliform density when using a constant chlorine dosage. In the spring and early summer when coliform densities were low, good disinfection was obtained. However, in late summer when coliform density increased, the effluent contained increased numbers of coliform organisms. Chlorine demand tests were run on some storms. The chlorine demand was generally in the range of 13 to 17 mg/l.

CONCEPTUAL DESIGN

The use of screening/flotation in full scale installations to treat combined sewer overflows requires integration of a variable rate pumping system, a screening/flotation system, and a solids storage and/or disposal system. The full scale design will be based on a modular concept. It is envisioned that a number of screening/flotation modules will be assembled and operated in parallel. A pumping system and solids storage/disposal system will complete each treatment site. All components in the integrated system can be automated 100%. Telemetry will probably be utilized to send the data and monitor the system from a central location. Figure 8 illustrates the complete treatment system concept. Raw wastewater enters the sump at a variable rate. The pumping system consists of a series of pumps of both fixed and variable capacity. A depth gauging system controls the pump output to match raw waste input. The feed pumps direct the waste flow to the proper screening/flotation module. It is anticipated that a single screen will feed two flotation cells. Screening/flotation modules will be put into service automatically as the flow rate increases. Each screening/flotation module is capable of a 50% hydraulic overload without a significant decrease in efficiency. This excess capacity will be utilized after all modules have been put into service. The solids removed from the flow will be stored or transported to the sewage treatment plant via an inceptor sewer. As the raw flow subsides various modules will be removed from service automatically until the overflow is terminated. The system is thus a floating system with modules automatically put into or taken out of service as required.

Based on the data taken during 30 overflows, Figure 9 presents the recommended design criteria for screening and dissolved-air flotation systems treating combined sewer overflow. This criteria is tentative, since the project has not yet been completed. The most important criteria associated with screen design include hydraulic loading and solids loading. The recommended values are those which

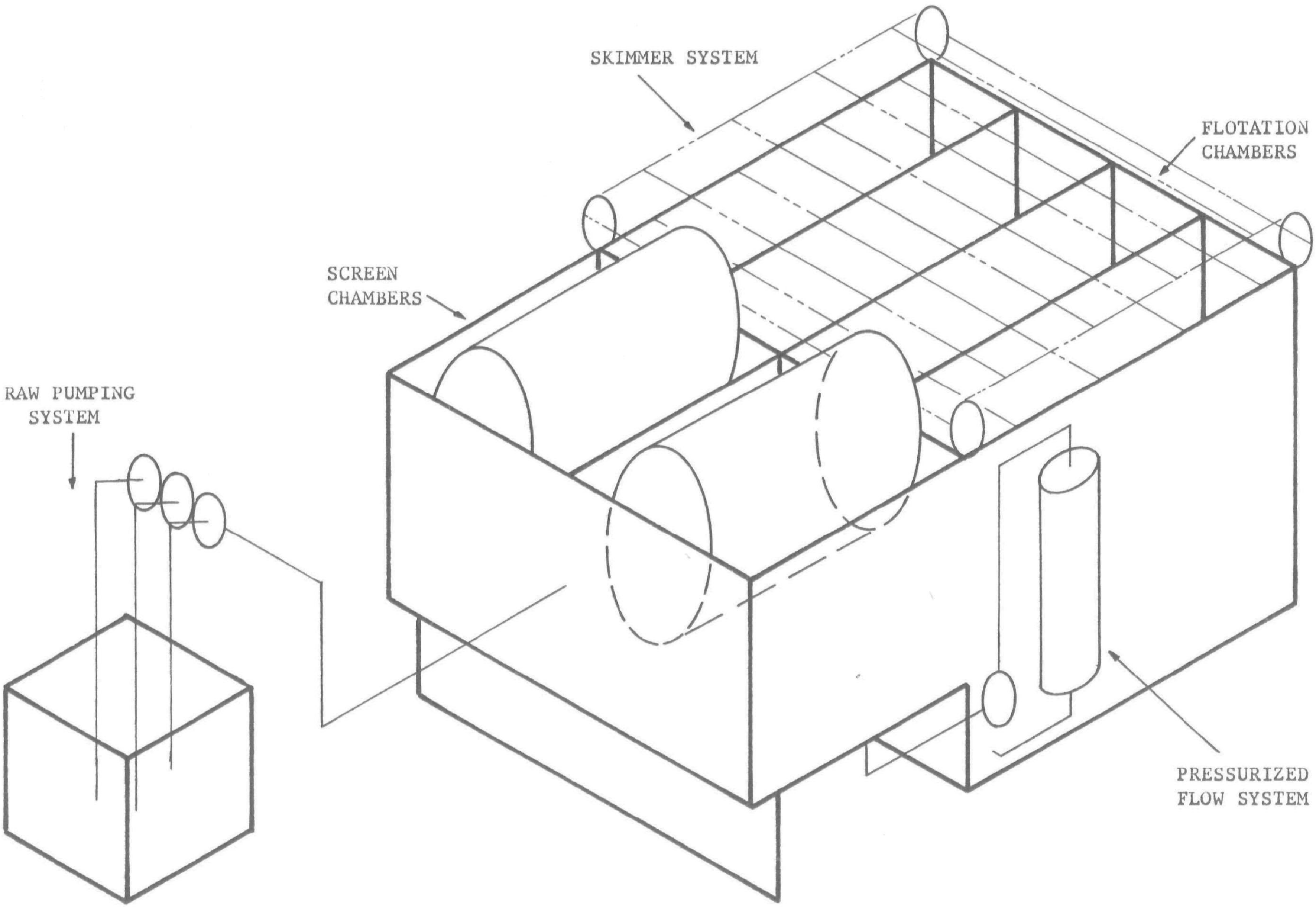


FIGURE 8

SYSTEM LAYOUT

FIGURE 9

RECOMMENDED DESIGN CRITERIA FOR SCREENING AND FLOTATION

Screens	Flotation
Media - 50 x 50 (297 Micron Openings)	Surface Loading - 3 gpm/sq ft[1]
Hydraulic Loading - 50 gpm/sq ft	Horizontal Velocity - 3 fpm
Head Loss Capability - 14 inches water	Pressurized Flow - 15%
Solids Loading - 1.4 # DS/100 sq ft	Operating Pressure - 50 psig
Cleaning Water - 0.75% Screened Flow	Floated Scum Volume -0.95% of Flow
	Provisions for Top and Bottom Skimming
	Chemical Flocculant Addition

(1) This value may be conservative, higher values now being tested.

were found satisfactory in the operation of the Hawley Road facility.

With regard to the flotation design criteria, the surface loading variable is the only one which has not been fully evaluated. Higher rates are now being investigated, and the affect of these rates on removal efficiencies will be evaluated. The other criteria for flotation shown in Figure 9 have been thoroughly evaluated and proven adequate for combined sewer overflow treatment.

ESTIMATED COSTS OF SYSTEM

There are many factors which must be considered when estimating costs for a combined sewer overflow treatment system. The basic areas of consideration are listed below:

1. Screening/flotation system (based on a particular storm intensity/frequency and runoff rates)

2. Variable rate pumping system

3. Solids storage and/or disposal

4. Land costs

5. Sewer interconnection costs (It is anticipated that a number of overflow points will be combined to reduce the number of treatment sites required.)

6. Instrumentation and data telemetry

Estimated costs discussed herein include those costs associated with items 1, 2, 3 and 6 listed above. Items 4 and 5 are particular to the individual treatment system and hence cannot be estimated in a general manner. These costs are therefore not included here. Total installed cost for the screening/flotation system is estimated at $8,000 per MGD capacity. Installed costs for solids storage, variable rate pumping system and instrumentation is estimated at $4,000 per MGD capacity. The total system costs less sewer interconnection and land cost is therefore $12,000 per MGD capacity. This cost estimate does not include consulting engineering fees nor cost of special design considerations if they are required. This cost is based on an overflow rate of 3 gpm/sq ft. If higher overflow rates are possible costs will be reduced.

A conceptual design and cost estimate has been made for a complete storm overflow treatment system in a small Wisconsin city. A number of overflow points were combined to reduce the number of treatment sites. All storm sewers were screened and all combined sewer overflows were treated by screening/flotation. A total of 294 acres was served by the system and the design was based on the once in two year storm. At a 50% overflow capacity the system will handle the once in 4.5 year storm. Total system costs including installation was $835,000. Total treatment capacity was 80 MGD design and 120 MGD peak flow. Of this 80 MGD approximately 40 MGD is combined overflow and the remaining is storm sewer overflow. Included in this cost was the combining of 12 overflow points into 5 treatment sites. The cost estimates also include engineering fees. All land was owned by the city so no land costs are included in these prices. The above prices stated on a per acre served basis is equivalent to about $2800/acre.

Operating costs for a screening and flotation system will be low due to the expected periodic usage when treating combined overflow. Chemical costs should be in the range of 2.0 to 2.5¢/1000 gallons, while operating, maintenance and power costs are expected to be less than 1.0¢/1000 gallons.

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SECTION 5

UNDERFLOW PLAN

FOR

POLLUTION AND FLOOD CONTROL

IN THE

CHICAGO METROPOLITAN AREA

STATE OF ILLINOIS
DEPARTMENT OF PUBLIC WORKS AND BUILDINGS
WILLIAM F. CELLINI, DIRECTOR

METROPOLITAN SANITARY DISTRICT
OF GREATER CHICAGO
BEN SOSEWITZ, ACTING GENERAL SUPERINTENDENT

CITY OF CHICAGO
DEPARTMENT OF PUBLIC WORKS
MILTON PIKARSKY, COMMISSIONER

MAY, 1970

ABSTRACT

To solve the problems of flooding and water pollution in the Chicagoland area, a number of plans have been proposed and studied. Three of these plans, the Underflow-Storage Plan, the Deep Tunnel Plan and the Chicago Drainage Plan, are still viable alternates for the total solution to meet the water quality standards established by the State and Federal Governments and the requirement of handling the runoff from a 100-year 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 FWQA. 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.

Further evaluation of the three plans indicates that portions of the Underflow-Storage Plan designated for the First Phase construction are compatible with future extensions along the general conceptual lines of any of the three plans. It is recommended that the final design of the First Phase work proceed and that all alternates for the Second Phase be thoroughly and systematically studied concurrently to determine the final plan. It is necessary to proceed at the earliest possible time to meet the water quality compliance date of 1978.

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DRAINAGE AND POLLUTION PROBLEMS
IN METROPOLITAN CHICAGO

THE FLOOD CONTROL PROBLEM

Since the end of the Second World War, Metropolitan Chicago has undergone a period of extensive urban development. This development has caused a tremendous increase in the impervious area and larger surface runoff during storm periods. To alleviate local flooding of basements and underpasses throughout Chicagoland, hundreds of millions of dollars have been expended in the construction of new sewerage. While greatly reducing the undesirable storage of water in basements and underpasses, a new and increasing problem of flood control in the rivers and canals is becoming apparent.

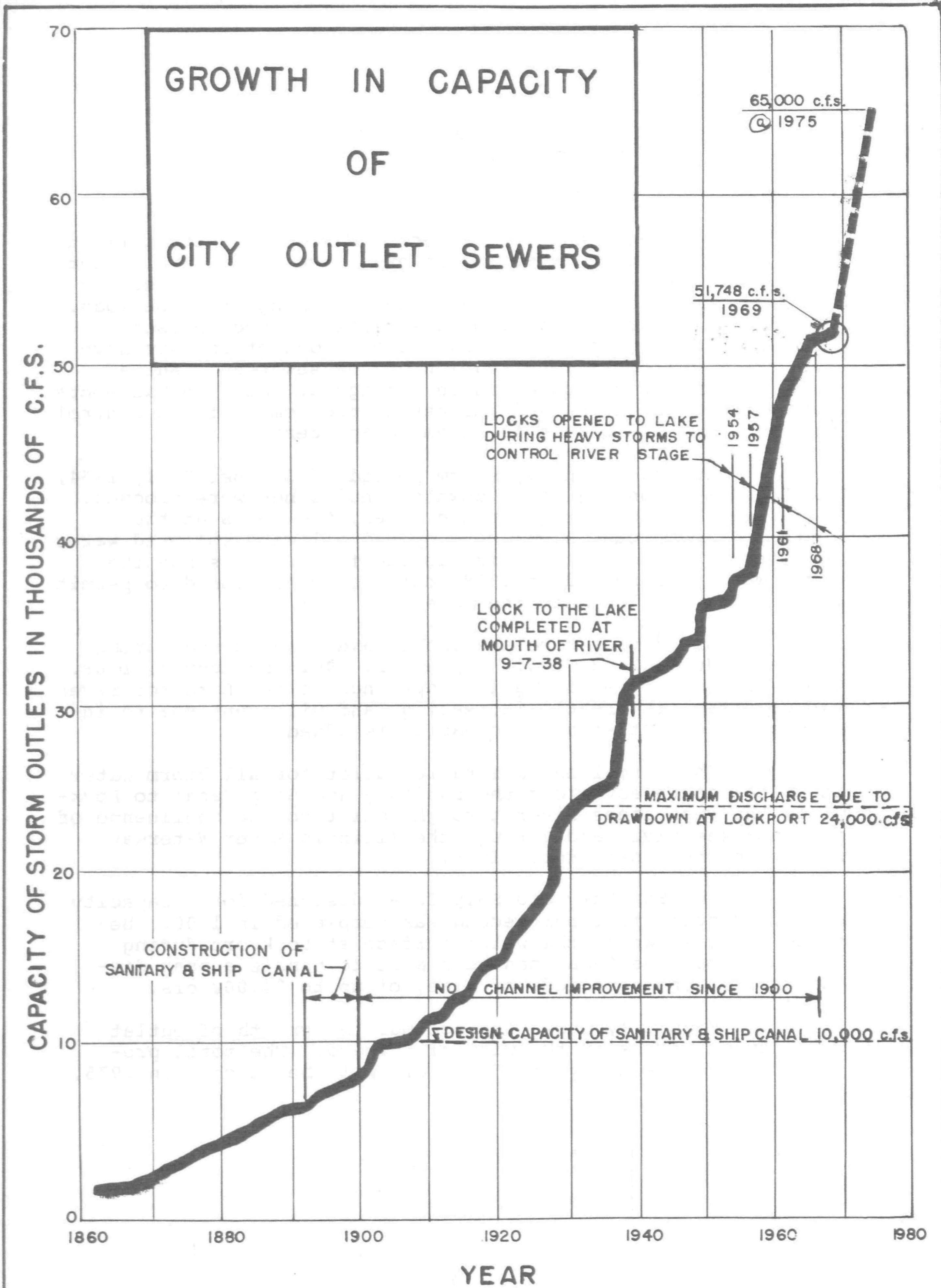
During the heavy storm period of October 9-11, 1954, the Union Station and other downtown buildings were flooded. To reduce the flood stage in the river, the locks at the mouth of the Chicago River were opened allowing polluted water to enter Lake Michigan. This was the first time since the locks were constructed in 1938 that they were opened to permit river water to flow into the lake.

Since that time the locks have been opened during storms of July 12-13, 1957, September, 1961 and August, 1968. The frequency of requiring lock openings to the Lake for river flood control is greatly increasing, and will continue to increase as new outlet sewer capacity is added.

The normal and desirable outlet for all storm water is to the southwest along the Sanitary and Ship Canal to Lockport, the DesPlaines River through Joliet to the confluence of the Kankakee River and through the Illinois River Waterway System to the Mississippi River.

The Sanitary and Ship Canal designed for a capacity of 10,000 cubic feet per second was completed in 1900. Because of drawdown of the water surface at Lockport during heavy storms, the Canal has been able to handle a peak discharge, for short periods of time, of up to 24,000 cfs.

Figure 1, shows the accumulated growth of outlet capacity of sewers in the City of Chicago. The total projected outlet capacity of 65,000 cfs will be reached in 1975.



GROWTH IN CAPACITY OF CITY OUTLET SEWERS
CAPACITY OF STORM OUTLETS IN THOUSANDS OF C.F.S.
YEAR
0
10
20
30
40
50
60
70
1860
1880
1900
1920
1940
1960
1980
65,000 c.f.s. @ 1975
51,748 c.f.s. 1969
LOCKS OPENED TO LAKE DURING HEAVY STORMS TO CONTROL RIVER STAGE
1954
1957
1961
1968
LOCK TO THE LAKE COMPLETED AT MOUTH OF RIVER 9-7-38
MAXIMUM DISCHARGE DUE TO DRAWDOWN AT LOCKPORT 24,000 cfs
CONSTRUCTION OF SANITARY & SHIP CANAL
NO CHANNEL IMPROVEMENT SINCE 1900
DESIGN CAPACITY OF SANITARY & SHIP CANAL 10,000 c.f.s.

FIGURE 1

This, of course, is not the required capacity of the waterway system because of channel storage and offsetting of the sewer discharge peaks; however, it is a good indicator of the future flood control problems that lie ahead.

The DesPlaines River north of Hofmann Dam in Riverside has inadequate capacity to drain its fastly urbanizing tributary area. Large storage reservoirs and/or increased conveyance capacity must be provided to handle the increasing runoff.

In the Calumet Area, large acreage is only a few feet above normal water level of the Calumet River and waterway systems. In many places this provides only small gradients for the tributary streams and sewer. During large storms, the O'Brien Locks must be opened permitting river water to flow through the Calumet River to Lake Michigan. But even this will not keep the stage sufficiently low in the largest storm periods.

THE WATERWAY POLLUTION PROBLEM

The pollution of the waterway system is another vital problem confronting the Chicago Metropolitan Area. This same problem exists for nearly every other large metropolitan area in the Country. Most of these urban concentrations are drained by systems of combined sewers which spill to the open water courses when the sanitary intercepting sewers or treatment plants are overloaded.

Combined sewers have been estimated to carry approximately 3 percent of the annual sewage volume to the waterways during storm overflow periods, thus 97 percent of the annual sewage volume is delivered to the treatment plants. The actual annual pollution load which is discharged from combined sewers to the waterways is somewhat greater. This is due to the cleansing of the sewer inverts during periods of high storm runoff.

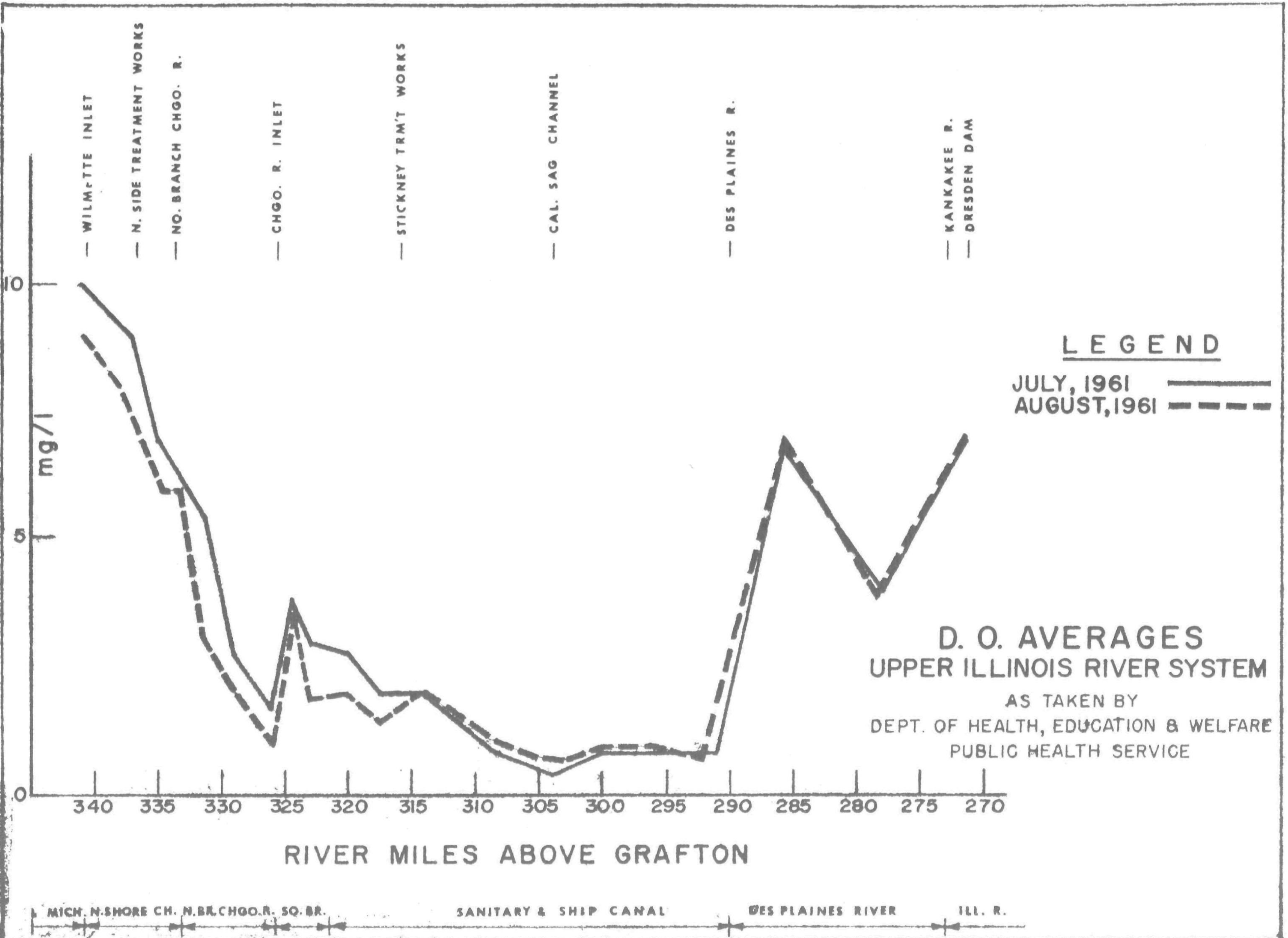
In addition to the pollution of the river caused by the combined sewer systems, other major contributors are the sewage treatment plants. Three major treatment plants handle the household and industrial wastes for the City of Chicago and much of the suburban area within the Metropolitan Sanitary District. These plants are the North Side Treatment Works, the West-Southwest Treatment Works and the Calumet Treatment Works.

The U.S. Public Health Service study, "Great Lakes, Illinois River Basin Project" (1) (GLIRBP) in two separate periods of study in 1961 found the combined effluent of the three plants was 1238 and 1682 MGD; at a population equivalent (PE) of 969,000 and 793,000; which is equal to 78.2 and 66.2 tons of 5 day B.O.D. per day, respectively. The average overall efficiency for these two periods was 88.3 percent. The reduction in the effluent PE during the second period was attributed to the heavy rainfalls occurring during that period resulting in the direct overflow of pollutants to the watercourses by combined sewers and therefore not measured at the plants.

Extensive sludge deposits are formed in the waterways downstream of the treatment plants and the many large outfall sewers. These sludge deposits have a significant oxygen demand and thereby use up a large part of the natural oxygen content in the waterways. At many places where these large sludge deposits occur, gaseous bubbles are released to dot the water surface and result in extensive odors along the river channel.

Other sources of pollution of the rivers are the discharges from industries, and leakage from boats and barges. Industries also use river water for cooling purposes increasing the temperature by several degrees; this reduces the amount of dissolved oxygen the water can hold.

The dissolved oxygen (DO) is one of the most important constituents of the waterway system. All of the above sources of pollution tend to deplete the dissolved oxygen. Figure 2 shows the DO in the North Shore Channel, Chicago River, Sanitary and Ship Canal and the DesPlaines River to the confluence with the Kankakee River for the two most critical months of the year. The DO at Wilmette near saturation during this period, shows a marked reduction below the North Side Treatment Works, down to near one mg/l just upstream of the main stem of the Chicago River where fresh lake water is introduced. The replenished DO is quickly reduced by the B.O.D. present in the water plus that of the sludge deposits on the bottom. Below the West-Southwest Treatment Plant, considerable DO is added along wth the B.O.D. in the effluent from the plant. The DO continues to diminish to near zero along the Sanitary and Ship Canal to Lockport. Aeration at Lockport and the flow from the DesPlaines River adds to the DO. Again at the Brandon



WILMETTE INLET
N. SIDE TREATMENT WORKS
NO. BRANCH CHGO. R.
CHGO. R. INLET
STICKNEY TRM'T WORKS
CAL. SAG CHANNEL
DES PLAINES R.
KANKAKEE R.
DRESDEN DAM
mg/l
10
5
0
340 335 330 325 320 315 310 305 300 295 290 285 280 275 270
RIVER MILES ABOVE GRAFTON
MICH. N.SHORE CH. N.BR.CHGO.R. SO.BR.
SANITARY & SHIP CANAL
DES PLAINES RIVER
ILL. R.
LEGEND
JULY, 1961
AUGUST, 1961
D. O. AVERAGES
UPPER ILLINOIS RIVER SYSTEM
AS TAKEN BY
DEPT. OF HEALTH, EDUCATION & WELFARE
PUBLIC HEALTH SERVICE

FIGURE 2

Road Dam, additional DO is entrained. Almost complete recovery is reached, to the saturation point, after confluence with the large quantity of good water from the Kankakee River.

The low dissolved oxygen throughout much of the length of the waterway system indicates the poor condition, especially in the summer season. Insufficient DO is available to support desirable fish and aquatic life in the stream.

The GLIRBP study has shown that the Waterway System through Chicago and downstream to the Kankakee River is in an extremely polluted condition and can be considered as a hazard to human health.

POSSIBLE SOLUTIONS FOR THE FLOOD CONTROL AND WATERWAY POLLUTION PROBLEMS

A number of alternates have been studied for solving the problems of waterway pollution caused by the spillages from combined sewers. Also for solving the problems of flood control of the waterways during severe storm periods. Among those advanced are the following:

SEPARATION OF SEWERS

A complete study of the separation of sewers has been made for the 300 square mile area of Chicago and vicinity, and would require nearly ten thousand miles of sanitary sewers, many lift stations and interceptors. It has been estimated that the cost of this separation would be in the range of 3½ to 4 billion dollars.

Even if separation were to be accomplished at this tremendous cost, and with its concomitant disruption of traffic in almost every street, the inconvience to all of the people, the reworking of house and building plumbing, and the adjustment and relocation of public and private utilities, it is questionable as to whether it would solve the problems associated with delivering all wastes to the treatment plants. Accidental or illegal connections to the wrong sewer and the possible leakage between sanitary and storm sewers, would make policing of the six to eight mile long sewer systems impractical. In addition, it has been shown that storm water itself carries considerable pollution to the waterways.

The separation of sewers would not provide any flood benefit to the waterway system.

STORAGE IN EXISTING SEWERS

Consideration was given to storing the runoff of the smaller storms in the existing sewer systems by the use of inflatable dams. Such storage, if entirely used, would amount to approximately 3,200 acre-feet or 0.2 inches over the 192,000 acres of the combined sewered area. The entrapment of combined flow for storms having a runoff of this magnitude would result in a reduction of spillage of approximately 65 percent. This storage would reduce the frequency of combined sewer spillages from an average of 60 per year to about 15 per year.

However, because of the flat slope of the sewers in the Chicago Metropolitan area, this method of reducing the spillages was not further considered. The velocity generated in the sewers in the post storm period would not be sufficient to scour the sediment deposited during the storage period, and would result in extensive maintenance problems. Also, this method would not contribute anything toward the solution of the flood control problem.

UNDERFLOW-STORAGE PLAN

This 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% reduction of pollutants entering the waterway from combined sewer spillages.

Subsequent paragraphs will provide the details of this plan.

DEEP TUNNEL PLAN (HARZA AND BAUER, CONSULTING ENGINEERS)

This plan is a multi-purpose plan, including hydroelectric power development, with a "pumped-storage" scheme, now widely used throughout the world as adjuncts to hydropower 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 to the Commonwealth Edison Company 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 Plan, with an interconnecting tunnel through Chicago's southside connecting the Mainstream and DesPlaines Tunnel System to the Calumet Tunnel System.

Two locations for storage and power development are presented; one near the Calumet Treatment Works, and one near the West-Southwest Treatment Works.

<u>CHICAGO DRAINAGE PLAN</u> (ILLINOIS DIVISION OF WATERWAYS)

This plan presented in a preliminary report in November, 1968, combines navigation, flood control and pollution control in the areas tributary to the Illinois Waterway upstream from Brandon Dam.

For flood control in the Lockport-Joliet Area, as well as for improved navigation, it is proposed to remove the Brandon Road Dam and Locks, and the existing Lockport Dam, Lock, and Controlling Works; to build new twin locks, dam and controlling works about two miles upstream from the existing Lockport Dam; and to deepen and widen the channel from Brandon Road to the new Lockport Locks, so as to lower water levels in this reach, about 34 feet below present water levels.

Upstream from the new Lockport Dam, the Sanitary and Ship Canal would be widened to 325 feet, with 150 feet of this width to be deepened 10 feet. The Lockport Dam would be designed to maintain dry weather water levels above Lockport, 10 feet lower than at present.

The widening would extend to Willow Springs Road and the 10-foot lowering of the surface water levels would extend along the Calumet-Sag Channel and Little Calumet River to the O'Brien Lock and Dam and along the Sanitary and Ship Canal to Throop Street. A new dam and lock would be built at Throop Street with a 10-foot differential head, and the O'Brien Lock and Control Works rebuilt to accommodate the lowered water surface.

For pollution control, the Division of Waterways proposed the installation of storm water detention and sedimentation tanks at combined sewer outlets. These would be of the flow-through type and would discharge all flows in excess of tank volumes as partially settled combined sewage into the surface waterways. Solids which settled in the tanks, together with the liquid retained at the end of each stormwater runoff period would be drained or pumped into the intercepting sewers of the Metropolitan Sanitary District. Screening and chlorination at the tank locations as well as mobile aeration of the waterways might be added to improve the pollution control.

DEMONSTRATION GRANT BY FWQA FOR THE LAWRENCE AVENUE UNDERFLOW SEWER SYSTEM

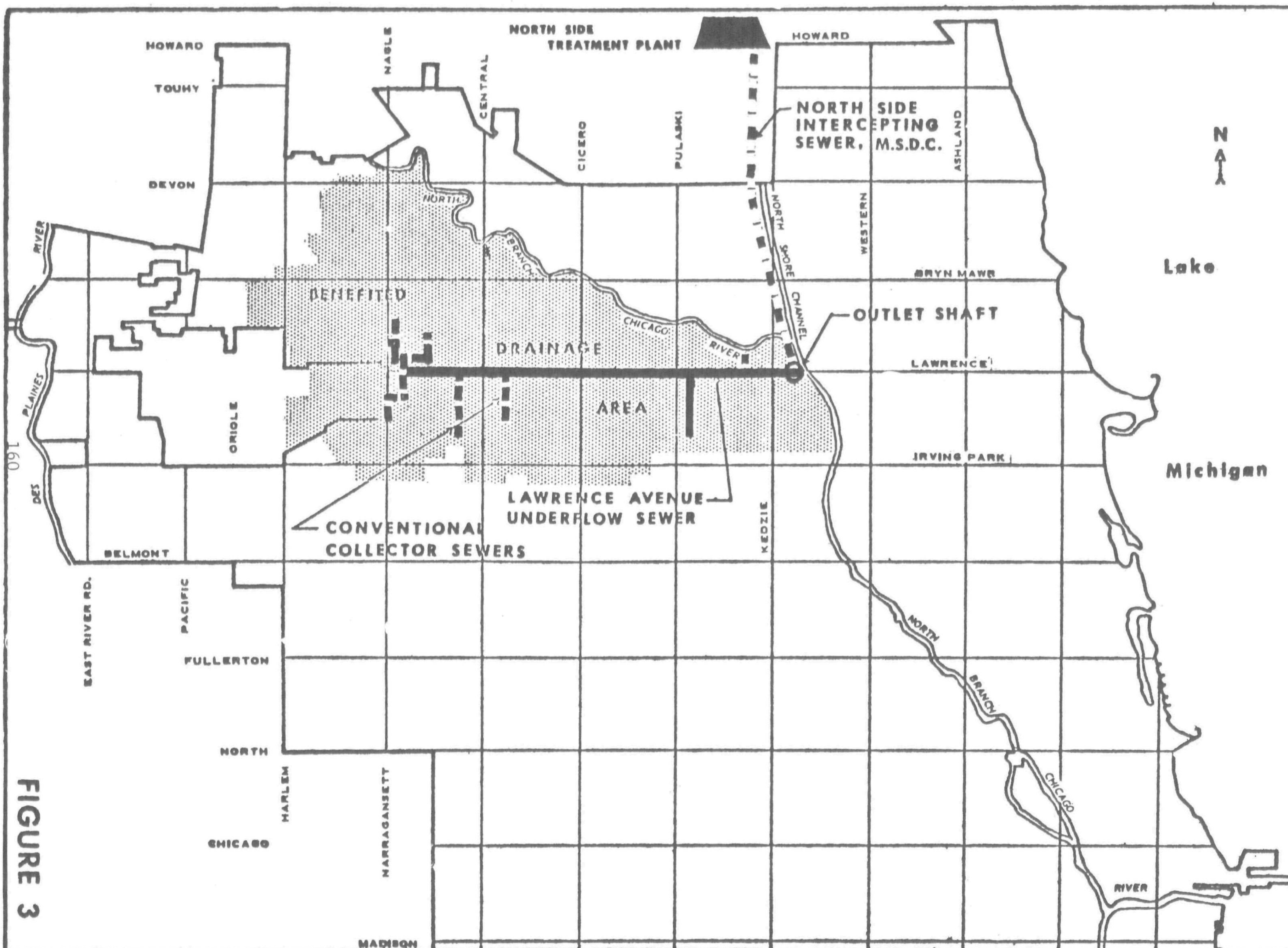
In 1966, the City of Chicago proposed a sewer project which would demonstrate the principles of the Underflow Plan but, of course, on a much smaller scale.

The City of Chicago's Five Year Capital Improvement Program called for the construction of a new Auxiliary Outlet Sewer System to provide relief from basement and underpass flooding of an area bounded by the North Branch of the Chicago River, Irving Park Road, Oriole Avenue and Devon Avenue. (See Figure 3). Preliminary hydraulic studies indicated that a trunk sewer in the vicinity of Wilson Avenue from the North Branch of the Chicago River to Melvina Avenue with branches extending north and south to intercept and unload existing trunk sewers would provide the necessary flood relief for a direct drainage area of 3,620 acres.

The proposed sewer system in that program was designated the Eastwood-Wilson Avenue Sewer System and varied in size from a 2 barrel 13-foot by 13-foot section at the lower end near the river to a 7.5-foot circular section at its upper end.

Consideration was given to lowering the profile of this sewer to increase the storage available during small storm periods and to cause it to flow full before discharging to the river. The storage thus generated would reduce the frequency of spillages from this combined sewer to the river. Lowering the profile would necessitate pumping of sewage to the existing sanitary intercepting sewer, increasing the overall cost. It would also require that more of the construction be performed by earth tunnel method. Recent development of earth mining machines has resulted in lower bid prices in earth tunnel contracts. However, preliminary soil investigations indicate that heavy primary steel lining and occasional rock sections would negate the savings from the use of such machines. Costs would greatly exceed that of the conventional open cut construction method.

Recent improvements of the rock mining machines (Moles) have reduced the cost of tunneling in various kinds of rock materials for large irrigation and hydroelectric projects throughout the world. Preliminary cost estimates revealed that mining in rock may be competitive with open cut methods.



NORTH SIDE TREATMENT PLANT
NORTH SIDE INTERCEPTING SEWER, M.S.D.C.
OUTLET SHAFT
BENEFITED DRAINAGE AREA
LAWRENCE AVENUE UNDERFLOW SEWER
CONVENTIONAL COLLECTOR SEWERS
Lake Michigan
N
HOWARD
TOUHY
DEVON
BRYN MAWR
LAWRENCE
IRVING PARK
BELMONT
FULLERTON
NORTH
CHICAGO
MADISON
NAGLE
CENTRAL
CICERO
PULASKI
ASHLAND
WESTERN
KEDZIE
ORIOLE
EAST RIVER RD.
PACIFIC
HARLEM
NARRAGANSETT
NORTH SHORE CHANNEL
NORTH BRANCH CHICAGO RIVER
DES PLAINES RIVER
CHICAGO RIVER

FIGURE 3

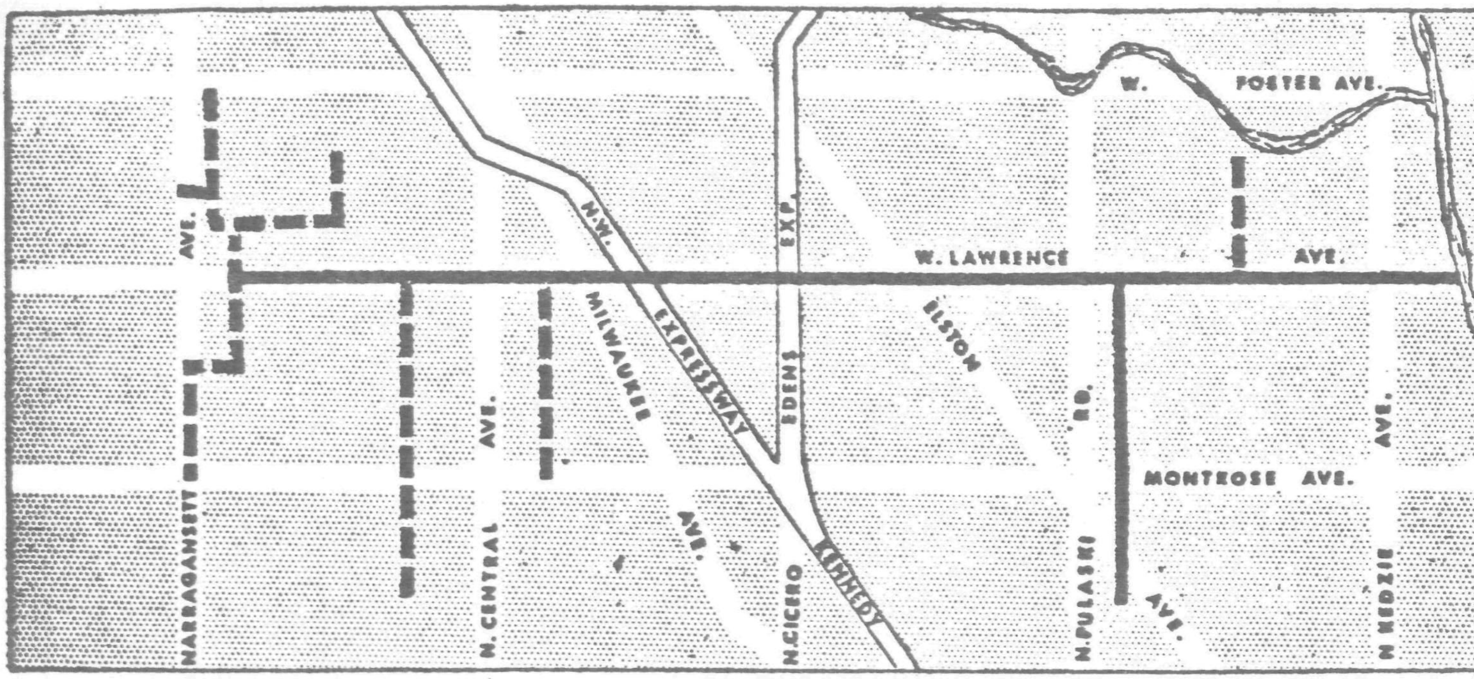
Lowering the profile of the Eastwood-Wilson sewer over one hundred feet into bed rock and constructing it as an "Underflow Sewer" looked promising. Sanitary flow would not normally, in dry weather periods, enter the tunnel and therefore would not be pumped on a continuous basis. Pumping would be required, however, for dewatering of the tunnel to the existing sanitary intercepting sewer in the post rainfall period.

The Department of Public Works retained the Harza Engineering Company to study alternate methods of constructing the proposed Eastwood-Wilson Auxiliary Outlet Sewer System. The studies were to include a comparison of costs of constructing the sewer by open cut and tunnels, the maintenance and operating costs, and their recommendations on the best method to fit the City's needs.

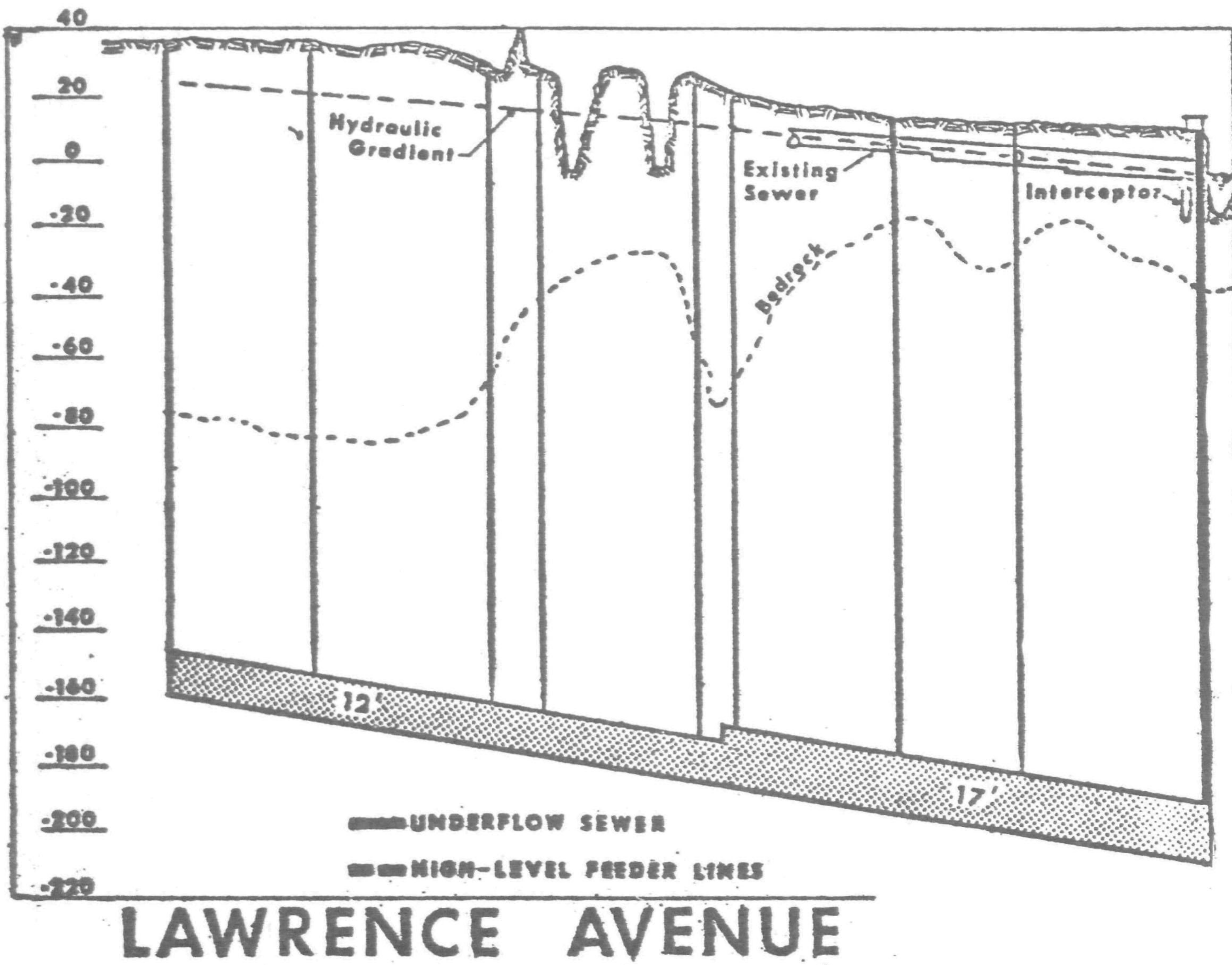
It was decided to construct a lined tunnel sewer in the Niagaran limestone formation approximately 250 feet under the surface of Lawrence Avenue and demonstrate the feasibility of the "Underflow" concept. See Figure 4. The rock tunnel would be excavated by a tunnel boring machine. Lawrence Avenue, an arterial street, was selected as the route of the sewer because of the requirement of the mole to travel in nearly a straight line. Because the tunneling would be so far below the surface, traffic in that arterial street and commercial activities would not be interrupted, as would be the case with the conventional open cut construction.

The tunnel would be 12,800 feet long at 12 feet in diameter and 9,300 feet long at 17 feet in diameter. A branch tunnel in Harding Avenue extending south from Lawrence Avenue to Berteau Avenue, a distance of 4,000 feet would also be 12 feet in diameter. Approximately 18,000 feet of new conventional branch sewers would relieve the overloaded existing sewers and convey the flow to the tunnel inlet shafts. Ten inlet shafts would be constructed to supply the tunnel and one 25-foot diameter outlet shaft for the discharge drainage.

The total storage in the tunnels and shafts will be about 4,000,000 cubic feet or about 0.30 of an inch over the 3,620 acre drainage area. This storage would provide space for the runoff from rainfall accumulation up to about 0.9 inches without overflowing to the river.



UNDERFLOW PLAN



LAWRENCE AVENUE UNDERFLOW TRIBUTARY

FIGURE 4

COMPUTER STUDIES

In order to analyze the Lawrence Avenue Underflow sewer system under actual operating conditions, a mathematical model of the system was simulated in a computer program. Each hour of rainfall during an entire year was analyzed. The amount of rainfall along with the corresponding hourly code was recorded on punched cards. The computer was programmed to determine the net runoff from the impervious and pervious areas for each hour of rainfall. For the impervious area, a small amount of depression storage was subtracted from the first hours of rainfall of each storm to obtain the net runoff supply. On pervious areas, depression storage and varying amounts of infiltration depending on wet or dry antecedent conditions were subtracted from the rainfall to determine the net runoff supply. The total runoff was then calculated by weighing the net runoff supply from the impervious and the pervious areas in accordance with the imperviousness ratio of the tributary drainage area.

A hydrograph with a mass equal to the net runoff supply was then developed. The base of this hydrograph can be varied for the time of concentration of the tributary sewer system. The hydrographs for adjacent hour periods having a net runoff supply were then added together, somewhat similar to the method used in summing the unit hydrographs in river hydrology.

Sanitary flow was added to these runoff hydrographs to obtain the combined flow hydrographs for every rainfall period of the year. For this study 0.01 cubic feet per second per acre was used as the sanitary flow rate. This rate has been verified as a good approximation for the quantity of sanitary flow by the U.S. Public Health Service studies on the Roscoe Street sewer system which serves a similar drainage area.

At each overflow point of the existing sewer system, it was assumed that up to two times the dry weather flow would continue to flow by the overflow weir and along its present route to the treatment plant. The excess flow over and above two times the dry weather flow spills down into the tunnel system.

The sanitary flow at these overflow points is assumed to be uniformly mixed in the total combined flow upstream of the controlling weir. Four factors were set for the pollutional load in the combined flow. These included suspended solids in sanitary sewage, suspended solids in storm water, B.O.D. in sanitary sewage and B.O.D. in storm water. The sewer was sized for only its necessary conveyance capacity to handle the calculated runoff from a five-year frequency on its tributary drainage area.

A graph of the storage volume was plotted against the water surface pool elevation in the tunnel. This data was placed in the computer with linear interpolation between sets of points. When the volume of inflow to the tunnel exceeded the total storage volume, the excess water was discharged to the river. Limitations were placed in the program, on the maximum discharge flowing through the system, since storms exceeding the design capacity of the existing sewer system and the new tunnel system would cause upstream basements to flood. This flooding would limit the maximum discharge through the system. This eventuality was provided for in the computer by flood routing procedures and limiting the maximum discharge to 1,500 cfs.

A set time after the last hour of rainfall of a storm period, the dewatering pumps were turned on. The pumps were set at 48 cubic feet per second which would provide complete dewatering of the tunnel to the interceptor in 24 hours.

The B.O.D. in the tunnel was assumed to be pumped to the interceptor or to overflow to the river at the instantaneous concentration in the system. The suspended solids were divided into two parts, that which would remain in suspension and that which would settle as a function of tunnel velocity. That portion which remained in suspension was pumped during dewatering or overflow to the river at the instantaneous concentration in the system. The volume of suspended solids that settled to the bottom was assumed to be removed by flushing and pumping after the tunnel was dewatered.

Table 1 shows the results summarized for five years of records using the rainfall as it occurred at Midway Airport, U.S. Weather Bureau Gage for 1956 to 1960 inclusive.

Table I

PERFORMANCE COMPARISON

LAWRENCE AVENUE UNDERFLOW SEWER - EXISTING CONVENTIONAL SEWERS IN AREA

A Summary of Computer Calculations Based on Hourly Precipitation Records for 5 Years Period 1956 - 1960

CRITERIA		1956	1957	1958	1959	1960	5 Year Average
HOURS OF PRECIPITATION		446	631	456	631	547	542
TOTAL PRECIPITATION (Inches)		22.23"	44.29"	26.35"	38.68"	27.84"	32"
EXISTING CONVENTIONAL SEWERS IN AREA OVERFLOW TO RIVER	Number	52	79	55	61	47	59
	Duration (Hrs.)	156	336	183	282	218	235
	Suspended Solids (Lbs.)	557,400	1,882,500	840,900	1,406,100	877,700	1,112,900
	B.O.D. (Lbs.)	81,600	236,000	114,700	182,700	124,200	147,800
LAWRENCE AVENUE UNDERFLOW SEWER OVERFLOWS TO RIVER	Number	4	6	6	4	6	5
	Duration (Hrs.)	9	22	17	15	25	18
	Suspended Solids (Lbs.)	73,600	763,300	156,200	542,300	199,400	347,000
	B.O.D. (Lbs.)	11,300	75,900	20,300	52,000	25,100	36,900
REDUCTION (Percent) IN OVERFLOWS DUE TO LAWRENCE AVENUE UNDERFLOW SEWER	Number	92%	92%	89%	93%	87%	91%
	Duration (Hrs.)	94%	93%	91%	95%	89%	92%
	Suspended Solids (Lbs.)	87%	60%	81%	61%	77%	73%
	B.O.D. (Lbs.)	86%	68%	82%	72%	80%	78%
MAXIMUM STORAGE TIME IN TUNNEL - STARTING DEWATERING PUMPS 4 HOURS AFTER RAINFALL (PUMP CAPACITY - 48 CFS)	Hours	87	70	63	68	89	75
	Days	3.6	2.9	2.6	2.8	3.7	3.1

Prepared by:
The City of Chicago, Department of Public Works, Bureau of Engineering

HYDRAULIC MODEL STUDIES

Ten drop shaft structures were required to connect the existing high level sewers and the connecting sewers to the rock Underflow Tunnel. These drop shafts were all slightly over 200 feet in length. Harza Engineering Company and the St. Anthony Falls Hydraulic Laboratory were retained to study and test various configurations with the aid of models, and to determine the best method to destroy the energy of water falling this distance. Another problem to be studied with the computer and a hydraulic model was the surges that may result when the fast filling tunnel suddenly becomes full during large storms.

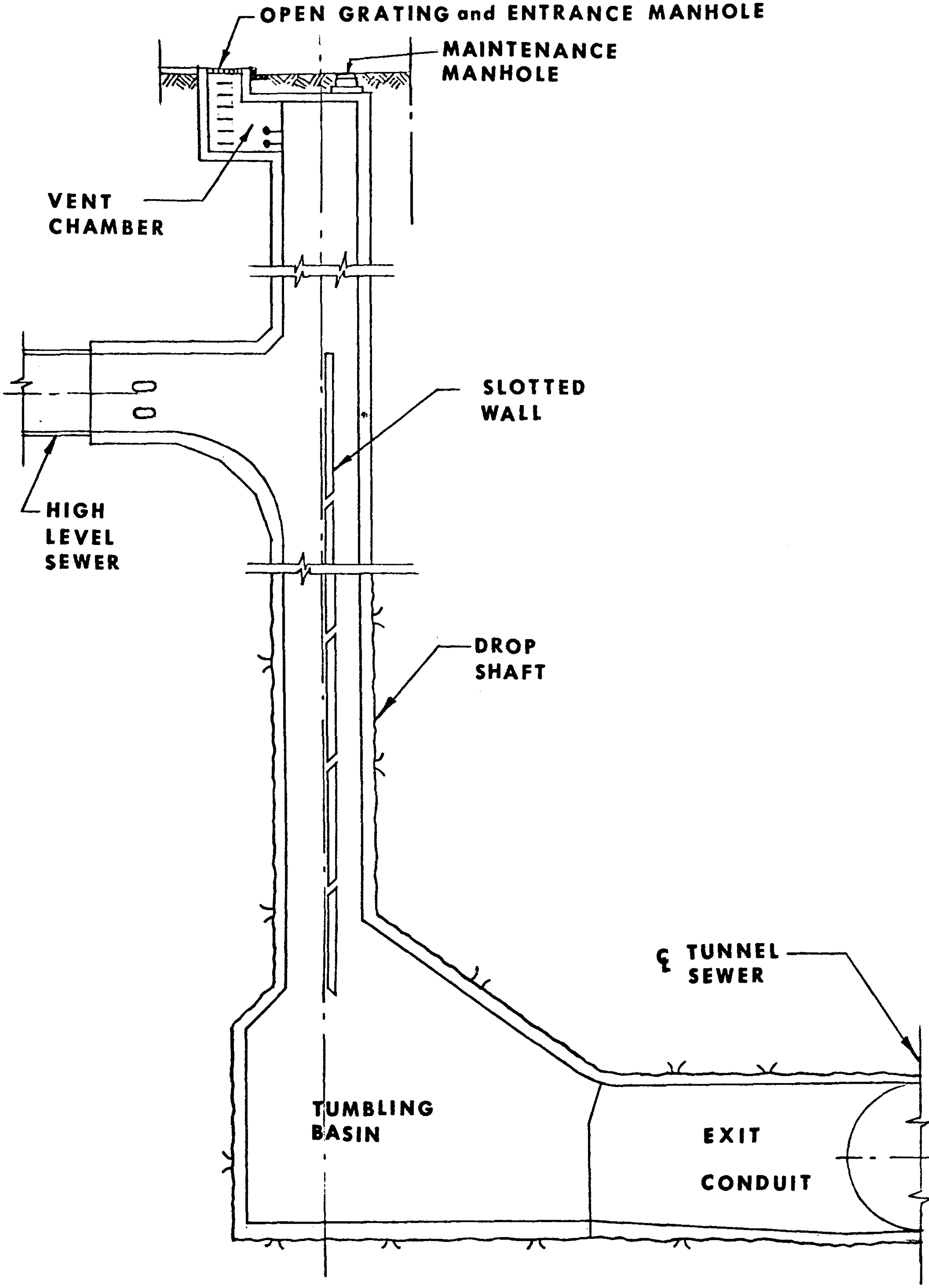
The final drop shaft configuration is shown in Figure 5. This scheme uses a slotted wall with one side for water and the other air, during low tail water. The slots in the wall suck in air to be mixed with the water. This air-water mixture having a much lower density, greatly reduces the impact on the bottom. A large chamber or tumbling basin permits the water and air to separate before the water enters the tunnel.

During heavy storms after the tunnel is full and high tail water exists, the water flows down both sides of the shaft so as to reduce the hydraulic losses through the structure.

CONSTRUCTION

The Lawrence Avenue Underflow Sewer System was proposed to be constructed under several contracts. The first contract was for the outlet shaft near the river, 9,126 feet of 17-foot and 16,638 feet of 12-foot concrete lined rock tunnel. Soil borings and rock cores showed that the dolomitic limestone rock was very dense and at the level of the tunnel, little water problems were expected.

Contract Number 1 was awarded to the low bidder in November 1967 at a price of $10,792,094. This cost was to be financed with $1,500,000 from a Federal Water Quality Administration grant and the remaining from the City of Chicago's Sewer Bond Program.



TYPICAL DROP SHAFT

FIGURE 5

The contractor elected to use the mole designed for the smaller 12-foot tunnel as a pilot tunnel for the 17-foot and later enlarge it to full size by drill and blast methods. A tunnel mining machine, built by the Lawrence Manufacturing Company of Seattle, Washington, was used. The photograph of the machine being placed in the tunnel is shown in Figure 6. The actual cut of the machine is 13'-8" so as to allow for the concrete lining. Figure 7 shows the very smooth walls of the portion mined by the machine. A summary of pertinent data for this contract is shown in Table 2.

An inspection of the mined portion of the tunnel revealed that the bedrock consists essentially of a light gray to gray massive fine grained dolomitic limestone with horizontal clay partings. The horizontal clay partings probably represent bedding planes, and were partially obscured by the machine operation. Occasional areas were washed clean by seepage from fractures and/or joints, and in these areas, the bedding planes had an average thickness of about 1/4 inch to 5 inches. There is no apparent opening or space along bedding planes throughout the excavated portion of the tunnel as is evidenced by the lack of obvious seepage of groundwater through the bedding planes.

No faults occurred in the inspected area. The major fractures and/or joints in the tunnel are basically vertical and run in either a northeast to southwest or northwest to southeast direction. The fracture and/or joint openings are generally 1/8 inch to almost 1 inch thick. A grayish green to green clay generally was found in the fracture and/or joint openings.

As mentioned above, there was no apparent seepage of groundwater from the bedding planes. Approximately 75 percent of the fractures and/or joints had at least some water seepage. Generally, the walls and ceiling of the tunnel were damp. The quantity of seepage water which collected in the tunnel amounted to only about 20 gallons/minute/mile.

During the construction of the first contract of the Lawrence Avenue Underflow System, it was learned that the mining machine had some difficulty maintaining line and grade. This was caused by the pilot shaft at the front of the machine which served to pull the machine along behind it. This problem was later rectified when jacks were added behind the cutting face of the second machine which was used.

FIGURE 6

FIGURE 7

TABLE 2
Pertinent Data For
LAWRENCE AVE. SEWER SYSTEM
CONTRACT NO. 1

Length of Tunnel: In Lawrence Ave. In Lawrence Ave. In Harding Ave.	 *9,126 feet of 15'-6½" x 19'-5" 12,670 feet of 12 foot dia. 3,968 feet of 12 foot dia. 25,764 feet *6,760 feet mined by machine to 13'-8" and enlarged by drill and blast method and 2,366 feet full face drill and blast with finished section of 8" liner to dimensions of 15'-6½" x 19'-5".
Depth below Ground	245 feet max., 220 feet min.
Slope of Sewer	2.5 per 1000
O.S. Diameter specified: (Mined by Machine): In Lawrence Ave. (1st 9,126') In Lawrence Ave. (2nd 12,670') In Harding Ave. (3,968')	 18'-4" 13'-4" 13'-4"
O.S. Diameter Actual: In Lawrence Ave. (1st 9,126') In Lawrence Ave. (2nd 12,670') In Harding Ave.	 16'-10½"x20'-9" D&B or enlarged from machine bore of 13'-8". 13'-9" dia. 13'-9" dia.
I.S. Diameter: In Lawrence Ave. (1st 9,126') In Lawrence Ave. (2nd 12,670') In Harding Ave. (3,968')	 15'-6½"x19'-5" (lined) 12'-0" dia. (if lined) 12'-0" dia. (if lined)
Tail Tunnel	61 feet
Shaft	27 feet dia. and 256 feet deep

TABLE 2 (cont.)
Pertinent Data For
LAWRENCE AVE. SEWER SYSTEM
CONTRACT NO. 1

Contract Costs (Bid):	
1. Shaft	$ 600,000
2. 12 foot dia. Tunnel	4,658.640
17 foot dia. Tunnel	3,732,534
3. 12 foot dia. Lining	998,280
17 foot dia. Lining	730,080
4. Rock Bolts	67,500
5. Wire Mesh	5,000
Total	$10,792,094
Increased Storage (without conc. lining in the 12' dia. section)	31% increase in Volume
In Lawrence Ave. (west of Sta. 91+50)	16,610 cubic yards
In Harding Ave.	5,202 cubic yards
Total	21,812 cubic yards
Award Date	November 1, 1967
Term of Contract	1,095 days
Specified date of completion	November 5, 1970
Normal Shifts	24 Hours Mon. through Sat.
Progress to Date:	
In Lawrence Ave.:	
Machined mined (13'-8")	6,760 feet (1-31-69)
Drill & Blast Enlargement	6,760 feet (9-22-69)
Full Face Drill & Blast	2,383 feet (Sta.91+43) (5-7-70)
In Harding Ave. (13'-9")	2,710 feet (5-7-70)
Progress Max. Week	347 feet (3 shifts, week ending 11-23-68)
Progress Max. Day	92 feet (2 shifts on 4-21-70)
Maximum Penetration ft./hr.	8.6 Maximum
Comp. Strength Rock p.s.i.	11,400 to 29,600

TABLE 2 (cont.)
Pertinent Data For
LAWRENCE AVE. SEWER SYSTEM
CONTRACT NO. 1

Mining Machine:	
Manufactured by	Lawrence Mfg. Co.
Thrust of Machine	1,300,000 lb.(Max.) 850,000 Lb.Op.
Drive of Machine	5-125 hp. Motors
Operation Voltage	480 Volts
Make of Bits	Lawrence Mfg. Co.
Number of Cutters	29 Disc-Type with carbide inserts.
Dia. of Cutterhead:	
Machine No. 1	13'-8" dia. in Lawrence Ave.
Machine No. 2	13'-9" dia. in Harding and Lawrence
Length of Machine:	
Assembly	19'-11"
Drawbar	15'-11"
Power Train	23'-7"
Auxiliary Power Train	25'-4"
Total	84'-9"
Tunnel Power Line	4,160 Volts
Conveyor System Manufacturer	Lawrence Mfg. Co. with a Good-year Belt 24" wide by 84' long.
Muck Cars	6 Cubic Yards
Length of Train	9 Cars
Track Gauge	36"
Locomotives	10 Ton, Plymouth Diesel, 86 hp.
Ventilation	28" Vent line 2-40 hp Vent fans made by the Joy-Axivane Co. 14,000 CFM each. One 15 hp fan at street level to prevent any line back pressures.
Contractor	J. McHugh Construction Co. S. A. Healy Co., and Kenny Construction Co. (a joint venture)
Resident Engineer	John Redmore

The most significant causes of time delays in this contract consisted of: the replacement of a burned-out transformer; replacement of burned out electrical cables and blown electrical switches; replacement of cutting wheels, conveyor rollers and muck buckets; repairs to the pilot shaft; and removing the original machine and installing its modified replacement.

In March, 1970, a second contract was awarded for 8,400 feet of earth tunnel, which will serve as the collecting sewers to intercept critical relief points in the existing City sewer system. Sizes of these tunnels, to be constructed in medium to hard blue clay, range from 5 to 9½ feet in diameter. This contract went for $2,393,645 and will be financed entirely by the City's Sewer Bond Program.

Another small contract for drilling three (3) monitoring wells was let in August, 1969 at $45,805 to test the groundwater quality and pressure at three levels of the aquifer. This contract includes level recorders and sample pumping facilities.

Three more contracts are required to finish the entire Lawrence Avenue Underflow System. These are for the ten drop shaft structures, pumping stations and outfall strutures, and at some future date a contract for additional collecting sewers. After the completion of these contracts, the total cost of the Lawrence Avenue Underflow System will be approximately $21,000,000.

Subsequent to placing the Lawrence Avenue Underflow Sewer System under construction, two more large Underflow Sewer Systems were started in the Chicago Area by the Metropolitan Sanitary District of Greater Chicago. One of these is a 15-foot diameter Underflow Sewer in Crawford Avenue from the Calumet Sag Channel to 105th Street, a length of 18,300 feet. The other is a 12-foot diameter Underflow Tunnel having a length of 17,600 feet, and serving the LaGrange-Brookfield suburban communities.

It is noteworthy that the three Underflow Sewers, all being constructed in the same dense Niagaran limestone strata, are being constructed by mining machines of different manufactures. Tables 2 and 3 show the pertinent data of the three jobs.

TABLE 3
Pertinent Data For Underflow
Sewers Being Constructed by MSDC

	Lawndale Ave. & 47th St. SWIS 13A		127th & Crawford Ave. Calumet 18E-Ext. A	
Length of Tunnel	17,634 feet		18,320 feet	
Depth below ground	235 Max. 201 Min.		223 Max.216 Min.	
Slope of Sewer	2.1 per 1000		1.5 per 1000	
O.S. Diameter Specified	13'-4"		16'-4"	
O.S. Diameter Actual	13'-10"		16'-10"	
I.S. Diameter (if lined)	12'-0"		15'-0"	
Tail Tunnel	250 feet		260 feet	
Shaft	30' and 28'x206' deep		29' and 27'x223' deep	
Contract Costs	Bid	Revised	Bid	Revised
1. Shaft	850,000	850,000	1,000,000	1,000,000
2. Tunnel	4,567,206	4,503,724*	4,763,200	4,763,200
3. Lining	793,530	0	1,190,476	0
4. Bulkhead	included above		1,000	1,000
Total	6,210,736	5,350,724*	6,954,675	5,764,200
	* includes credit for rock material and refund on Elec. Agreement ($360/lin.ft. credit on rock).			
Increased Storage (without Conc. lining)	36% increase in Volume 24,000 Cubic Yards		26% increase in Volume 31,000 Cubic Yards	
Award Date	June 6, 1968		May 17, 1968	
Term of Contract	930 Days		933 Days	
Specified Date of Completion	Jan. 5, 1971		Dec. 16, 1970	
Normal Shifts	24 hrs. Mon. thru Fri. 16 hrs. Sat.		24 hrs. Mon. thru Sat.	
Progress to Date	5,845 ft.(1-7-70)		4,860 ft.(1-6-70)	

TABLE 3 (cont.)

	Lawndale Ave. & 47th St. SWIS 13A	127th & Crawford Ave. Calumet 18E-Ext. A
Progress Max. Week	467 feet	480 feet
Progress Max. Day	113 feet	129 feet
Max. Penetration ft./hr.	5.5 avg. 7.2 max.	4.9 avg. 7.2 max.
Comp. Strength Rock psi.	15,000 to 24,900	23,500 to 39,000
Mining Machine Manufactured by	James S. Robbins & Assoc. Inc.	Jarva, Inc.
Thrust of Machine	890,000 lb. (Max.)	2,200,000 lb.(Max.)
Drive of Machine	6-100 hp motors	8-125 hp motors
Operation Voltage	460 Volts	480 Volts
Make of Bits	James S. Robbins & Assoc. Inc.	Reed Drilling Tools
Number of Cutters	27 Disc-Type plus Tri-Cone	54 Reed Type OKC Tungsten Carbide insert
Length of Machine	37 feet	35 feet
Dia. of Cutterhead	13'-10"	16'-10"
Conveyor System Manufactured by	Moran Eng. Co. 96' Bridge Conveyor(20" widebelt) to 132'(18" wide belt)car loader	Card Corp. 260' conveyor Supporting a 30" belt
Muck Cars	4.4 Cubic Yards	10 Cubic Yards
Length of Train	10 Cars	10 Cars
Track Gauge	24"	36"
Locomotives	10 Ton, Plymouth Diesel, 70 hp	15 Ton, Plymouth Diesel, 160 hp
Ventilation	30" Vent line 2-100 hp Vent fans @ 12,000 CFM ea.	36" Vent line Joy-Axivane fans 31,000 CFM max.
Contractor	S.A.Healy Company & Kenny Cons. Co. (a joint venture)	S.and M. Contractors, Inc.
Resident Engineer	Geo. A. Taylor	Thomas P. Vitulli
Tunnel Power Line	7,200 Volts	7,200 Volts

Although the mining machines have had considerable problems in the 11,000 to 29,000 psi rock, they are now making over 500 feet per week. There is no question that these machines can perform in this type of rock. It is the writer's opinion that the maximum size limitation in this rock will be in the 20 to 25 feet diameter range for the next several years. Sizes above 25 feet diameter, contemplated in the area-wide Underflow Plan, will be constructed by the drill and blast method.

The construction of these Underflow Sewer Systems will demonstrate the feasibility of constructing, economically, a detention reservoir to greatly reduce the pollution caused by overflows from combined sewers, far below the surface in public right-of-way, while providing the conveyance capacity to reduce basement and underpass flooding. It will also demonstrate the practicability of constructing a much enlarged Underflow System beneath the waterways to serve the entire 300 square mile combined sewer area in the City of Chicago and the surrounding Metropolitan Area. When the enlarged Underflow System is completed, the Lawrence Avenue Underflow Sewer and the two being constructed by the Metropolitan Sanitary District will become branches to the trunk lines under the waterways. At that time the pumping stations serving these three initial Underflow Sewers will be abandoned.

RECOMMENDED SOLUTION TO THE PROBLEMS OF FLOODING AND POLLUTION

INTRODUCTION

Because of the urgency of meeting the water quality standards, SWB-15, (2) established by the Illinois State Sanitary Water Board and approved by the Federal Government, studies were continued on the various solutions to the flooding and pollution problems.

In February 1968, a Technical Advisory Committee of prominent engineers in the field of sanitation and drainage was established to review the several plans advanced for solving the pollution and flood control problems of the Chicago Metropolitan Area. The Technical Advisory Committee was charged with establishing criteria by which each plan would be evaluated, and finally to make recommendations to the Flood Control Coordinating Committee on the plan or composite plan which would be best suited to economically solve these problems.

Such a plan would be primarily aimed at solving the pollution of the waterway system caused by the overflow from combined sewers during rainfall periods and eliminating flooding of the waterway in time of heavy storms. The plan must provide for meeting the criteria established by the State Sanitary Water Board for each water course not only as to the quality of water but as to time of implementation. The recommended plan must be adequate to handle a recurrence of the greatest storm of record without requiring the discharge of river or canal water to Lake Michigan.

The final plan selected, although meeting the criteria established for pollution abatement and flood control, should be as broad and as multiple-purpose as possible. Such other areas as recreation, esthetics, navigation and power generation should be considered if economically justified.

STORAGE-ENTRAPMENT STUDIES

Since there was considerable difference of opinion as to the effect of storage on entrapment of pollutants, a subcommittee with members selected to represent the three principle local agencies of the Technical Advisory Committee made some detailed computer studies involving the relationship between the volumes of underground storage (acre-feet) and the trap efficiency (percent of total B.O.D. spillage that would be trapped in underground storage).

These computer studies resulted in the relationship shown in Figure 8.

From this study, it appeared that the percent of entrapment varies greatly with the amount of storage until reaching between 15,000 and 20,000 acre-feet when applied to the 300 square mile combined sewer area. Above 20,000 acre-feet, the increase in entrapment is at a far slower rate per increment of volume. If capital cost per acre-foot were constant for all volumes, then to achieve a trap efficiency greater than 98 percent might appear uneconomical and unwarranted. However, many other compensating factors in the cost of providing such storage volume must be carefully evaluated.

PRESENT STATUS OF PLAN FOR POLLUTION AND FLOOD CONTROL

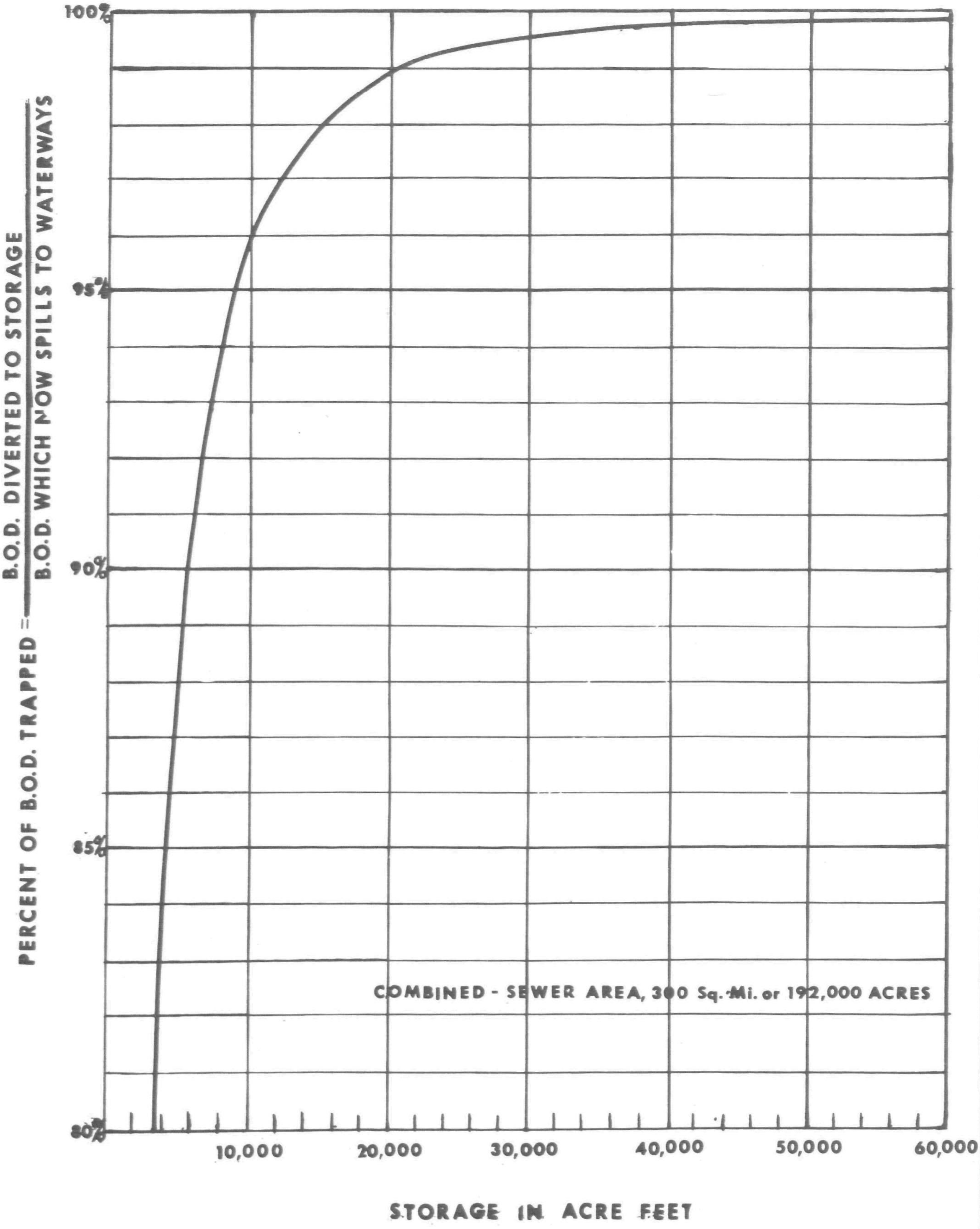
The authors have generally agreed that the First Phase Construction, to be outlined in the subsequent paragraphs of this report, is compatible with the Metropolitan Sanitary District's, the City of Chicago's and the State Division of Waterways' proposed plans. This is with the complete understanding that as detailed design progresses on this First Phase, conveyance tunnel configuration, size, elevation, storage volume, treatment of the overflows and locations may require some modifications to provide the most economical system. The extension of the underflow tunnel to the DesPlaines River at Lockport, lowering of the waterways for navigation and flood control, or generating hydro-electric power to offset a part of the capital cost under the Second Phase work, require further evaluation.

The elected officials of the various agencies, however, have not adopted the First Phase Construction, nor any of the three plans as of this writing. Implementation of the projects will depend on policies established and commitments made by these agencies.

In the following paragraphs, the complete Underflow-Storage Plan is presented only to provide the reader with the relationship of the First Phase to the overall plan.

COMBINED UNDERFLOW-STORAGE PLAN

A modified form of the initial Underflow Plan proposed in the 1966 report has been developed and is referred to



RELATION BETWEEN VOLUME
OF ENTRAPMENT FACILTIES
AND PERCENT OF B.O.D. TRAPPED

FIGURE 8

as the Combined Underflow-Storage Plan.

Since the Illinois Division of Waterways has solidified their recommendations regarding waterway improvements, and have prepared specific recommendations and requested action by the U.S. Corps of Engineers to reconstruct that portion of the Illinois Waterway through Joliet. As already stated, their recommendations include the removal of the Brandon Road Dam and Lock and the existing Lockport Lock and Controlling Works, and the construction of a new dual lock and control gates in the waterway about 2 miles upstream from the existing Lockport Lock. This will extend the water levels of the Dresden Pool upstream to the new Lockport Lock, where the low-water differential water surface will become about 74 feet. The corresponding differential, during maximum flows, has not been accurately determined but will probably be about 70 feet. The proposed relocation of the locks and deepening of the channel is shown on Figure 9.

The completion of the above described work will move the possible point of low-level discharge for an underflow tunnel, or tunnels, about 10 miles upstream from that shown in the 1966 Chicago Underflow Report. The utilization of this low-level point of discharge for peak flows becomes more attractive.

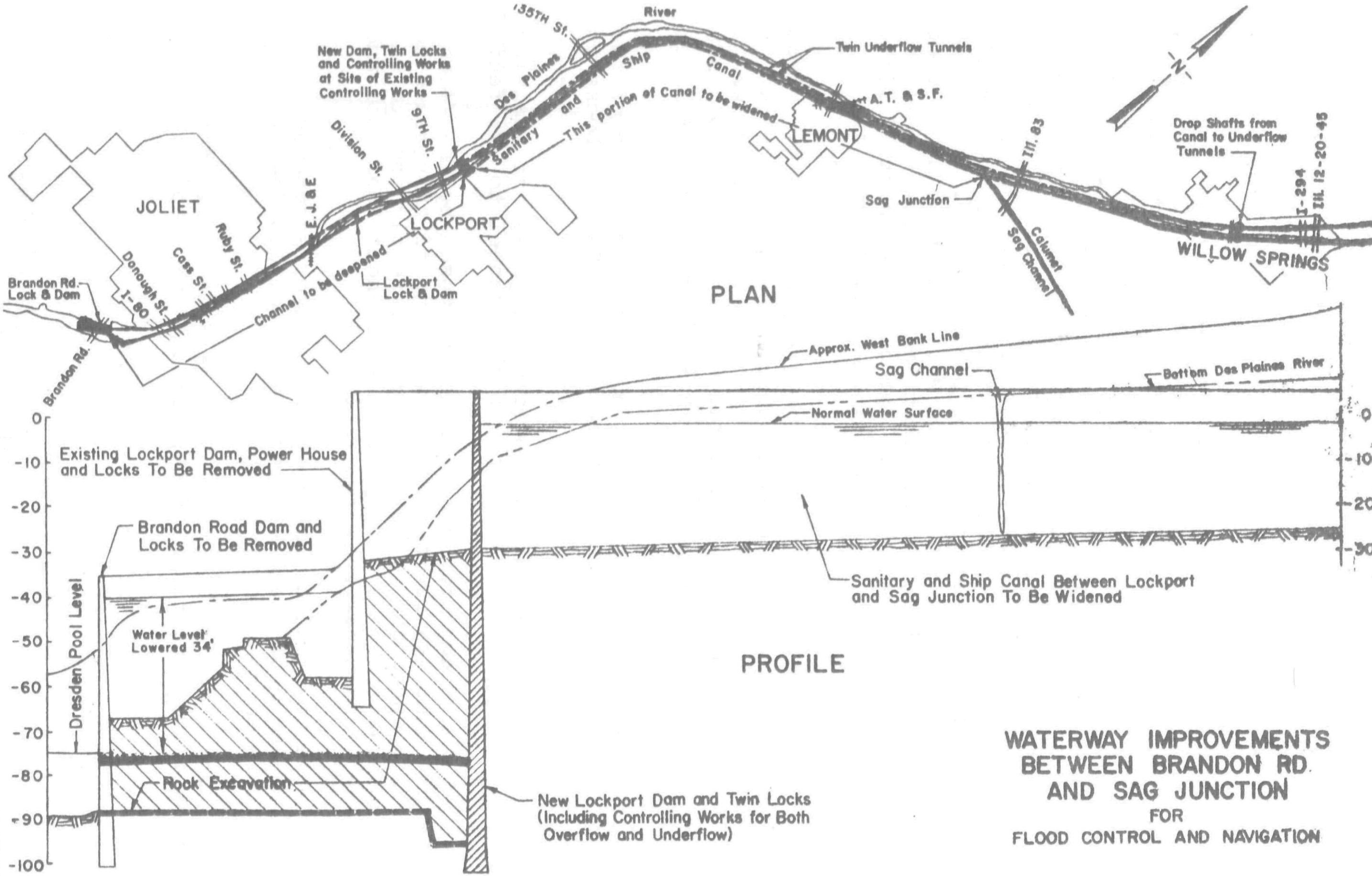
Also shown on Figure 9 is a proposed widening of the Sanitary and Ship Canal from the new Lockport Dam and Locks to Sag Junction.

A project, already approved by Congress and awaiting funding, provides for the widening of the channel to 225 feet.

The Chicago Drainage Plan now proposed by the Illinois Division of Waterways recommends the widening of this reach to 325 feet to accommodate barge tows which currently operate on the Illinois Waterway as far upstream as Brandon Road Pool and to increase flood conveyance capacity.

The Division of Waterways Plan also recommends a 10-foot deepening of the Canal for a width of 150 feet to further increase conveyance capacity.

The Combined Underflow-Storage Plan herein recommended assumes a widening to 325 feet, without any deepening.



WATERWAY IMPROVEMENTS BETWEEN BRANDON RD AND SAG JUNCTION FOR FLOOD CONTROL AND NAVIGATION

FIGURE 9

TABLE 4

STORAGE VOLUME IN MAIN TUNNELS

LOCATION	SIZE	LENGTH (MILES)	VOLUMES IN ACRE FEET			
			PER MILE	PHASE 1	PHASE 2	TOTAL
Mainstream	Twin 26'x50'	11.99	302.0	3,621		
Mainstream	Twin 26'x50'	20.38	302.0		6,155	
Mainstream	Single 26'x50'	13.53	149.0	2,016		
Mainstream	17' ¢	3.00	27.3	82		
Mainstream	12' ¢	12.80	13.7	175		
TOTALS		61.70		5,894	6,155	12,049
Calumet Br.	Single 26'x50'	8.27	149.0	1,232		
Calumet Br.	Single 26'x50'	12.45	149.0		1,855	
Calumet Br.	14' ¢	2.00	18.8	37		
Calumet Br.	10' ¢	2.29	9.2	21		
TOTALS		25.01		1,290	1,855	3,145
DesPlaines Br.	Single 26'x50'	19.50	149.0		2,906	2,906
GRAND TOTALS		106.21		7,184	10,916	18,100

COMBINING STORAGE WITH CONVEYANCE

The City of Chicago presented, in September, 1968, the Composite Drainage Plan, which considered the possibility of providing mined storage areas at four locations along the main tunnel, plus one in the Calumet Area and one along the DesPlaines River Tunnel. This was proposed in order to provide temporary detention storage closer to the various origins of spillage water and thereby reduce conveyance distances, and consequently the cost of conveyance tunnels. This concept of geographical spreading of the underground storage volume is further extended in the Underflow-Storage Plan.

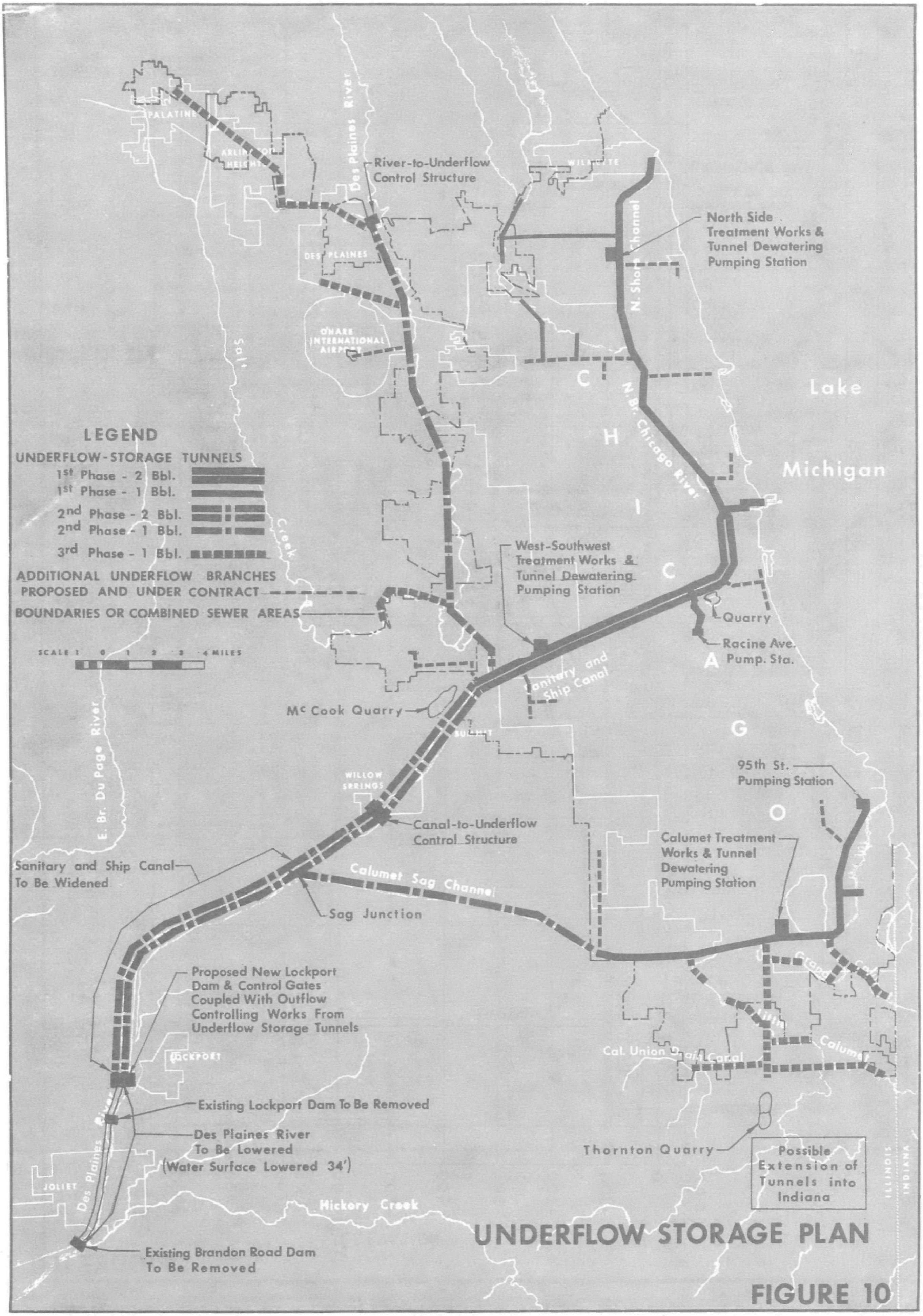
It is now suggested that the main tunnels be re-sized so as to serve both as conveyance tunnels and as continuous storage reservoirs. The revised pattern is shown in Figure 10 attached hereto. The total length of underflow-storage tunnels is about 106 miles.

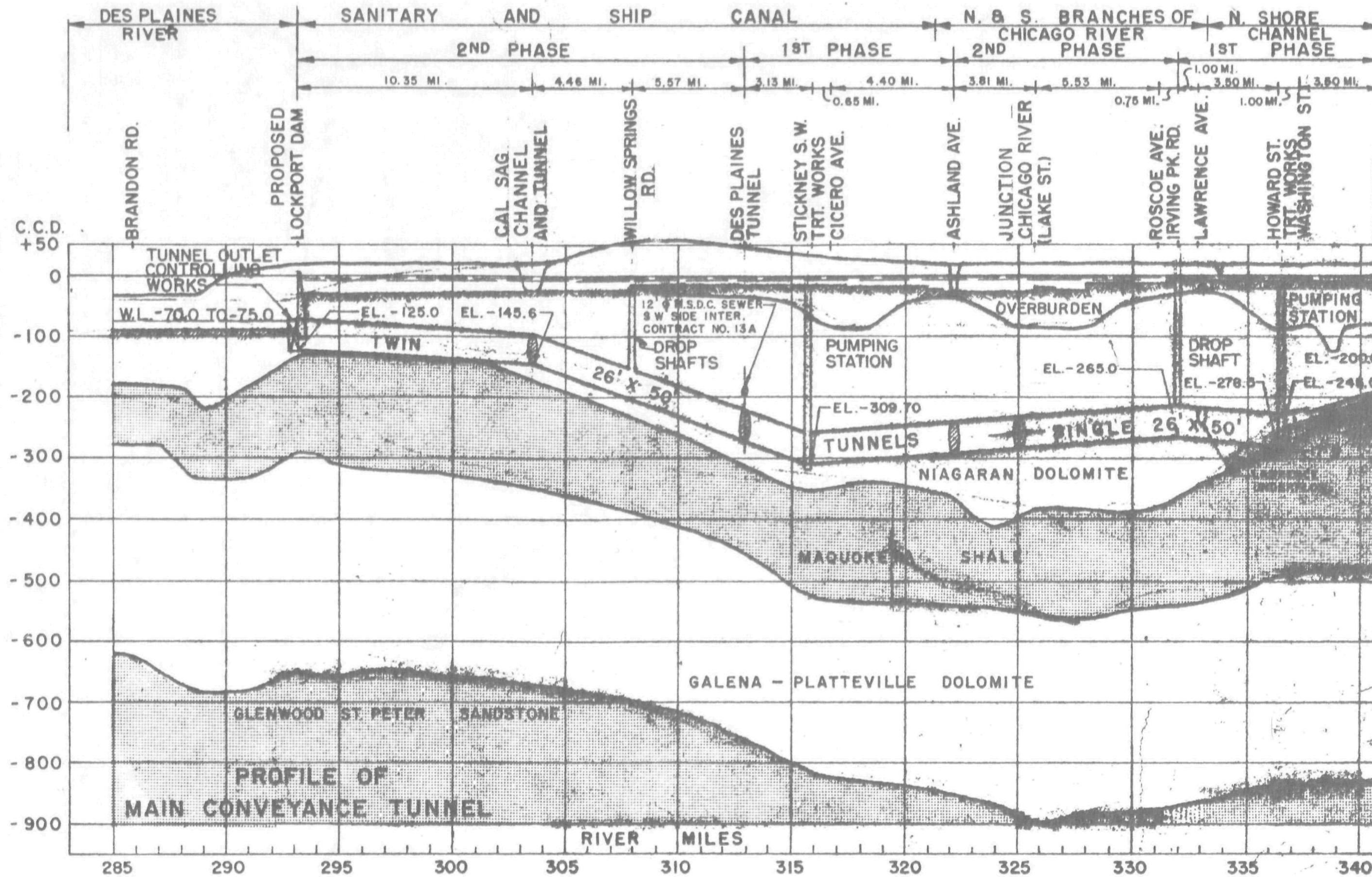
It is proposed that the principal tunnels be 26 feet wide and 50 feet high and have paved inverts plus sidewall lining (in their lower portions only). The principal or main-stream tunnels from Lockport to Lake Street would be Twin Tunnels, as shown in Figure 10.

It is also proposed that the inverts slope to low points opposite each of the three major existing treatment works as shown on the profiles in Figures 11 and 12. Pumping stations are proposed at these points having a combined pumping capacity of about 2,000 cfs, which is about equal to 3/4 of the total ultimate dry weather average flow through the three major treatment works.

A cross-section of the proposed principal tunnels is shown in Figure 13.

The total storage volume of the tunnels shown for construction on Figure 10 is equal to 18,000 acre-feet, or 1.12 inches of runoff from the entire 300 square miles of combined sewer area. The distribution of this storage volume is shown in Table 4. Again, referring to Figure 8, it would accomplish, on a long term basis, an average of more than 98.5 percent entrapment of combined sewer B.O.D. spillages.


PALATINE
ARLINGTON HEIGHTS
Des Plaines River
River-to-Underflow Control Structure
WILMETTE
North Side Treatment Works & Tunnel Dewatering Pumping Station
N. Shore Channel
DES PLAINES
O'HARE INTERNATIONAL AIRPORT
Salt
C
H
I
C
A
G
O
N. Br. Chicago River
Lake
Michigan
LEGEND
UNDERFLOW-STORAGE TUNNELS
1st Phase - 2 Bbl.
1st Phase - 1 Bbl.
2nd Phase - 2 Bbl.
2nd Phase - 1 Bbl.
3rd Phase - 1 Bbl.
ADDITIONAL UNDERFLOW BRANCHES PROPOSED AND UNDER CONTRACT
BOUNDARIES OR COMBINED SEWER AREAS
SCALE 1 0 1 2 3 4 MILES
Creek
West-Southwest Treatment Works & Tunnel Dewatering Pumping Station
Quarry
Racine Ave. Pump. Sta.
Sanitary and Ship Canal
Mc Cook Quarry
E. Br. Du Page River
SUMMIT
WILLOW SPRINGS
95th St. Pumping Station
Canal-to-Underflow Control Structure
Calumet Treatment Works & Tunnel Dewatering Pumping Station
Sanitary and Ship Canal To Be Widened
Calumet Sag Channel
Sag Junction
Proposed New Lockport Dam & Control Gates Coupled With Outflow Controlling Works From Underflow Storage Tunnels
LOCKPORT
Cal. Union Drain Canal
Little Calumet
Existing Lockport Dam To Be Removed
Des Plaines River To Be Lowered (Water Surface Lowered 34')
Thornton Quarry
Possible Extension of Tunnels into Indiana
ILLINOIS
INDIANA
JOLIET
Des Plaines River
Hickory Creek
UNDERFLOW STORAGE PLAN
Existing Brandon Road Dam To Be Removed
FIGURE 10



DES PLAINES RIVER
SANITARY AND SHIP CANAL
N. & S. BRANCHES OF CHICAGO RIVER
N. SHORE CHANNEL
2ND PHASE
1ST PHASE
2ND PHASE
1ST PHASE
10.35 MI.
4.46 MI.
5.57 MI.
3.13 MI.
0.65 MI.
4.40 MI.
3.81 MI.
5.53 MI.
0.75 MI.
1.00 MI.
3.50 MI.
1.00 MI.
3.50 MI.
BRANDON RD.
PROPOSED LOCKPORT DAM
CAL SAG CHANNEL AND TUNNEL
WILLOW SPRINGS RD.
DES PLAINES TUNNEL
STICKNEY S.W. TRT. WORKS
CICERO AVE.
ASHLAND AVE.
JUNCTION CHICAGO RIVER (LAKE ST.)
ROSCOE AVE.
IRVING PK. RD.
LAWRENCE AVE.
HOWARD ST.
TRT. WORKS
WASHINGTON ST.
WILMETTE
C.C.D.
ELEVATION
+50
0
-100
-200
-300
-400
-500
-600
-700
-800
-900
TUNNEL OUTLET CONTROLLING WORKS
W.L. -70.0 TO -75.0
EL. -125.0
EL. -145.6
TWIN
26' X 50'
12' Ø M.S.D.C. SEWER S.W. SIDE INTER. CONTRACT NO. 13A
DROP SHAFTS
PUMPING STATION
EL. -309.70
TUNNELS
OVERBURDEN
EL. -265.0
SINGLE 26' X 50'
DROP SHAFT
EL. -278.5
PUMPING STATION
EL. -200.0
EL. -246.0
NIAGARAN DOLOMITE
MAQUOKETA SHALE
GALENA - PLATTEVILLE DOLOMITE
GLENWOOD ST. PETER SANDSTONE
PROFILE OF MAIN CONVEYANCE TUNNEL
RIVER MILES
285
290
295
300
305
310
315
320
325
330
335
340

FIGURE 11

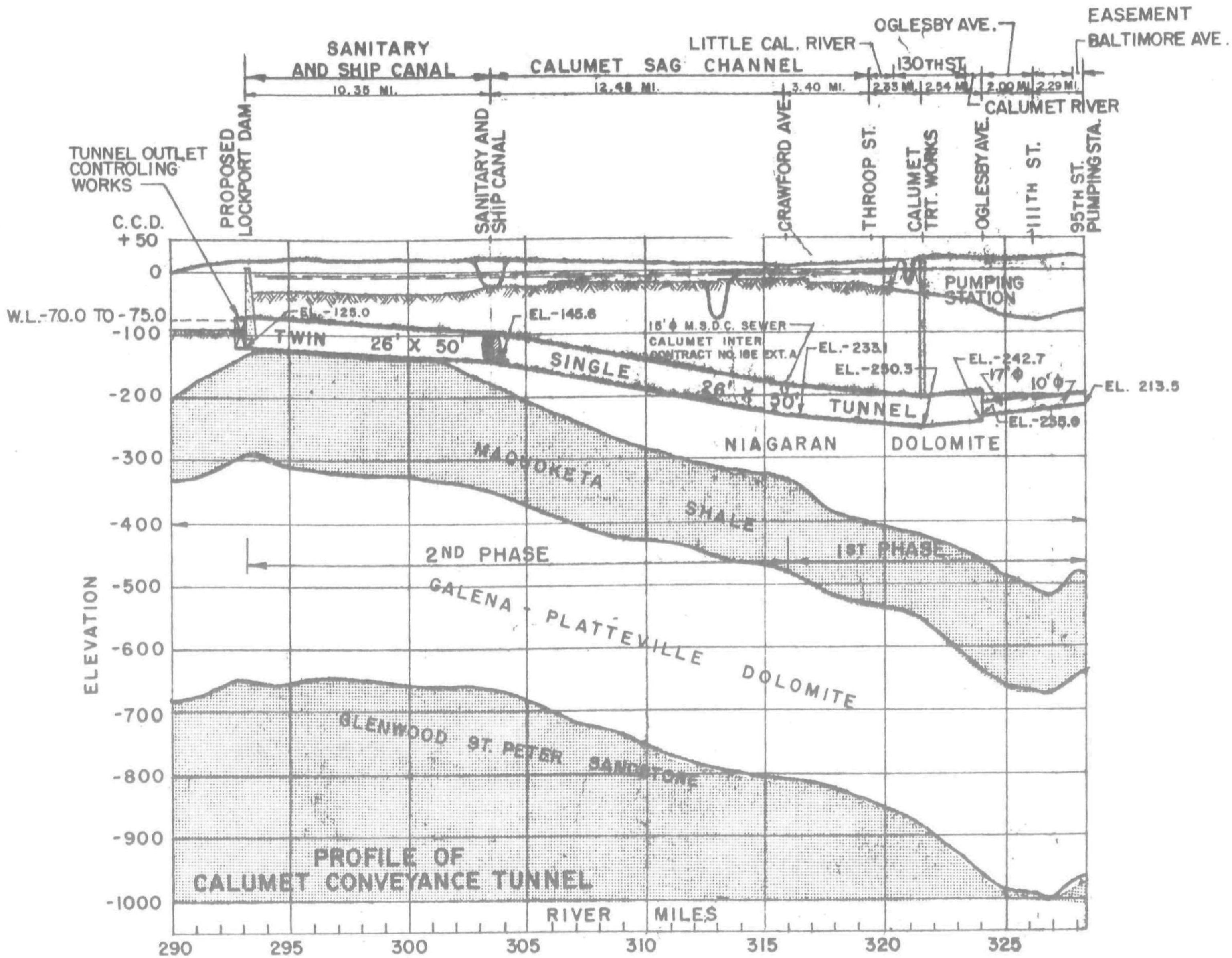
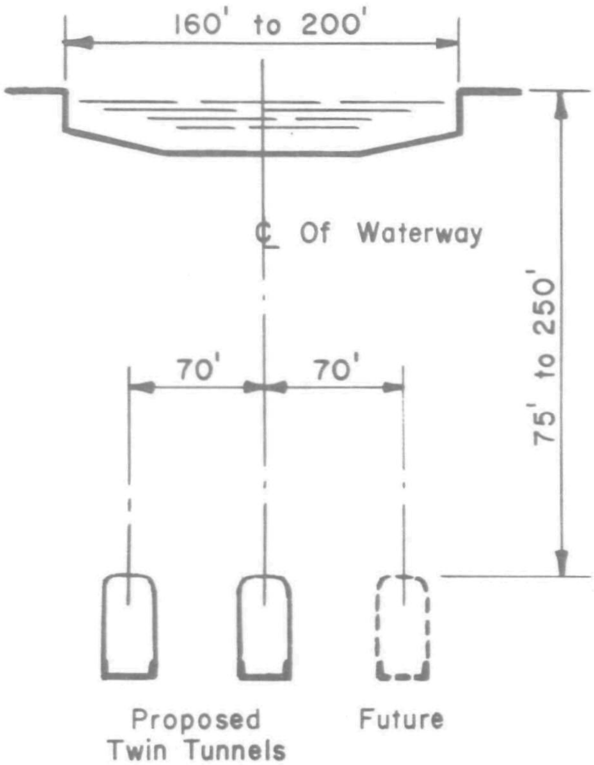
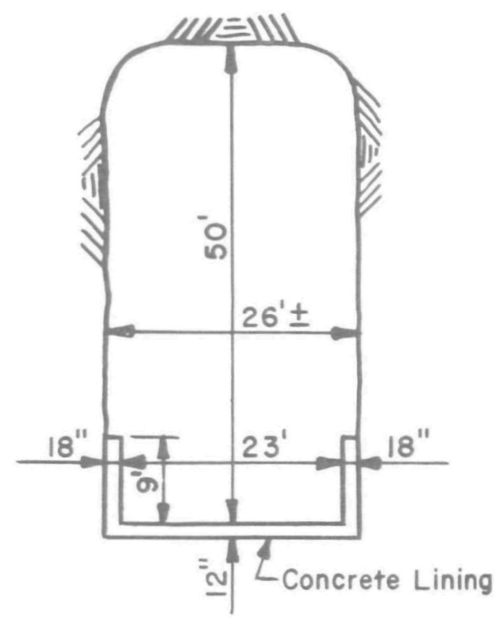

SANITARY AND SHIP CANAL
10.35 MI.
CALUMET SAG CHANNEL
12.48 MI.
3.40 MI.
LITTLE CAL. RIVER
OGLESBY AVE.
130TH ST.
2.33 MI.
2.54 MI.
2.00 MI.
2.29 MI.
EASEMENT
BALTIMORE AVE.
CALUMET RIVER
TUNNEL OUTLET CONTROLING WORKS
PROPOSED LOCKPORT DAM
SANITARY AND SHIP CANAL
CRAWFORD AVE.
THROOP ST.
CALUMET TRT. WORKS
OGLESBY AVE.
111TH ST.
95TH ST. PUMPING STA.
C.C.D.
+50
0
W.L.-70.0 TO -75.0
-100
-200
-300
-400
-500
-600
-700
-800
-900
-1000
ELEVATION
EL.-125.0
EL.-145.6
TWIN 26' X 50'
SINGLE 26' X 50' TUNNEL
15' Φ M.S.D.C. SEWER
CALUMET INTER CONTRACT NO. 18E EXT. A
EL.-233.1
EL.-230.3
EL.-242.7
17' Φ
10' Φ
EL. 213.5
EL.-235.0
PUMPING STATION
NIAGARAN DOLOMITE
MAQUOKETA SHALE
2ND PHASE
1ST PHASE
GALENA - PLATTEVILLE DOLOMITE
GLENWOOD ST. PETER SANDSTONE
PROFILE OF CALUMET CONVEYANCE TUNNEL
RIVER MILES
290
295
300
305
310
315
320
325

FIGURE 12



TYPICAL ARRANGEMENT
OF WATERWAY & TUNNELS



TUNNEL SECTION

ARRANGEMENT AND
SECTION OF MAIN TUNNELS

FIGURE 13

The pumping rate of 2,000 cfs. will completely dewater the full volume of these tunnels in about four and one-half days. It is intended that dewatering will commence at the end of each storm runoff period at the treatment works.

The distribution of the pumping capacity for dewatering stations would bear the following comparative relationship to the presently projected ultimate rates of flow.

Treatment Works	Ultimate Dry Weather Average (C.F.S.)	Ultimate Max.Rate of Flow (C.F.S.)	Tunnel Dewatering Capacity (C.F.S.)
North Side	500	1,000	350
West-Southwest	1,500	3,000	1,100
Calumet	750	1,500	550
Totals	2,750	5,500	2,000

The rate of delivery of entrapped combined sewer spillage to the treatment works would therefore be equal to 73 percent of the ultimate dry weather average flow or 96 percent of the estimated dry-weather flows in the year 1978. The present design basis for treatment works enlargement is based upon maximum flows, 100 percent greater than the ultimate dry weather flows.

Dewatering the entire storage volume of the tunnel system at the above rate would require 109 hours, or 4 days, 13 hours. This would represent the maximum detention period in the tunnel system and could be expected to occur at a frequency of about once in two years.

Assuming mass runoff rainfall ratios varying between 1/10 at 0.1 inch of mass rainfall to 4/10 at 2.5 inches of mass rainfall, and based on average frequencies various mass rainfall quantities, the probable storage detention periods and frequency of occurrence would be about as follows:

Frequency	Runoff	Vol.Stored	Detention
1 time per year	1.0 in.	16,000 Ac.Ft.	4 days
8 " " "	0.25 "	4,000 " "	1 day
24 " " "	0.08 "	1,300 " "	8 hours
40 " " "	0.04 "	650 " "	4 "

To minimize escape of odors from the pumpage, especially during the 4-day pumping periods, expected to occur about once per year, the pump discharge should be connected to the existing intercepting sewer in such a way as to obtain blending with the fresher sewage without undue turbulence.

If desired, at the end of each pumping period, flushing water can be admitted to the upper end of the sloping tunnel inverts in quantities not exceeding the pump capacity of their respective pumping stations, so that the tunnels will be both empty and free of sludge deposits after each storm runoff period. The depth of flow in the 26'x50' tunnels, during flushing, would be between 4 and 6 feet.

The remaining 1½ percent or less of combined sewer spillage, which is not accommodated by storage, represents the excess runoff from storms of small frequency -- less than once in two years. At such times, the tunnels will serve as flow-through conveyance tunnels operating as submerged conduits or flowing traps and delivering the excess volume to the extended Dresden Pool, and to the highly aerated flow at the base of the new Lockport Lock and Dam. During these heavier storm periods, flow over the Lockport Dam and from the DesPlaines River below Hofmann Dam will provide adequate dilution water to keep the dissolved oxygen through the extended Dresden Pool above the requirements of SWB-8 (3).

UTILIZATION OF SURFACE WATERWAYS

At all times, except when combined sewer spillages in any one storm period, exceed the volume of entrapment in the storage-conveyance tunnels, the surface waterways would be conveying only the treatment works effluents, surface runoff from areas not connected to the Underflow-Storage System, wastewater from industries, and dilution waters from Lake Michigan. With the proposed upgrading of treatment works and industrial plant effluents, and the allowable but greatly reduced dilution quantities, the stream quality in dry weather will readily meet prescribed standards (SWB-15).

When the spillage volume exceeds the underground storage, the hydraulic gradient in the tunnels will rise rapidly toward the water levels in the surface waterways. This would generally occur first in the upstream ends of the tunnels. The underflow tunnels which, for lesser storms, serve as underground storage, would become a major auxiliary conveyance facility delivering the excess flood flows to Lockport. The combined conveyance to Lockport via the Sanitary and Ship Canal (with widening below Sag Junction, but without any deepening) plus the conveyance via the underflow tunnel would be sufficient to permanently prevent overbank flooding or release of polluted water to Lake Michigan which, since 1954, has occurred with increasing frequency. Under the plan proposed herein, the combined discharge capacity of the surface canal plus the twin underflow tunnels is about 43,000 cubic feet per second or 3,580 acre-feet per hour. Compare this with the existing Sanitary and Ship Canal, completed in 1900, which was designed for 10,000 cubic feet per second, but under flood conditions has discharged up to 24,000 cubic feet per second.

RESIDUAL DIRECT SPILLAGE TO WATERWAYS

The initial portions of underflow to Lockport will deliver substantial quantities to Lockport by below-surface conveyance prior to the discharge of any pollution from the combined sewer outlets to the surface waterways upstream from Lockport. In some storm periods, this additional feature of the underflow-storage plan will entirely prevent spillage of pollution to surface waterways. Even in the infrequent more extreme storm periods, it will greatly reduce surface waterway pollution in the areas, upstream from Lockport.

In order to evaluate the effect of this pre-pollution flow through the tunnels, a separate computer analysis of all combined sewer spillages for a 16-year period was made. In this analysis for the mainstream tunnel alone, with 12,000 acre-feet of storage and a dewatering pumpage rate of 1,000 cfs. following each storm period, (the pumpage proposed herein for the mainstream tunnel would be 1,450 cfs., however, this capacity would also serve the DesPlaines River Underflow Tunnel) there were only 6 storm periods in 16 years, 1949-64, which would overtax the storage volume of 12,000 acre-feet.

The hydrographs of estimated flow at combined sewer outlets along the mainstream in excess of the storage volume for a future recurrence of these 6 storms are shown in Figure 14.

Backwater computations for the underflow tunnels shown in Figure 11, indicate that, before the hydraulic gradient would reach river levels at Wilmette, quantities of spillage rates could accumulate to a rate of flow of 12,000 cfs. passing through the tunnels at McCook (just west of West-Southwest Treatment Works).

Using 12,000 cfs. as a dividing line on Figure 14, the underflow and overflow quantities were determined for 6 storm periods mentioned above. The data shown in Figure 14 is tabulated in Table 5. It will be seen in this tabulation that, if the proposed underflow-storage plan had been in operation, the total hours of spillage in excess of storage for the 16-year period would have been approximately 43 hours or an average of 3 hours per year. Also, that the hours of spillage directly to the river at the combined sewer outlets would be only 15 hours, or an average of 1 hour per year.

HYDROLOGIC ANALYSIS OF TWO MAXIMUM STORMS

A further analysis has been made of the impact of the possible recurrence of the two maximum storms of record, which occurred in October, 1954 and in July, 1957. Data obtained by computer runs of the storms are plotted on Figures 15 and 16.

Conditions of maximum probable future development and land use affecting runoff conditions were assumed in the study of the combined adequacy of the proposed underflow tunnels and the improved surface channels downstream from Sag Junction to the proposed new Lockport Controlling Works. It was calculated that after filling 18,000 acre-feet of underground storage, the flow in the underflow tunnels could reach a rate of 14,000 cfs. before rising hydraulic gradients upstream would cause spillage into the surface waterways at the combined sewer outlets, and that the maximum combined outlet capacity of tunnels and the Sanitary and Ship Canal would be 43,000 cfs.

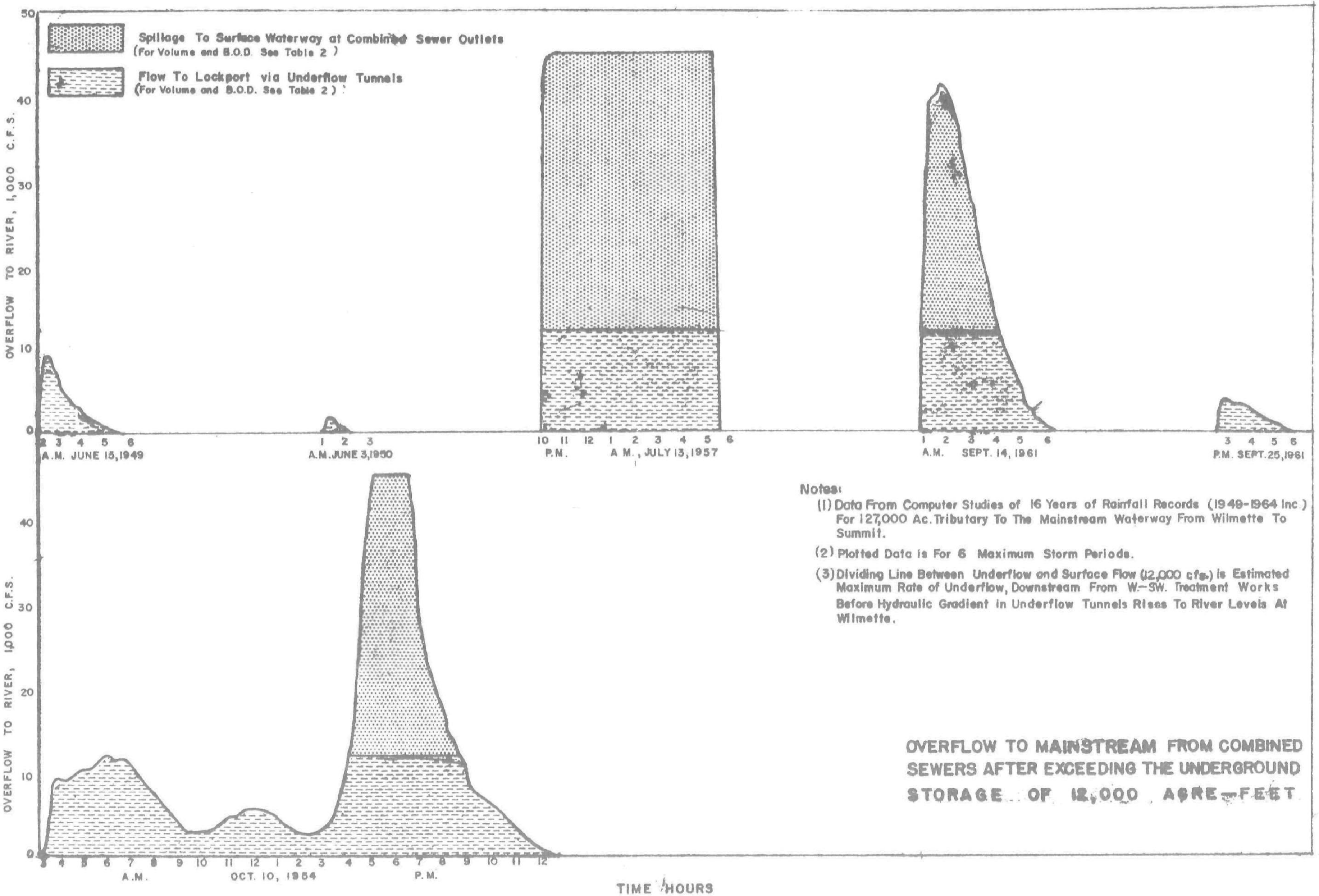

Spillage To Surface Waterway at Combined Sewer Outlets
(For Volume and B.O.D. See Table 2)
Flow To Lockport via Underflow Tunnels
(For Volume and B.O.D. See Table 2)
OVERFLOW TO RIVER, 1,000 C.F.S.
50
40
30
20
10
0
2 3 4 5 6
A.M. JUNE 15, 1949
1 2 3
A.M. JUNE 3, 1950
10 11 12 1 2 3 4 5 6
P.M. A.M., JULY 13, 1957
1 2 3 4 5 6
A.M. SEPT. 14, 1961
3 4 5 6
P.M. SEPT. 25, 1961
3 4 5 6 7 8 9 10 11 12 1 2 3 4 5 6 7 8 9 10 11 12
A.M. OCT. 10, 1954 P.M.
TIME HOURS
Notes:
(1) Data From Computer Studies of 16 Years of Rainfall Records (1949-1964 Inc.) For 127,000 Ac. Tributary To The Mainstream Waterway From Wilmette To Summit.
(2) Plotted Data is For 6 Maximum Storm Periods.
(3) Dividing Line Between Underflow and Surface Flow (12,000 cfs.) is Estimated Maximum Rate of Underflow, Downstream From W.-SW. Treatment Works Before Hydraulic Gradient in Underflow Tunnels Rises To River Levels At Wilmette.
OVERFLOW TO MAINSTREAM FROM COMBINED SEWERS AFTER EXCEEDING THE UNDERGROUND STORAGE OF 12,000 ACRE-FEET

FIGURE 14

TABLE 5

ESTIMATED DURATION, VOLUME AND B.O.D. OF SPILLAGE AT COMBINED SEWER OUTLETS AND UNDERFLOW TO LOCKPORT. ASSUMING RECURRENCE OF YEARS OF RECORD (1949-64 INCLUSIVE) AND 12,000 ACRE-FEET OF MAINSTREAM UNDERGROUND STORAGE

(For Graphical Presentation, See Figure 14)

	Spillage to Waterway Upstream from Lockport			Underflow to Lockport		
Storm	Time Hrs.	Volume Ac.-Ft.	B.O.D. K.Lbs.	Time Hrs.	Volume Ac.-Ft.	B.O.D. K.Lbs.
June 15, 1949	-	-	-	3.50	1,016	227
June 2, 1950	-	-	-	1.25	107	40
Oct. 10, 1954	4.50	7,215	356	22.25	12,383	1,295
July 13, 1957	7.25	19,058	1,255	7.50	7,879	497
Sep. 13, 1961	3.25	4,518	458	5.25	4,342	474
Sep. 25, 1961	-	-	-	3.00	553	125
Totals	15.00	30,791	2,069	42.75	26,280	2,658

The results of these analyses are shown on Figures 15 and 16. Figure 15 indicates that the proposed underflow-storage plan will be able to handle all flow rates expeditiously with some margin of safety. Figure 16 indicates a narrow margin of over-run between the expected runoff hydrograph under the assumed future land use and development and the storage-plus-conveyance capacity of the proposed plan. It indicates that under the ultimate development, there would be some spillage into Lake Michigan.

The storm in July, 1957, far exceeded in total volume of rainfall in such a short time period than any other storm of record and has been variously classified as having a probability of recurrence interval well in excess of 100 years. It also had exceptional uniform rainfall rates and volumes over the entire Chicago Metropolitan Area. We have assumed, therefore, that the cost/damage ratio is too high to warrant expansion of present planning to develop complete protection against the possible repetition of this storm. Since the ultimate land use as assumed may not occur for several decades, expenditures to meet this ultimate possibility can be deferred until after a decade or two of experience with the facilities proposed herein. Future increase in both storage and conveyance can be accomplished by additional lengths of large tunnels, paralleling the initial tunnels. Such an added tunnel, for example, under the mainstream from Willow Springs to Lockport would add 15.8 miles of 26'x50' tunnel, and would provide an additional 2,350 acre-feet of storage and 525 acre-feet per hour of additional outlet capacity at an overall cost of 95 million dollars. To spend this additional amount at this time is not advisable.

POLLUTION MODEL OF THE MAINSTREAM SURFACE WATERWAY

In order to determine, with a reasonable degree of certainity, whether the Underflow-Storage Plan would meet the requirements of the State Sanitary Water Board's Rules and Regulations SWB-15, a Pollution Model of the waterway was programmed for the electronic computer.

The computer model is essentially that which was presented at the December 2, 1968 Technical Advisory Committee meeting. Input data, required for this program, include the initial conditions and all incremental data such as the flow rates, B.O.D., D.O., and temperature of all flows entering

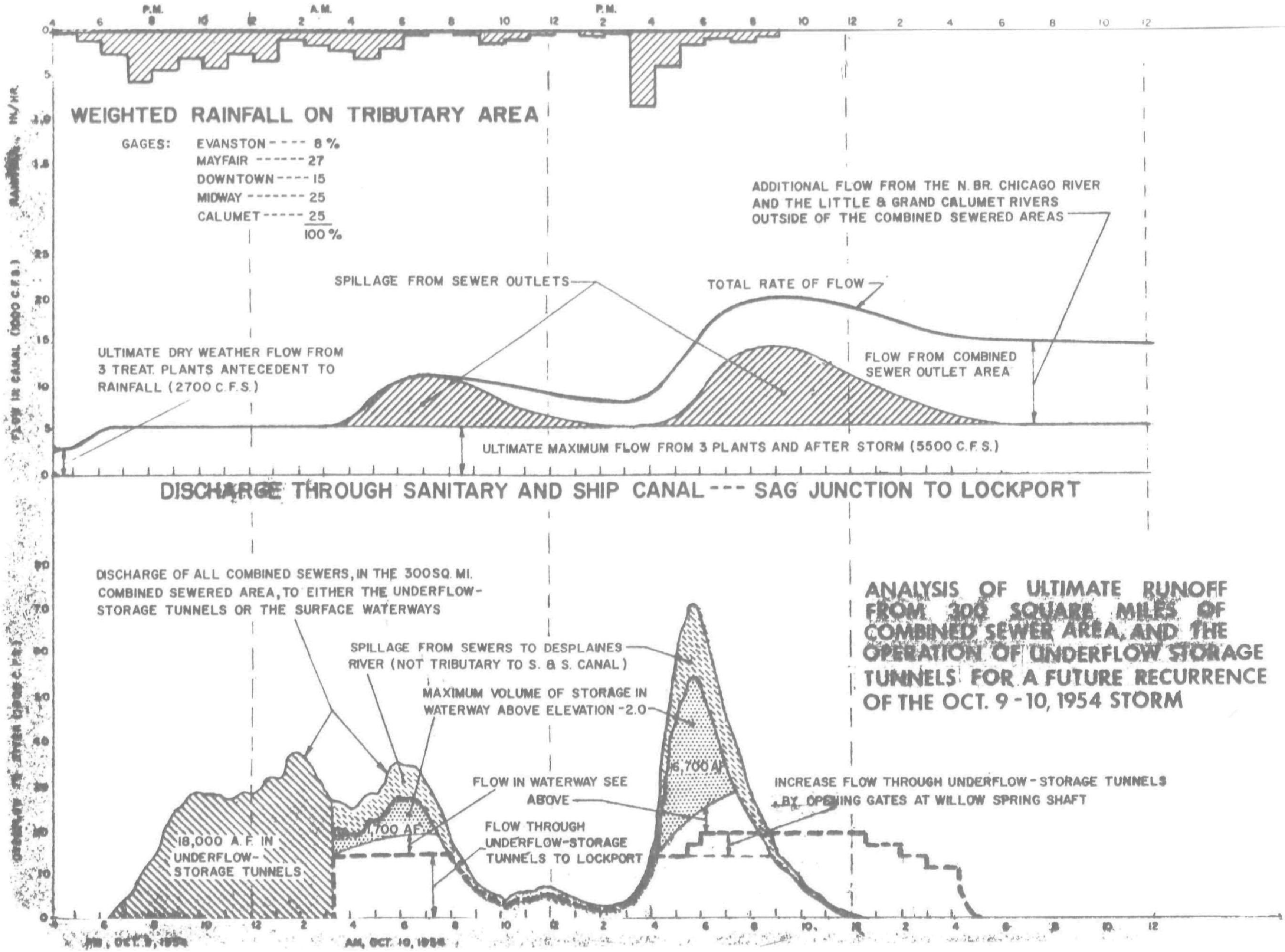
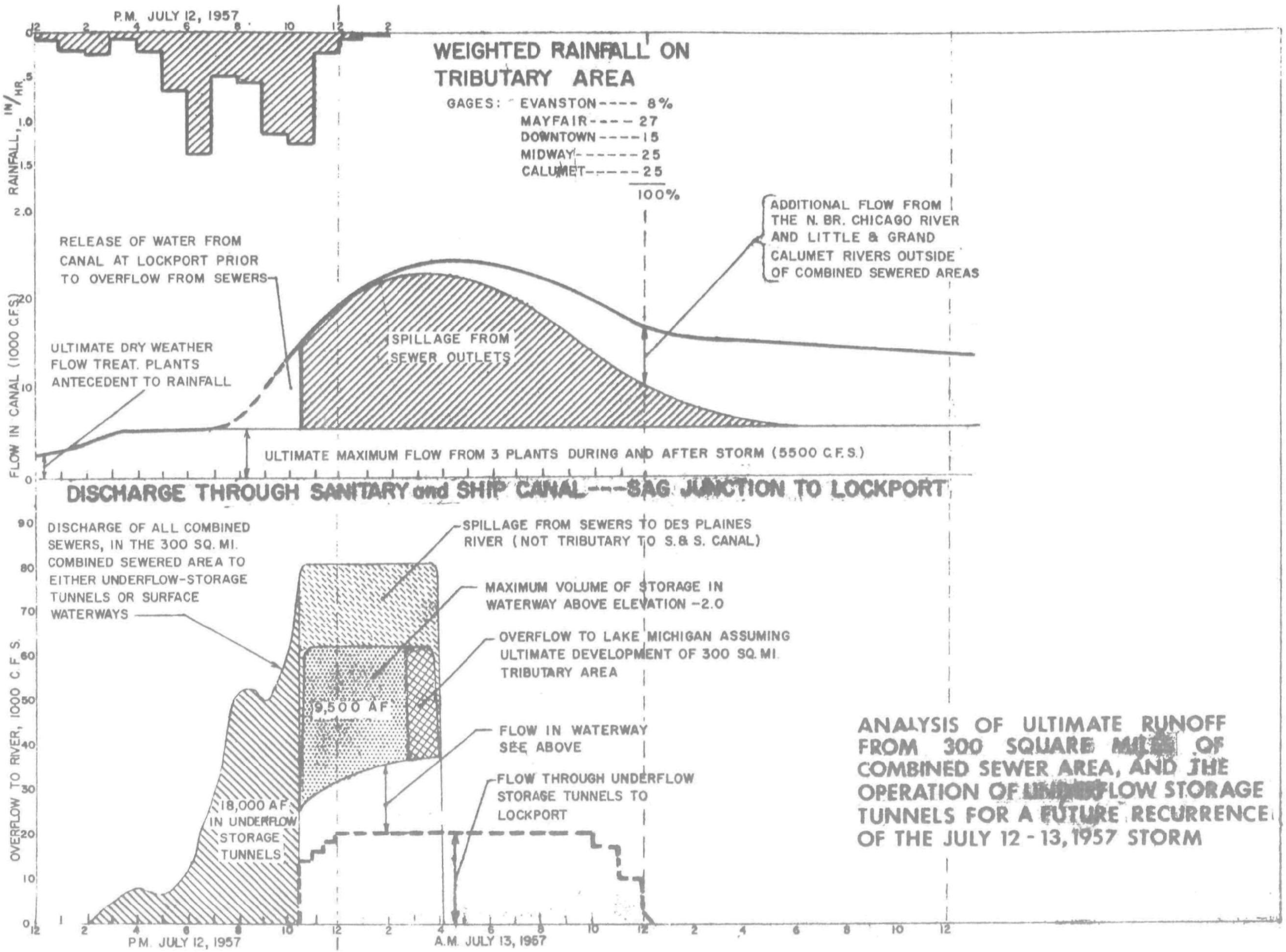

WEIGHTED RAINFALL ON TRIBUTARY AREA
GAGES: EVANSTON ---- 8 %
MAYFAIR ------ 27
DOWNTOWN ---- 15
MIDWAY ------- 25
CALUMET ----- 25
100 %
P.M.
A.M.
P.M.
IN./HR.
ADDITIONAL FLOW FROM THE N. BR. CHICAGO RIVER AND THE LITTLE & GRAND CALUMET RIVERS OUTSIDE OF THE COMBINED SEWERED AREAS
SPILLAGE FROM SEWER OUTLETS
TOTAL RATE OF FLOW
ULTIMATE DRY WEATHER FLOW FROM 3 TREAT. PLANTS ANTECEDENT TO RAINFALL (2700 C.F.S.)
FLOW FROM COMBINED SEWER OUTLET AREA
ULTIMATE MAXIMUM FLOW FROM 3 PLANTS AND AFTER STORM (5500 C.F.S.)
DISCHARGE THROUGH SANITARY AND SHIP CANAL --- SAG JUNCTION TO LOCKPORT
DISCHARGE OF ALL COMBINED SEWERS, IN THE 300 SQ. MI. COMBINED SEWERED AREA, TO EITHER THE UNDERFLOW-STORAGE TUNNELS OR THE SURFACE WATERWAYS
SPILLAGE FROM SEWERS TO DESPLAINES RIVER (NOT TRIBUTARY TO S. & S. CANAL)
MAXIMUM VOLUME OF STORAGE IN WATERWAY ABOVE ELEVATION -2.0
ANALYSIS OF ULTIMATE RUNOFF FROM 300 SQUARE MILES OF COMBINED SEWER AREA, AND THE OPERATION OF UNDERFLOW STORAGE TUNNELS FOR A FUTURE RECURRENCE OF THE OCT. 9 - 10, 1954 STORM
16,700 AF
1,700 AF
FLOW IN WATERWAY SEE ABOVE
INCREASE FLOW THROUGH UNDERFLOW - STORAGE TUNNELS BY OPENING GATES AT WILLOW SPRING SHAFT
18,000 A.F. IN UNDERFLOW-STORAGE TUNNELS
FLOW THROUGH UNDERFLOW-STORAGE TUNNELS TO LOCKPORT
PM, OCT. 9, 1954
AM, OCT. 10, 1954

FIGURE 15



ANALYSIS OF ULTIMATE RUNOFF FROM 300 SQUARE MILES OF COMBINED SEWER AREA, AND THE OPERATION OF UNDERFLOW STORAGE TUNNELS FOR A FUTURE RECURRENCE OF THE JULY 12 - 13, 1957 STORM

FIGURE 16

TABLE 6

ESTIMATED DISSOLVED OXYGEN CONDITIONS IN MAINSTREAM DURING SEVERE STORM PERIODS

Reach Number And Location		October 9-10, 1954			July 12-13, 1957			September 12-13, 1961			Total Hours Below Indicated D.O. in 16 Year Period		Percent Of Time Above Indicated D.O. In 16 Year Period	
		Min. D.O.	Hours Below 5 PPM	Hours Below 4 PPM	Min. D.O.	Hours Below 5 PPM	Hours Below 4 PPM	Min. D.O.	Hours Below 5 PPM	Hours Below 4 PPM	5 PPM	4 PPM	5 PPM	4 PPM
1	Wilmette To Evanston	6.59	0	0	5.40	0	0	6.90	0	0	0	0	100	100
2	Evanston To N. Side Treat. Wks.	5.34	0	0	4.24	15	0	5.61	0	0	15	0	99.99	100
3	N. Side Treat. Wks. To Lawrence Ave.	5.01	0	0	4.47	7	0	5.15	0	0	7	0	99.99	100
4	Lawrence Ave. To Addison St.	4.28	4	0	3.78	7	1	4.26	3	0	14	1	99.99	99.99
5	Addison St. To North Ave.	4.21	12	0	3.72	20	3	4.17	13	0	45	3	99.97	99.99
6	North Ave. To Chicago River	3.96	35	3	3.62	44	13	3.66	37	10	116	26	99.92	99.98
7	Chicago River To Ashland Ave.	3.71	26	10	3.05	34	22	3.11	30	15	90	47	99.94	99.97
8	Ashland Ave. To Cicero Ave.	3.32	38	18	2.41	46	31	2.57	44	25	128	74	99.91	99.95
9	Cicero Ave. To Harlem Ave.	4.40	29	0	3.31	38	22	3.94	33	4	100	26	99.93	99.98
10	Harlem Ave. To Hodgkins	4.28	32	0	3.09	42	25	3.78	38	12	112	37	99.92	99.97
11	Hodgkins To Willow Springs	4.16	36	0	2.92	44	28	3.63	43	18	123	46	99.91	99.97
12	Willow Springs To Sag Junction	3.97	41	7	2.76	50	33	3.40	50	24	141	64	99.90	99.95
13	Sag Junction To Lemont	4.18	41	0	3.32	50	31	3.68	50	19	141	50	99.90	99.96
14	Lemont To Lockport	4.03	48	0	3.26	57	36	3.47	57	25	162	61	99.88	99.96

the mainstream from combined sewers, treatment works, industries, power plants and Lake Michigan dilution water, and from branch waterways. (N.Branch and Cal.-Sag Channel)

Of the 6 storms that would exceed the underground storage of 12,000 acre-feet in the mainstream underflow tunnels, only three would exceed the underflow conveyance to Lockport, See Figure 14.

Combined flow from the sewer outlets would spill to the mainstream waterway in a future recurrence of the three exceptionally heavy storms. The Pollution Model was used to determine the impact that these spillages would have on the dissolved oxygen in the mainstream waterway between Wilmette and Lockport, and a separate analysis was made for the Des Plaines River from Lockport to the Kankakee River.

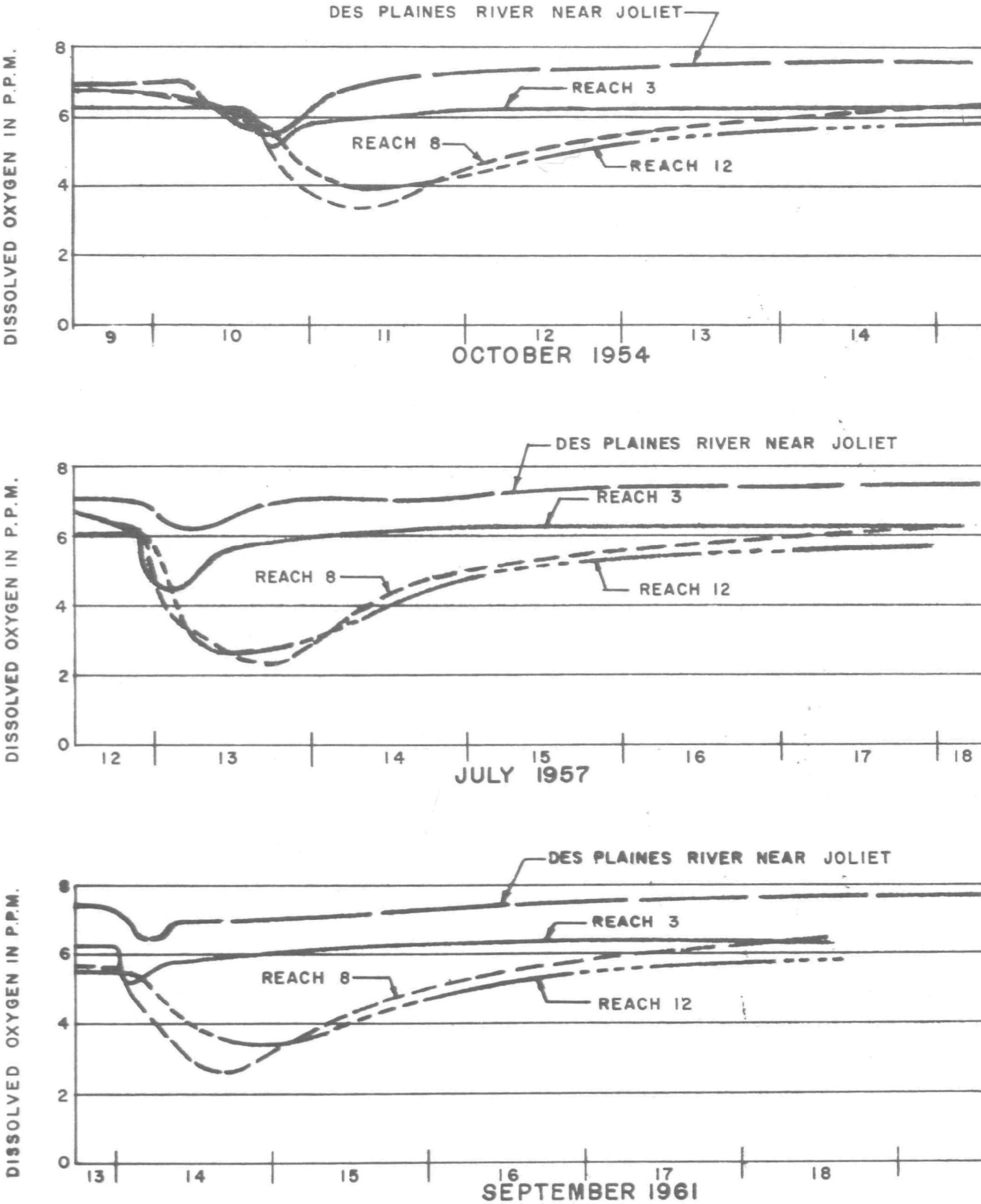
Table 6 gives the minimum dissolved oxygen during each storm for all 14 reaches between Wilmette and Lockport. Also, the time in hours is recorded, during which the D.O. was below 5 ppm and 4 ppm. The most critical reaches appear to be numbers 8 and 12. Figure 17 shows the oxygen sag curves for certain reaches of the waterway, including the DesPlaines River, for a recurrence of the October 9-12, 1954, July 12-13, 1957 and the September 13-14, 1961 storms.

It can be seen in Table 6 that for reach number 14 oxygen was below 5 ppm for 162 hours and below 4 ppm for 61 hours in the entire 16 years analyzed. Thus, the dissolved oxygen in this reach of the waterway would be above 4 ppm more than 99.96 percent of the time. This can be accepted as being in compliance with the standards established by the Sanitary Board's Rules and Regulation, SWB-15.

PROTECTION OF GROUNDWATER AQUIFERS

It is important in any plan which contemplates the storage of polluted stormwater in underground tunnels or reservoirs, that necessary precautions be taken to adequately protect the groundwater resources of the Chicago Region.

There are four aquifers in the Chicago Metropolitan Area which are arranged successively with regard to depth below the ground surface as follows: 1) the sand and gravel aquifer in the glacial drift, 2) the shallow dolomite or Silurian dolomite aquifer, 3) the Cambrian-Ordovician aquifer, and 4) Mt. Simon aquifer.



DISSOLVED OXYGEN SAG CURVES FOR RECURRENCE OF STORMS CAUSING SPILLAGE TO MAINSTREAM UPSTREAM OF LOCKPORT

FIGURE 17

The sand and gravel aquifer is widely scattered through the region and in many cases is hydraulically interconnected with the silurian dolomite aquifer. The upper two aquifers, referred to as the shallow aquifers, are the source of well water for many homes, industry and some villages in the suburban areas which are not supplied Lake Michigan water by the Chicago Water System. The shallow aquifer extends to a maximum depth of about 400 feet and is recharged locally by the downward percolation from streams and rainfall infiltration. As shown in Figures 11 and 12, the tunnels herein proposed are located in the dolomite or lower portion of this aquifer.

A relatively impermeable shale formation, having a thickness of 150 to 300 feet, called the Maquoketa Group, lies below and separates the shallow Silurian aquifer from the Cambrian-Ordovician aquifer. The Cambrian-Ordovician and the Mt. Simon aquifers are partially separated by another lower shale formation called the Eau Claire Group. The Cambrian-Ordovician and the Mt. Simon aquifers are referred to as the Deep aquifers. The Galena-Platteville Dolomite, shown in Figures 11 and 12, forms the upper portion of the Cambrian-Ordovician Aquifer.

The greatest sources of water for deep well pumps which supply many communities and industry, are the Ironton-Galesville and the Glenwood-St. Peter sandstone formations within the Cambrian-Ordovician aquifer. These sandstone strata lie below the Galena-Platteville Dolomite, in the Chicago Region, but are recharged from rainfall infiltration and surface water percolating through the outcroppings of this aquifer in Western Illinois and Southern Wisconsin. Because pumping from this deep aquifer is exceeding the recharge rate, the piezometric level in this lower aquifer is dropping at the rate of 10 to 15 feet per year.

As already stated, the Underflow-Storage tunnels herein proposed are to be located in the Niagaran dolomite formation in the Silurian or shallow aquifer. The tunnels will be 75 to 300 feet below the normal piezometric water table in that aquifer. Since the tunnels will be located generally, in the lower part of the Niagaran rock formation, which is more dense and less permeable than the weathered and fractured, near-surface, rock of the same formation, it is expected that leakage into or out of the tunnels will be small.

Computer calculations of runoff for every hour of rainfall in 16 years of records, 1949 to 1964 inclusive, as previously mentioned show that the 6 largest storms, during the 16-year period, would --- if they recurred after completion of the plan proposed herein --- cause overflow to the waterway at Lockport. During the time of overflow, the hydraulic gradient in the Underflow-Storage tunnelw would be at Elevation -70 or above. The calculated summation of this time of overflow for all 6 storms is less than 43 hours, with about 22 hours of the total occurring during an assumed recurrence of the October 9-10, 1954 storm.

Since there are approximately 140,000 hours in 16 years, the hydraulic gradient or pressure level in the Underflow-Storage tunnels would be below Elevation -70.0 more than 99.97 percent of the time.

Since the upper portion of the Underflow-Storage tunnel section is not proposed to be concrete lined, the percolation of groundwater into the tunnels will slowly lower the piezometric head in the aquifer in the immediate vicinity of the tunnels. This is similar to what occurs at the many quarry sites in the Chicago area. After a period of several months, the water table within the aquifer would stabilize forming a groundwater valley over and along each of the tunnels.

Within this relatively dry valley, all free water will have been removed from the pores, joints and cracks and crevices of the rock creating a considerable volume of available storage. During heavy rainfall periods when the surface runoff exceeds the storage volume of the Underflow-Storage tunnels, the higher pressures in the tunnels will cause water to move outward into the rock at a slow rate filling those voids. Studies were made to determine the distance the water would travel as a function of the duration of surcharge by several methods. The most severe results from the standpoint of groundwater pollution were obtained by a study which assumed that the water would travel outward from the tunnel along radial paths filling the voids as it moves. Values of void ratio of 0.0002 and a permeability coefficient of 1.0 gpd/ft^2 were used for this dense rock at the level of the proposed tunnels.

Figure 18 shows the radial distance the water would travel as a function of the duration and magnitude of the surcharge.

This graph was applied to the storm causing the longest period of overflow. (Storm of October 9-10, 1954). Assuming a static level of the piezometric water level above the tunnels to be at Elevation -200, the water table would rise less than 50 feet above the roof of the tunnels during the approximately 24 hours of overflow to the waterway. See Figure 19. Soon after the conclusion of the overflow period, dewatering of the tunnels would commence, internal pressure in the tunnels would dimish to the roof levels and infilatration would again re-establish the groundwater valley to its pervious shape. It is apparent that no temporary storm surcharge period, which causes outward flow from the tunnel, will refill the normal groundwater valley established by the much more prevalent condition of infiltration into the tunnels.

It can therefore be concluded that no pollution of the aquifers will be caused by the construction and operation of the Underflow-Storage tunnels in Niagaran limestone formations, and that this valuable water resource of groundwater supply will be preserved.

Conditions in the Galena-Platteville Dolomite present a different picture. Here, as previously stated, the groundwater resources are steadily being depleted and the piezometric level has already fallen to more than 300 feet below ground surface and is expected to continue downward at 10 to 15 feet per year.

Unlined tunnels or reservoirs at these lower levels would now or in the near future be relatively higher than the steadily receding groundwater level. In order to prevent pollution of the aquifer, in this case, would require continuous recharging of the rock strata in the vicinity of the tunnels or reservoirs, or limitation of withdrawals to the natural recharge rates.

PROJECT COST

The largest single item of cost of the proposed project is the excavation, hauling and disposal of rock.

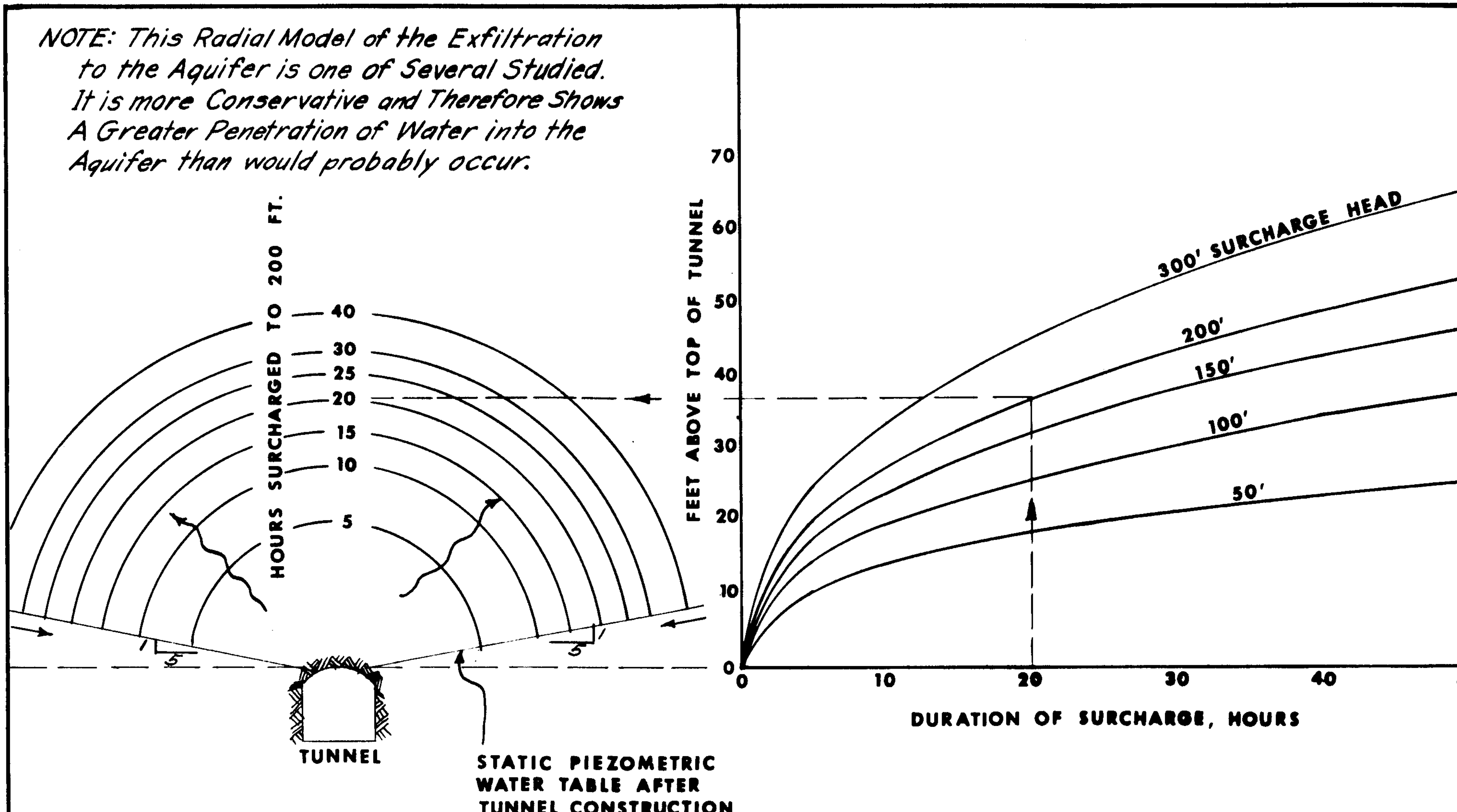


FIGURE 18

STUDY OF EXFILTRATION OF WATER DUE TO INTERNAL SURCHARGE IN TUNNEL SYSTEM DURING STORM PERIODS

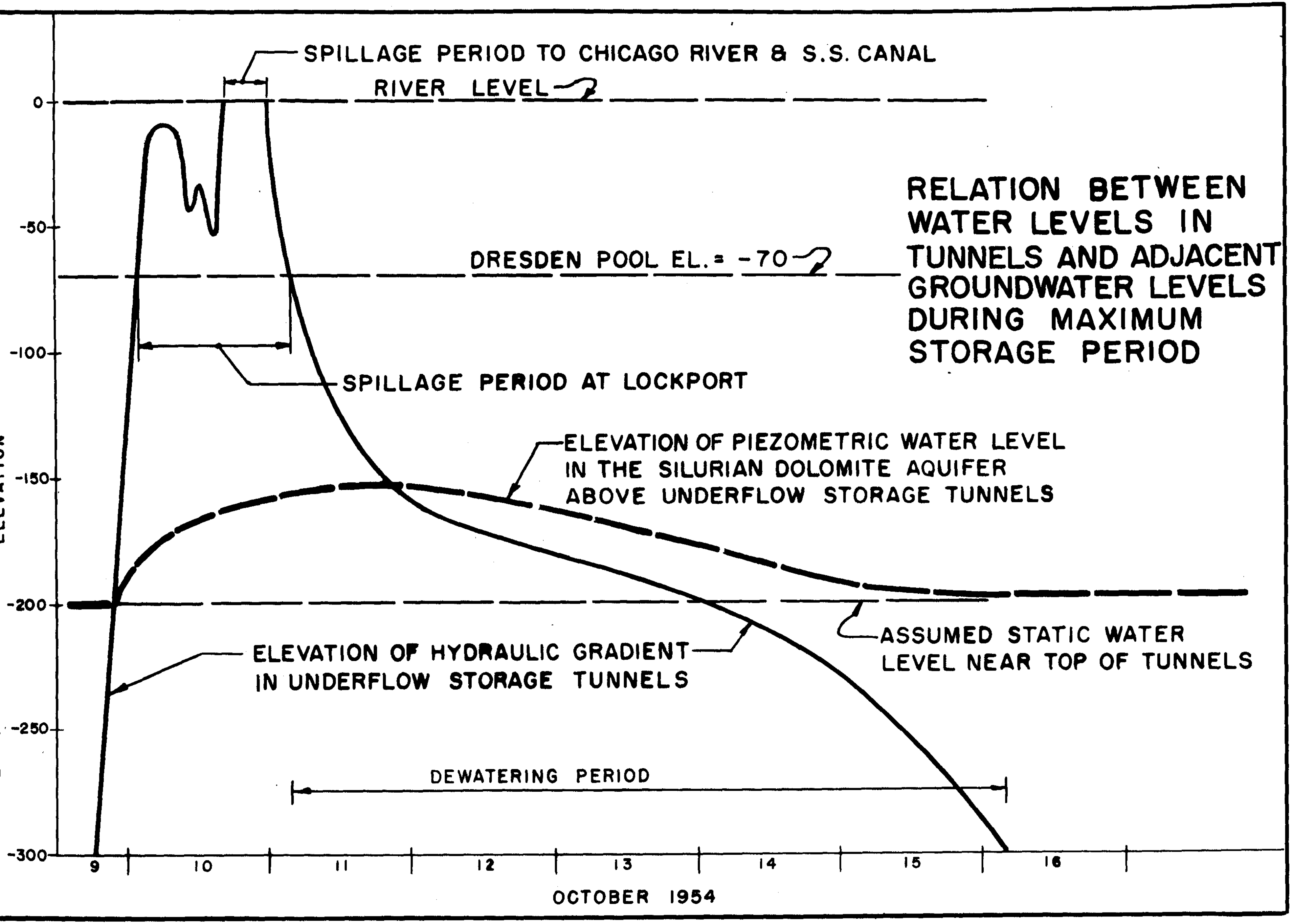

SPILLAGE PERIOD TO CHICAGO RIVER & S.S. CANAL
RIVER LEVEL
DRESDEN POOL EL. = -70
RELATION BETWEEN WATER LEVELS IN TUNNELS AND ADJACENT GROUNDWATER LEVELS DURING MAXIMUM STORAGE PERIOD
SPILLAGE PERIOD AT LOCKPORT
ELEVATION OF PIEZOMETRIC WATER LEVEL IN THE SILURIAN DOLOMITE AQUIFER ABOVE UNDERFLOW STORAGE TUNNELS
ASSUMED STATIC WATER LEVEL NEAR TOP OF TUNNELS
ELEVATION OF HYDRAULIC GRADIENT IN UNDERFLOW STORAGE TUNNELS
DEWATERING PERIOD
ELEVATION
0
-50
-100
-150
-200
-250
-300
9
10
11
12
13
14
15
16
OCTOBER 1954

FIGURE 19

To arrive at a conservative estimate for the rock excavation (including hauling and disposal) conferences have been continued with manufacturers of rock drilling, blasting and hauling equipment. These companies have had wide experience in working with contractors engaged in rock tunneling throughout the United States and elsewhere.

These conferences and studies were a continuation of similar conferences and inspection trips, participated in by the Bureau of Engineering, as well as the Metropolitan Sanitary District and their consultants.

Figure 20 shows graphically the variation in unit cost of rock excavation with the size of the tunnels and the number of simultaneously worked tunnel headings. Prices per cubic yard 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.

Table 7 shows the quantities of rock excavation required for the various tunnel sizes and locations.

Table 8 shows the overall contract cost of the tunnels required, classified as to size, location and construction phase.

Table 9 is a summary of Total Project Cost, including contingencies, and engineering and supervision.

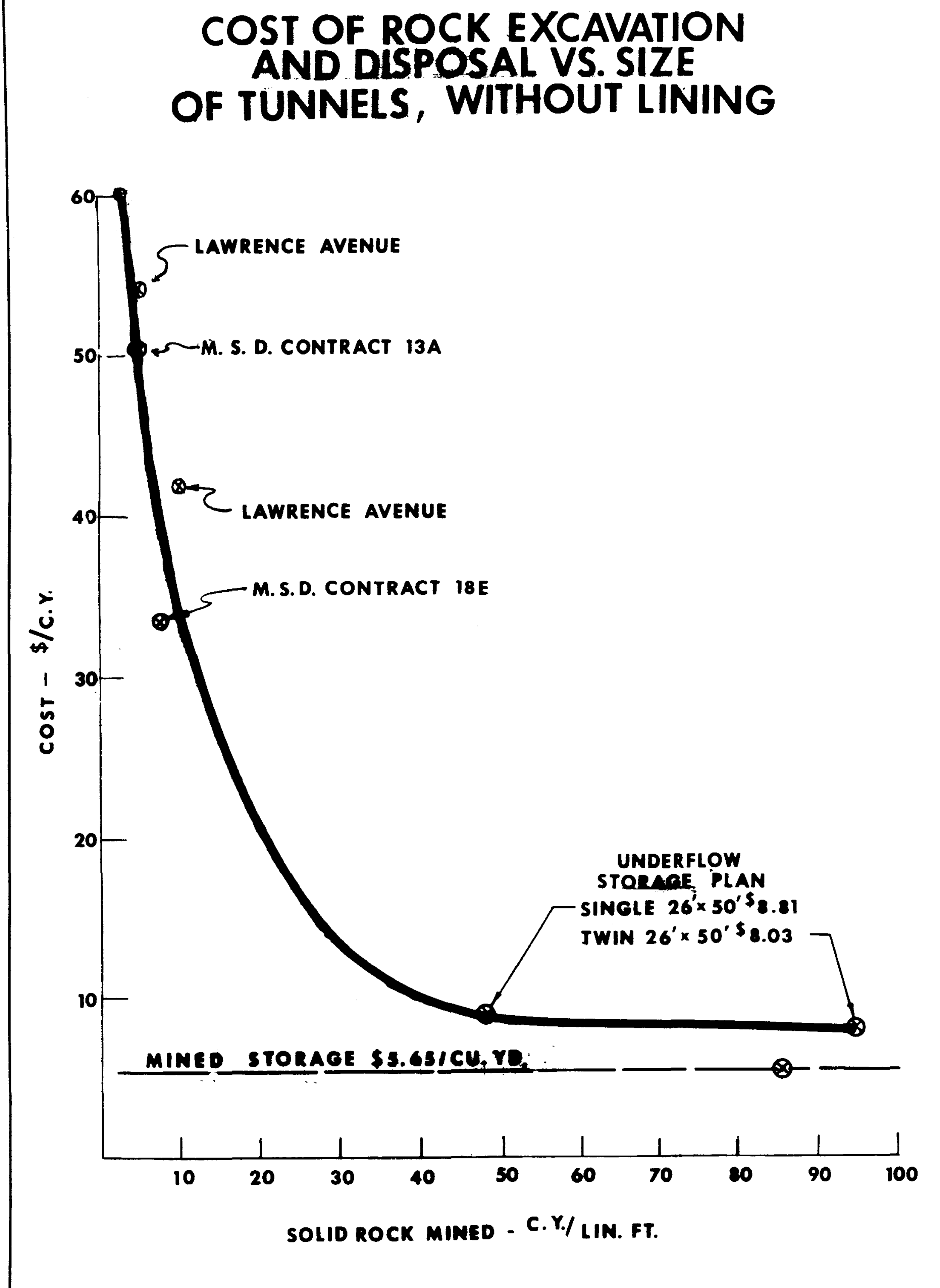

COST OF ROCK EXCAVATION AND DISPOSAL VS. SIZE OF TUNNELS, WITHOUT LINING
LAWRENCE AVENUE
M. S. D. CONTRACT 13A
LAWRENCE AVENUE
M. S. D. CONTRACT 18E
UNDERFLOW STORAGE PLAN
SINGLE 26'× 50' $8.81
TWIN 26'× 50' $8.03
MINED STORAGE $5.65/CU. YD.
COST — $/C.Y.
60
50
40
30
20
10
10 20 30 40 50 60 70 80 90 100
SOLID ROCK MINED - C.Y./ LIN. FT.

FIGURE 20

TABLE 7

QUANTITIES OF ROCK EXCAVATION (Main Tunnels Only)
(Rock "in situ", or solid rock)

LOCATION	SIZE	LENGTH (MILES)	ROCK EXCAVATION - THOUSAND CU.YDS. PER MILE	PHASE I	PHASE 2	TOTAL
Mainstream	Twin 26'x50'	11.99	502.0	6,030		
Mainstream	Twin 26'x50'	20.38	502.0		10,250	
Mainstream	Single 26'x50'	13.53	251.0	3,406		
Mainstream	17' ϕ	3.00	51.6	155		
Mainstream	12' ϕ	12.80	26.9	344		
TOTALS		61.70		9,935	10,250	20,185
Calumet Br.	Single 26'x50'	8.27	251.0	2,090		
Calumet Br.	Single 26'x50'	12.45	251.0		3,120	
Calumet Br.	14' ϕ	2.00	36.0	72		
Calumet Br.	10' ϕ	2.29	19.8	45		
TOTALS		25.01		2,207	3,120	5,327
Des Plaines Br.	Single 26'x50'	19.50	251.0		4,880	4,880
GRAND TOTALS		106.21		12,142	18,250	30,392

Note: -
Loose rock to be handled is estimated at 140% of volume shown or approximately 42 million cubic yards.

TABLE 8

ESTIMATED CONTRACT COST OF TUNNELS

PHASES 1 AND 2

LOCATION	SIZE	LENGTH (MILES)	COST IN THOUSAND DOLLARS			
			PER MILE	PHASE I	PHASE 2	TOTAL
Mainstream	Twin 26'x50'	11.99	5,500	65,945		
Mainstream	Twin 26'x50'	20.38	5,500		112,000	
Mainstream	Single 26'x50'	13.53	3,000	40,590		
Mainstream	17' ϕ	3.00	2,700	8,100		
Mainstream	12' ϕ	12.80	2,000	25,600		
TOTALS		61.70		140,235	112,000	252,235
Calumet Br.	Single 26'x50'	8.27	3,000	24,810		
Calumet Br.	Single 26'x50'	12.45	3,000		37,350	
Calumet Br.	14' ϕ	2.00	2,300	4,600		
Calumet Br.	10' ϕ	2.29	1,700	3,893		
TOTALS		25.01		33,303	37,350	70,653
Des Plaines Br.	Single 26'x50'	19.50	3,000		58,500	58,500
GRAND TOTALS		106.21		173,538	207,850	381,388

COST OF BRANCHES - Phase 3

Tributary to Calumet Branch - 36,650
Tributary to DesPlaines Branch -35,000

Total Phase 3 71,650

Total Phases 1, 2 & 3 $453,038

TABLE 9

SUMMARY OF PROJECT COST

Tunnels (See Table 8)	$453,038,000
Pumping Stations	50,000,000
Pump Discharge Conduits	3,000,000
Drop Shafts From Combined Sewer Outlets to Tunnels	65,000,000
Drop Shafts From River to Tunnels	6,000,000
Tunnel Outflow Controlling Works @ Lockport	5,000,000
Subsurface Exploration	5,000,000
TOTAL CONTRACT COST	$587,038,000
Miscellaneous Work (Use 5%)	29,352,000
	$616,390,000
Engineering & Supervision, 10%	58,610,000
TOTAL PROJECT COST	$675,000,000

Cost as of May, 1970

E.N.R. Construction Cost Index 1,417.41

CONSTRUCTION PHASES

RECOMMENDED FIRST PHASE CONSTRUCTION

The authors recommend that the First Phase construction be started at the earliest possible date in order to meet the requirements of Water Quality Standards of the State and Federal Governments. These standards require compliance by 1978.

The First Phase construction of the Underflow-Storage Plan would include the large conveyance tunnels under the North Shore Channel; Chicago River and its North and South Branches, and South Fork; and the Sanitary and Ship Canal; all between the Wilmette controlling works and the outlet of the Southwest Side Intercepting Sewer, Contract 13-A, an Underflow Sewer, just west of Harlem Avenue. The First Phase construction would also include the conveyance tunnels beneath the Calumet Sag Channel, Little Calumet and Calumet Rivers, all lying between the outlet of the Calumet Intercepting Sewer, Contract 18-E, an Underflow Sewer at Crawford Avenue and the 95th Street Pumping Station.

A new Underflow Sewer to the west through Skokie, Illinois, and extensions of the Lawrence Avenue Underflow Sewer to the north, would intercept combined sewer overflow outlets along the North Branch of the Chicago River upstream of its confluence with the North Shore Channel.

Drop shafts, connecting all combined sewer overflow outlets along the route of the First Phase tunnels should be constructed. Also the three major pumping stations for dewatering the tunnels to the treatment plant facilities, must be constructed under the First Phase work.

The First Phase would serve a tributary area of about 240 square miles and have an underground storage of about 7,000 acre-feet or the equivalent of 0.56 inches of storage. Referring to Figure 8, and prorating for area (300/240 x 7,000 AF = 8,000 AF), it appears that 95 percent of the pollutants would be entrapped from the tributary 240 square mile combined sewer area.

The construction under the First Phase of the Combined Underflow-Storage Plan, would greatly reduce the spillages from 75 percent of the combined sewer outlets, covering

80 percent of their tributary drainage area. It is anticipated that the First Phase construction, coupled with effluent improvements at the treatment plants will substantially meet the waterway standards for many reaches of the waterway system.

It is understood that the tunnel configuration, size, elevation, storage volume and location may be modified somewhat as detailed engineering design progresses. However, the following cost estimate for the First Phase construction is believed to be conservative.

Underflow-Storage Tunnels	$173,538,000
Three Pumping Stations	53,000,000
Drop Shaft Connections	50,000,000
Subsurface Exploration	4,000,000
Miscellaneous Facilities	12,000,000
	$292,538,000
Engineering & Supervision 10% ±	28,962,000
	$321,500,000

It appears that considerable benefit will be accomplished at less than one-half of the total cost of the Combined Underflow-Storage Plan by the construction of the First Phase. It should be emphasized, however, that the First Phase will not accomplish the basic criteria established. It will not provide complete flood relief to prevent discharge of river water to Lake Michigan, the needed relief from flooding along the DesPlaines River and Calumet Waterways, nor will it intercept combined overflow outlets from some 20 percent of the combined sewer area. All of these requirements must be included in the Second Phase work.

The schedule for completing the First Phase work is becoming very critical. In order to have any possible chance of meeting the compliance date of 1978, it is imperative that the First Phase engineering design be started immediately.

SECOND PHASE STUDY

Concurrently with the engineering design work of the First Phase, a study should be made of the Second Phase. There still remains a difference of opinion regarding the most economical method of handling the runoff during excessive storm periods.

The Combined Underflow-Storage Plan proposes storage in transit and underflow conveyance through large tunnels 200 to 300 feet below the Calumet Sag Channel and Sanitary and Ship Canal to a lower water surface at Lockport. The Deep Tunnel Plan proposes to store all of the storm water runoff in large underground mined storage chambers 600 to 800 feet below the surface, and in surface reservoirs, and to produce and sell peaking power to defray a part of the cost of the pumping-generating facilities and reservoirs. The Chicago Drainage Plan proposes channel lowering and widening to provide both surface storage and to improve conveyance capacity. Also, such widening and lowering of water surfaces would be of benefit to navigation.

The First Phase construction as set forth herein, will be compatible with expanding under the Second Phase along the lines of either of the three general plans proposed, with only minor modifications thereto.

Mined storage chambers could be added at 600 to 800 feet below the ground level in the McCook and Calumet Areas to form the lower reservoirs of a pump-storage system, as proposed in the Deep Tunnel Plan. Siphon overflow shafts could connect the Underflow-Storage tunnels in the shallower Niagaran strata limestone formations. The Underflow-Storage tunnels and pumping stations at the treatment plants under the First Phase work, would handle the runoff from the small to medium storms. In the excessive rainfall periods, runoff that exceeds the Underflow-Storage tunnels would spill through the siphon shafts to large mined reservoirs. The combined storage of the First Phase Underflow-Storage tunnels, the low level mined chambers and surface reservoirs would meet the criteria set forth for both the flood control and pollution problems.

The First Phase construction would also eliminate the need of providing retention tanks at the combined sewer outlets as proposed in the Chicago Drainage Plan. Further, studies with the computer water quality model would determine the amount of additional underground storage that may be required to meet the waterway standards (SWB-15) for all reaches.

The waterway improvements for navigation and flood control proposed in the Chicago Drainage Plan, would provide the necessary outlet capacity. Studies may show that large pumping facilities at the western terminus of the First Phase Underflow-Storage tunnels are needed to lift the water to the enlarged Sanitary and Ship Canal and Sag Channel, during the larger storm periods.

The First Phase construction, of course, can be expanded, as outlined herein, as the Combined Underflow-Storage Plan.

Extension of some of the Underflow-Storage tunnels and Underflow Sewers in the Second Phase, will be required in any plan, to intercept the combined sewers along the DesPlaines River and northwest communities, and through the Little Calumet River area.

A systematic study of all alternates should be undertaken to determine the merits of each of the plans proposed. It is by such a study that the best and most economical scheme from a benefit/cost standpoint can be determined for the Second Phase construction.

SUMMARY AND CONCLUSION

Having outlined some of the major problems associated with flooding of basements and underpasses and the inadequacy of present rivers and canals to carry off flood flows, it becomes apparent that major flood control facilities, at great expenditure of monies, are required. The polluted condition of these open watercourses must also be eliminated to meet the standards established by the State and Federal pollution control agencies. A primary source of pollution, namely, the spillages of polluted water from combined sewers in time of storms, has been the subject of this study and report.

Three separate schemes have been described for solving these flooding and pollution problems in the Chicago Metropolitan Area. These are the Underflow Storage Plan, the Deep Tunnel Plan and the Chicago Drainage Plan.

A large relief sewer system proposed in the City's Capital Improvement Program has been redesigned as an Underflow Sewer along the conceptual plan of the metropolitan area-wide Underflow-Storage Plan. This Underflow Sewer in Lawrence Avenue is now being constructed with the aid of a FWQA demonstration grant for $1,500,000. Since the time this project was started, two other Underflow Sewers were placed under construction by the Metropolitan Sanitary District.

The construction of these sewers has already demonstrated the anticipated quality of the dense dolomitic limestone rock which is prevalent throughout the Chicagoland area, and the structural ability of such rock to adequately support the proposed large tunnels.

A complete description of the Underflow-Storage Plan has been presented which will reduce the spillage of pollutants to the surface waterways by over 98.5 percent and provide the necessary flood control to handle the 100-year frequency storm. Preliminary runs with a mathematical computer water quality model indicate that the Underflow-Storage Plan, together with improved sewage treatment plants, will clean up the waterways so that they will be in compliance with the State and Federal Standards.

It is recommended that the First Phase of the Underflow-Storage Plan, which would include the construction of tunnels along the North Shore Channel, Chicago River System and Sanitary and Ship Canal between Wilmette Controlling Works and Harlem Avenue be started immediately. Also, this First

Phase should include tunnels along the Sag Channel, Little Calumet and Calumet Rivers between Crawford Avenue and the 95th Street Pumping Station. Drop shaft connections and pumping facilities would be constructed along the route of the tunnels under the First Phase work.

It is estimated that the First Phase would cost approximately $322,000,000 and would provide a direct benefit to a 240 square mile tributary combined sewer area. Pollution quantities which now overflow to the waterways in time of storm from that area, would be reduced by 95 percent.

The First Phase work can be expanded in the Second Phase along the lines of either the Underflow-Storage Plan, the Deep Tunnel Plan, or the Chicago Drainage Plan. It is further recommended that a complete conceptual study of the Second Phase work be done concurrently with design and construction of the First Phase.

ACKNOWLEDGEMENTS

This report was prepared by the Bureau of Engineering, Department of Public Works, City of Chicago; and reviewed by the Division of Waterways, Department of Public Works and Buildings, State of Illinois; and the Engineering Department of the Metropolitan Sanitary District of Greater Chicago.

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DETROIT SEWER MONITORING and REMOTE CONTROL

Research Project, Aiming at the Reduction of Combined Sewer Overflow Pollution in Detroit using System Monitoring & Remote Control Techniques

FEDERAL WATER QUALITY ADMINISTRATION
DEPARTMENT OF THE INTERIOR

by

Detroit Metro Water Department
735 Randolph Street
Detroit, Mich. 48226

F.W.Q.A. RESEARCH & DEVELOPMENT PROJECT 11020 FAX

June, 1970

SEWER MONITORING AND REMOTE CONTROL-DETROIT

ABSTRACT

Detroit is faced with the problem of preventing pollution of the Detroit and Rouge Rivers from its combined sewer system overflows. As an alternative to undertaking a dubiously effective sewer separation program, estimated to cost in excess of two billion dollars, the Detroit Metro Water Department has installed the nucleus of a sewer monitoring and remote control "system" for controlling the pollution from the combined overflow from many small storms at a cost of slightly over two million dollars.

The "system" includes telemetering rain gages, sewer level sensors, overflow detectors, a centrally located computer and data-logger, and a centrally located operating console for controlling pumping stations and selected regulating gates. Installation has been virtually completed and now enables applying such pollution control techniques as "storm flow anticipation", "first flush interception", selective retention" and "selective overflowing".

An evaluation of the effectiveness of this initial installation will serve as the basis for determining what additional pollution control facilities are required, what suburban monitoring and remote control is essential, what computer related equipment for pump and valve control can be used for more effective pollution control, what automatic sampling and analysis will be most valuable in the synchronous operation of the sewerage system and what design parameters should be used in the construction of new or supplemental sewers or treatment facilities.

KEY WORDS:
1. Combined Overflows
2. System Monitoring
3. Remote Control
4. Storm Flow Anticipation
5. First Flush Interception
6. Selective Retention
7. Selective Overflowing

"This interim report is submitted in partial fulfillment of Research and Development Project 11020 FAX between the Federal Water Quality Administration and the Detroit Metro Water Department.

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SECTION I

INTRODUCTION

The pollution resulting from combined sewerage systems is the one common problem which has plagued all sanitary engineers doing work in the older cities of this country. Overflows from these combined sewers (those which carry sanitary sewage, storm water runoff and industrial wastes) have been increasing and they may now be classified as one of the primary pollution problems of this era. Our forebears built these combined systems, which economically carry dry weather flow to interceptors, with the thought in mind that overflows would be infrequent, of relatively short duration and sufficiently dilute so as not to harm receiving waters. However, with the growth and development of our urban centers (which appear to have ever expanding impervious surfaces) these same sewers now spill a high portion of this mixed flow, untreated, to receiving streams during runoff from major storms. This report deals with one approach toward controlling this major cause of degradation to our nation's vital natural resource.

The Detroit Metro Water Department (DMWD) is a regional agency that serves 70 communities in Southeastern Michigan with drinking water and provides wastewater interception and treatment service to 55 communities. (These figures as of JANUARY 1970) It has contracted to serve dozens of additional communities in the near future. Basically, the pure water needs of 40% of the population of the State of Michigan are served by DMWD and the same agency is providing wastewater disposal service to 30% of the State' populace.

The agency also has the responsibility for constructing, operating and maintaining the sewer collection, the drainage and the water distribution system within the City limits of Detroit. The suburban communities which are served by DMWD have each retained responsibility for the operation and maintenance of their local sewer collection and water distribution networks.

The map on Figure 1 shows the status of wastewater disposal service in Southeastern Michigan as of January 1970.

The entire DMWD service area is presently connected to a 1200 M.G.D. single treatment plant located near the junction of the Detroit and Rouge Rivers. The capacity at the plant is being expanded and the facilities are being upgraded under an agreement with the Michigan Water Resources Commission. This agreement is one of the resulting actions to come from the 1965 Conference on Pollution of the Detroit River, Lake Erie and their Tributaries. Construction at the plant site began in July of 1969 and the advanced treatment facilities now being installed will remove between 80 and 90% of all impurities such as suspended solids, dissolved organics, phosphates, phenols and oils as well as keeping coliform bacteria limits to less than 1000 per 100 ml.

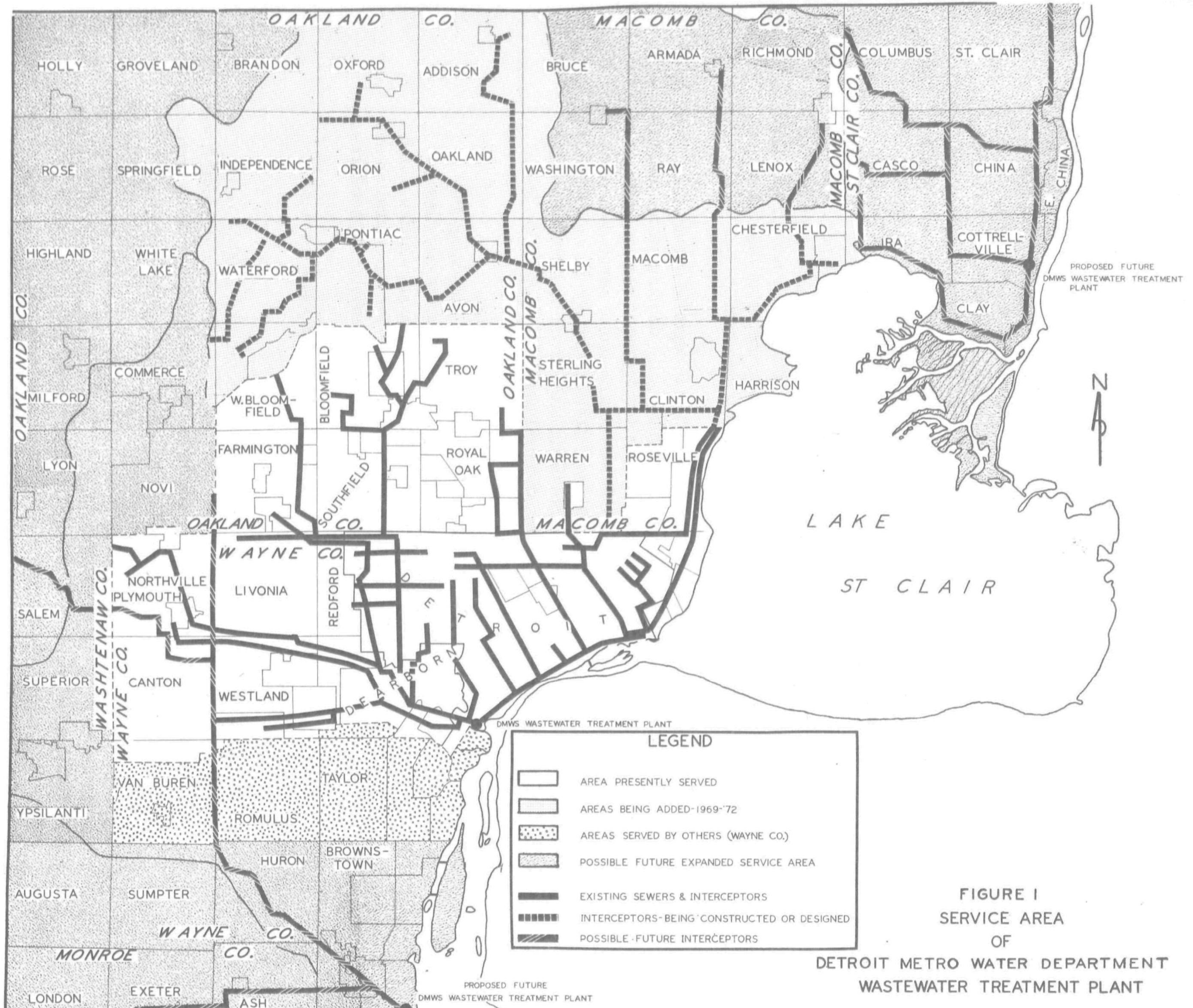


FIGURE I
SERVICE AREA
OF
DETROIT METRO WATER DEPARTMENT
WASTEWATER TREATMENT PLANT

The Agreement with the Michigan Water Resources Commission also calls for the City of Detroit to take immediate steps to decrease the frequency, magnitude, and pollutional content of all overflows of combined sewage, industrial wastes, and storm water from the City's sewerage system to the Detroit and Rouge Rivers. The Agreement further stipulates that a study be made of methods and costs of achieving these desired reductions and this project is a direct part of that study.

POSSIBILITIES OF MONITORING AND REMOTE CONTROL

In order to utilize the potential of such pollution control techniques as "storm flow anticipation", "first flush interception", "selective retention", and "selective overflowing", one needs to have instantaneous and accurate information about the behavior of the overall sewerage system. This information must include rainfall events taking place within (and without) the contributory drainage area, sewer and interceptor levels, and the status of pumps, valves, and backwater gates. It is of equal importance to be able to remotely control the pumps and valves so that one may react in accordance with the data being received.

The Detroit Metro Water Department has been monitoring water pressures and remotely operating water pumping stations and valves throughout the metropolitan area for eight years. Utilizing this experience, DMWD studied the possibilities of installing a sewer monitoring system with remote control of sanitary and storm pumping stations and regulating gates. With the aforementioned pollution control techniques in mind, the following factors related to the decision to install a monitoring and remote control system.

1. There are large areas served by pumping stations whose tributary lines could be used as storage areas during small storms.

2. The grades of the sewers, either rectangular boxes or cylinders, are relatively flat which would permit substantial storage under level conditions, near the outfalls.

3. Interceptors along the Detroit and Rouge are fed through float-controlled regulators equipped with sluice gates which appear to be adaptable to conversion to remote controlled power actuated regulators.

4. Most of the 71 outfall points are equipped with backwater gates and/or dams which serve as automatic retention devices.

5. Interconnections exist throughout the system which could be used for flow routing if remote controlled gates are added.

6. From knowledge of the particular industries connected to certain sewers, there apparently would be a wide variation in the quality of dry weather effluent.

7. To be able to evaluate any method which attempts reduce combined overflows it is first necessary to establish the existing conditions.

Monitoring would allow the collection of these data.

POTENTIAL BENEFITS

With central system monitoring and remote control, the following benefits appeared possible:

1. The sewerage system could be operated to completely contain a small spot storm.

2. Runoff could be anticipated, sewers could be emptied and in readiness. Grossly contaminated first flushes in areas adjacent to the interceptor could selectively be captured, especially during large storms.

3. All flow near the end of a large storm could be held in the system for subsequent treatment.

4. Regulators could be adjusted to get the most efficient use of the interceptor and set to favor the most grossly contaminated inlets.

5. Backwater from floods entering unprotected outfalls in the northwestern part of the City could be selectively controlled.

6. Information on the level of flow within the sewers would provide sufficient lead time so that pumps could be operated to minimize basement flooding in the east side areas which have no gravity relief outlets.

7. The flow to the wastewater plant from various segments of the City could be better balanced.

8. The data collected could be used for deriving new design criteria which would be of benefit for future improvements to the sewerage system.

MONITORING VS COST OF COMPLETE SEPARATION

Complete separation of the combined sewerage system of the City of Detroit into separated storm and sanitary sewers has been estimated to cost over $2 billion and would probably take from 30 to 50 years to accomplish. Separation would require excavation at nearly every house in the City to change the connections, and appears also to have the following drawbacks.

1. There is no assurance that all cross connections can be eliminated by separation. Detroit has a separated area on its east side in which a single storm water connection to the sanitary system can go undetected for 2 or 3 years even after the agency is aware that it probably exists and is causing problems.

2. The first flush in a 100% storm sewer is highly contaminated. Selective interception of this flush will undoubtedly be required in the future.

3. Time is a factor. System Monitoring and Remote Control could be installed and in operation within two years -- rather than 40 years. The benefits would begin immediately.

Detroit believes that storm anticipating, monitoring, storm storage and remote control will be somewhat effective (but to a lesser degree) on larger storms, such as rainfall rates of 2" per hour or greater. However, the major advantage will be in retaining more of the runoff from smaller storms which contribute the most highly contaminated overflows.

DETROIT'S TOPOGRAPHY

The terrain in Detroit is gently sloping to flat. There are no hills within the City limits. From a high elevation of 667 ft. (USGS) above sea level at Wyoming and Eight Mile to the mean elevation of 576 at the Detroit River, there is a fall of 91 ft. in a sewer distance of 14.5 miles. This gives an average fall of only 1.17 ft. in a 1,000 ft. length. The net hydraulic fall is even less since the lateral sewers at the upper extremity of the system are a minimum of 8 ft. deep. This flat condition makes for nearly horizontal slopes to the gravity sewer system with particularly flat grades in the lower reaches where possible volumetric storage can be affected.

The ground slopes largely from the north to the south except in the area of the Rouge River Valley where the ground slopes to the east and to the west toward the river.

Figure 2 shows the original watershed within Detroit.

The original sewers generally followed the slope of the drainage basins. The north and northeast sections slope gently into what were known as Conner Creek and Fox Creek, respectively. The central portions of the city slope gently directed to the Detroit River. The western portion of the City is drained by the Rouge River which meanders through northern suburbs, through Detroit and through southwestern suburbs to reach the Detroit River. When relief of the original sewers was needed, the relief lines were constructed at right angles to the rivers in order to get the shortest lengths and steeper grades.

FIGURE 2 - DETROIT'S ORIGINAL WATERSHED

Detroit's sewers were installed by tunnel or open cut method through glacial till or lacustrine clays. No rock ledges were encountered in building these tunnels.

DETROIT SEWERAGE SYSTEM CHARACTERISTICS

Approximately 85% of the 139 sq. miles within Detroit can be drained by gravity sewers and 15% of the area requires lift pumps. Nearly 98% of the sewerage system is of the conventional combined type.

Figure 3 shows the areas of the City of Detroit served by large sewerage lift stations. Stormwater from the Bluehill Pumping Station even requires repumping by the Freud or Conners Stations for discharge to the Detroit River.

The diked area in southeast Detroit, which was reclaimed from the Detroit River by enterprising real estate men over forty years ago during a period of low water, is several feet below River elevation.

Figure 4 shows a small area north of Mack Avenue which was developed as a separated sewer system. The sanitary flow outlets by gravity to the Detroit River Interceptor at Alter and Jefferson. However, to accelerate the street flow and roof conductor runoff, all storm flow from the district is drained into the deeper Conner-Freud storm pumping complex.

The Detroit River Interceptor is located adjacent to the river. It varies in size from 8 ft. at the east city limits to 16 ft. I.D. at the treatment plant. It flows by gravity from the Grosse Pointe area to the treatment plant with only one lift, at the Fairview Sanitary Lift Station.

Along the Rouge River the Oakwood-Northwest Interceptor in the Rouge Valley intercepts sanitary flow from the various combined outfalls along the Rouge River and brings this flow to the treatment plant. It crosses under the River at several points by inverted siphon. This interceptor varies in size from 4 ft. to 12'-9" I.D.

Figure 5 shows a typical float controlled regulator which allows flow into the interceptors. These regulators are normally set to close when flow in the interceptors nears the 7/10ths point.

Some 77% of the regulators were manufactured by Brown and Brown Co. and are actuated by tell-tale pipes connected to the interceptor. When water levels in the interceptor rise, the float rises in the float chamber, which in turn, through a series of chains or cables and a transmission shaft, allows the regulator gate to close, shutting off combined flow into the interceptor before the interceptor floods out. When the regulator closes, all of the combined flow is then carried to the outfall and into the receiving stream.

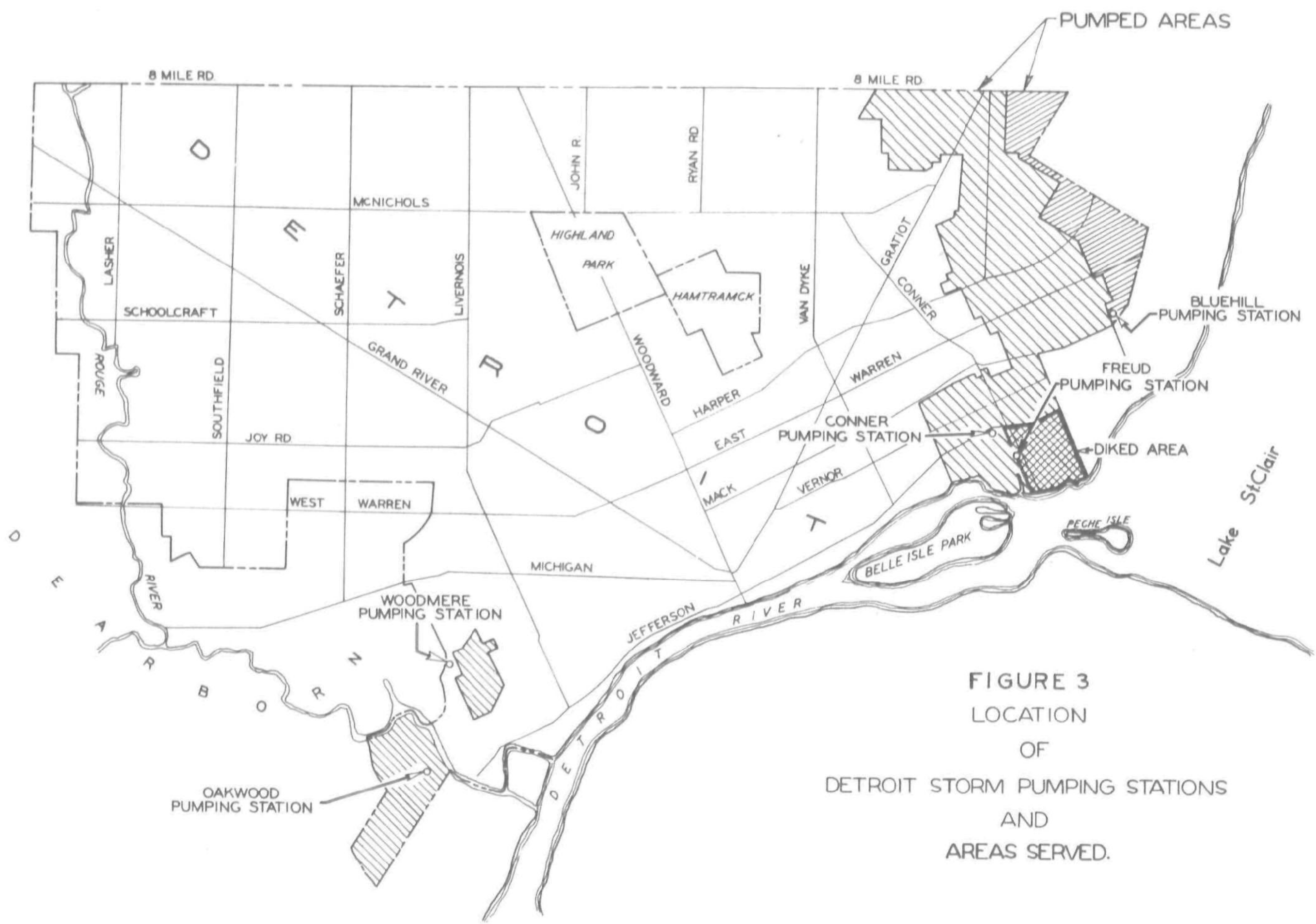


FIGURE 3
LOCATION
OF
DETROIT STORM PUMPING STATIONS
AND
AREAS SERVED.

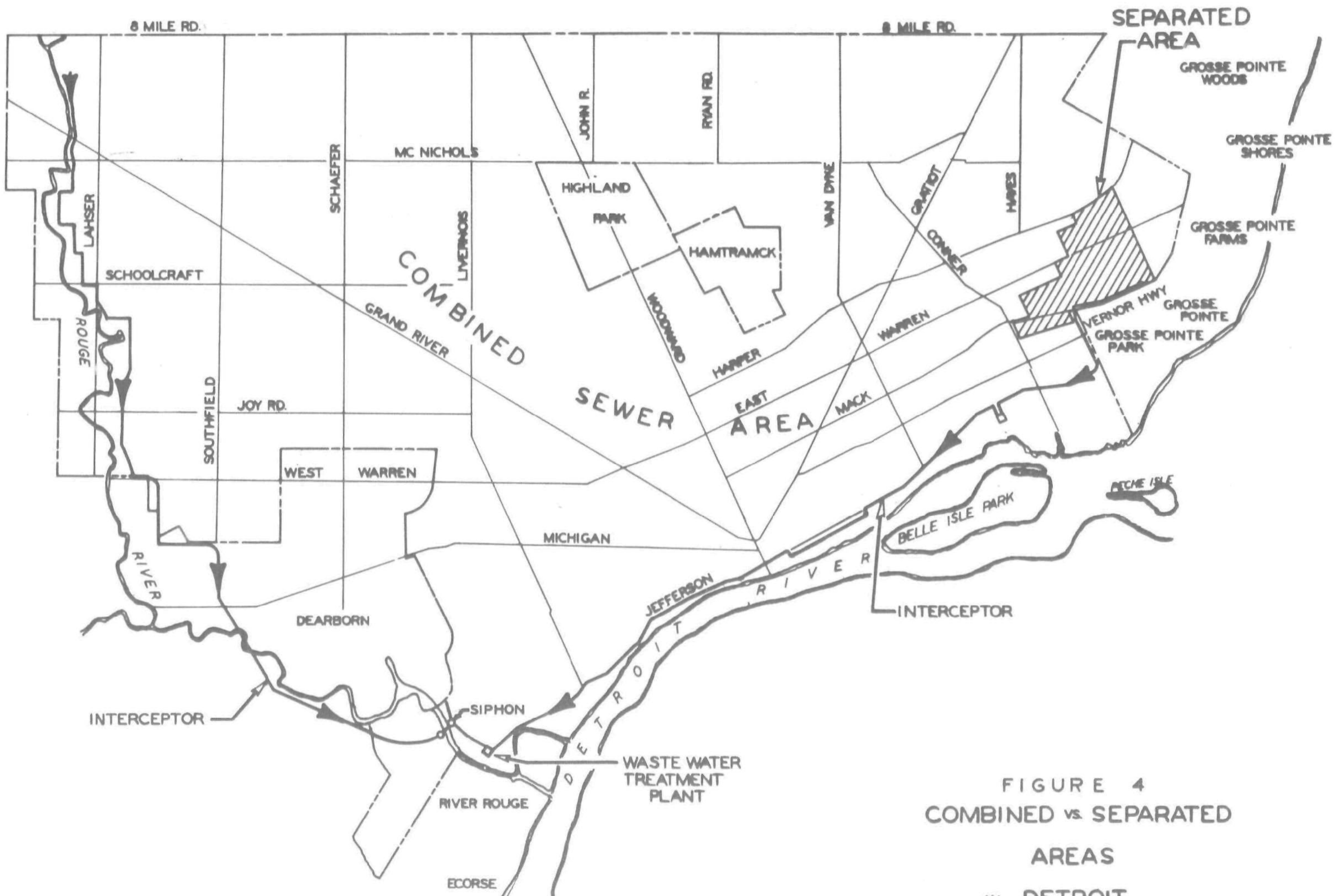


FIGURE 4
COMBINED vs. SEPARATED
AREAS
IN DETROIT

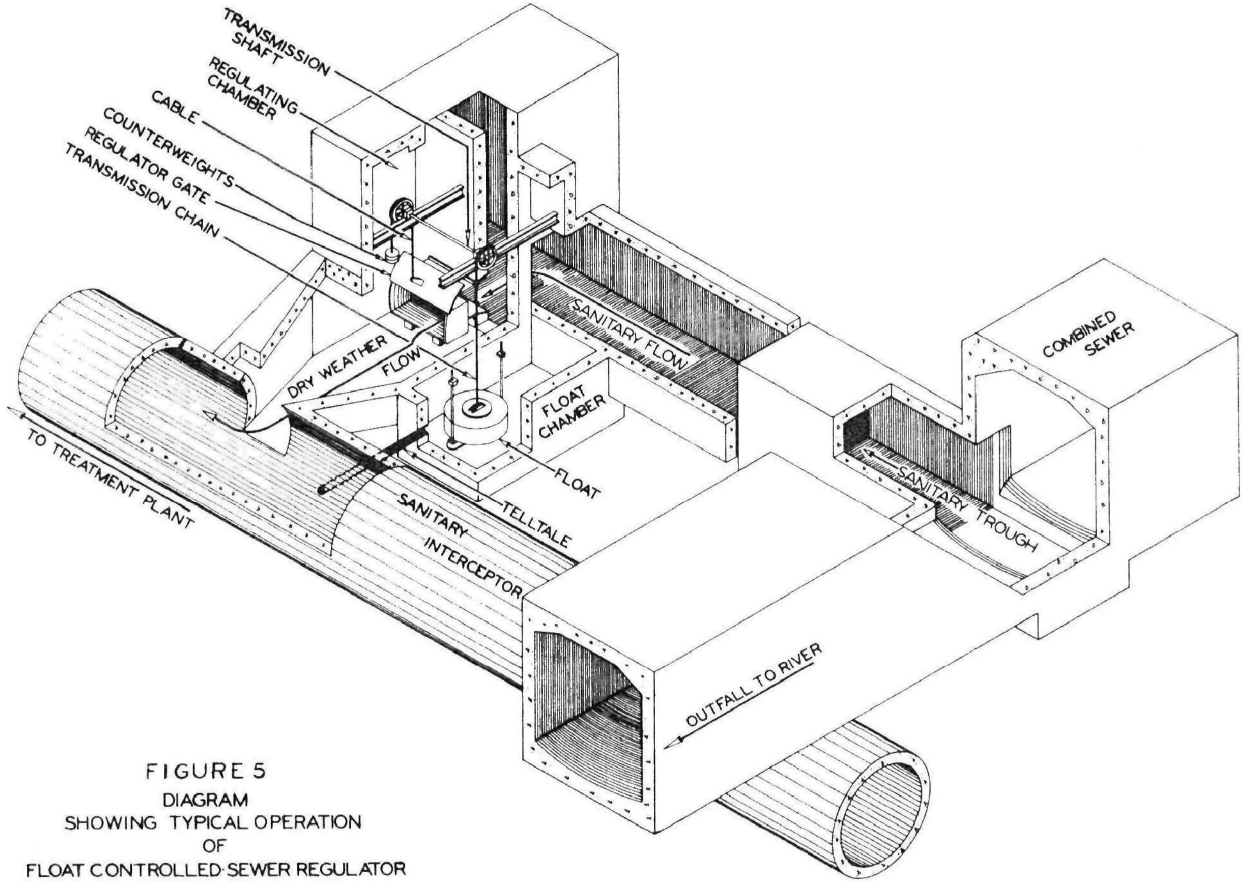


FIGURE 5
DIAGRAM
SHOWING TYPICAL OPERATION
OF
FLOAT CONTROLLED SEWER REGULATOR

Wherever needed, Detroit has installed protective backwater gates or dams downstream of the regulator to prevent backflow from entering the interceptor from the rivers. Some outfalls are lower than the average river levels and during periods of high water many outfalls are completely submerged.

SECTION II.

MONITORING AND REMOTE CONTROL EQUIPMENT

The following equipment[1] has been installed by this project:

(a) 14 telemetering rain gages supplied by Belfort Instrument Co., Baltimore, Maryland.

(b) 80 telemetering sewer level sensors, 40 telemetering interceptor level sensors, and 4 telemetering river level sensors. The sensor cells were fabricated by the contractor to DMWD specificat ions and the transmitters and receivers were supplied by Quindar Electronics, Inc., Springfield, N. J.

(c) 30 telemetering proximity sensors on backwater gates supplied by Minneapolis - Honeywell.

(d) 38 telemetering probe-type dam overflow sensors, which consist of a set of electrodes with an amplifier, supplied by B/W Controller Corp., Birmingham, Michigan.

(e) 1 central digital computer with drum and disc memory Model PDP8 supplied by the Data Master Division of the Bristol Co., Glen Cove, New York.

(f) 3 data loggers (computer controlled typewriters) with 30 inch platens supplied by I.B.M. Corp.

(g) 1 teletypewriter for input, output and alarm supplied by Teletype Corporation, Skokie, Illinois.

(h) 1 operator console supplied by the Data Master Division of the Bristol Co., Glen Cove, New York.

(i) Central Control Panel containing the following equipment:

8 sets of equipment for the remote control and monitoring of pumping stations supplied by Quindar Electronics Inc., Springfield, New Jersey.

5 sets of equipment for the remote control and monitoring of sluice and flushing gates, also supplied by Quindar Electronics. The gates were supplied by the Rodney-Hunt Co. of Orange, Mass. and the motor

1. All commercial products were purchased on a low bidder basis and mention by name does not imply endorsement by the Federal Water Quality Administration or the Detroit Metro Water Departm ent.

operators for these gates were from Philadelphia Gear Corp., Philadelphia, Pennsylvania.

18 recorders for pump station suction and discharge lines supplied by the Bristol Co., Glen Cove, New York.

Details of the aforementioned equipment are explained below.

(a) Rain Gages

Figure 6 shows the locations of the telemetering rain gages. These are tipping bucket gages which telemeter a pulse signal to the central-control office for every 1/100 of an inch of rain. The gages, one of which is shown on Figure 7a are for the most part installed on the flat roofs of DMWD buildings. Figure 7b shows a close-up view of the gage mechanism.

These roofs are not perfect sites, as an exposed location in an open field would possibly be less influenced by wind effects, but all such open sites in a big city would be subject to vandalism, so the compromise of a location on a low flat roof was made.

Whereas the tipping bucket gages are primarily for operation, the existing set of 16 spring wound 8-day clock weighing rain gages (which are generally adjacent to the new gages) are being kept in service as an accuracy check and for the purpose of historical record.

(b) Level Sensors

The location of telemetering level sensors are shown on Figure 8. These are located on all the larger trunk line sewers 10 ft. diameter and larger, plus on certain critical smaller upstream lines. Level sensors are also installed at all wet wells of all pumping stations.

Detail of Detroit Level Sensor

The sewer level sensor, as shown on Figure 9 consists of a 2" I.D. polyvinylchloride tube some 11 inches long to which is attached a 1/4" O.D. nylon tube. Dry air is entrained in the cell and tubing. When the sewer water level rises, it slightly compresses the trapped air in the pressure cell which in turn compresses the air in the 1/4" nylon tube which actuates a transmitting bellows located in pedestal cabinet on the surface.

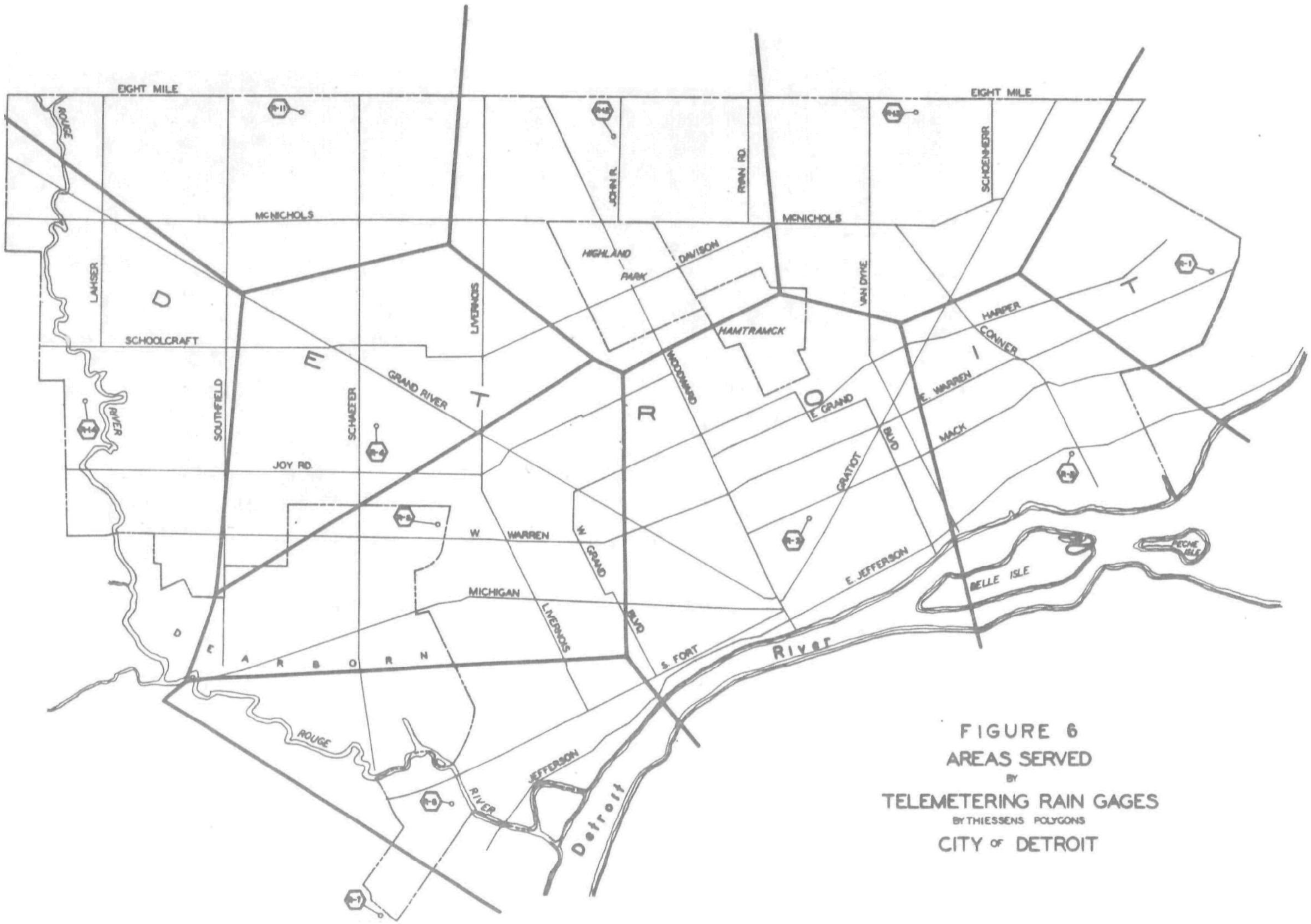


FIGURE 6
AREAS SERVED
BY
TELEMETERING RAIN GAGES
BY THIESSENS POLYGONS
CITY OF DETROIT

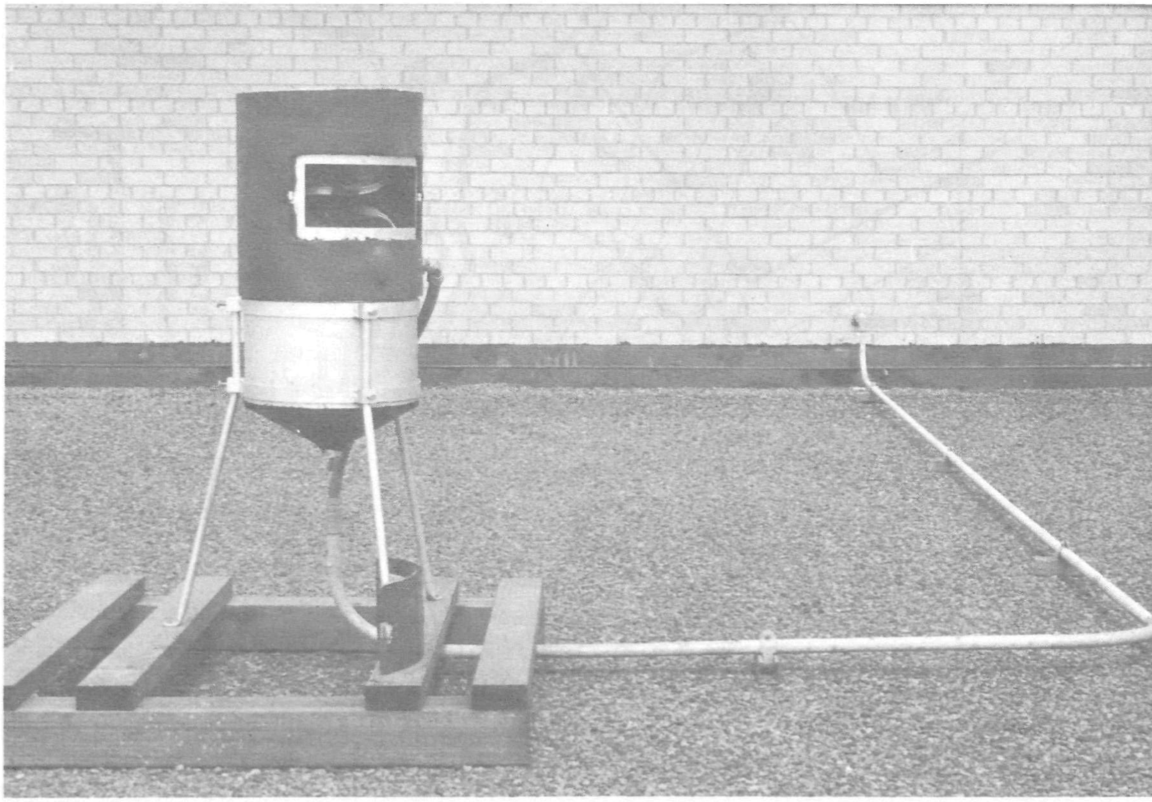


FIGURE 7a - RAIN GAGE INSTALLATION

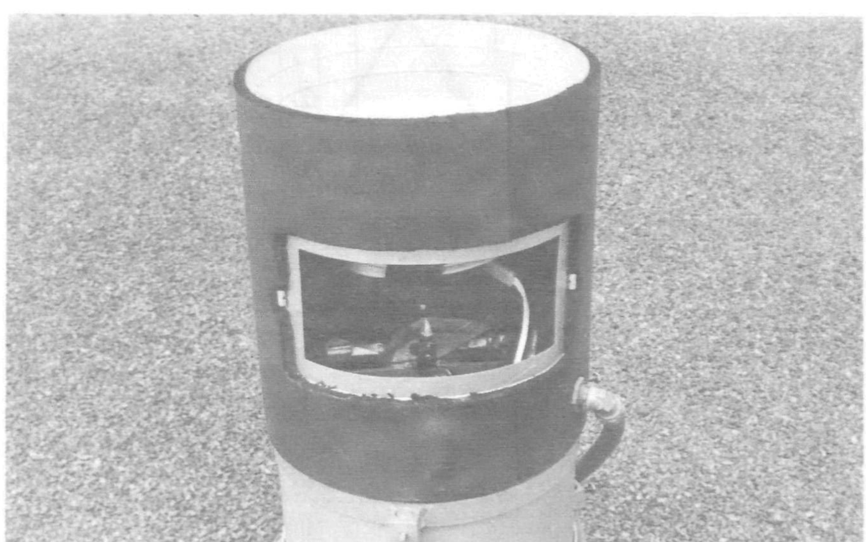
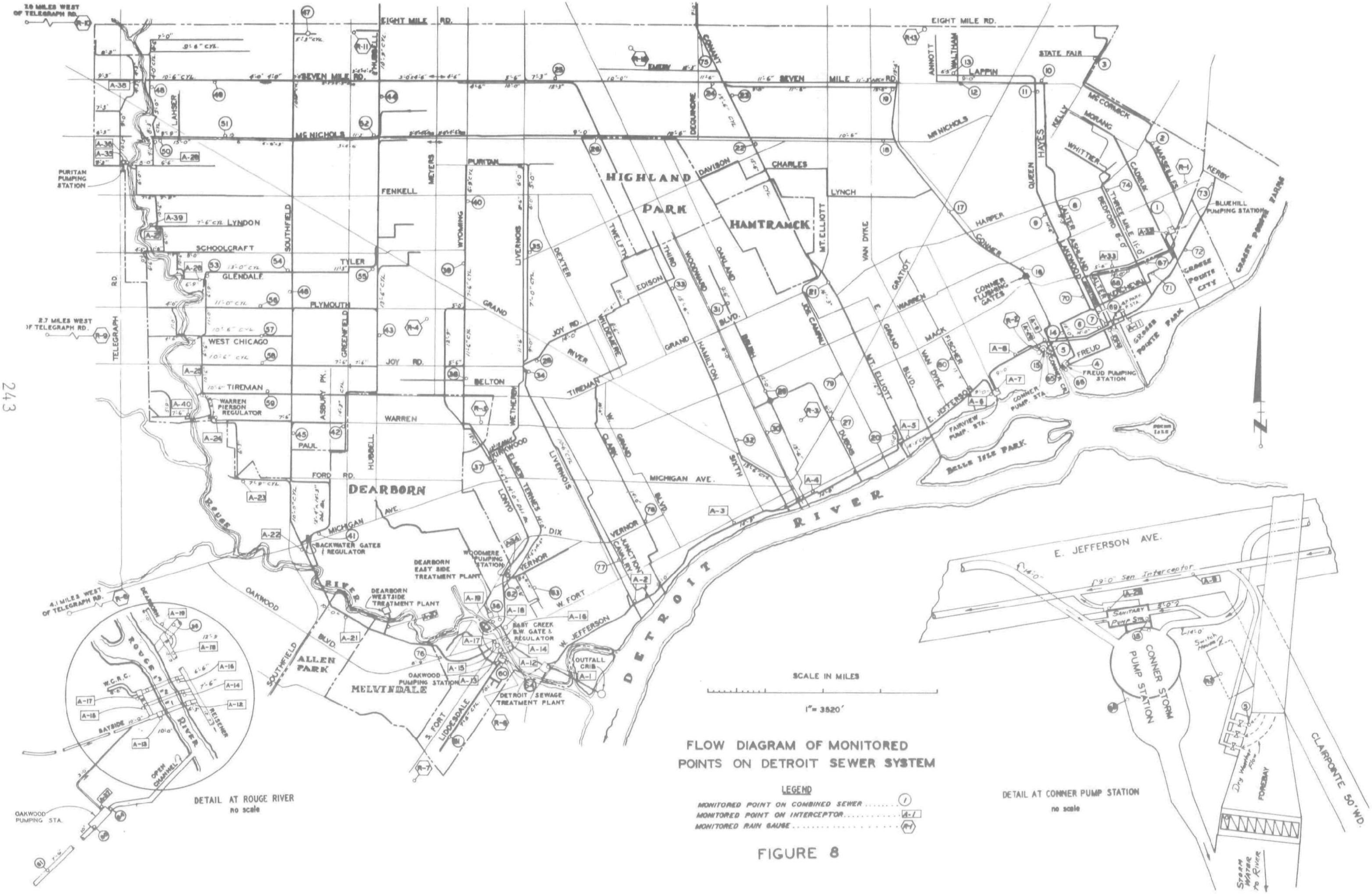


FIGURE 7b - RAIN GAGE MECHANISM



FLOW DIAGRAM OF MONITORED POINTS ON DETROIT SEWER SYSTEM

LEGEND

MONITORED POINT ON COMBINED SEWER 1

MONITORED POINT ON INTERCEPTOR A-1

MONITORED RAIN GAUGE R-1

FIGURE 8

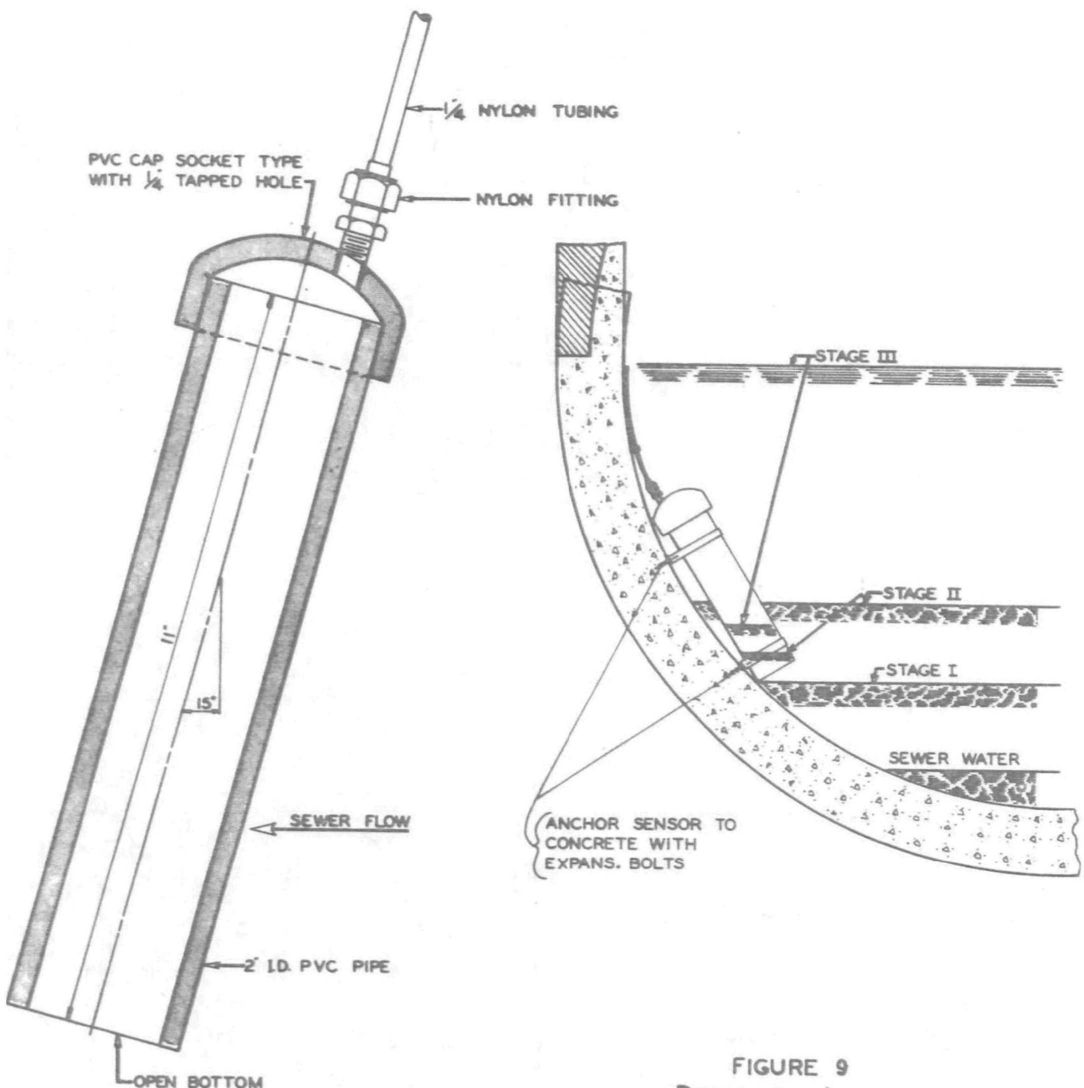


FIGURE 9
DETAIL OF LEVEL
SENSOR
INSTALLATION

These sensors are installed as shown in Figure 10 in existing manholes along sewers and interceptors. The cell is slanted about 15^{o} downstream to avoid being fouled by debris. Near the surface, the nylon tubing is inserted in a 3/4" metallic conduit which has been installed some 3" deep in a cut slot in the asphalt or concrete pavements. The slots were filled with either hot asphalt or epoxy concrete, respectively, to quickly restore the street to service.

Figure 11a shows a typical sensor pedestal located about 3 ft. in back of the curb. Figure 11b shows the power and telephone service drops brought in at the nearest utility pole and Figure 11c shows a typical slot cut in the pavement running from the pedestal to a manhole. Figures 12a and 12b show a typical view of the equipment inside the cabinet on the pedestal.

The equipment consists of the pressure bellows, a signal transmitter, and level indicator. The 1/4" nylon tubing from the manhole plus the power line bringing in 120 V. A.C. current and the telephone leased line all enter the pedestal from underground. Power is purchased on a yearly basis, therefore no meter is required at these installations.

Telemetering Signals

The tone transmitter is a single rectangular box which was shown in the upper left hand of Figure 12b. The signal cycle is 5 seconds in length with the first second being a null tone to clear the receiver.

At the level sensing points the cells are calibrated so that if it had 40 feet of water above it, the transmitter would generate a 4-second analog signal. This 4-second signal is considered to a 100% reading and coupled with the 1 second null tone we have a 5 second cycle. Therefore, if the height of the water was 20 feet the transmitter would send out a 2-second signal which would be equivalent to a 50% reading. The signal would then be 3 seconds long with the first second again being the null tone. The transmitter continuously broadcasts this signal on an assigned frequency. As many as 10 different signals are multiplexed over a single leased telephone line from the field installations into the central computer.

(c) Proximity Sensors

At the locations shown on Figure 13, a sensor block of ferrous metal is attached to the backwater gate and placed in series through a magnetic coupling with the proximity sensor. A continuous discrete signal is transmitted until the gate is opened. When the gate is opened, the circuit is broken and an "open" signal is recorded on the circuit.

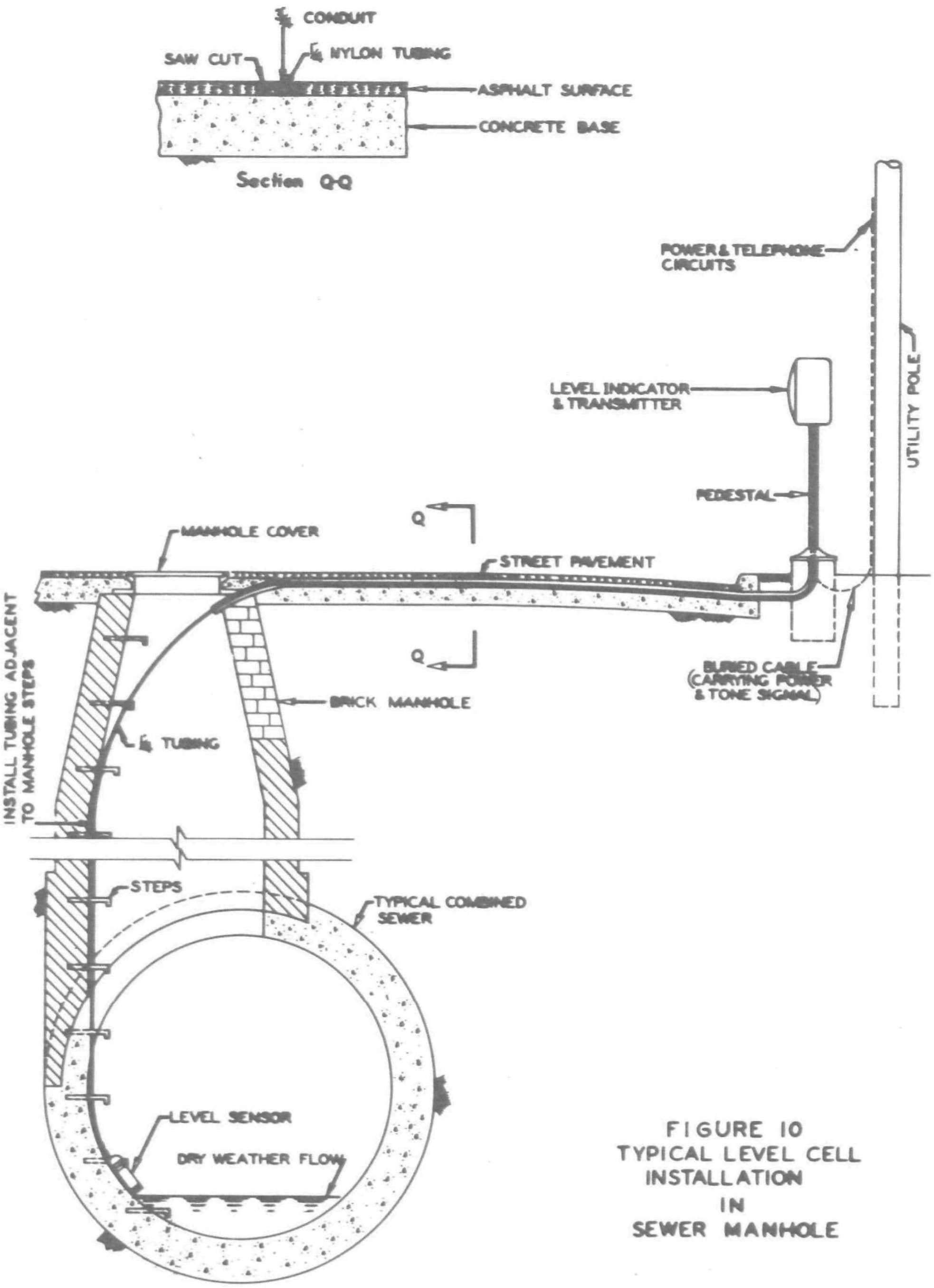


FIGURE 10
TYPICAL LEVEL CELL
INSTALLATION
IN
SEWER MANHOLE

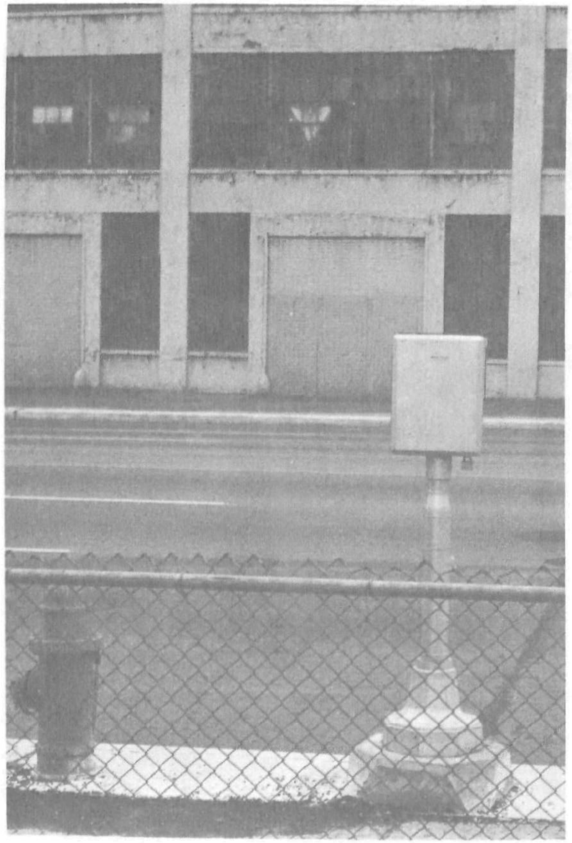


FIGURE 11a - PEDESTAL FOR LEVEL SENSOR

FIGURE 11b - POWER & TELEPHONE SERVICE DROP ON UTILITY POLE

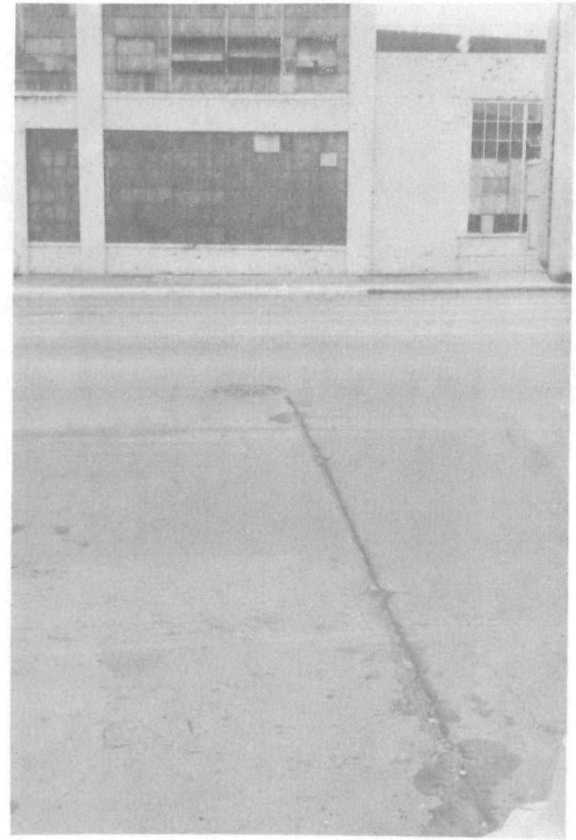


FIGURE 11c - SLOT IN PAVEMENT FOR CONDUIT FROM PEDESTAL TO THE MANHOLE

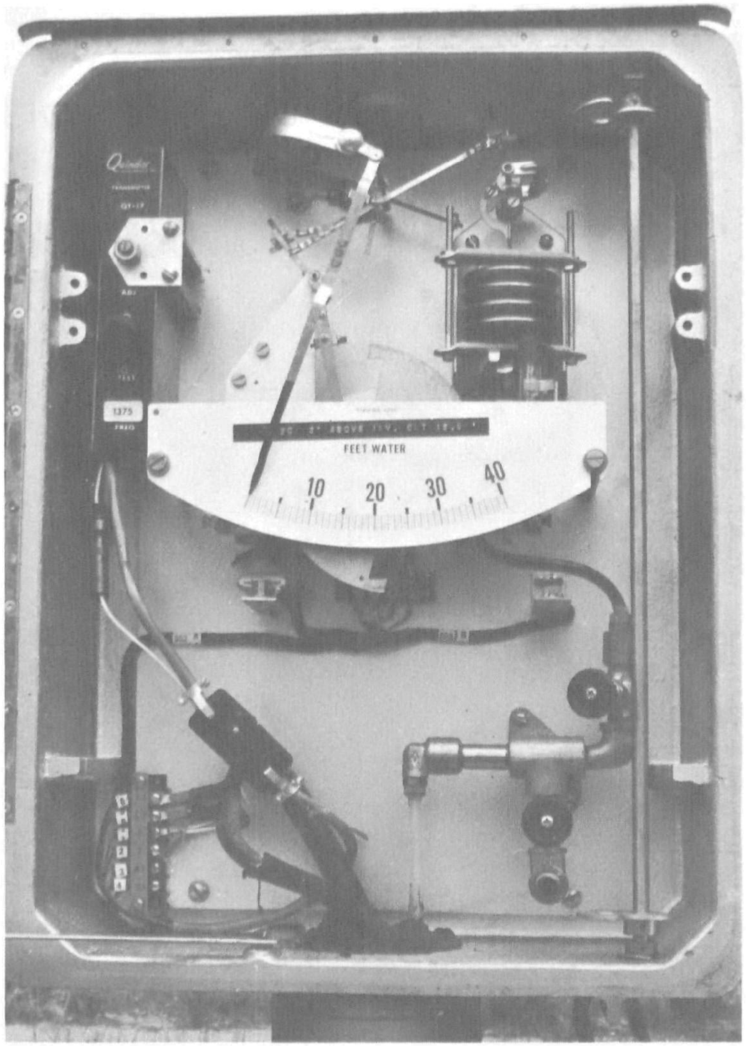


FIGURE 12a

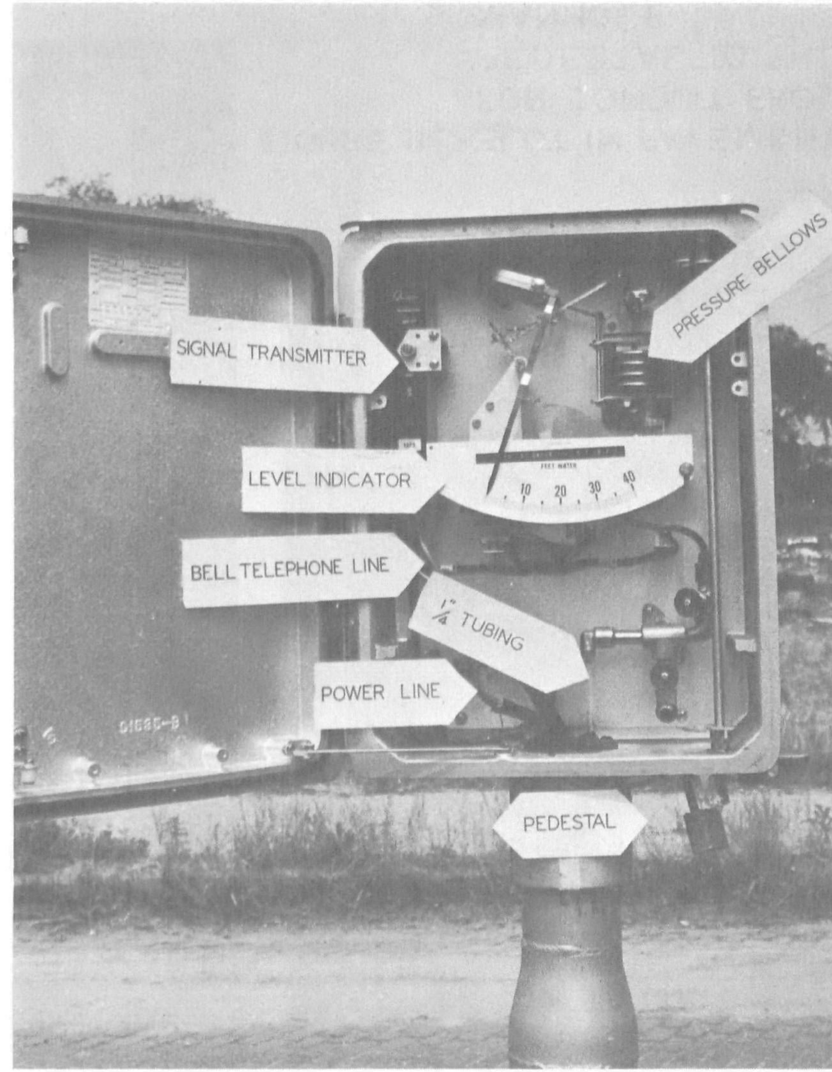


FIGURE 12b

TYPICAL VIEW OF EQUIPMENT INSIDE OF A PEDESTAL CABINET

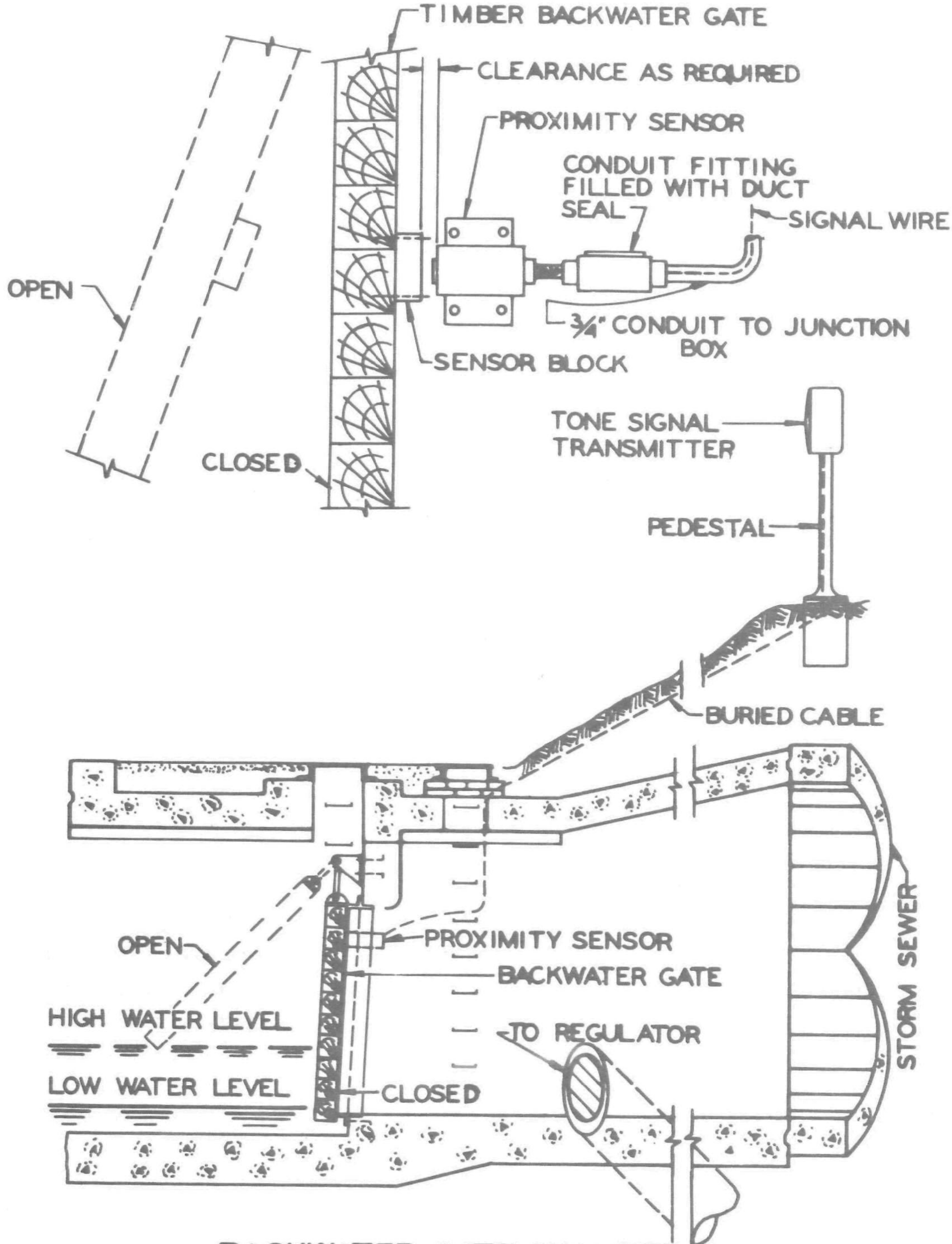


FIGURE 13 - PROXIMITY SENSOR INSTALLATION

(d) Probes

At dams or weirs, as shown on Figure 14, a two-element probe is anchored upstream of the dam and slightly above the dam crest. When both elements are wet, the circuit is completed, the continuous discrete signal is interrupted and an "open" signal is recorded on the circuit.

(e) Digital Computer

The tone receivers are arranged as a battery of interchangeable modules directly adjacent to the computer as shown on Figure 15. There is a receiver for every transmitter, which is tuned to the same frequency as the transmitter. The signal interpreted by the tone receiver is next transmitted through relays to the computer.

Since the tone signals are transmitted on differing frequencies and several signals are multiplexed on one leased phone line, they must be unscrambled by the tuned receiver units.

The computer shown on Figure 16 receives these various signals. This device in turn computes and totalizes rainfall data from the rain gages and depth of flow data from the level sensors.

This is a program type of digital computer with a disc-drum memory. There is space to add three additional memory discs at a later time. It has a present capacity of 36,000 words of memory.

(f) Data-Loggers

The computer actuates a bank of three special long platen electric typewriters as shown on Figure 17.

The sewer levels and status of outfall sensors are tabulated in double space columns. The rainfall data are tabulated in two lines of figures (single space) under the heading for the given gage. The upper figure is the calculated rainfall intensity in inches per hour during the preceding five minutes, with the cumulative total rain in inches tabulated directly below the calculated rate. These data are routinely tabulated every hour during dry weather, but are actuated to 15 minute or 5 minute tabulations either by rainfall of .01" or by higher flow conditions in the combined sewers.

Typical sewer or interceptor levels are tabulated on the typed sheets showing levels, say at points 3, 2 and 1. The typed levels show the actual depth of flow at these sensor points. These tabulated data inform the central control operator that storm flow will be arriving at a pump station or remotely operated regulator. As an example, Figure 19 shows the profile of the Rivard sewer which flows into the Bluehill Storm Pumping Station. With the level information and the profile the operator can visualize the incoming flow and decide on a course of action. The central control office has been furnished a file of such profiles for each main trunk line sewer flowing to a pump station or remotely operated regulator.

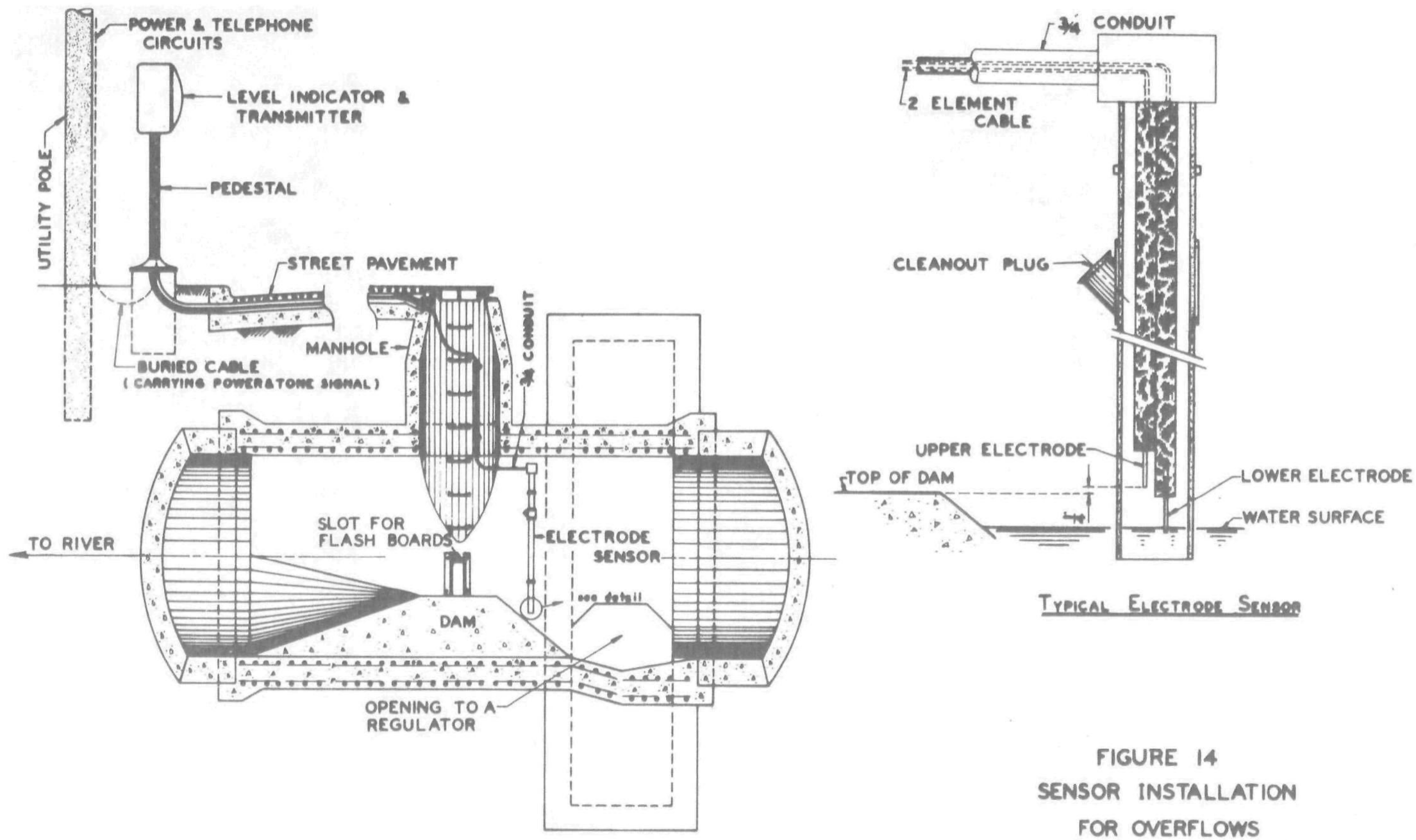


FIGURE 14
SENSOR INSTALLATION
FOR OVERFLOWS
AT
DAMS OR WEIRS.

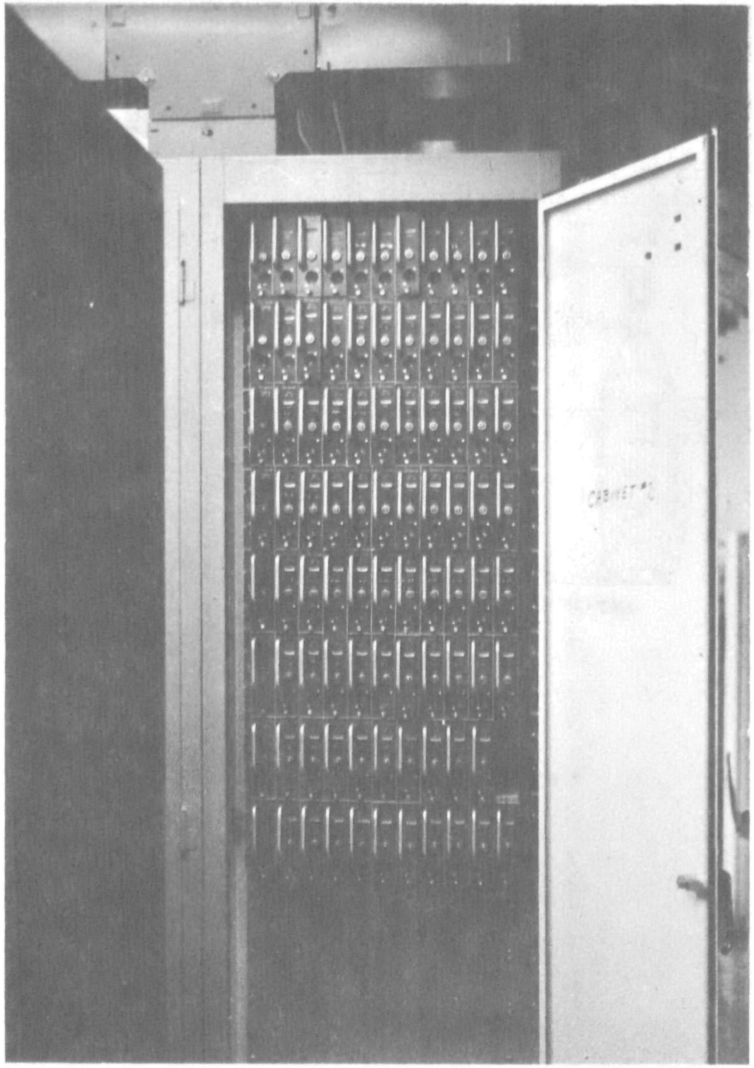


FIGURE 15-TONE SIGNAL RECEIVERS

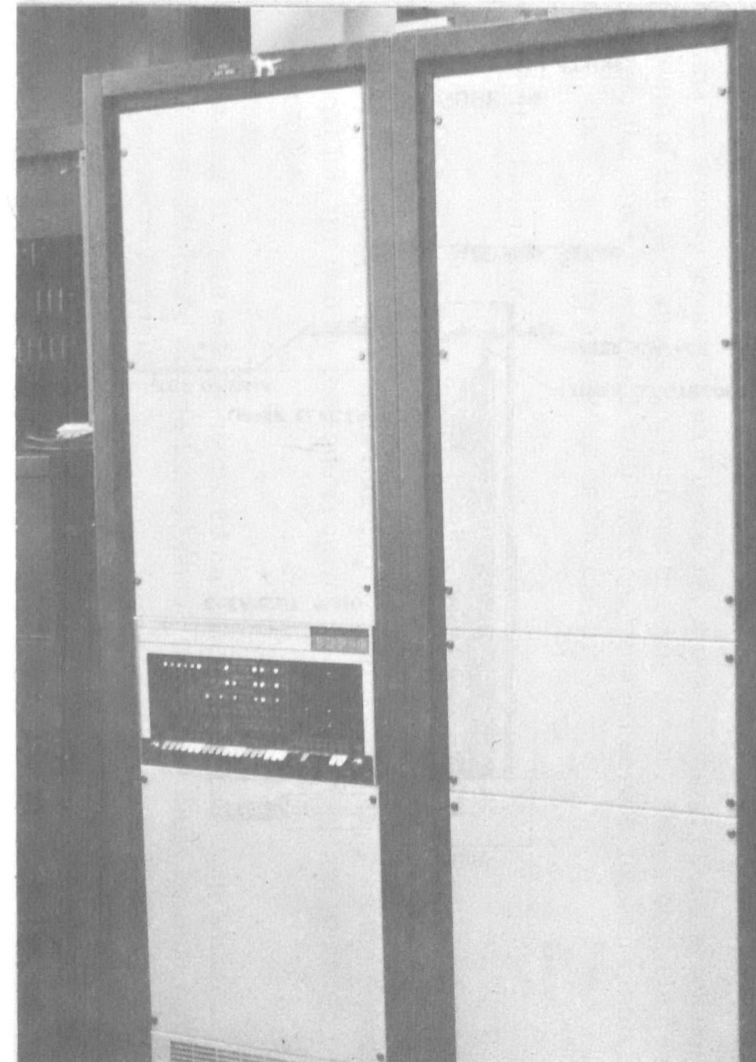


FIGURE 16-DIGITAL COMPUTER

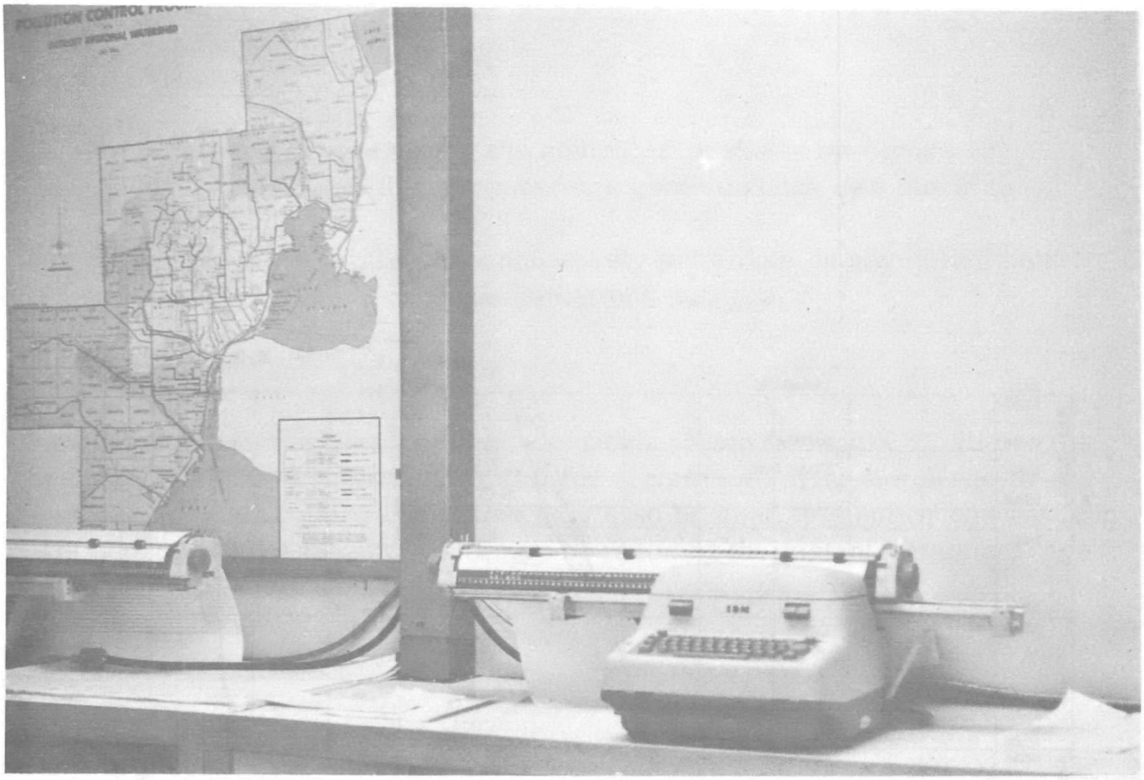


FIGURE 17- TYPEWRITER FOR DATA LOGGING

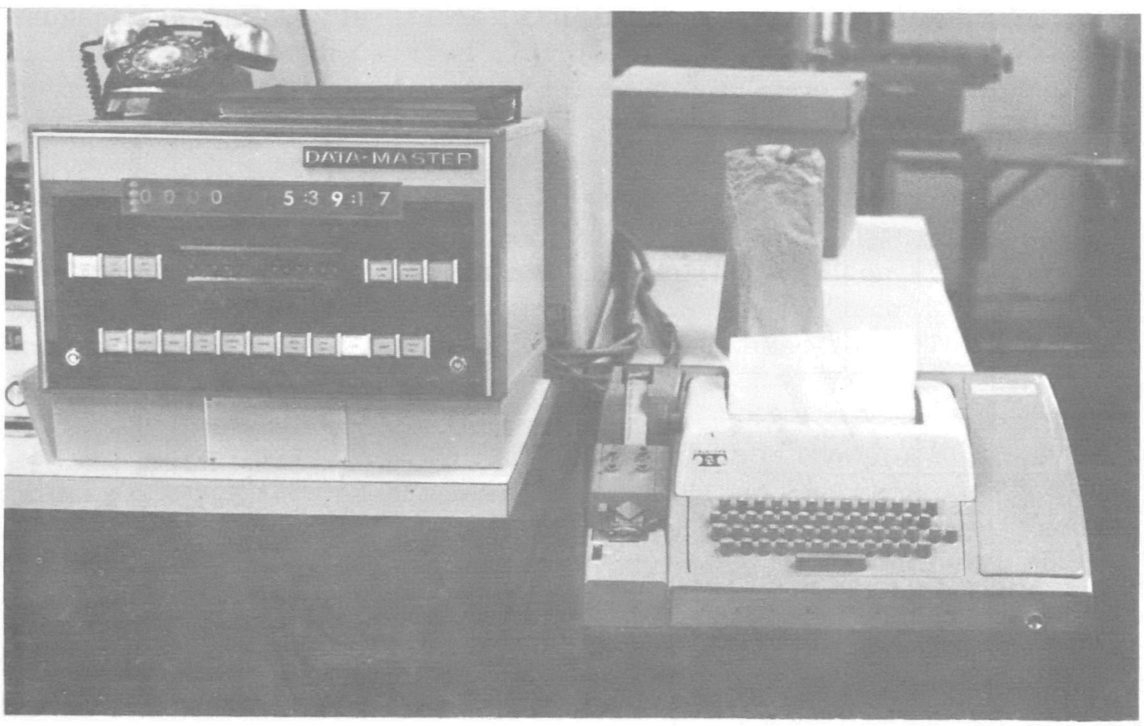


FIGURE 18- MONITOR AND TELETYPEWRITER

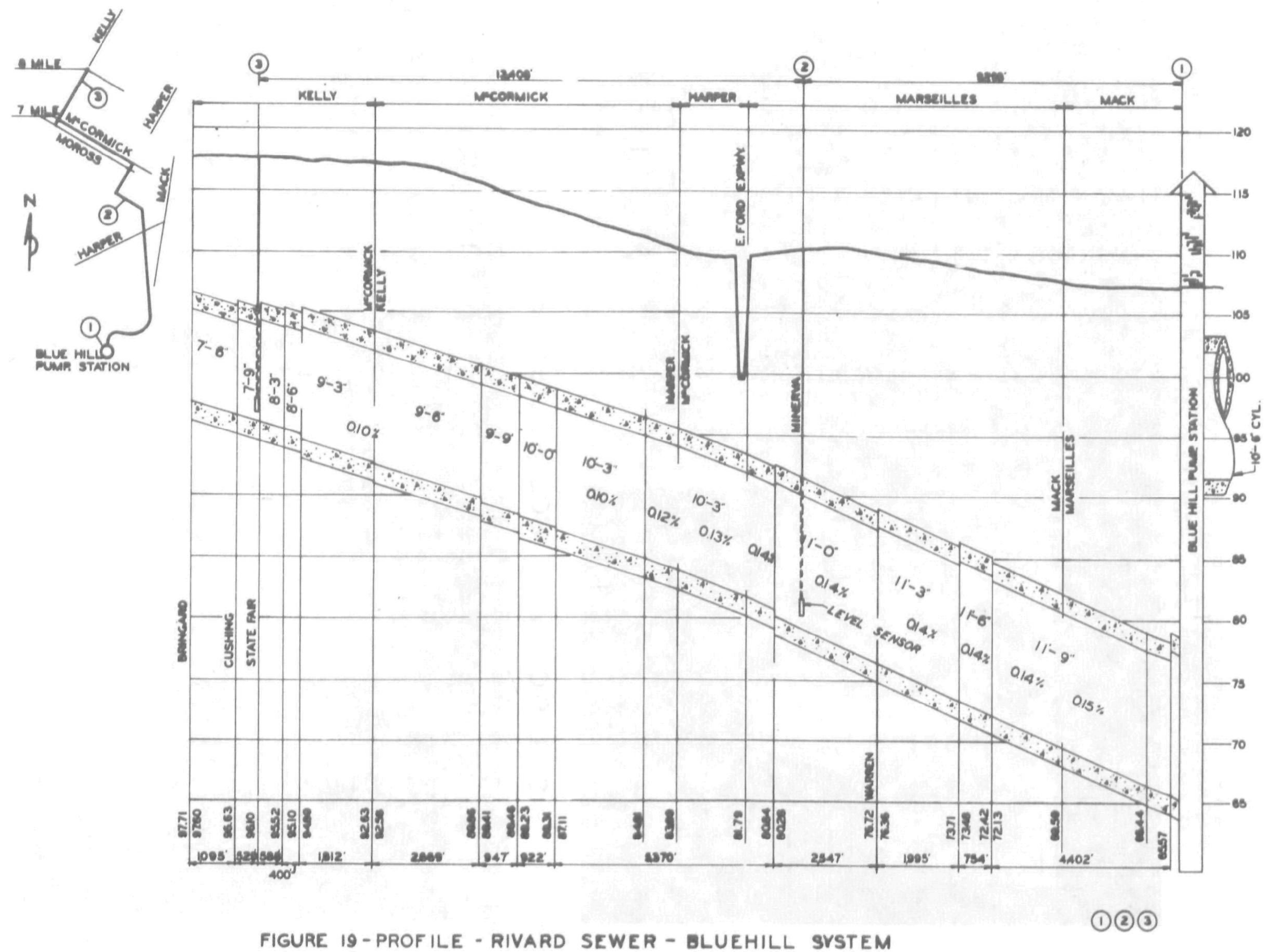


FIGURE 19 - PROFILE - RIVARD SEWER - BLUEHILL SYSTEM

(g) Teletypewriter

The typewriter for programming the computer is shown on Figure 18. Changes of instruction to the computer are given through this machine.

This teletypewriter will also instantaneously print data on any called point, high and low level alarms or communications outages.

(h) Operator Console and Monitor

The operator console which is also a monitor shown in Figure 18 allows the personnel at the control center to call for a complete print out on the data logger at any time. This equipment may also be used to monitor one specific level sensor so that observations may be made of the instantaneous changes taking place between data logger printouts.

(i) Central Control Panel

All present operation of the sewerage system pumps and valves is manual from the control center switchboard panel as shown on Figure 20.

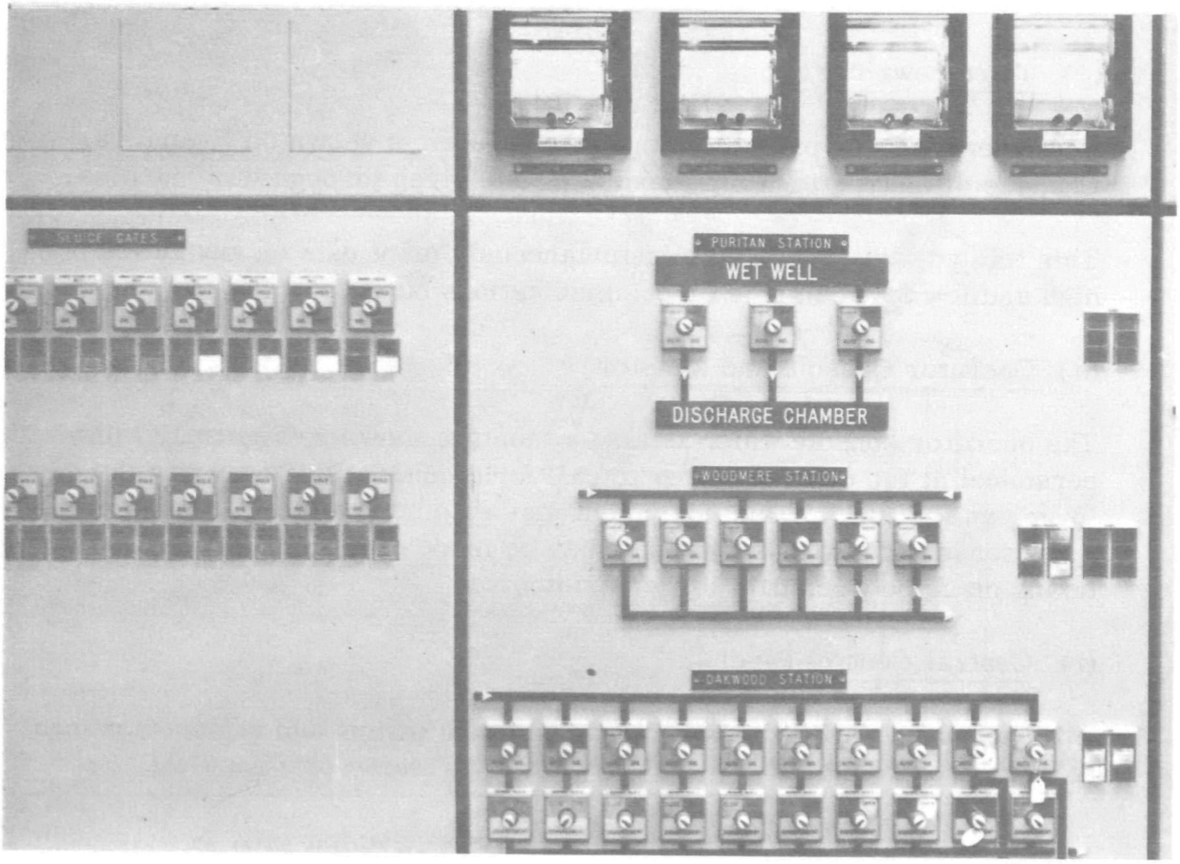
Experience and parameters are being developed which will, in time permit automated operation of the pumps. One program would control storm operation, another would be for dry weather conditions and a third program would control in-system storage.

A flow diagram is superimposed on the board to aid the operators in controlling pumps and valves. Directly above each pump control is a recorder showing the recorded elevation of the wet well and the discharge level of each station. These telemetered elevations are furnished by a completely independent system of transmitters and receivers, therefore the recorders continue to function even if the computer is temporarily out of service. This allows the operators to act in the event of an emergency.

The central portion of the pump control panel which is shown in Figure 20 allows the remote control of a sanitary pump station, a storm station and also monitors which pumps are operating at the wastewater plant. Adjacent and to the left of the pump control switchboard is the switchboard which controls the regulating gates and flushing gates. The operation of these gates is controlled from the central office based on the telemetered information which comes in on the three computer operated typewriters.

Remotely Operated Pumping Stations

Figure 21 shows the inside of one of the remote pumping stations. The conversion involved only the addition of relays and some minor electrical work. There is a roving maintenance man who visits the remotely operated stations daily when they are not pumping and once every shift during periods of pumpage.



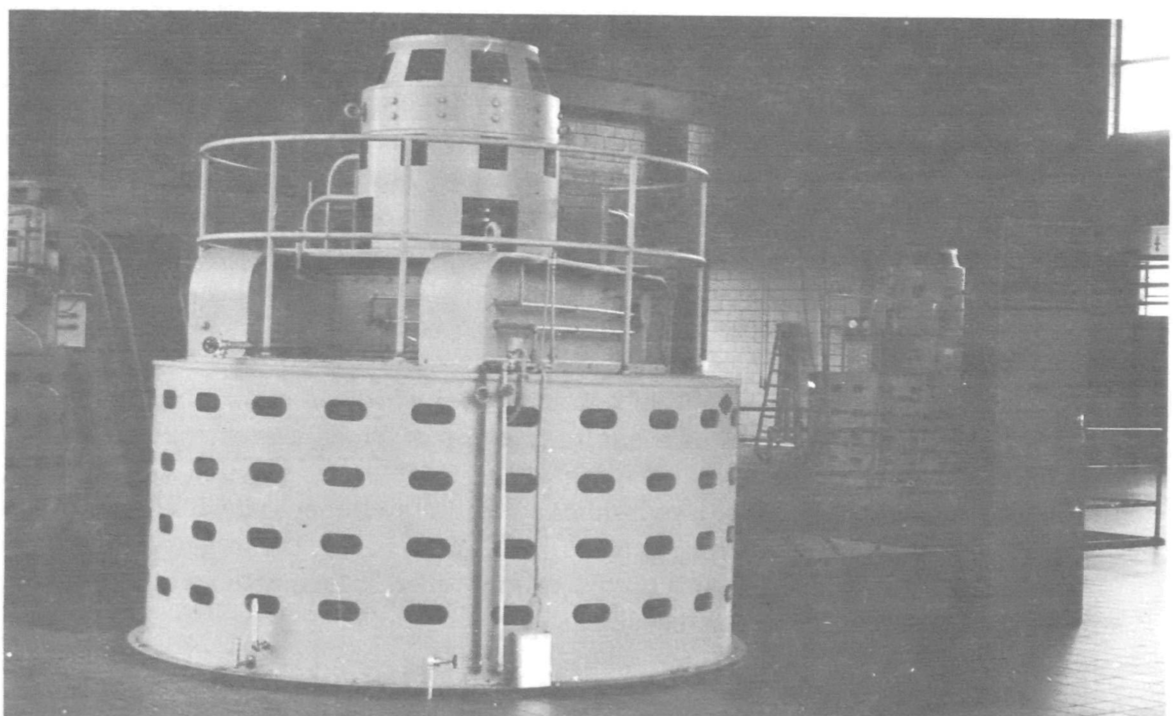


FIGURE 21-FREUD STORM PUMPING STATION

Seven storm and/or sanitary pumping stations are operated remotely from the central control panel. The pumps of one stormwater station and the wastewater plant are not remotely operated but are monitored by the central control office.

There is no early plan to operate the Conner Storm Pumping Station by remote control as this station is over 40 years old. The pump impellers are raised and require manual priming. Remote operation will undoubtedly be considered at a later date. Operation of the pumps at the Wastewater Plant and Conners Storm Station are coordinated with the other stations in the system by telecommunication from central control with the operators on duty at these locations.

Remote Operated Gates

In order to selectively load the interceptor system, it was desirable to remotely operate regulators at Warren-Pierson, at Michigan-Southfield, and at Baby Creek. The existing sluice gates at these locations are being modified, to accept motor operators and the work should be complete by the spring of 1970. These locations were shown on Figure 8. These three points regulate flow from approximately 37% of the City.

The installation of flushing gates at the 3 barrel Conner sewer is discussed later.

At the Conner's Station, provisions will be made to remotely operate the regulating gates which control the flow entering the interceptor from the Conner's Gravity Sewer.

SECTION III

OPERATION OF THE SYSTEM

The basic concept of Monitoring and Remote Control is that as a storm moves across the City from west to east (the usual route), the operators at the Central Control Center begin to act in accordance with the data received to apply the pollution control techniques that were mentioned earlier. (The interceptor may be well pumped down to accept more flow, a small storm may be retained, storage may be effected within the pumped system, and the first flush of a storm may be better retained. All of these steps would be centrally controlled.) This concept of actually operating a combined sewerage system is snow possible since the above described equipment provides not only the data required to make decisions but also provides the means by which affirmative action can be taken to better utilize the full potential of the available storage capacity. It is no longer necessary to build a system and leave it buried underground, more or less forgotten, until problems arise from overloading, sludge build up, or flooding.

The following discussion is based on the limited experience that DMWD has had through 1969 with the components of the system that are in operation. The remote controlled sluice gates have not been placed in service at this time (January 1970) and therefore that portion of the discussion will be theoretical. Although the primary aim of this project is the reduction of combined overflows it must be kept in mind that this goal must be accomplished in such a manner as to avoid endangering the health and property of the local citizens. The use of basements as detention basins is not a justifiable alternative to preventing the occurence of a combined sewer overflow. Therefore, the discussion of the different safety factors and back-up systems of this project should be of interest to all readers.

ANTICIPATING MAJOR STORMS

The network of 14 rain gages gives the ability to anticipate the impact of a major storm. The four western gages are from 3 to 7 miles west of the City of Detroit city limits since a majority of storms come from this direction. This gives the central control operators from 3 hours to 6 hours of lead time to have the system pumped down in order to store a small storm (by taking advantage of storm travel time and the running time in the barrel).

During May and June of 1969, the control center operators began utilizing these rainfall data to pump down the levels in interceptors and selected storm barrels to determine how much storm overflow they could prevent. The gate monitors were already in operation at this time. The operators found that they were able to entirely contain certain spot storms, plus holding several scattered 1/4" to 1/2" city wide rains on various occasions. It appears that the concept of "anticipating" a rain is easily understood by the operators and in-line storage can be accomplished.

When storing water in the low level barrels and in the wet wells, there is a hazard to the storm system in the event of a sudden intense storm.

This possibility is illustrated on Figure 22 which shows a condition which can occur at several of the storm pumping stations. The flow from an intense storm tends to suddenly fill the barrel with the higher level flow, which travels faster than lower level flow, thus overtaking the earlier part of the storm. V_5 travels faster than V_4 and V_4 travels faster than V_3, and so on. Thus one can have a sudden wall of water, or translatory wave, as shown in Stage III arriving at the pump station, which without monitoring, would not be expected at the station and could cause extensive flooding. In certain cases, this phenomenon could lift manholes in the street with the surge pressures. The monitoring system should give the central control office enough lead time to enable the operators on duty to anticipate this water. The operators can then be prepared to turn on an adequate number of pumps when the water arrives. (Pumps cannot be placed in operation until the starting water level is reached in the wet well of the station. Severe damage can occur if the pumps are started in a dry situation.)

ANTICIPATING SMALL STORMS

In order to safely practice storm storage in the sewer barrels, it is necessary to determine the correlation between the various storm intensities and the recorded downstream stormflow. Thiessen's polygons, which were shown earlier in Figure 6, are being used by DMWD rainfall analysts to establish the relationships. From precipitation and flow data, the sewer hydrographs of the maximum storm that can be stored in the various combined systems are being developed for each area. It is quite obvious that each sewer system will have a different storage capacity depending on imperviousness of the tributary area, sizes of storage barrels, depth of storage in the trunk line, slope of the barrel, depth of tributary arms, depths of basements and other factors.

For any program of planned storage, it is assumed that the wastewater plant will be operating to capacity for a considerable time prior to, during and after a storm.

SMALL STORM STORAGE

The in line storage of small storms within the barrel of the existing sewers is dependent upon the following factors:

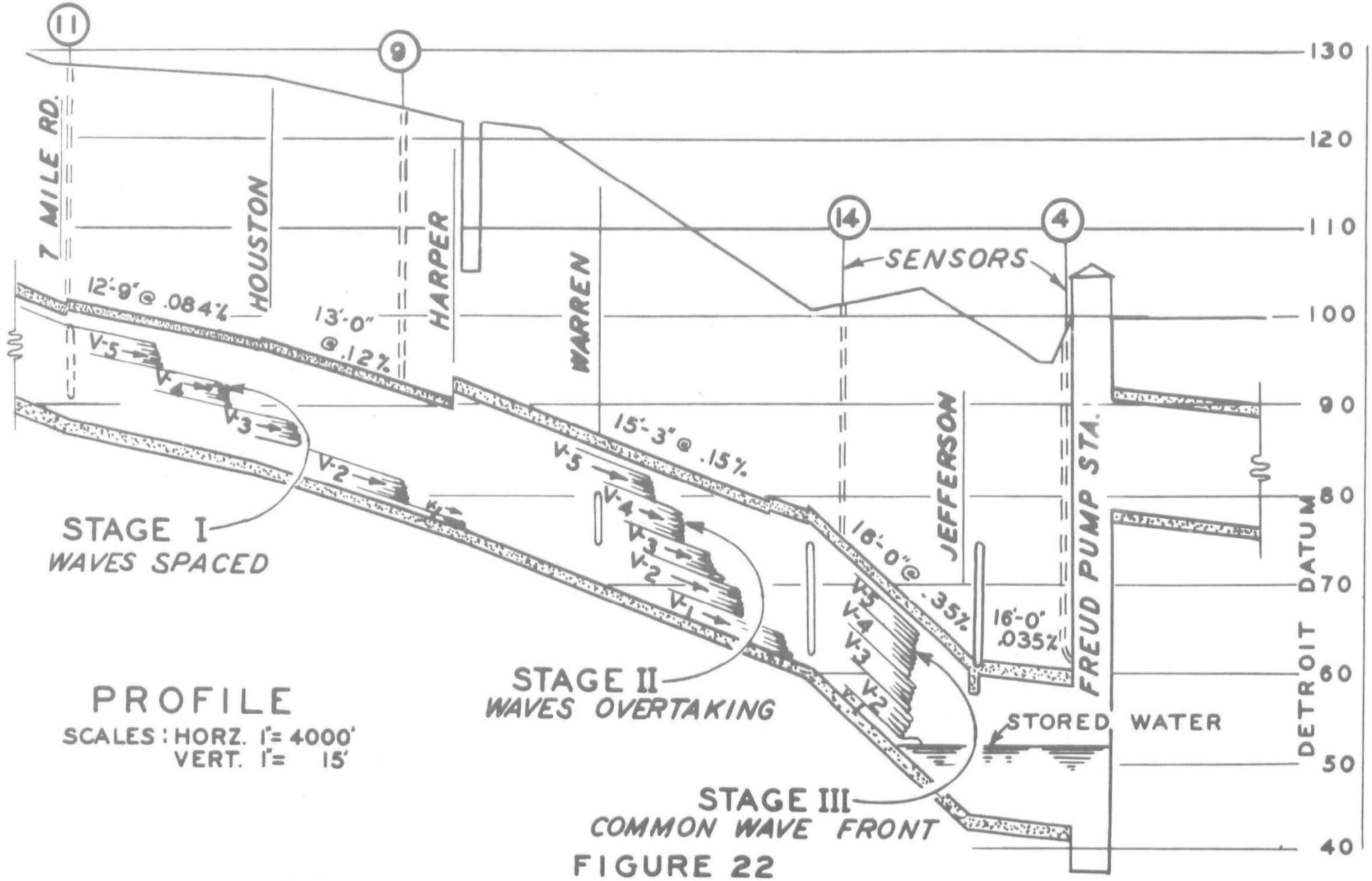


FIGURE 22
TRANSLATORY WAVE IN STORM SEWER
DURING INTENSE STORMS
ASHLAND-ALGONQUIN STORM SYSTEM

1. Size of box or cylinder
2. Slope of the conduit
3. Imperviousness of tributary area
4. Time elapsed since previous rain
5. Available height in sewer before gates open
6. Intensity or length of storm
7. The level of the river
8. Available capacity in the interceptor

Figure 23 shows a typical arrangement at an outfall which illustrates certain of these factors. The depth of water (15 ft.), the flat grade, the height the water must be raised before gates open, and the effect of added storage if the river level is raised even one foot should be noted. At the time the backwater gate opens, there is a depth of 15.01 feet of water in the outfall with a total quantity of stored water of 6.3 million cu. ft. in this outfall (assuming dead level conditions). The water surface actually is not quite level but curves as a backwater curve so there is somewhat more water upstream of the gate, depending on the amount being diverted into the interceptor. In these situations the backwater gates are the safety valves of the storage operation which could open before there is danger of basement flooding. Available storage at the various outfalls either upstream of pumps or backwater gates are being calculated and tabulated for use by the system control operators.

STORAGE IN SEWERS - vs. - SEDIMENTATION

Any storage of runoff in larger trunk line sewers results in reduced velocity. Velocities below 2 ft. per second usually cause graded sedimentation with coarse deposits occurring upstream where the velocities are still relatively high and finer deposits downstream where the velocities are still relatively high and finer deposits downstream where the velocities approach zero.

DETROIT EXPERIENCE WITH SEWER DEPOSITS

Initially, storage is to be attempted at:

(1) The Hubbell-Southfield Outfall
a double box with each box 14'-6" wide x 12'

(2) The Baby Creek Outfall
a triple box each box being 14'-6" wide x 17'-6"

(3) The Conner Gravity Outfall
a triple box with each box 15'-9" wide x 17'-6"

(4) The five storm pumping stations

Detroit has had the following experience with sewer deposits:

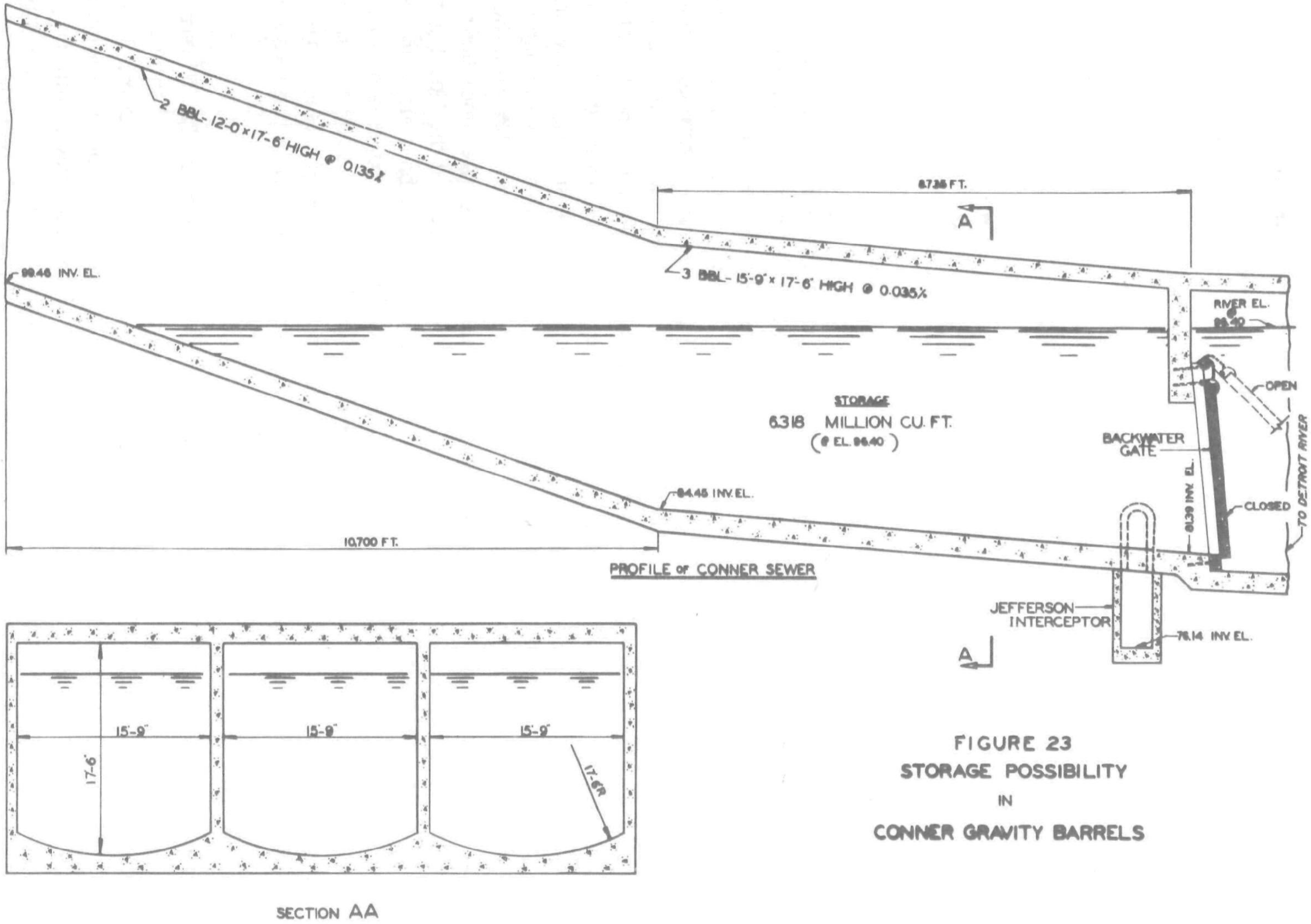


FIGURE 23
STORAGE POSSIBILITY
IN
CONNER GRAVITY BARRELS

About ten years ago the Baby Creek barrels and the Conner barrels were used to store dry weather flow in order to moderate the variations in the hydraulic load on the treatment plant. Sedimentation occurred in both the Conner and Baby Creek barrels. About four years ago, when the treatment plant began to operate the system at a lower hydraulic gradient, the deposits in the Baby Creek were gradually moved to the treatment plant. Some initial problems of overloading the grit chamber and breaking flights in the primary tanks did result. The sediment deposits in the Conner barrels were not dislodged by the lower gradient and the deposits still exist in the Conner barrel in substantial amounts.

Therefore to take these expected sedimentary deposits into account, storage in gravity sewers is being initially attempted only at locations where there are double or triple barrels. This will give a built-in safety factor and also will allow any required flushing to be accomplished by diverting flows.

Figure 24 shows the arrangement at the Conner sewer barrels, where flushing gates will be installed at the point where the sewer changes from two barrels to three barrels. This is the point where the deposits have not been moved along by the lower hydraulic gradients at the wastewater plant. Slots and guides have been installed in the transition chamber to permit lowering and raising cable supported 7-ft. high gates in the 17-ft., 6-inch high combined sewer barrels. A level sensor has been installed in a manhole located about 300 ft. upstream of the flushing gates. After a storm has occurred (during which time the gates are up) and the level in the barrel is showing below the 7-ft. level on the sensor, two of the 7-ft. gates are lowered (say the number 3 and number 2 gates) as shown in Figure 24 which forces all flow into the No. 1 barrel and will flush this barrel. Correspondingly, after four hours of flushing of the east barrel, the No. 3 gate can be raised, and the No. 1 gate lowered in order to flush the opposite west barrel. After about eight hours of flushi ng, all gates are raised in order to be ready for the next storm. The flushing at present is deliberately shortened in order not to overload the grit chamber at the treatment plant. After all deposits have been cleaned from all barrels, it will be possible to routinely flush each barrel in regular sequence by a predetermined program.

There are two reasons for using only 7 foot high gates in these 17'-6" barrels. The first was dictated by the available space between the crown of the sewer and the street surface. The second reason is again the safety factor. If for some reason the electronic remote control equipment should become inoperable when the gates are in a down position it is possible for the flow to top the gates without causing flooding upstream. A secondary system for raising the gates utilizing power take-off equipment from a truck is also being installed.

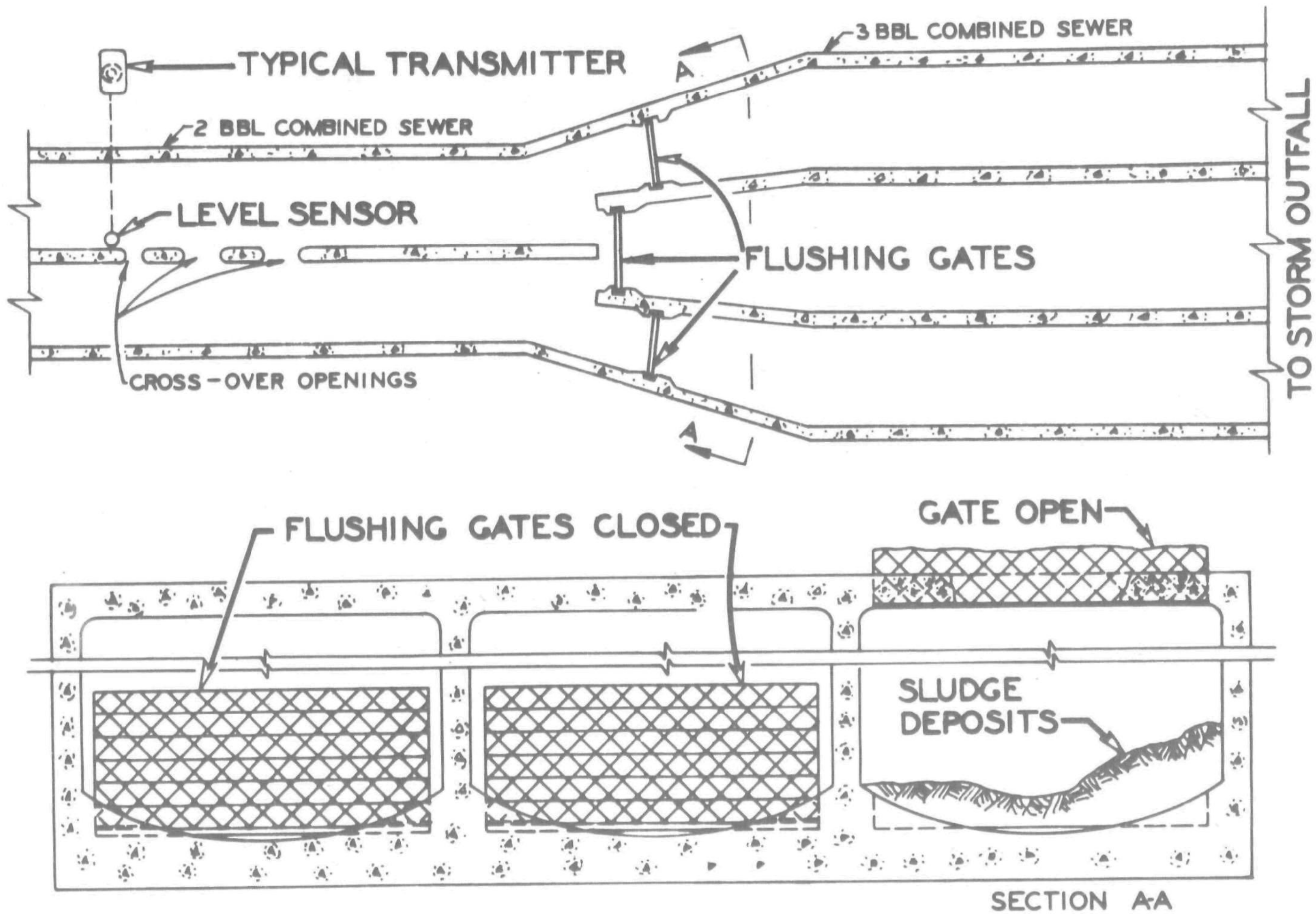


FIGURE 24- FLUSHING INSTALLATION

By using the "storm flow anticipation" technique coupled with in line storage we feel that "first flush interception" is possible especially with the smaller storm events and in the case of some scattered spot storms. Before discussing the ideas of "selective retention" and "selective overflowing" it is first necessary to have some background on the quality and quantity of the overflows.

QUALITY OF OVERFLOW

The sanitary engineer and governmental agencies must establish criteria for overflow quality. The pollutants that each sewer line carries vary from outfall to outfall. These variations are rather wide. The system operator must discriminate between outfalls in order to secure the greatest benefit to the river.

Figures 25 and 26 graphically show two sets of the averaged variations in B.O.D. which occur along certain trunk line sewers, as revealed by our program of manual sampling. These data are tabulated in Tables I-A, I-B, 2-A and 2-B located in APPENDIX I. For example, it is obvious that some consideration must be given to intercepting as large a portion as possible of the high B.O.D. in the Orleans sewer during a storm.

Figures 27 and 28 show two sets of the averaged variations in the oils and greases from samples collected in 1968 and 1969 from the trunk sewers. There are rather striking variations from sewer to sewer. Again it is obvious that special consideration should be given the greases in the Orleans, the Baby Creek and Flora sewers.

Figures 29 and 30 shows two sets of averaged variations of suspended solid from the same samples. It can be seen from these figures that there are many lines with a high concentration of solids and one would be hard pressed to single out any particular line for special attention.

The tables also give data on the averaged variations of phenols and total phosphate. It can be seen that two lines, Baby Creek and Dearborn (Miller Road) carry a high concentration of phenols while the phosphate concentrations are fairly uniform throughout the system.

These charts represent variations during dry weather flow. The mass diagrams have not been adjusted as to quantity of flow and therefore, these are concentrations. Corresponding data on storm overflow will be collected later.

QUANTITY OF OVERFLOWS

DMWD has developed a program for a computer which is independent of the one used in the monitoring system. This program will determine the actual volume of a combined overflow, in cubic feet, that is discharged from each major outfall at the time of a storm overflow. The integration is rather complicated under either open channel or full conduit conditions. The sensor points on backwater gates and dams provide the elapsed time the outfall is spilling.

FIGURE 25

CITY OF DETROIT SEWER SYSTEM

MAP

SHOWING VARIATION OF UNIT AMOUNTS

OF

BIOCHEMICAL OXYGEN DEMAND

FOUND IN DRYWEATHER FLOW AT THE OUTFALLS SHOWN.

JUNE-DEC.'68

FIGURE 26

CITY OF DETROIT SEWER SYSTEM

MAP

SHOWING VARIATION OF UNIT AMOUNTS

OF

BIOCHEMICAL OXYGEN DEMAND

FOUND IN DRYWEATHER FLOW AT THE OUTFALLS SHOWN.

JAN.–JULY '69"

FIGURE 27

CITY OF DETROIT SEWER SYSTEM

MAP

SHOWING VARIATION OF UNIT AMOUNTS

OF

OIL AND GREASES

FOUND IN DRYWEATHER FLOW AT THE
OUTFALLS SHOWN.
JUNE - DEC. '68

FIGURE 28

CITY OF DETROIT SEWER SYSTEM

MAP

SHOWING VARIATION OF UNIT AMOUNTS

OF

OIL AND GREASES

FOUND IN DRYWEATHER FLOW AT THE
OUTFALLS SHOWN.
JAN-JULY '69

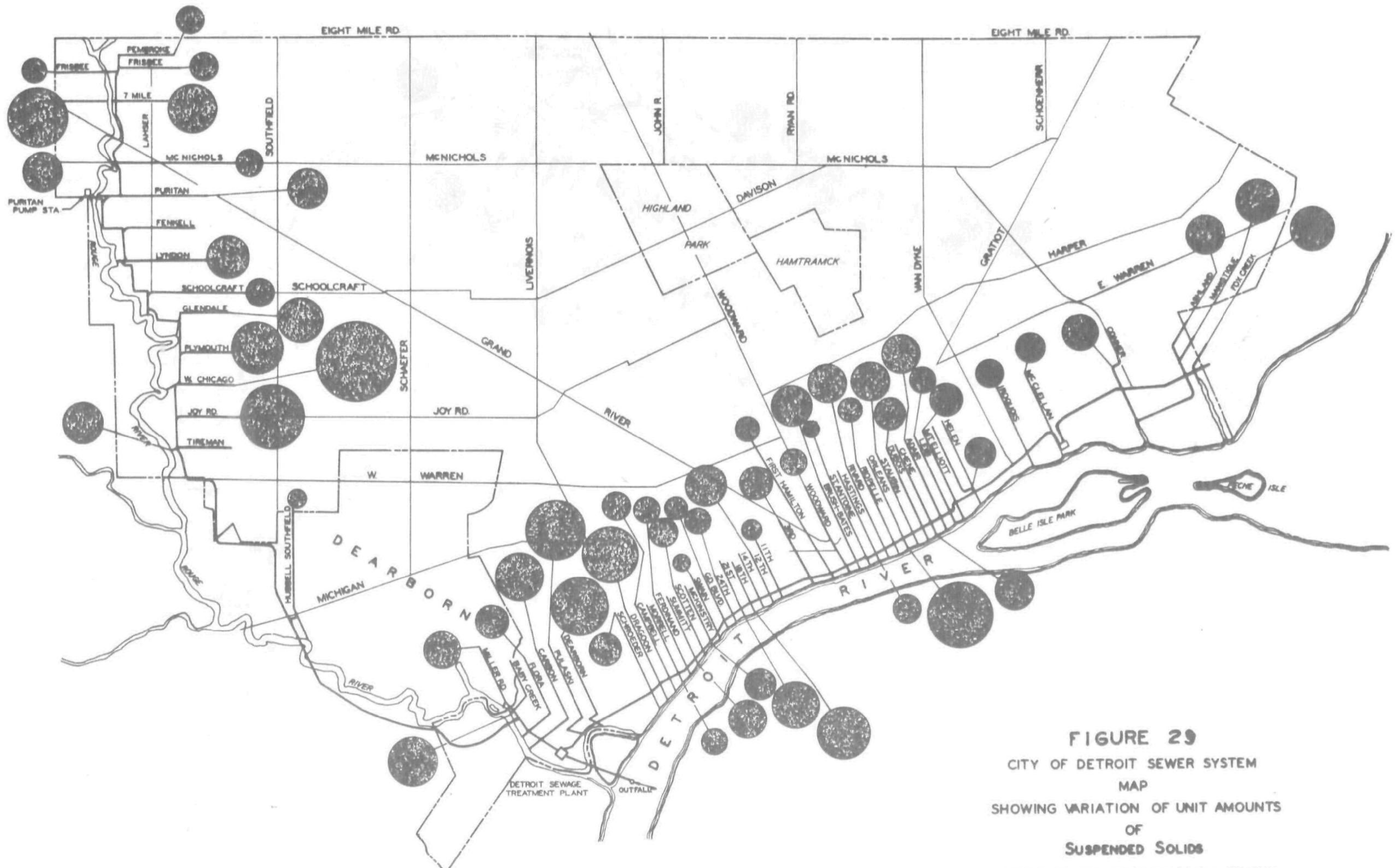


FIGURE 29
CITY OF DETROIT SEWER SYSTEM
MAP
SHOWING VARIATION OF UNIT AMOUNTS
OF
SUSPENDED SOLIDS
FOUND IN DRYWEATHER FLOW AT THE
OUTFALLS SHOWN.
JUNE DEC. '66'

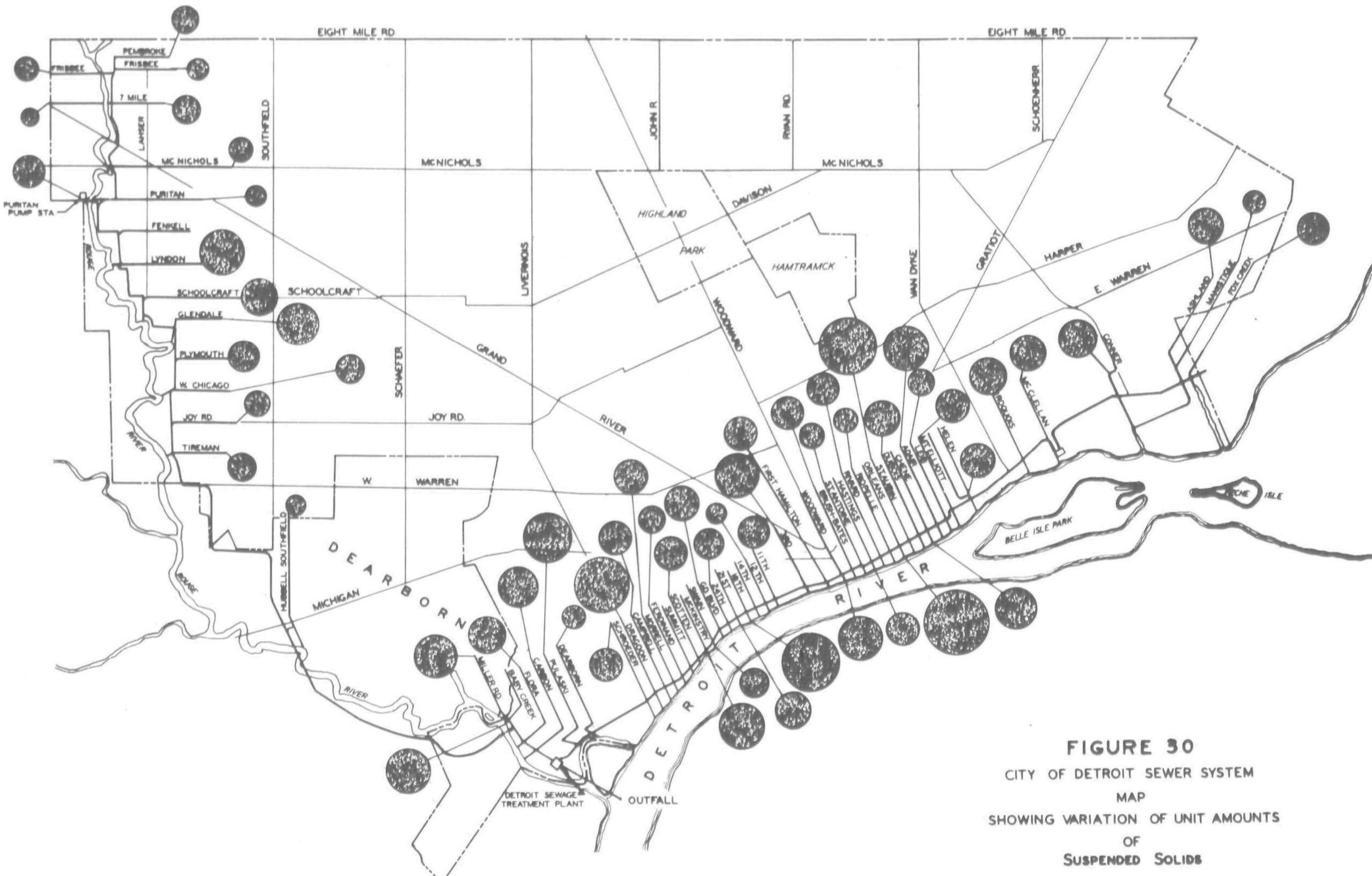


FIGURE 30

CITY OF DETROIT SEWER SYSTEM

MAP

SHOWING VARIATION OF UNIT AMOUNTS

OF

SUSPENDED SOLIDS

FOUND IN DRYWEATHER FLOW AT THE

OUTFALLS SHOWN.

JAN.- JULY '69"

The upstream sewer level sensors give the varying hydraulic gradients above the outfall. From these data, the total discharge is integrated to determine the total overflow to the river.

The present level sensors on 25 of the larger outfalls will permit calculation of the runoff from 86% of the area of the city. Measurement of the flow from the balance of the smaller outfalls has been deferred because of the capital cost for equipment. However, some very reasonable estimates of the overflow can be secured since elapsed time of spilling is known, plus average runoff per square mile from other comparable areas.

"SELECTIVE RETENTION" AND "SELECTIVE OVERFLOWING"

The preceding charts and the accompanying tables reflect a high level of pollutants in sewers from industrial areas, slaughterhouses, laundries, refineries or breweries. By contrast, sewers draining large parking lots, parks or residential areas carry relatively low quantities of these same pollutants. The levels of pollution will be expected to vary during each period of overflow. To secure the greatest benefit to the river, consideration must also be given to the duration of overflow as well as the major type of pollutant being spilled. By developing criteria for each outfall the schedule would call for allowing a higher percentage from the least polluted outfall to overflow to the river in order to route the more polluted flow to the treatment plant.

At present with three remote control sluice gates and the five storm pumping stations it will be possible to begin using "selective retention" and "selective overflow".

During storm events after the operators have learned the "storm flow anticipation" technique they then can close down the east side pumping stations allowing storage to take place in the wet wells while leaving interceptor capacity for the westerly sewers. As the storm moves across the City the remote sluice gates in the west can be closed causing storage to begin in the westerly sewers while the interceptor is then utilized to carry flow from the east.

In some storm events this may cause overflows to occur but it is expected that much of the gross pollution can be entrapped for treatment. In the future as more regulators are set up for remote operation these techniques could be expanded.

SAMPLING

Under the present manual sampling program, grab samples collected during a regular work week are refrigerated and delivered to the wastewater plant once daily for standard analysis. One automatic mechanical sampler has been installed for around the clock collection and several others are to be added later. These and supplemental samples collected manually during storms will be utilized in comparing the degree of pollution under varying conditions. It will also be necessary to collect samples from the receiving waters so that comparisons can be made to see what improvements in water quality have occurred.

Figure 31a shows the type of vehicle used by a 2 man crew for picking up grab samples. This particular vehicle is also equipped to service the automatic sampler located in the shed on the right side of the picture.

This shed is set over an open manhole and chained to the manhole steps to prevent vandalism.

Figure 31b shows the hoist that is mounted on the truck for lowering the sampler into underground locations. Figure 31c shows a closer view of the automatic sampler which takes a continuous sample but cycles every half hour to fill a new bottle. With this machine 48 bottles will be filled in a 24 hour period. Variable time cycles and flow rates are possible. The sampler may be operated on 120 volt AC or on 12 Volt DC current. The location shown here uses two 12 volt automotive batteries wired in parallel to provide the necessary amperage. Figure 31d shows the portable battery charger mounted in the truck. This charger is plugged into a 120 volt AC power source each night so that fresh batteries are available for each days operation.

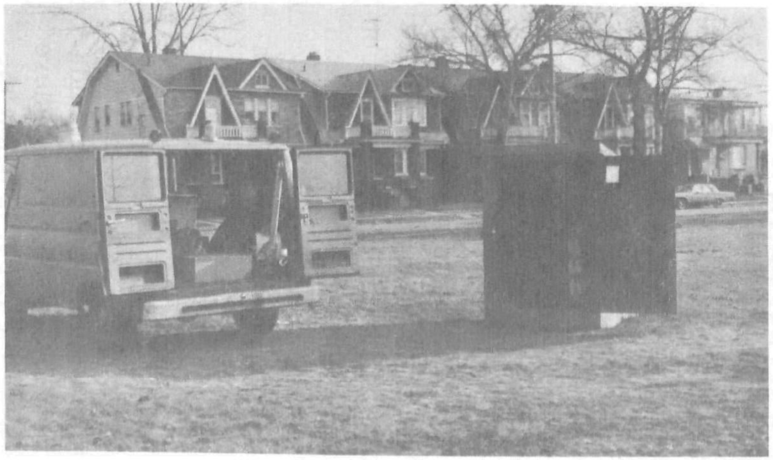
The pumping unit for the automatic sampler is mounted on the manhole steps below the sampling unit which shown in figure 31c. Since the pump is a vacuum type the maximum lift is approximately 18 feet.

Different sampling heads are under investigation since the debris (paper, rags, plastic from disposal diapers, etc.) in a combined line has a tendency to wrap around them and cause blockages.

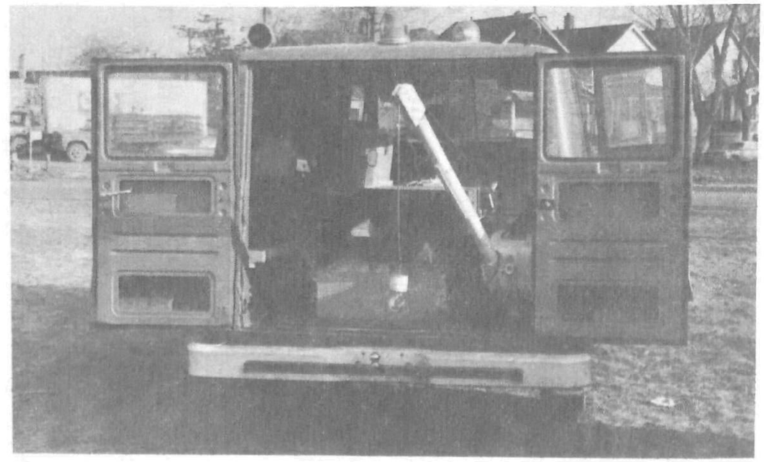
The type of suction pump used by this sampler will continue to operate without causing any damage to itself even if no flow is available to be drawn up into the collection unit. The collection unit will continue to cycle each half hour yielding empty bottles. Therefore it is expected that we will be able to sample combined overflows at different outfalls by placing the unit in operation in a dry overflow chamber and having it await a rain of sufficient size to cause an overflow to occur.

This particular sampler is manufactured[2] by Rock and Taylor of Birmingham

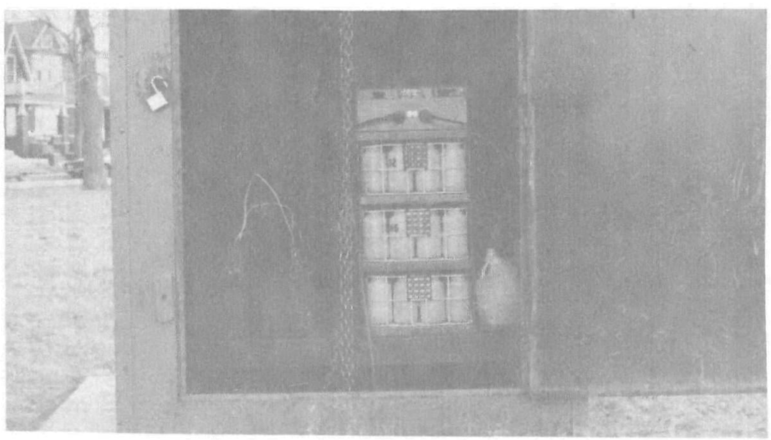
2. All commercial products were purchased on a low bidder basis and mention by name does not imply endorsement by the Federal Water Quality Administration or the Detroit Metro Water Department.



a. SAMPLING VEHICLE AND SAMPLING STATION



b. HYDRAULIC HOIST



c. AUTOMATIC SAMPLER

d. PORTABLE BATTERY CHARGER

FIGURE 31 – SAMPLING EQUIPMENT

England and is distributed in the United States by Megator Corp. of Pittsburg, Pennsylvania. Other samplers of U.S. manufacture are now available.

MONITORING BENEFIT - BETTER REGULATOR SETTINGS

Prior to the installation of the monitoring system on the outfalls, there was no way of knowing which regulators were slow or overloaded, sluggish, blocked by debris or affected by peculiar conditions in their district. All backwater gates and regulators, were however, inspected weekly but with the monitoring system if an overflow continues to occur after all others have ceased, or if an overflow occurs during a dry period, action can be taken immediately to alleviate the problem.

Optimum condition

The optimum setting of a particular regulator is a setting which will take in the maximum portion of the highest suspended solids, B.O.D. and other pollutants and conversely to set other regulators on the cleaner water outfalls to take in minimum portions without overloading the treatment plant. Based upon the outfall sampling program, the regulators are being re-set to more closely fit the optimum condition.

MONITORING BENEFIT - EFFECT ON ROUGE INTERCEPTOR

The Northwest-Rouge Interceptor lies within the Rouge River Valley and actually makes four separate crossings of the Rouge River. As mentioned before, only where the interceptor passes under the Rouge shipping channel is there a normal inverted siphon crossing. The other crossings are made by slightly lowering the grade for the crossing. These crossings do not create true inverted siphons except under high storm conditions.

Along the upper three miles of the interceptor at outfalls which are not yet protected by backwater gates, a major flood along the Rouge can flow back across the diversion dams into the interceptor. This causes undue load on the wastewater plant. The Lyndon, Schoolcraft, Puritan, Seven Mile Road and Frisbee outfalls still require backwater gates. (See Figure 8 for locations) In order to exclude this load of high river water, a sluice gate at Warren-Pierson is being connected for remote operation so that the Rouge River water could be shut off during flood stage.

Three river level sensors are installed at three critical points along the Rouge River to enable the central control office to follow the height and progress of a flood crest as it moves down the section of the river where there is no backwater gate protection. As long as the crest is below the lowest dam at Lyndon, (which is more than 99% of the time) the Warren-Pierson sluice gate is held in the open position to take intercepted dry weather flow.

SUBURBAN FLOW

A factor which at this time limits additional in-system storage is the rather high volume of stored flow which is received from suburban communities for treatment at the Detroit Wastewater Plant. Five large detention basins in suburban areas retain storm runoff for gradual release to the interceptor over as much as a three day period. It appears that de-watering priorities and schedules may be desirable or necessary in the future. This type of scheduling would be a type of flood routing and has caused DMWD to consider other possible locations for remotely operated sluice gates to implement such a program. Figure 32 shows a possible gate location in the junction between a trunk sewer and a relief line. Other ideas are also being investigated.

START-UP PROBLEMS

There have been numbers of unexpected problems in placing this monitoring system in operation. Many difficulties were experienced in getting the leased lines properly connected. The computer program required several adjustments to produce usable data on the typewriters. Slight variations in voltage affected the memory of the computer. During early stages, there were unexpected down times due to failures of transistors. There has been a problem of calibrating each level sensor or rainfall sensor to get exact correspondence of sewer level or rainfall data with the computer typed data. As these problems are solved, the engineering staff is now getting data which are now much more useful and dependable as a basis for developing parameters for use in future operations.

CONSTRUCTION, CONTRACT, AND EQUIPMENT PROBLEMS

Miscellaneous problems have arisen during and after the construction phase of the project, especially with the sensor installations. One problem that came up three times involved the placing of pedestals in locations that made local property owners unhappy. Although the pedestals are all on public property, care should be taken to anticipate such situations as having a pedestal installed too near a driveway or having it fall directly in front of a homeowner's front door. The location of the manhole, of course, is the governing factor but the pedestal can usually be shifted a few feet laterally to satisfy most citizen complaints.

The original plan for sensor installation (See Figure 10) did not call for the use of conduit but it was included at FWQA's request.

This has proven to be a wise decision since it allows for the replacement of the 1/4" nylon tubing with a minimum of such problems as traffic disruption. There have been a few locations where the conduit has risen out of its slot in the pavement. To eliminate this problem in later installations, we have

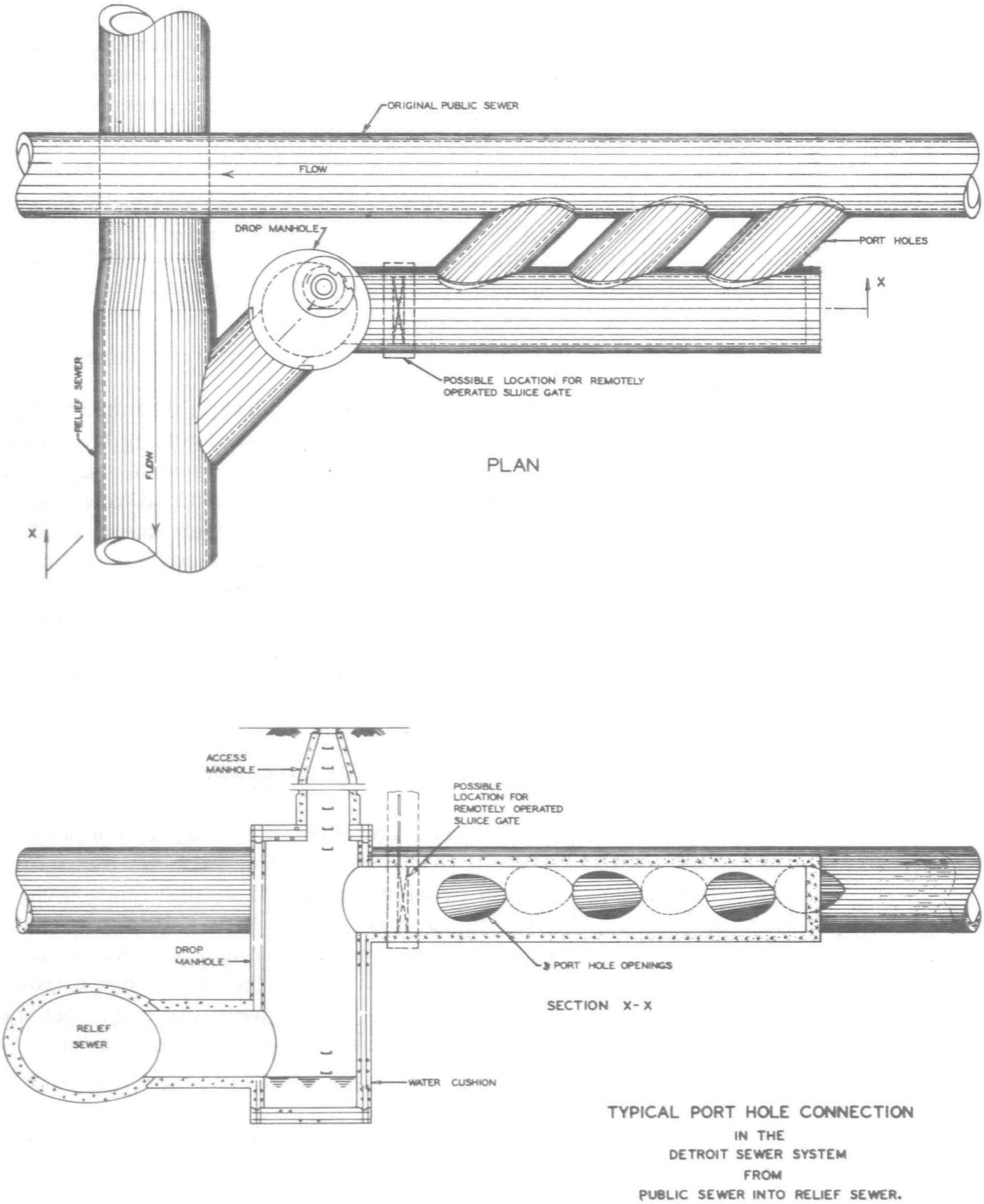


FIGURE 32 - POSSIBLE REMOTE SLUICE GATE LOCATION

been cutting the conduit and placing a sleeve approximately half way between the manhole and the pedestal. This should allow the necessary movement required to keep the conduit from buckling because of the traffic loads.

Traffic volumes should be considered when preparing a contract for installation of this type of equipment. In Detroit, as well as many other cities, a traffic department will limit the time that certain streets or parts of streets may be closed. This information should be made available to potential bidders so that problems do not arise after contract execution has taken place.

The use of air rather than a bubbler gage in the Detroit level sensor has not caused any unusual problems. Some temporary installations have been operating nearly two years without any maintenance. Tests were conducted during an extremely cold period of December 1968 to see if any freezing problems might arise. The float controlled gate at one regulator was manually closed causing sewage to rise in the line and actuate the sensor at temperatures as low as $-6^{o}F$ with no adverse affects. Blockage from oil and debris in these combined lines has not occurred. It is envisioned that an annual preventive maintenance program will be set up and each fall each of the air lines will be blown down with dry air. The valving set-up inside of each pedestal cabinet (See Figure 12) has been so designed to allow such an operation.

It has been found that once the 1/4" nylon tubing has been cut that an effective splice cannot be made. One tube was cut with a pavement breaker and all attempts at splicing have failed and an air leak continued to occur. In the future when a break occurs the complete piece of tubing from the level cell to the valve inside of the pedestal cabinet will be replaced. We at DMWD feel that the problems arising from use of the low maintenance air type level sensor are far out weighed by the benefits of the lower capital and operating costs.

SECTION IV

POST CONSTRUCTION EVALUATION PLAN (1970)

1. Complete computer programming of mathematical models of each outfall so that in conjunction with pump station records the volume of overflows may be calculated.

2. Correlate volume of overflows with rainfall data and develop hydrographs.

3. Allow and record overflows by using the "Monitoring and Remote Control" system so that comparisons may be made and improvements documented.

4. Complete calculation and tabulation of all available storage volumes that may be used during times of overflows.

5. Collection and analysis of additional storm and river samples.

6. Evaluation by weight of all types of pollution from each overflow point to provide data for a study on regulator float settings.

7. Storm flow routing studies to allow for maximum benefit from monitoring data and to pinpoint locations for future flow controllers.

8. Cost - Benefit Study.

9. Publication of Final Report.

SECTION V

FUTURE OBJECTIVES (1971-75)

1. Further storm routing studies with emphasis on suburban flow entering the system.

2. Development of new design criteria to aid in future expansion and improvement to the system.

3. Study the possibilities of off-line storage operated in conjunction with the monitoring program.

4. Study needs for and usefulness of supplemental computer related equipment for control center (i.e. lighted panels, graphic printouts, tapes, cards, self instrumentation, water quality inputs, etc.).

SECTION VI

ACKNOWLEDGEMENT

F.W.Q.A. RESEARCH AND DEVELOPMENT PROJECT 11020 FAX

DETROIT - SEWER MONITORING AND REMOTE CONTROL

by

DETROIT METRO WATER DEPARTMENT

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C. Schultz - Field Engineer
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ACKNOWLEDGEMENT (CONTINUED)

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W. Callfas	-	Chief Water System Supervisor
J. Urban	-	Chief Engr. - Wastewater Plant
T. Standen	-	Supervisor of Mech. Maintenance
E. Fisher	-	Superintendent of Building Maintenance
E. Kline	-	Regulator Foreman

REPORT - DELINATION AND PRODUCTION

S. Beer
Q. Washington
C. Porter
E. Tulecki

We wish to acknowledge at this time that the development of the Detroit system of Monitoring and Remote Control was a "Team Project", rather than an individual brain child. It was a team project in the finest sense of word with the engineers, the operating personnel, the field crews, the contractors forces, and the governmental representatives all working together towards developing an operating system. It has been a real pleasure to have worked on such a team.

APPENDIX A

TABLE I-A

Average of Daily Grab Samples - 1968

Sewer Location	Susp. Sol. mg/l	BOD mg/l	Tot.P mg/l	Phenols mg/l	Oil & Grease mg/l
Pembroke	195	111	14.8	78	34
Frisbee E.S.	194	90	10.2	79	32
Frisbee W.S.	192	148	16.1	104	70
7 Mile E.S.	532	458	15.0	177	75
7 Mile W.S.	790	267	16.8	151	83
McNichols	180	158	11.2	117	95
Puritan E.S.	332	181	18.2	163	58
Puritan W.S.	335	234	16.6	144	86
Fenkell-Lyndon	423	184	15.4	137	49
Schoolcraft	180	192	16.3	214	48
Glendale	482	113	11.4	81	67
Plymouth	592	298	16.3	89	230
W. Chicago	1350	197	15.9	89	100
Joy Road	915	202	16.4	111	100
Tireman	387	149	15.3	195	26
Hubbell-Southfield	78	43	3.1	75	22
Dearborn (Miller Rd.)	270	144	7.4	9700	55
Baby Creek	502	227	7.9	2775	2775
Flora	238	162	8.3	235	1395
Carbon	452	109	11.3	200	62
Pulaski	817	181	10.4	276	140
Dearborn Avenue	733	222	10.6	129	77
Schroeder	223	97	10.5	193	116
Dragoon	702	203	4.3	236	20
Campbell	185	107	8.0	348	122
Morrell	166	125	4.3	109	65
Ferdinand	146	111	8.8	144	84
Summitt	327	183	4.4	238	82
Scotten	195	95	4.7	145	443

TABLE I-B

Sewer Location	Susp. Sol. mg/l	BOD mg/l	Tot.P mg/l	Phenols mg/l	Oil & Grease mg/l
McKinstry	203	92	5.4	183	177
Swain	69	21	4.7	227	11
W. Gd. Blvd.	117	88	5.1	60	25
24th	149	217	6.9	78	104
21st	498	270	7.5	113	858
18th	648	390	7.0	125	99
12th	387	207	4.4	98	116
11th	92	116	5.9	58	14
3rd	239	174	4.9	111	163
First-Hamilton	140	186	2.7	39	31
Woodward	155	38	2.9	110	38
Brush-Bates	342	221	3.6	136	84
St. Antoine	53	120	3.1	81	23
Hastings	180	90	7.1	147	52
Rivard	342	201	5.4	115	39
Riopelle	142	68	4.1	98	36
Orleans	1005	723	12.7	87	301
St. Aubin	335	299	8.6	76	127
Dubois	234	305	5.9	220	66
Chene	336	233	8.8	185	408
Adair	333	233	8.0	223	60
Leib	165	109	5.8	190	25
Mt. Elliott	232	208	6.6	166	71
Helen	209	170	9.5	164	45
Iroquois	182	108	6.9	156	34
McClellan	220	148	6.3	169	51
Conner	348	84	2.7	278	123
Ashland	327	156	6.1	220	40
Manistique	384	146	8.8	613	52
Fox Creek	411	204	6.7	232	88

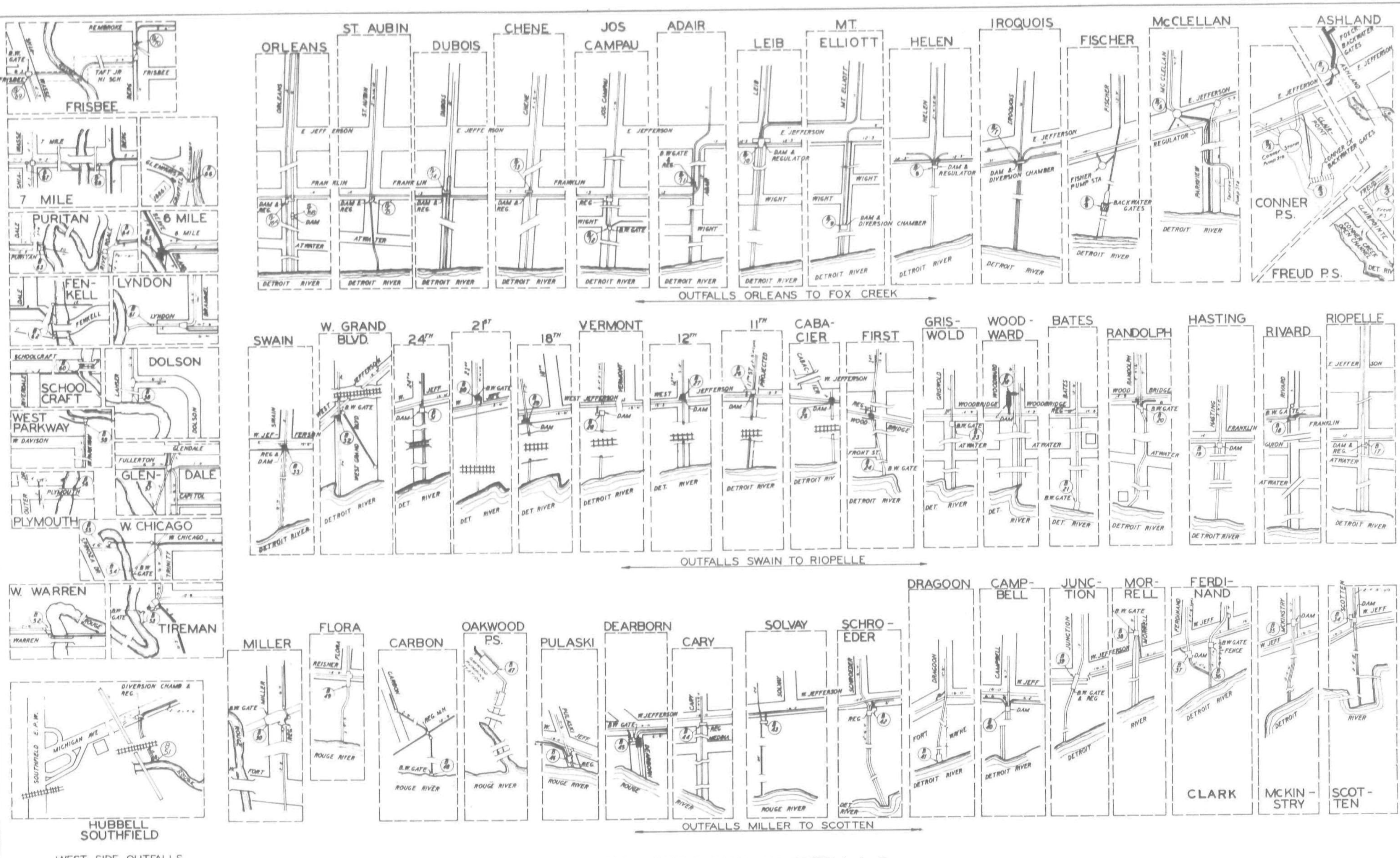
TABLE II-A

Average of Daily Grab Samples - 1969

Sewer Location	Susp. Sol. mg/l	BOD mg/l	Tot.P mg/l	Phenols mg/l	Oil & Grease mg/l
Pembroke	149	198	13.3	130	57
Frisbee E.S.	105	192	12.9	222	53
Frisbee W.S.	117	120	9.6	180	29
7 Mile E.S.	172	80	11.4	212	14
7 Mile W.S.	70	211	11.4	326	60
McNichols	115	98	6.8	163	24
Puritan E.S.	85	110	11.5	212	19
Puritan W.S.	253	99	10.6	218	19
Fenkell-Lyndon	411	134	13.6	239	31
Schoolcraft	302	104	15.0	286	26
Glendale	399	64	12.5	159	28
Plymouth	200	260	17.3	247	42
W. Chicago	163	111	9.0	239	25
Joy Road	147	107	13.5	148	24
Tireman	170	122	15.4	149	19
Hubbell-Southfield	78	43	3.1	75	19
Dearborn (Miller Rd.)	402	166	8.3	1250	64
Baby Creek	446	109	7.6	963	121
Flora	294	52	6.8	257	182
Carbon	309	110	8.5	214	34
Pulaski	453	528	7.3	99	689
Dearborn Avenue	115	230	9.3	249	41
Schroeder	239	251	7.5	269	81
Dragoon	625	246	8.0	142	680
Campbell	205	151	7.8	165	87
Morrell	197	154	5.9	181	74
Ferdinand	213	259	8.2	109	550
Summitt	163	205	3.1	152	89
Scotten	204	29	8.8	125	29

TABLE II-B

Sewer Location	Susp. Sol. mg/l	BOD mg/l	Tot.P mg/l	Phenols mg/l	Oil & Grease mg/l
McKinstry	471	213	6.5	199	188
Swain	189	84	3.2	329	23
W. Gd. Blvd.	210	128	6.4	191	44
24th	169	86	4.8	179	91
21st	271	210	5.0	131	162
18th	762	328	6.1	188	132
12th	80	24	2.0	81	13
11th	227	139	8.5	49	38
3rd	422	152	4.3	107	157
First-Hamilton	184	76	3.7	18	45
Woodward	413	103	4.6	122	65
Brush-Bates	225	206	3.1	115	33
St. Antoine	119	49	3.5	73	14
Hastihgs	218	163	5.4	206	57
Rivard	142	124	5.2	120	21
Riopelle	224	169	8.2	243	133
Orleans	1005	730	11.9	70	240
St. Aubin	729	424	10.5	112	73
Dubois	267	350	6.7	265	81
Chene	338	233	8.9	185	409
Adair	460	226	7.0	236	91
Leib	148	109	5.8	173	25
Mt. Elliott	229	157	7.6	179	44
Helen	268	157	11.5	105	51
Iroquois	240	199	8.1	96	55
McClellan	268	198	9.1	95	35
Conner	248	84	3.7	322	43
Ashland	257	195	5.8	146	27
Manistique	101	111	4.0	130	22
Fox Creek	228	146	4.8	228	26



COMBINED SEWER OUTFALLS
CITY OF DETROIT
FIGURE 33

SECTION 7

STREAM POLLUTION & ABATEMENT FROM COMBINED SEWERS BUCYRUS, OHIO

BY

RICHARD F. NOLAND

DALE A. DECARLO

BURGESS AND NIPLE LIMITED

CONSULTING ENGINEERS

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COLUMBUS, OHIO 43212

ABSTRACT

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

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

A method of controlling the pollution from combined sewer overflows is presented along with the degree of protection, advantages, disadvantages and estimate of cost. The method presented is an interceptor sewer and lagoon system. The most economical method of providing a high degree of protection to the Sandusky River is by collecting the combined sewer overflows with a large interceptor and using an aerated lagoon system to treat the waste loads from the overflows.

The report from which this paper was prepared was submitted in fulfillment of Contract 14-12-401 between the Federal Water Pollution Control Administration and Burgess & Niple, Limited, Columbus, Ohio.

STREAM POLLUTION AND ABATEMENT
FROM COMBINED SEWER AND OVERFLOW
BUCYRUS, OHIO

Bucyrus, Ohio is located near the headwaters of the 1421 square mile drainage area of the Sandusky River Basin in northern Ohio. The drainage area above the City is 90 square miles and consists of mostly level, agricultural land. The current estimated population of the City is 13,000 with an incorporated area of about 2,340 acres. The topography of the City is generally flat to slightly rolling. The study area is shown on Figures 1, 2 and 3. The mean annual precipitation is 36 inches.

The Sandusky River downstream from Bucyrus is a source of water supply for the cities of Upper Sandusky, Tiffin and Fremont. The principal pollution problems in the Sandusky River are sediment, oxygen consuming materials, bacteria, phosphates and nitrates. The stream drains rich agricultural lands which contribute significant amounts of sediment and nutrients. The area around Bucyrus is intensely cultivated. At the present time all communities discharging sewage effluent to the Sandusky River provide secondary treatment facilities.

Future water management plans for the principal cities in the watershed are based on utilizing the natural flow in the River and upground storage reservoirs as the major source of water for the area. Reduction in the pollution discharging to the River thus becomes very important if the desired water quality is to be achieved and maintained for the intended uses.

This is a study to determine the possibility of interception of all or part of the combined sewer overflow for treatment prior to release to the stream; also to determine the relationship of rainfall events to overflow events and the volume of flow and waste load discharged to the Sandusky River. Weirs for measuring overflows during rainfall were installed at the overflow points of three selected sewer districts. Samples were collected during selected

LEGEND

1421	Drainage areas enclosed by shaded lines (sq. miles)
353	Drainage areas enclosed by unshaded lines - - - ——— (sq. miles)
90	Drainage areas above points indicated by arrows – sq. miles
980	Approximate low-water elevation in feet above sea level

Figure No. 1

SANDUSKY RIVER

Drainage Area

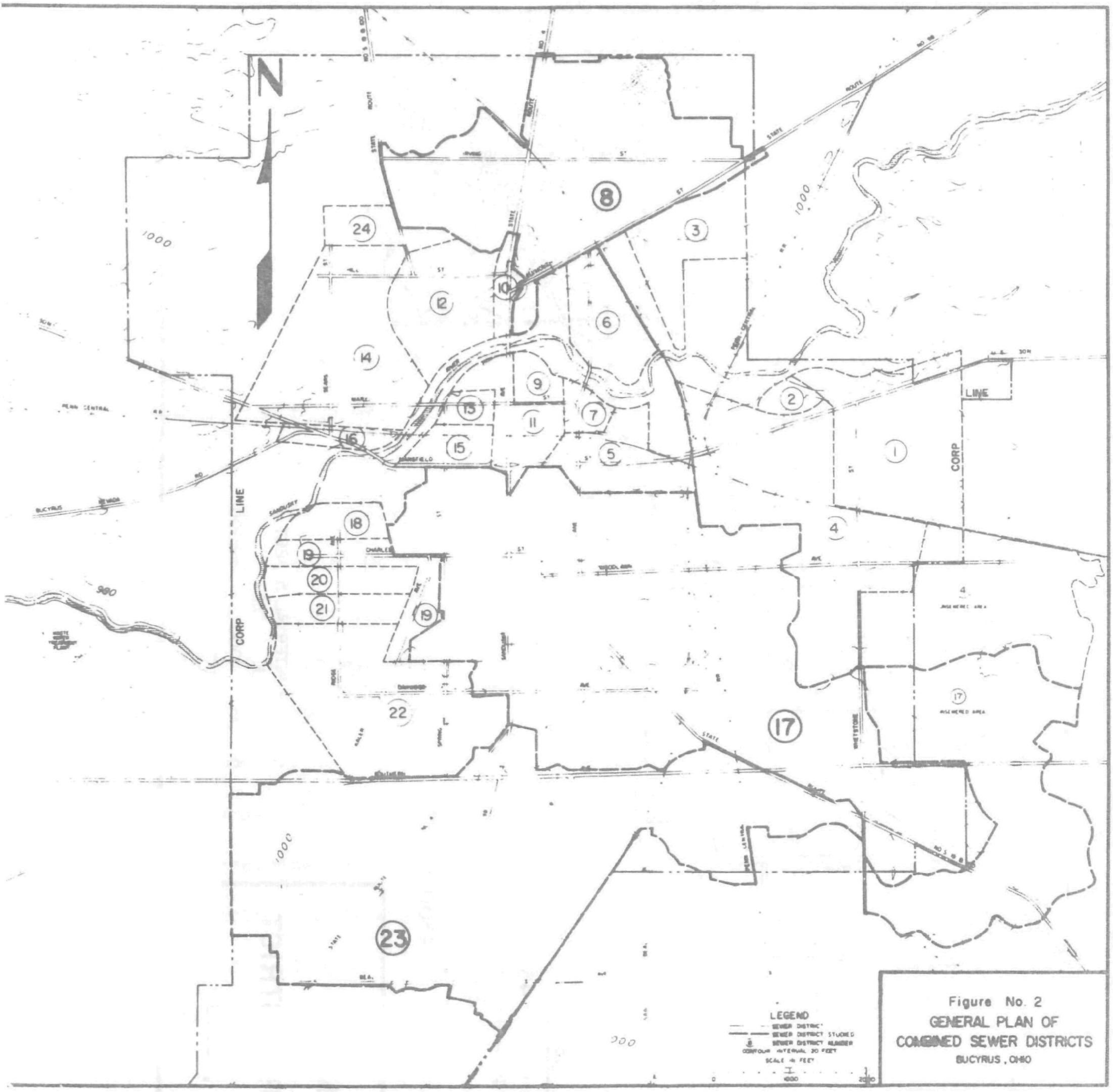
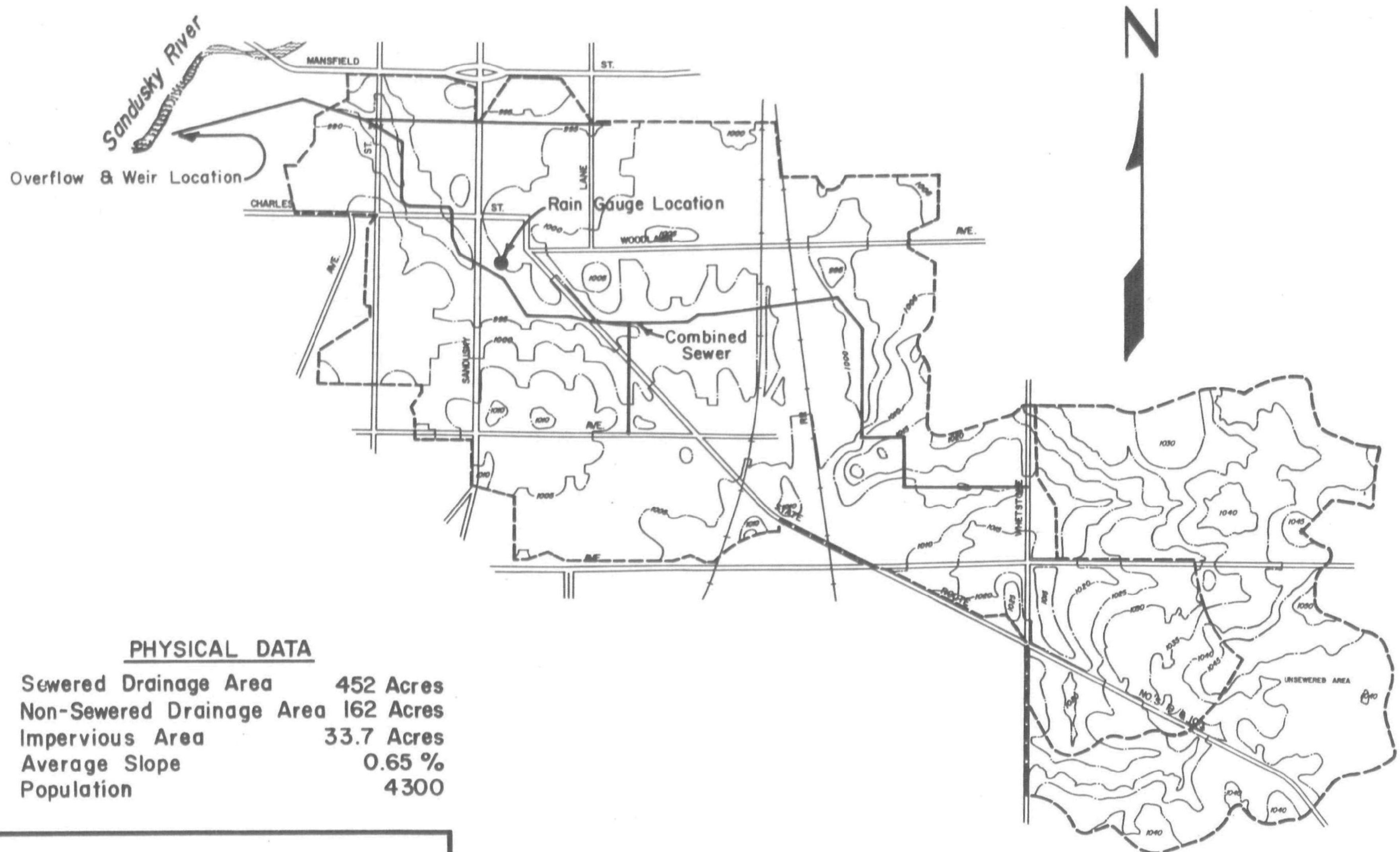


Figure No. 2
GENERAL PLAN OF
COMBINED SEWER DISTRICTS
BUCYRUS, OHIO



PHYSICAL DATA

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

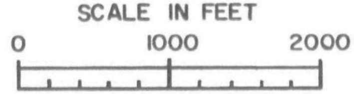
Figure No. 3

NO. 17 SEWER DISTRICT

Bucyrus, Ohio

CONTOUR INTERVAL 5 FEET

SCALE IN FEET



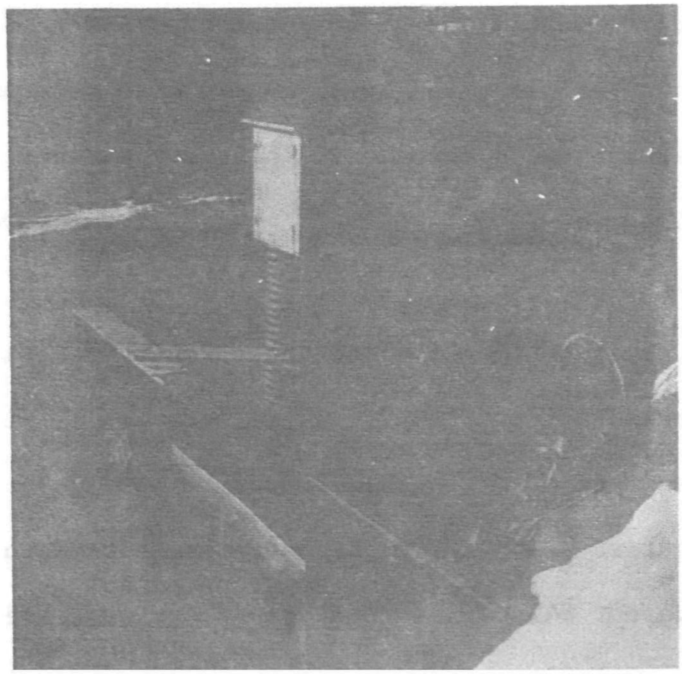
overflow events to determine overflow characteristics and the effects on the stream.

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

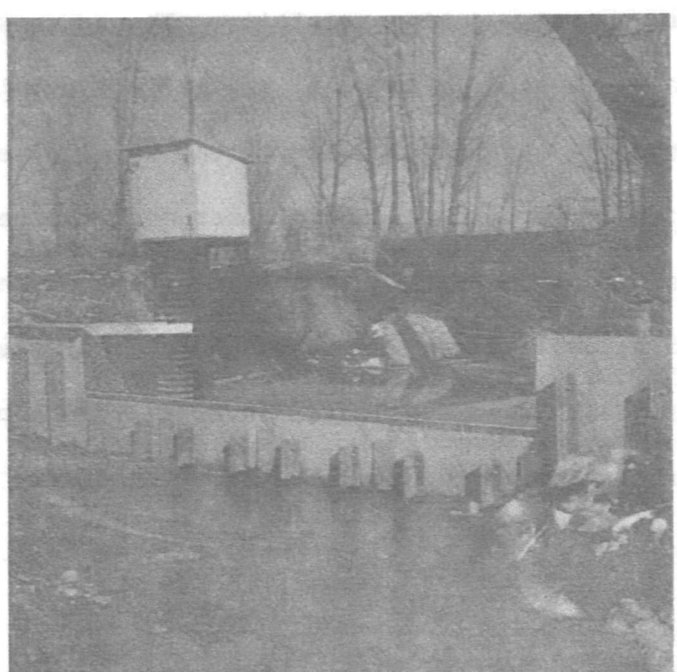
A detailed study of the City and the sewer system was made and three districts were selected for study. These were No. 8, with 179 acres, No. 17 with 452 acres, and No. 23 with 378 acres. No. 17 is shown in the accompanying figures. These are the three largest drainage districts in Bucyrus and represent 64% of the total sewer drainage area. A complete detailed hydraulic analysis was made of the sewer system in these districts.

A rectangular Weir (See Figure 4) was built at each of the three overflow points to measure overflow during rainfall. The weirs were constructed of 1" plywood which was bolted on to steel angles imbedded in concrete. The weirs were sized to pass the maximum capacity of the trunk sewer at the overflow points. Water level recorders were installed in instrument shelters behind the weirs. All recorders were equipped with automatic starters which would start the clocks at predetermined water levels. An automatic starter was devised for the samplers that started the clocks when the water level reached a predetermined height behind the weirs. Samplers could therefore be left unattended prior to and during an overflow.

A continuous record of the flow in the Sandusky River, both above and below Bucyrus, was obtained during the study period. This was accomplished by using the records from an existing recording gage operated by the U. S. Geological Survey



NO. 8 OVERFLOW WEIR



NO. 17 OVERFLOW WEIR

Figure No. 4

and a stream gaging station installed by us prior to the beginning of the study.

The combined sewer overflows and the River were sampled during many storms throughout the study period to determine the quality of the overflow and the pollution loads. Only the overflows at No. 8, 17, and 23 sewer districts were sampled. From July, 1968 through January, 1969 samples were collected manually. After February 1, 1969 automatic samplers were installed and supplemented by manually sampling. Laboratory analyses were performed on all overflows and river samples selected for 18 different physical and chemical tests.

Project personnel made 16 trips during wet weather to Bucyrus from July 1968 until September, 1969 to collect samples of predicted overflows. There were 10 days out of 16 that overflows actually occurred and were sampled. Grab samples were collected manually during five overflow events that occurred prior to February 8, 1969. Samples of the remaining 5 overflow events were collected by automatic samplers and project personnel.

The relationship of rainfall and runoff was studied by the use of several different methods. Hyetographs and Unit Hydrographs were developed for the design storm. The graphs for No. 17 district are shown in Figure 5. A straight line relationship was found between maximum rainfall intensity for a given duration and peak overflow rate. The least amount of deviation was produced by a rainfall of 20 minute duration.

Following is the mathematical formula for the relationship for No. 17 overflow:

<u>Maximum Twenty Minute Rainfall versus Peak Overflow Rate</u>

Q = Peak flow, cfs

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

<u>No. 17 Overflow</u>

1) No Antecedent Rainfall

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

$I > 0.39$, Same as with Antecedent Rainfall

2) Rainfall within 24 hours

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

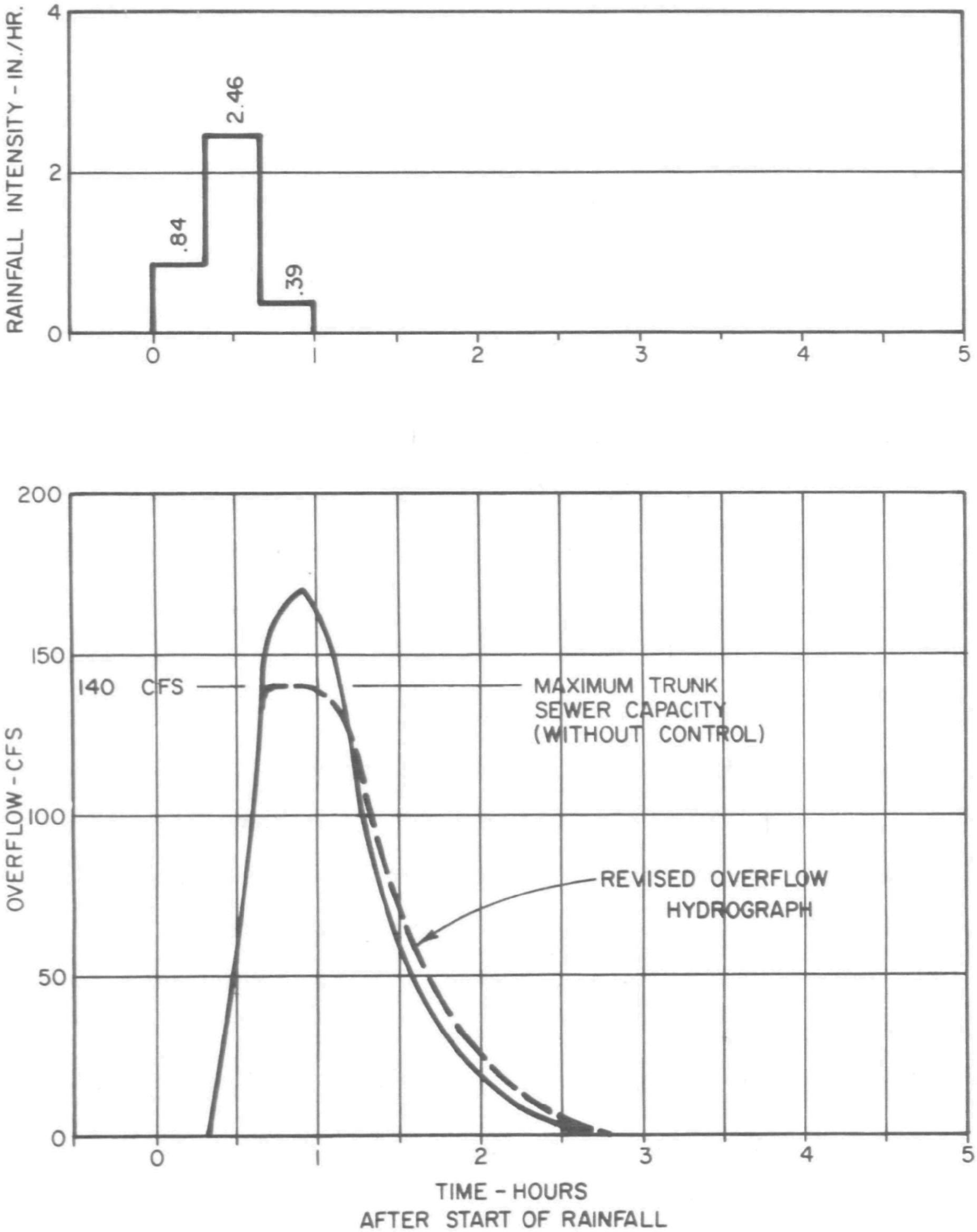


Figure No. 5
Rainfall and Overflow
Two Year, One Hour Storm
No. 17 Overflow

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

<u>Twenty Minute Rainfall versus Overflow Volume</u>

O = Overflow Volume, Depth on sewer district in inches

P = Rainfall, inches

<u>No. 17 Overflow</u>

1) No Antecedent Rainfall

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

$P \quad 0.13$, Same as with Antecedent Rainfall

2) Rainfall within 24 hours

$$O = 0.28 (P - 0.01)$$

Part of the laboratory results of the overflow samples from the three selected districts have been summarized and presented in Table 1. This table presents the average, minimum, maximum and medium values of the chemical and bacteriological characteristics of all the individual overflow samples collected during this study. Sewer District 8 had an average BOD Concentration of 170 milligrams per liter which is considerably higher than the average BOD Concentration of Sewer Districts #17 and 23, each of which had an average of 107 mg. and 108 mg. per liter, respectively. The average suspended solids concentration of 480 mg. per liter for the overflow samples is much higher than the average of 160 mg. per liter for the dry weather samples.

The significant water quality characteristics of the overflow samples which include BOD, suspended solids, total solids, and nitrogen series, total phosphates and chlorides have been graphed in comparison to time after start of overflow and are shown in Figures 6 to 12. These curves very clearly show the first flushing effect of the storm water. A summary of the waste loads discharged into the Sandusky River from each of the five overflows sampled and measured have been calculated and summarized in Table 2. This Table shows that the August 9, 1969 overflow event (No. 5 in the table) discharged into the Sandusky River from just

TABLE 1

SUMMARY OF LABORATORY ANALYSES
ON OVERFLOW SAMPLES

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

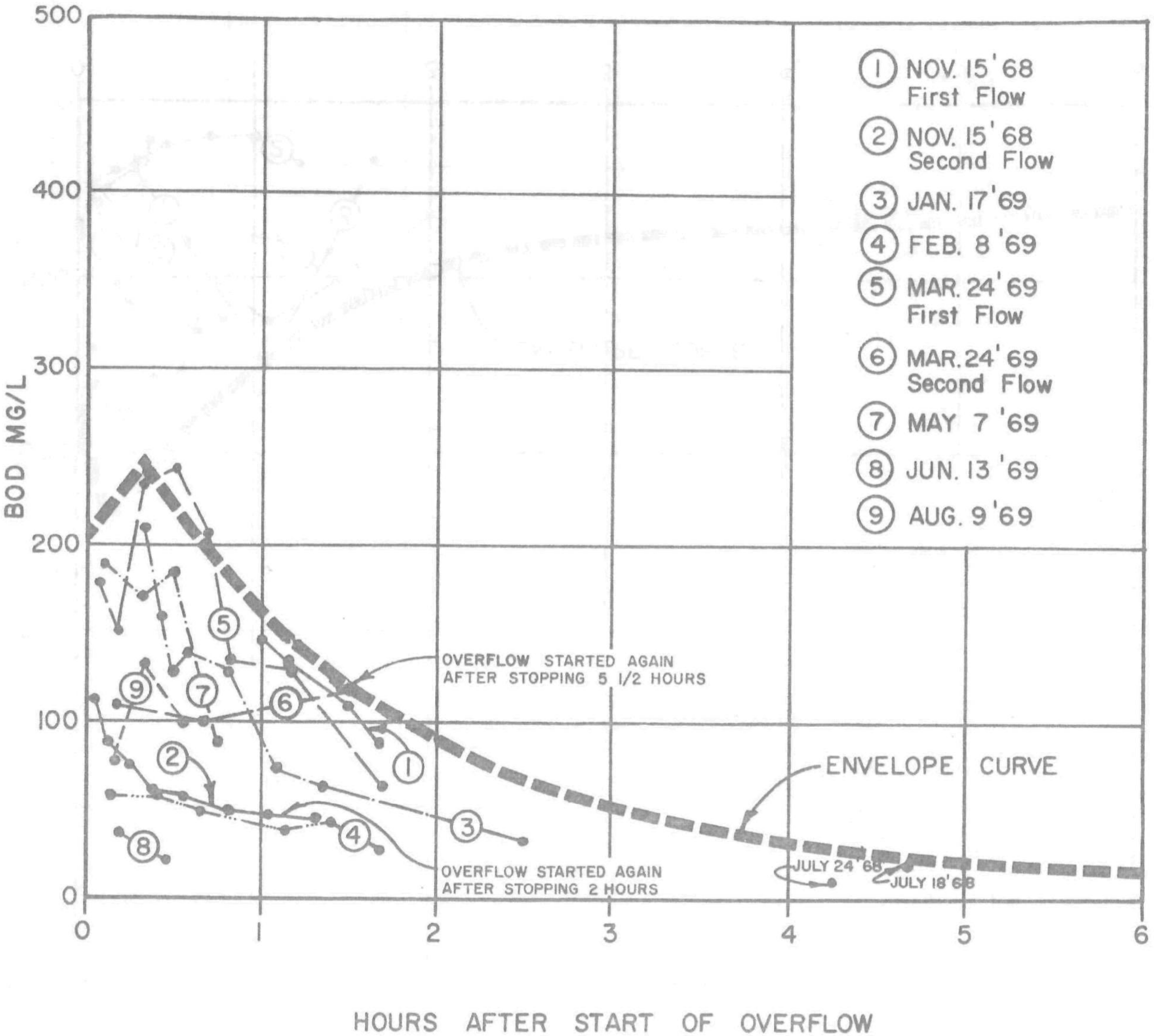


Figure No. 6
BOD Concentration vs. Time
No. 17 Overflow

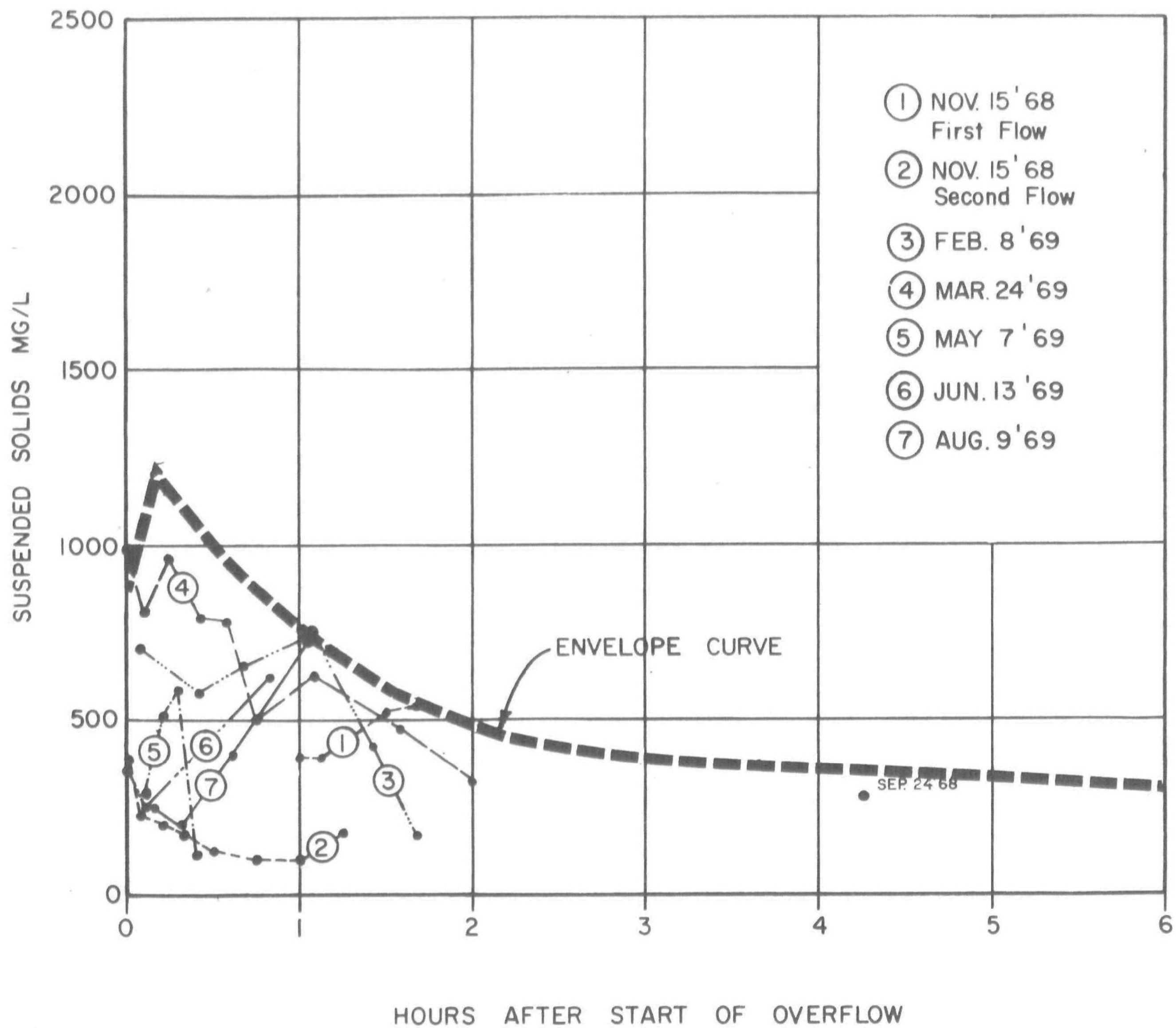


Figure No. 7
Suspended Solids Concentration vs. Time
No. 17 Overflow

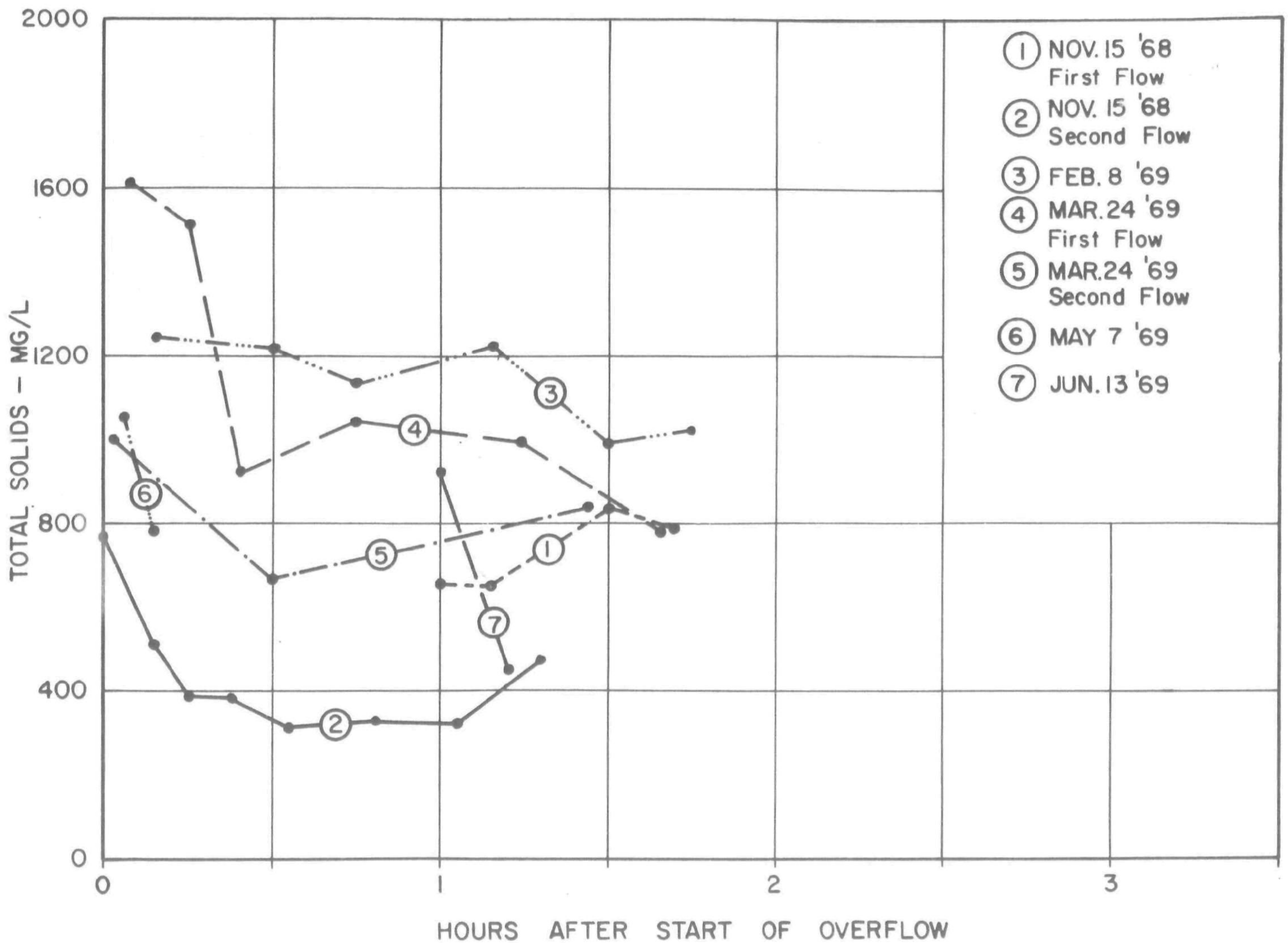
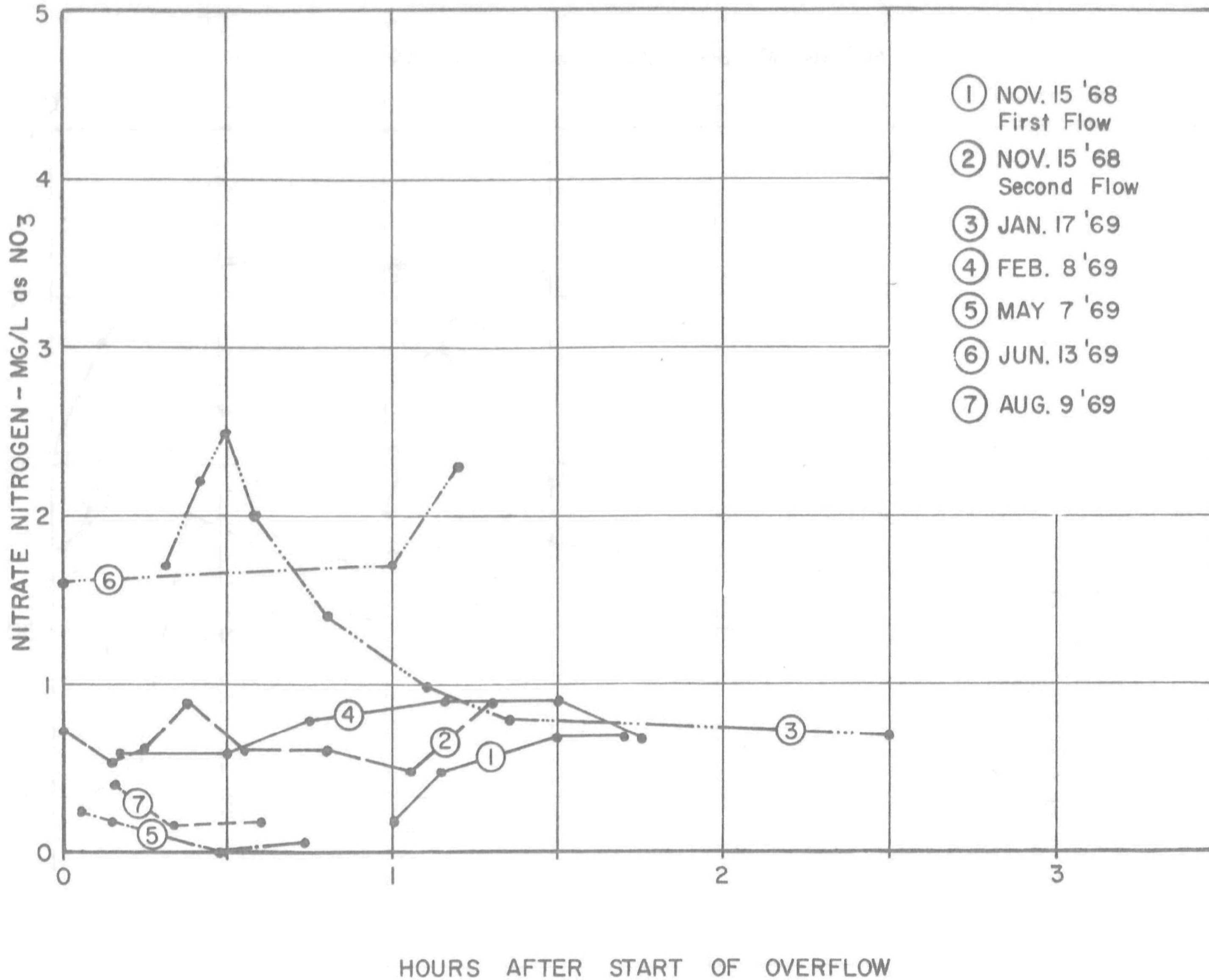


Figure No. 8
Total Solids
No. 17 Overflow

Figure No. 9
Nitrate Nitrogen
No. 17 Overflow



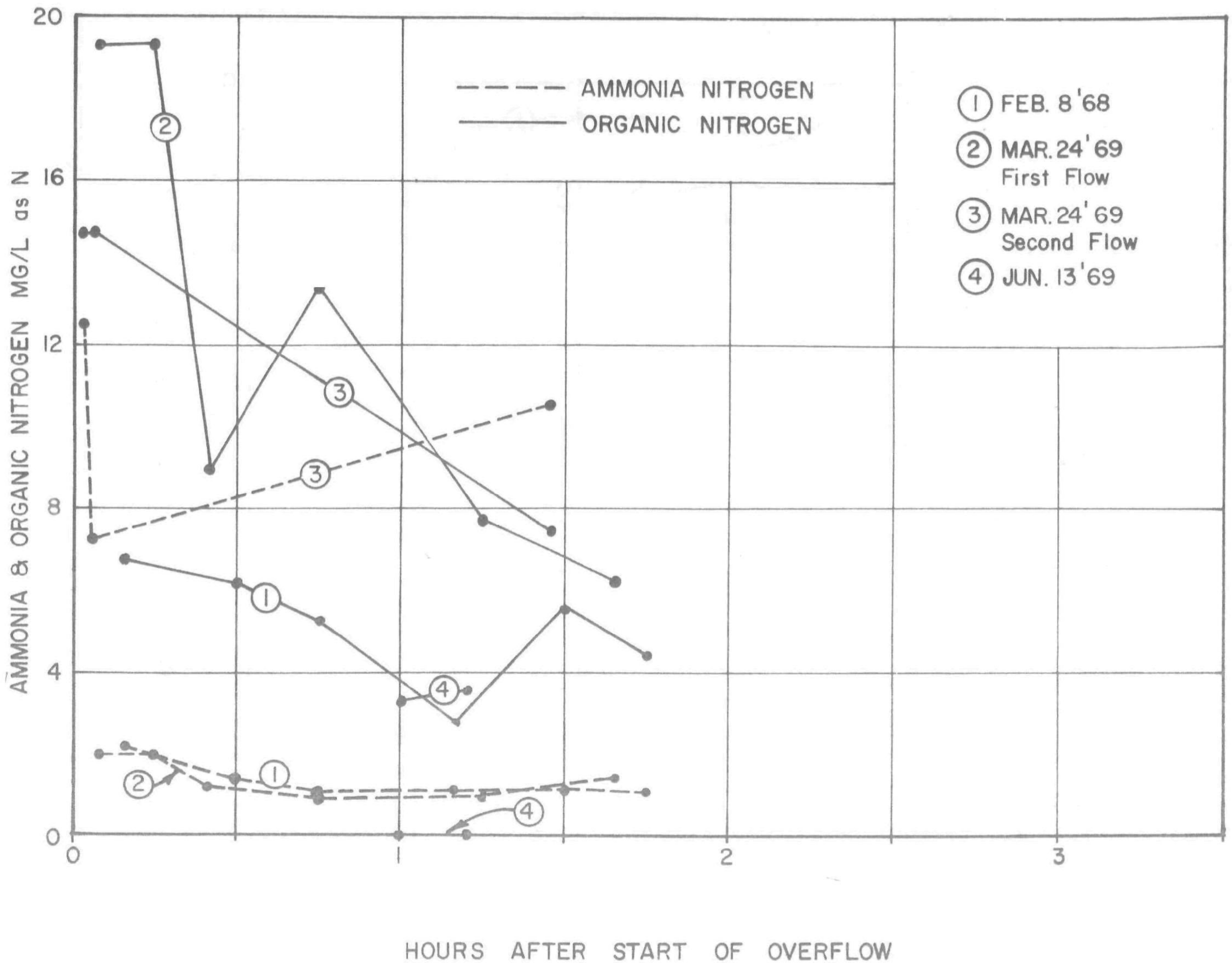
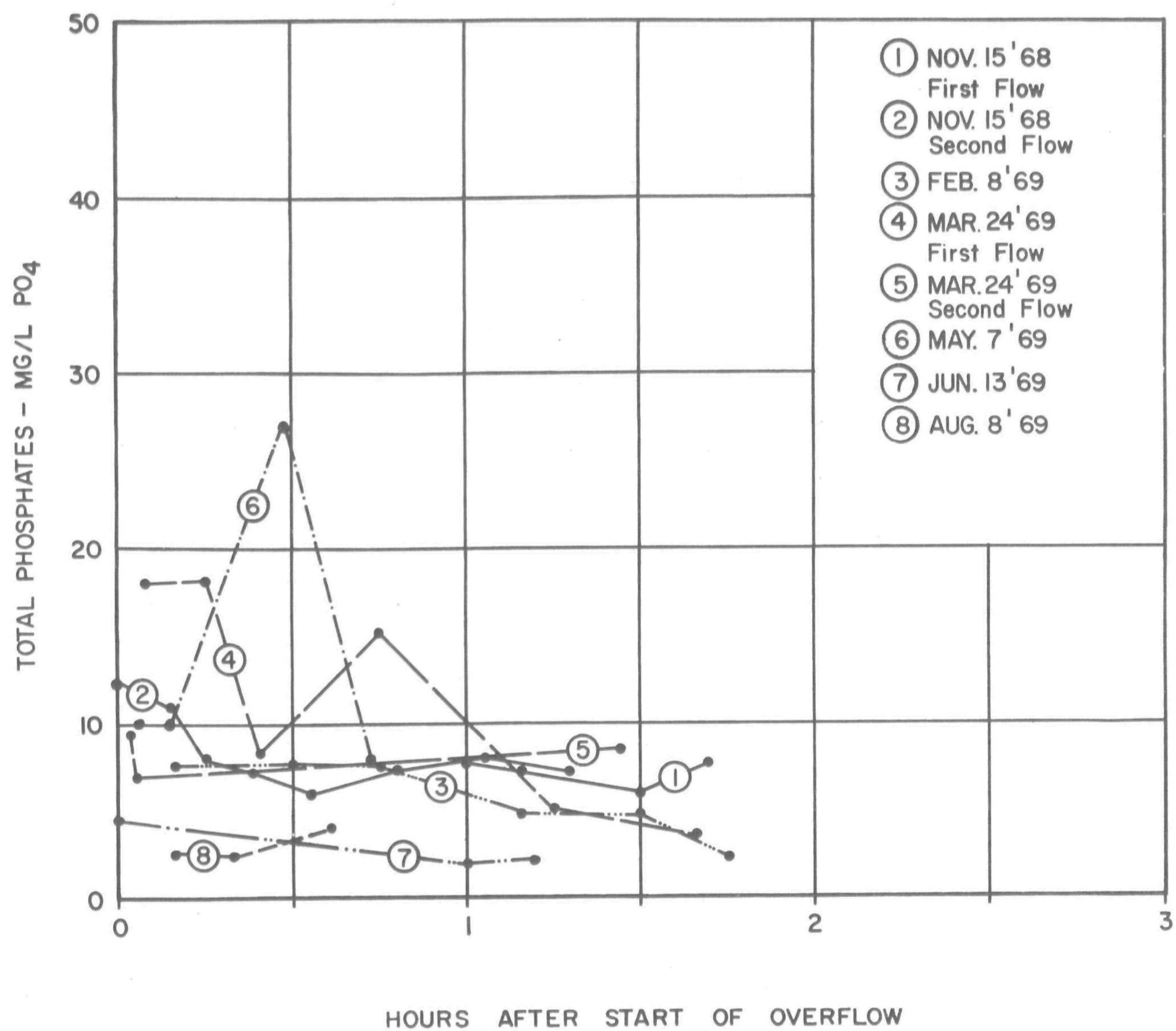


Figure No. 10
Ammonia & Organic Nitrogen
No. 17 Overflow

Figure No. 11
Total Phosphates
No. 17 Overflow



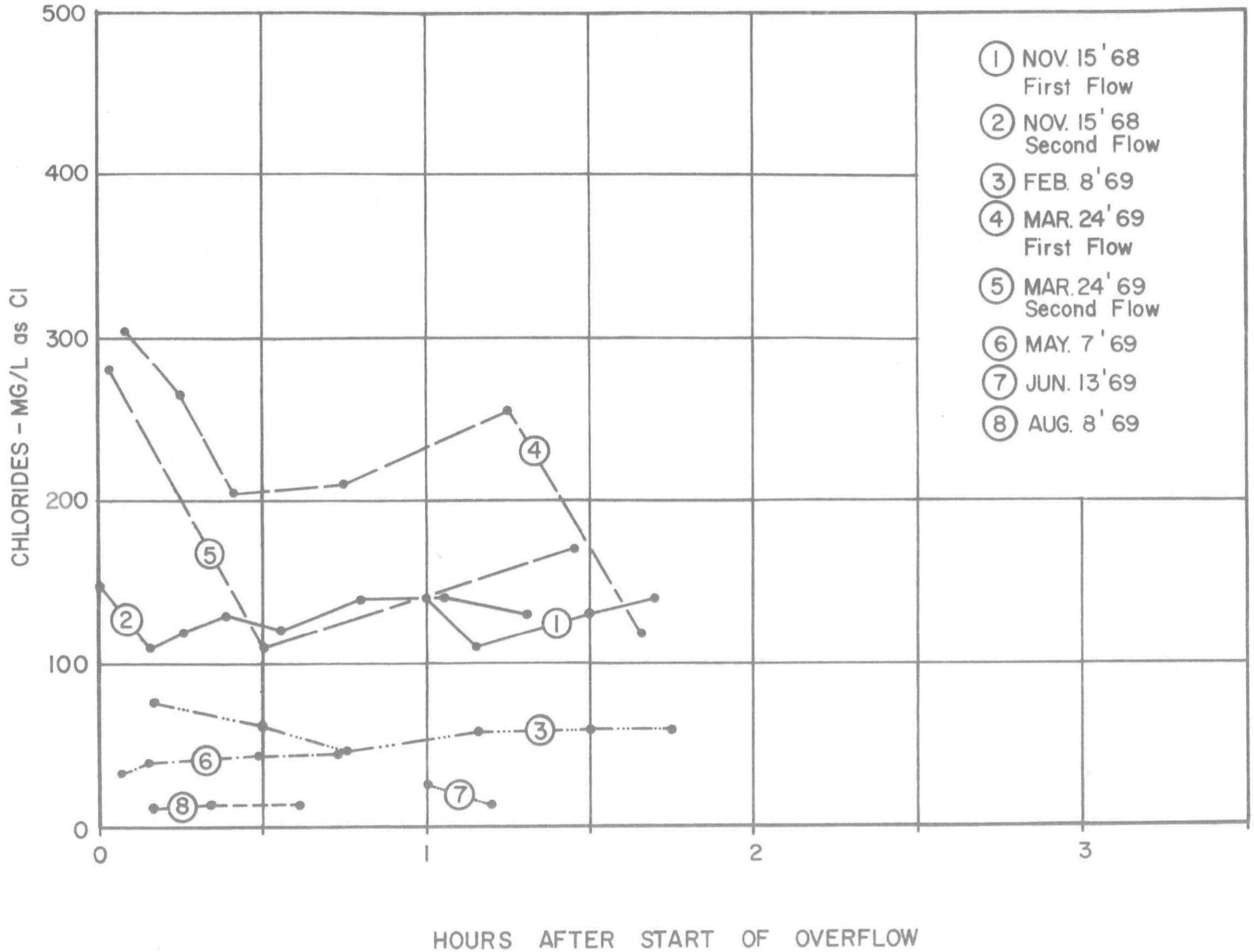


Figure No. 12
Chlorides
No. 17 Overflow

TABLE 2

SUMMARY OF WASTE LOADS FOR EACH OVERFLOW EVENT

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

three districts, 2300# of BOD in approximately 2 hours, this is more BOD than was received at the wastewater treatment plant in 24 hours of dry weather flow. Extrapolating the 2300# of BOD to include all 24 sewer districts, gives the total of 3500# of BOD discharged to the river.

In studying the river response to rainfall, an urban runoff hydrograph was developed which showed the distinct effect of the runoff from the urban area on the river. The significant runoff reaches the downstream gage approximately one hour after the start of the rain. The river reaches a peak flow two hours later, as shown in Figure 13. In 7 hours the river returns to its prestorm level and remains until the runoff from the upstream drainage area arrives. The time of arrival depends entirely on the velocity in the River. The lag time of the peak flow following the end of the rains, varies from 40 hours to 17 hours for river flows of 4 to 300 C.F.S. at the upstream gage.

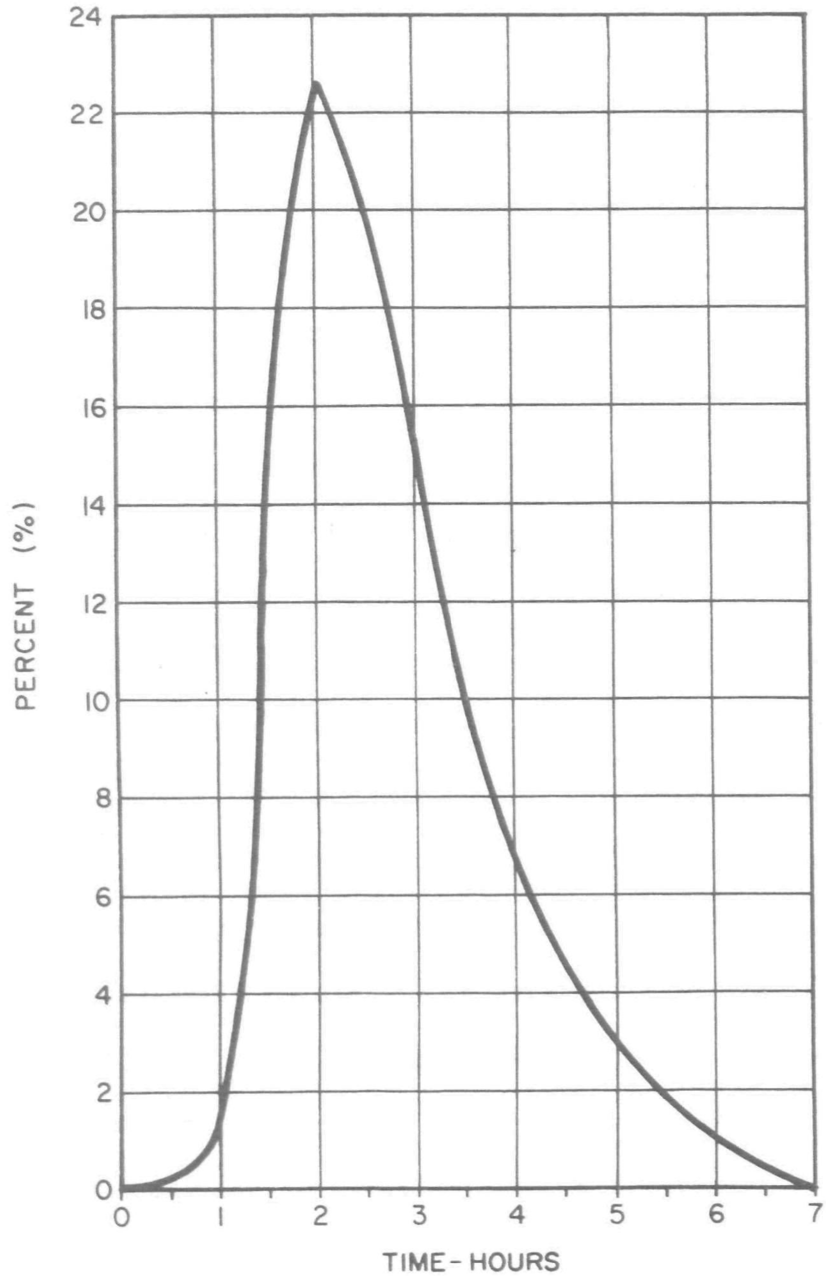
The wastewater loads, discharged from combined sewer overflows, depends on a number of factors, including duration and intensity of rain fall, volume of runoff, number of days between overflow events, efficiency of street cleaning operations and design characteristics of the sewer system. The relationship between BOD and suspended solids and load discharged per 100 acres is shown in Figures 14 and 15. Generally the longer the period of time between overflows, the larger the waste load for a particular overflow volume.

Some of the conclusions from this study were:

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

2. A typical summer thunder shower occurred on June 13, 1969 and produced 1.1 inches of rain with a duration of 78 minutes and an average intensity of 84 hundreds of an inch per hour. The runoff from this storm discharged into the Sandusky River through the combined sewer overflows, 5.2 million gallons of combined sewer wastewater; 1580# of BOD and 23,000# of suspended solids.

3. A storm on August 9, 1969 which produced .5 inches of rain in about 75



(PERCENT BASED ON 30 MINUTE TIME INTERVALS)

Figure No. 13
Distribution Graph for Urban Runoff
Downstream Gauge

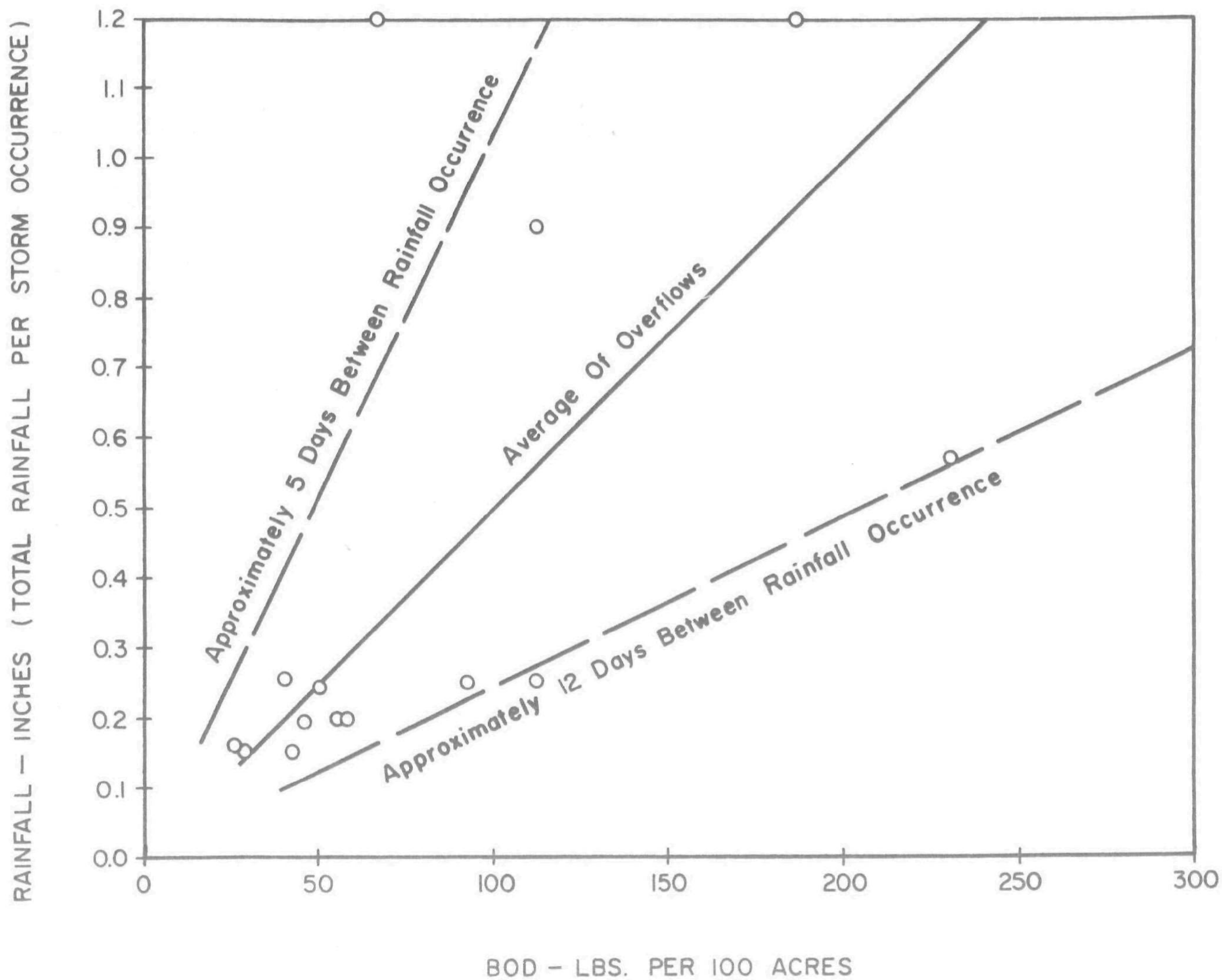


Figure No. 14
Rainfall vs. BOD

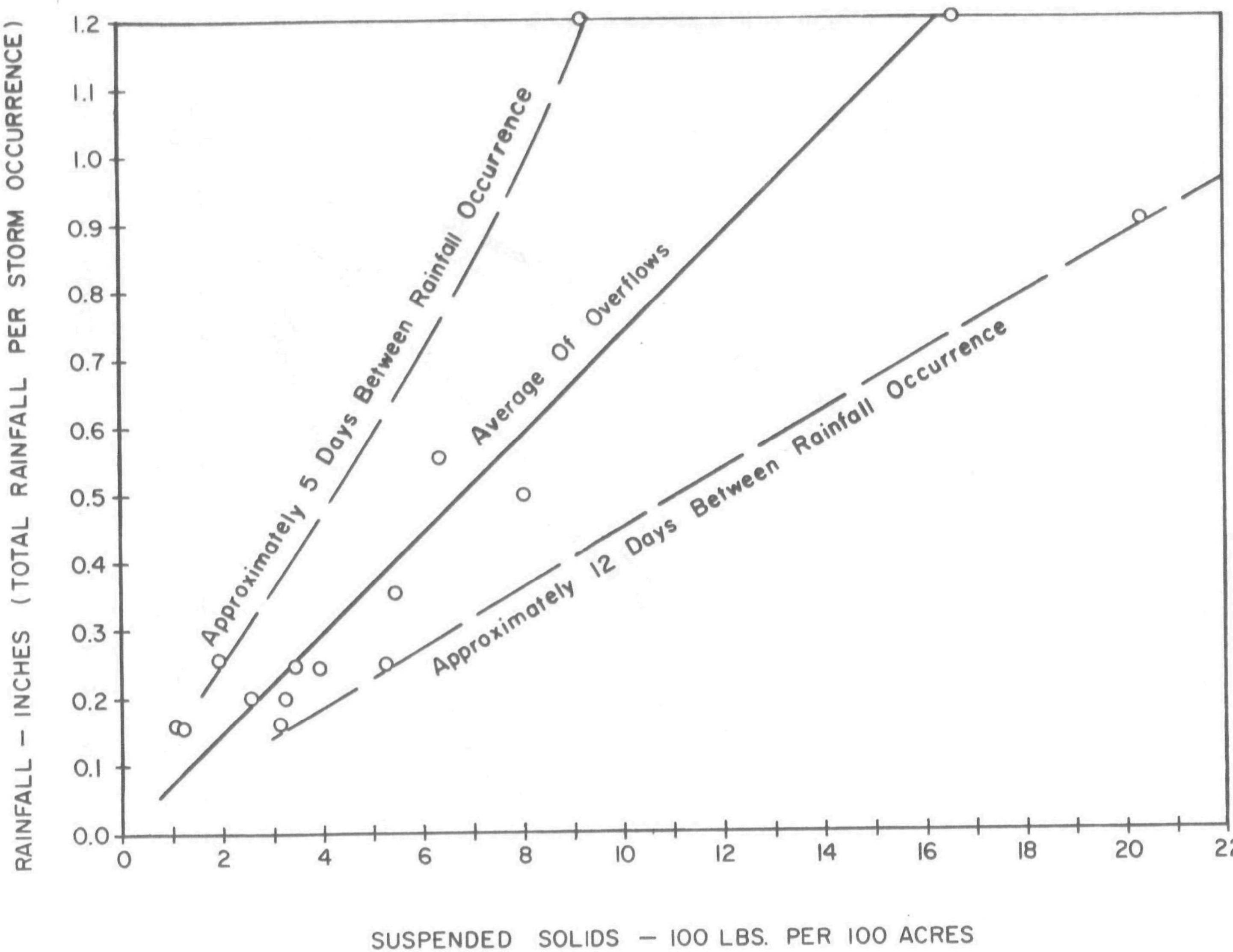


Figure No. 15
Rainfall vs. Suspended Solids

minutes, increased the BOD concentration of the Sandusky River downstream ffom Bucyrus from 11 mg per liter at river flow of 9 c.f.s. to 51 mg per liter at a river flow of 130 c.f.s.

4. The combined sewers will overflow about 73 times each year, discharging an estimated total annual volume of 350,000,000 gallons, or about 1,000,000 gallons per day.

5. The combined sewer overflows have an average BOD of 120 mg per liter, suspended solids of 470 mg per liter. Total coliforms of 11 million per 100 ml and fecal coliforms of 1.6 million per 100 ml.

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

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

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

9. The nitrate nitrogen concentration of the Sandusky River, upstream from Bucyrus, varied from a low of 0.4 mg/l as NO_3 to a high of 32 mg/l. The

high concentrations occurred during high river flows in the spring of the year. The estimated nitrate nitrogen discharged from the upstream drainage area is 2,300,000 pounds annually.

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

From this investigation and study the design storms and waste loads have been determined for Bucyrus and are shown in Table 3. An interceptor Sewer and Lagoon System is proposed. The benefits from controlling pollution due to combined sewer overflows by the use of an "Interceptor and Lagoon System" are many.

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

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

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

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

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

The total cost of the proposed Interceptor Sewer and Lagoon System is about \$5,200,000 compared to \$9,000,000 for sewer separation.

TABLE 3

DESIGN STORMS AND WASTE LOADS

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

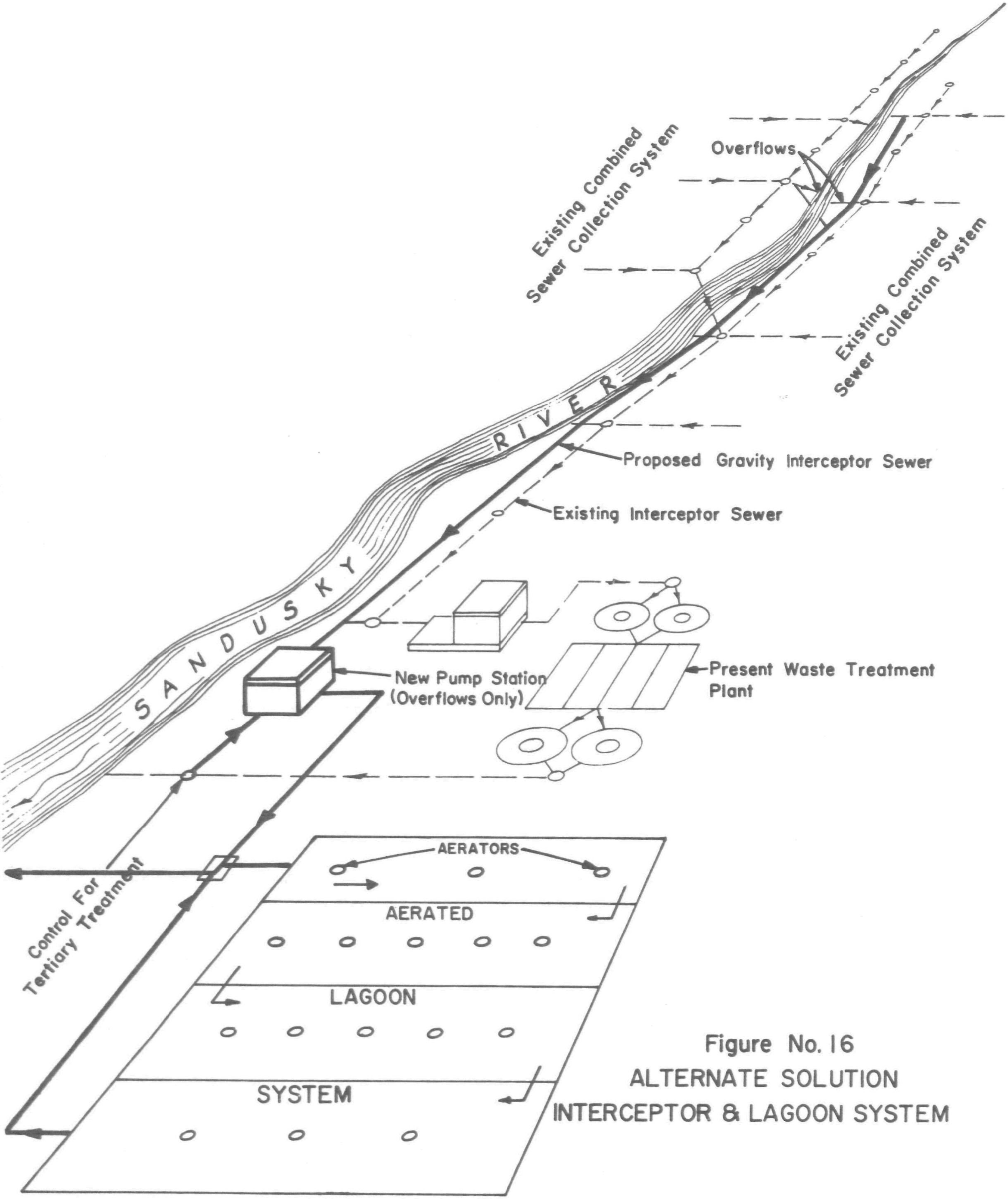


Figure No. 16
ALTERNATE SOLUTION
INTERCEPTOR & LAGOON SYSTEM

SECTION 8

ORGANIZING FOR SOIL EROSION AND SEDIMENT CONTROL IN OUR NATION'S URBAN AREAS

by

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National Association of Counties
Research Foundation

for the

STORM AND COMBINED SEWER SEMINAR

United States Department of the Interior
Federal Water Quality Administration

Chicago, Illinois

June 22-23, 1970

America has reached a point where it can no longer ignore the very real environmental dangers that are created as by-products of technological growth. In few places are these dangers to the environment more real than in the watersheds and river basins within which the nation's massive urban and suburban growth is taking place.

In the nation's suburban areas, extensive alternation of the landscape and intensive use of the land have resulted in serious imbalance between soil and water. As a result, erosion and sediment, traditionally considered as exclusively agricultural problems, have become serious problems in urban and suburban America.

Despite the fact that suburban erosion and sediment cause extensive pollution of water bodies, cost millions annually in damages to homes, roads and recreational areas and in some areas even threaten domestic water supplies, very few localities in the nation have organized and implemented erosion and sediment control measures.

As a means of encouraging positive action at the local level, the Federal Water Quality Administration awarded the National Association of Counties Research Foundation a grant to investigate the problem of sedimentation and to develop a guidebook for local policy making officials. The guidebook published in March 1970 describes the administrative, legal and organizational tools available and necessary to successful soil erosion and sediment control programs. The concepts and strategies recommended are based on actual programs in effect in various localities. Our findings, therefore, are based on numerous on-site examinations and interviews with state, county and municipal officials, soil and water conservation people, as well as homebuilders and roadbuilders.

The purpose of the guidebook is to assist local policy-making officials in making sound resource management decisions by providing comprehensive information on the various aspects of control programs in the belief that local officials share much of the responsibility for proper resource management.

Erosion and Sediment are Local Problems

Erosion and sediment inflict heavy damages upon local governments, businesses and citizens. The financial costs to local communities caused by these problems has been staggering in many areas. These costs are born by the local communities either through higher taxes or direct expenditures to repair damage to private property. Many of these costs are unnecessary in that through proper planning and organization, much of the damage can be prevented.

I think we can agree that citizens elect local government officials to take the necessary steps to prevent damages to their property. Local officials, therefore, share much of the responsibility for preventing such damage.

Increasingly, local officials are being reminded that they must assume their share of the responsibility for managing the natural resources of their community. The federal and state governments can and do offer guidelines and assistance to local areas. However, the bulk of the responsibility for proper management of the environment must come from local officials, as well as concerned citizens, professional conservationists, businesses and industries. The role of local officials is important in this connection because they are primarily responsible for making the basic policy decisions with regard to how local resources are allocated and used. As the demand of local resources, such as land, increases, the demand for wise decisions with regard to their use will also increase.

Organizing for Control

There are two important organizational features of local control programs which we might note. First, organizational approaches may vary considerably from place to place in accordance with local variations in physical, historical, legal, political, financial and demographic characteristics. To be effective, a control program will have to be organized on the basis of these local conditions. The first condition that should be considered is the nature and extent of the local problem. This may mean that research will be needed to determine where and how much soil is being eroded, what the effects of sediment are, and what measures can be taken.

Second, control programs should be organized in such a way as to include all interested and relevant groups and agencies. There are several reasons for this, the most important being that this approach encourages cooperation. The experience of several on-going control programs suggests that the importance of cooperation cannot be overemphasized. It has been a key factor in the success of many county efforts. An example is available in Montgomery County, Maryland, which organized and implemented a pioneer sediment control program in 1965. Their program, which is now being used as a model by other counties, made cooperation among local agencies the key factor.

Local leaders emphasize that the program would be ineffective in its present form without a high level of involvement and cooperation by numerous local, state and federal agencies and groups. Local groups involved in this cooperative program include the local homebuilding association, the county soil and water conservation district, citizens' groups, conservation societies, a water management agency, a planning commission and a department of public works, among others.

The Multi-disciplined Approach

Because so many groups and individuals are intimately associated with the problems, it may be appropriate to organize a control program on a basis which will encourage the participation of all groups and agencies which have an interest in, or which will be affected by, the control program once it becomes operational. There are several reasons for this which have been substantiated by the experiences of several communities. First, by involving as many groups as possible, the chances are lessened that the program will divide those who contribute to the erosion and sediment problem and those who are attempting to correct it. In most cases these positions are not clear, and in many cases they are interchangeable; that is, those who create sediment, such as homebuilders, may also be attempting to control sediment, and those who are responsible for controlling sediment, such as local governments, are also causing it. Thus, a value of the task force approach is that it helps to promote a unified and realistic recognition of the nature of the erosion and sediment problem.

Another reason for obtaining involvement of diverse groups in the control program is that the manpower resources available to the program can be increased by utilizing personnel from various participating groups. To be effective, a control program requires professionally trained personnel. The task force approach makes available personnel trained in various disciplines when they are needed. For example, soil scientists and other technically trained personnel are often available on a cooperative basis from local soil conservation districts; hydrology experts are available from state departments of water resources or their equivalents; departments of public works are normally staffed with professional engineers, as are the homebuilding organizations; and planning agencies can contribute professional planners from their staffs. Citizens' groups are also sources of manpower and can carry out important responsibilities in connection with public education programs.

Another reason for encouraging involvement is to reduce resistance. Where several groups work together, the resulting control program is likely to be more effective.

The Role of Local Government

Establishing control over erosion and sediment involves making decisions with regard to how local resources are to be planned, allocated and used. Specifically, a control program will frequently affect land use policy, the quality of water resources, and will require the use of local funds to support the control effort.

A control program therefore represents a form of resource management which must be regarded as a responsibility of local government officials. But the responsibility of local officials is to develop the organizational and procedural forms for gaining control over erosion and sediment. The following four (4) steps suggest some of the important aspects of organizing a control program:

(1) formal recognition by local elected officials of the need for erosion and sediment control;
(2) formulation of administrative and legal controls;
(3) assignment of specific responsibilities to local agencies;
(4) provisions for on-site inspection of erosion and sediment sources, including procedures for evaluating the effectiveness of the control program and for maintenance of control devices.

Looking at step one closely, formal recognition of the need for erosion and sediment control by local elected officials serves several purposes, each of which is important to effective control. First, formal recognition represents an official statement that erosion and sediment problems do exist. In order for local agencies to exercise effective control, they will need the support of local officials who have the responsibility for making policy decisions related to land-use activities. Second, a formal recognition and acknowledgement of a need for control by local officials serves the purpose of establishing the position of the "public interest" in favor of control. This, in effect, forms the justification for specific follow-up legislation. A third purpose served by a formal recognition of control is related to timing. Once erosion occurs, and sediment is yielded, the damage is done. It is not possible to halt damage at this point, and costly repairs usually result, frequently at public expense. A formal recognition of control serves to notify the general public that, henceforth, efforts will be made to control erosion and sediment.

Step two--It is not usually possible to develop a control program within a short period of time. Hastily developed provisions may be ineffective if they are either too demanding or not demanding enough. One characteristic of successful control programs now in operation is that the legal provisions and administrative procedures, which constitute the backbone of control programs, have not been abruptly imposed but instead have been developed to their present form over a long period of time.

Compliance with control legislation will be difficult to enforce unless the community is aware of and understands the need for regulation. A period of time will be needed for the members of the community to become acquainted with the control program and how it affects them. This time period can also be used to make adjustments in the program to meet unanticipated problems. Accordingly, it may prove helpful to launch the program on a voluntary basis for a period of time in order that various parties can make appropriate adjustments. Legal provisions and administrative procedures can then be made firm at a later date when initial difficulties have been worked out. A balanced and flexible approach to this aspect of the program's development process may be important to overall success.

For many types of suburban erosion, controls may be implemented by placing stipulations within subdivision regulations. These stipulations set in motion a series of administrative and operational activities designed to control sediment yields by limiting erosion from subdivision developments and other construction activities. Some local governments have curtailed erosion and sediment from housing developments by stipulating within their subdivision regulations that homebuilders must include within their preliminary subdivision plans adequate provisions for control. The preliminary plans are reviewed by appropriate local agencies for approval and recommendation. In this way, protection is built into the subdivision planning process and protection is provided, in most cases, before construction begins.

It may be necessary to make use of grading regulations in cases where grading of land will contribute to the problem. Grading permits may be used to regulate the timing of development, the extent to which grading operations may disturb the soil, and may also regulate sloping operations and vegetation removal. Normally, the issuance of grading permits is a function of a department of public works. Specific control specifications are usually technical in nature and generally are of interest only to those parties having a direct interest in controlling sediment. These detailed standards can be printed and made easily available to builders, local agencies and other interested parties. These standards, however, should be supported by general standards which are part of local legal codes.

Other methods that local governments can use to control erosion and sediment include land-use planning policies and certain types of zoning.

Step 3--Once appropriate legislation is incorporated into legal codes, it will be necessary to implement the regulations by assigning administrative responsibilities to appropriate local agencies.

Under the task force approach, responsibility for administering controls is shared by several local agencies. Specific responsibilities may be assigned according to the capabilities of each respective agency.

A frequently used approach is to require that the local planning commission review the subdivision plans to evaluate the probable effectiveness of control measures proposed by the builder. Frequently, copies of the subdivision plans are made available to the department of public works where they are reviewed for erosion control. In some control programs, subdivision plans are forwarded to local soil conservation districts where soils experts and other professionals review proposed erosion control measures and make recommendations for improvement when necessary. Other local agencies, such as sewer and water agencies, are also included.

There are sources of erosion and sediment other than those which are caused by private developers. Construction of public facilities, such as highways, sewers, and public buildings are major causes of suburban soil erosion. To be effective, an erosion and sediment program will need to control these causes as well as others.

Highway erosion is widespread in the United States, and various technical means have been developed to help control this costly problem. In many instances highway departments can receive technical assistance from local conservation agencies. This is sometimes arranged by intergovernmental agreements between appropriate local agencies. Intergovernmental agreements can also be used in efforts to control erosion from construction of public facilities, such as schools and other public buildings, and sewers, and they can also be employed to coordinate municipal and county control efforts.

In order to achieve comprehensive control, sediment from public as well as private causes will need to be curtailed. It is important that public agencies take the leadership in controlling sediment caused by their own construction activities.

Step 4--To be effective, a control program will need to provide for on-site inspection of construction activities. In most cases, this function is carried out by a local agency, such as the public works department. The inspection function serves the purpose of identifying problems, examining control devices, and evaluating the effectiveness of various control techniques. Evaluation of the program's effectiveness is necessary in order that modifications in the program can be made to improve control.

Leadership

If a community chooses to use the task force as an organizational approach, it is important that the task force group select one organization or individual to provide leadership. This step is necessary in order to provide the coordination needed in the program, to schedule and conduct meetings, to help orchestrate various activities, and to ensure continuity in the overall program. One important function of the leadership may be to ensure that an objective evaluation of the program is conducted periodically. The leadership should see to it that the program proceeds on a continuous basis and that guidelines and needs are provided for future task force group activities.

I have attempted to describe some considerations involved in organizing a control program and I have briefly discussed some specific organizational structures that may be suitable, with modifications, to a variety of local situations.

The multi-disciplined approach to organization outlined represents only one pattern that may be employed in efforts to control sediment. However, the experiences of several communities using this approach indicates that it is an attractive organizational format which helps to promote full community involvement and cooperation so necessary for effective control.

Let me reemphasize that erosion and sediment problems in suburban and developing areas can be brought under control, in most instances. What is most needed at this time is an effort toward building a widespread understanding of the problem at the community level, and the desire and energy to construct a workable community-wide system to administer available human resources.

Community Action Guidebook for Soil Erosion and Sediment Control

We have divided the guidebook into ten chapters. The first nine chapters address a specific aspect of control. We have attempted to refine, for local officials, their role in relation to erosion and sediment control specifically, and their role with regard to overall resource management generally. Following is a complete copy of Chapter Ten, which is, in effect, an action plan presented in outline format and incorporating the key elements of the previous nine chapters.

Action Guide for Erosion and Sediment Control

LOCAL GOVERNMENT'S ROLE

Environmental quality has deteriorated so seriously that local governments now have only two choices: to conduct effective environmental control programs at the local level, or to pass local responsibility and authority for control programs to state and federal levels by default. Local officials should provide leadership to their departments and to their communities in maintaining a clean environment and managing local resources.

Very few local governments have accepted responsibility for developing sedimentation control programs, and consequently, experience with erosion and sediment control in urban areas has not been extensive. However, citizens are beginning to demand that community resources, including soil and water, be properly managed. Since the county is an areawide unit of government, serving urban, suburban, and rural citizens, county officials are in an excellent position to respond to the public's demand by establishing effective areawide sedimentation control programs.

The guidebook is based on 10 months of research, including on-site visits to local sedimentation control programs across the nation, to state level control operations, and to various federal agencies. In addition, the guidebook is based on the recommendation made by 200 experts in water quality and soil conservation at the National Conference on Sediment Control, held in Washington, D.C., September, 1969.

This chapter represents a synthesis of sedimentation control concepts, principles, and techniques, which can be converted into general action plans by local, state and federal levels of government.

WHAT SHOULD LOCAL GOVERNMENT DO?

Local elected officials can establish a sedimentation control program by taking the following basic steps:

I. Appoint a Task Force.

Local elected officials should begin their sediment control program by appointing a sediment control task force to develop recommendations for the program.

In most existing urban programs, this task force was made up of individuals from the planning commissions, water and sewer agencies, home builders associations, soil conservation districts, professional engineers associations, contractors groups, U.S. Soil Conservation Service, State Department of Water Resources, and others concerned with the problem.

II. Establish Task Force Objectives. The Task Force should fulfill the following basic objectives:

(1) Determine through physical and demographical studies the nature and extent of the local sedimentation problems.
(2) Determine existing erosion and sediment control practices exercised by local public agencies, and private developers contractors.
(3) Determine what state and local laws exist regarding water pollution and land use.
(4) Decide what should be done by local governments, areawide government, and private industry, and how they can best cooperate in carrying out the program.
(5) Insure that development and construction activities do not result in environmental pollution.

III. How to Proceed

(1) See that the program is premised on providing control for the totality for every watershed lying, whole or in part, within local jurisdictions.

Frequently, the county is the areawide unit which meets this requirement. Where a single county is not large enough to solve the areawide sediment control problem, the multi-county approach may be best. In some large metropolitan areas where erosion and sediment problems cross jurisdictional boundaries, councils of government may offer an excellent vehicle to stimulate local officials to think, plan, and act in broad terms of mutual problem areas and to encourage jurisdictions to effect a mutually complementary system for sedimentation control.

Sometimes special purpose governments may be used because of their expertise in erosion control. If a special purpose government must be used, it is better to work through existing special purpose governments (where possible) rather than to create new ones.

Jurisdictions can cooperate through various techniques: by jointly performing some or all aspects of the control program; by contracting between cities and counties; and by transferring responsibility for a function from one level of government to another. Through these and other techniques, local governments can take advantage of economies of scale to implement an areawide control program.

(2) Determine whether necessary legal authority has been delegated by the state. If state enabling legislation is not adequate, officials should do as much as possible within existing law and decide what changes are needed. Then, they can work through their state association of counties and other interested groups for passage of comprehensive sedimentation control enabling legislation.

The legal basis for local governments to control land use is state enabling law. Without this enabling authority, local governments cannot acquire land, develop facilities, or spend public funds to regulate and control erosion and sediment. To ensure that local governments have the necessary powers, legislation should allow political subdivisions to manage sediment in coordination with other .environmental protection programs.

Home rule cities and counties must closely examine their charters to be sure they have the authority to plan, regulate, and operate a sedimentation control program.

State legislation should give local government authority to:

(a) acquire land, buildings, and facilities by purchase, lease, eminent domain, and donation;
(b) plan and zone for the protection of watersheds and natural drainage courses;
(c) adopt and enforce necessary ordinances, rules, and regulations;
(d) use various sources of revenue such as bonds, taxes, general appropriations, fees and service charges, and state and federal assistance programs;
(e) make intergovernment agreements and contracts;
(f) regulate private contractors and developers through the issuance of permits and licenses;
(g) prohibit any type of environmental pollution.

(3) Require that soil and water conservation considerations be incorporated incorporated in community plans. Plans may be prepared by an interagency committee of interested departments, by a single department, by a consultant, or by a combination of local departments and consultants.

Community plans should include:

(a) data on population, land use, transporation, and public facilities and utilities;
(b) considerations of the climate, topotraphy, geology, and related factors, with the technical assistance of any needed specialists so that development and construction activities are not detrimental to the community's land and water resources;
(c) presentation and evaluation of feasible immediate and long-range solutions.

(4) Require that development and construction project plans be prepared in coordination with community plans.

Project plans should include specifications for needed erosion and sediment control measures.

(5) To prepare the best possible plans and achieve implementation, elected officials should:

(a) solicit cooperation on an areawide basis from city and county planners, public works agencies, health officers, engineers, soil conservation districts, other appropriate departments, and interested citizens;
(b) plan to inform the public about the need for a comprehensive erosion and sediment control program;
(c) provide leadership and initiative to ensure acceptance and implementation of the plan.

(6) Decide what type of organization is needed and assign operating responsibilities.

No one organizational pattern for erosion and sediment control can be said to be best. Local conditions and custom will determine which one or combination of agencies can be assigned responsibility for administration of the control program. The sedimentation control agency or agencies must be responsible to elected officials of general purpose governments. Regardless of organization, the following functions must be performed: policy making; public information; budgeting; planning and review; drafting, adoption, and enforcement of standards; and operation of the system.

The main criterion for determining what place a sedimentation control program should have in the orgainzational structure of a local government that existing agencies should be used to carry out the program rather than creating a new agency.

(7) Obtain technical information on current community plans, and the community's geological, topological, and soil conditions.

The program should stress the physical limitations of every development and construction site. Also, this should be considered in all land-use decisions. Basic principles would include the development of large areas in small, workable increments, the holding of exposure time to a minimum and adapting site plans to the natural topography.

Timely installation or structures, storm drains, streets, and gutters is necessary plus applicable conservation measures, such as the use of mulch (as a temporary cover), temporary seedings, early installations of permanent vegetation, and the use of temporary structures, terraces, waterways, and debris basins.

(8) Prepare a financial plan and capital budget so that both immediate operating expenditures and long-range capital financing needs are provided for.

Although much of the cost for providing sedimentation control will be assumed by private industry (i.e., developers and builders), local government will still be responsible for providing control related to public improvements and for their maintenance, e.g., parks, reservoirs, open channel linings, etc.

Since the system must be financed within the constraints of state laws and local charters these should be thoroughly examined during the planning process. Local governments can finance the system when necessary following methods: taxes, bond issues, loans, and/or service charges. The local capital improvement budget should schedule the financing of all necessary control facilities and equipment.

If the sedimentation control program is operated on an areawide basis, economies of operation will often benefit each jurisdiction.

(9) Find out what federal, state, and private technical and financial assistance is available and take advantage of it.

Technical assistance from federal state, and private sources is available to local governments to develop measures related to sediment control. On the federal level, the primary sources of financial and technical assistance are the Department of Agriculture the Department of the Interior and Department of Housing and Urban Development. Imaginative use of assistance from other federal agencies may provide help for local sedimentation control.

Many states provide technical assistance for soil and water conservation through conservation districts and other special purpose governments such as flood control districts. While financial assistance is currently limited, recent appropriation trends indicate a growing response to environmental needs.

The home building industry, universities, professional societies and private organizations also can provide information and assistance.

(10) Direct the program's agencies to respond quickly to all citizen complaints and conduct a continuing educational program to inform the public about the need for land use control in relation to water pollution control.

(11) Use as many public information tools as possible to reach citizens.

Among these tools are meetings at which slides and films are shown; creation of events such as "go-see" trips; speakers bureaus; brochures and flyers; radio, T.V., newspapers, and newsletter coverage and announcements; exhibits; and communications media endorsement.

(12) Employ a qualified committee of representatives from public agencies, citizens groups, and industry to periodically review, evaluate, and report on the effectiveness of the program.

(13) Survey recruitment needs. Where they exist, solicit personnel from other levels of government, professional organizations, and universities. Also, technical manpower may, in many cases, be involved in the program as a form of technical assistance from other local, state and federal government agencies.

(14) In-house training will be needed for program personnel, especially for planners, and regulatory and maintenance personnel. It should be noted that during the development of these ordinances and the program, local soil conservation districts are available to work with local public agencies, consultants, and engineers in the design and installation of erosion control practices.

IV. Make the Sedimentation Control Program Developed by the Task Force State Local Government Policy.

Charge local government department heads with responsibility for developing policies and procedures designed to implement the program, and solicit the voluntary cooperation of the building industry.

Sedimentation control programs to date that appear to work best are those that initially evolve from some type of voluntary action. Urban sedimentation control is a new field and all concerned need an opportunity to test their ideas. Where developers, planners and conservationists have an opportunity to cooperate voluntarily on erosion control projects, a solid foundation for future regulatory program is provided for.

V. Make Sediment Control Mandatory Through Adoption of an Ordinance or Land-Use Regulations.

The responsibility for developing the ordinance or land-use regulation can best be assumed by the Task Force. Also Task Force members know the existing regulations and they have developed the basic guidelines for the voluntary program.

The ordinance or land-use regulation, when developed, would set the local standards. They should be conceptual in scope; flexible in methods; positive in direction; prohibitive of any type of land or water pollution; and above all, they must be clearly understandable. They should be designed to control the occasional irresponsible developer.

The ordinance or regulation should designate the local agencies to be responsible for enforcing the standards, e.g., plan, review and inspection.

WHAT SHOULD STATE GOVERNMENT DO?

I. Provide comprehensive state enabling legislation to permit counties to manage soil and water resources. Also, counties should be permitted and encouraged to contract with internal municipalities and other counties to develop areawide sedimentation control programs.

Develop clear state guidelines with regard to sedimentation standards. Water quality standards, based on federal guidelines (Federal Water Pollution Control Act of 1956, the Water Quality Act of 1965, and the Clean Water Restoration Act of 1966) have been adopted by all 50 states. States should ensure that criteria for sedimentation control be included in these standards.

III. Provide financial and technical assistance to local sedimentation control programs. Such assistance can be delivered through state agencies, should help local programs conduct watershed research, conduct soils studies, and provide major capital improvements, etc.

IV. Develop and execute an information distribution program. Local governments and their agencies, planning commissions, soil conservation district personnel, etc., need to be informed on state laws and their interpretation, what state assistance is available, state policy guidelines, state planning programs and other state activities.

V. Offer traning to local government and private industry in sedimentation control techniques and principles.

VI. Develop and enforce a state sedimentation control program to help control erosion and sediment on all state projects and activities including highway construction and maintenance, and state building projects.

WHAT SHOULD THE FEDERAL GOVERNMENT DO?

I. Help to promote national recognition of urban erosion and sediment as constituting a major threat to environmental quality.

II. Continue to contribute to technical and non-technical research programs related to all aspects of urban erosion and sediment problems.

III. Continue and improve upon financial and technical assistance programs for state and local governments.

IV. Develop and enforce a federal sedimentation policy to help control erosion and sediment on all federal projects, and federally sponsored projects, including federal buildings, federal highways, and on all federal lands and waters. Sedimentation control policy should be enforcible on all appropriate federal contracts, whether carried out by public or private agencies.

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