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EVALUATION OF FLOW EQUALIZATION IN MUNICIPAL WASTEWATER TREATMENT

by

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

Variations in flow rate and composition are characteristics of wastewaters treated at municipal treatment facilities. Improved efficiency, reliability, and control of various physical, chemical and biological treatment processes are believed possible at or near constant plant conditions. This publication presents data gathered from all sewage treatment plants having equalization facilities that could be located throughout the United States and analyses of plant operations are made. Analysis procedures and design principles are presented to cover the spectrum of conditions to which equalization may be applicable.

Francis T. Mayo, Director
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EXECUTIVE SUMMARY

The executive summary presents a detailed overview of the major segments of the manual including purpose and scope, procedures, supporting data, and major recommendations.

<u>Section Contents Description</u>

- Section 1: Introduction; establishes the purpose and intent of the manual. Basic problems are defined and included with major problem subcategories. The purpose of providing the manual is identified, and the scope of subject material coverage is established.

- Section 2: Design and Operation Practices Recommendations; typical wastewater system situations requiring evaluation for applicability of equalization are described. Conditions favoring applicability of equalization are summarized according to magnitude of input variations, characteristics of sewer systems, size and type of treatment facilities, etc. Recommendations are made concerning size, type, location, and required appurtenances of equalization facilities appropriate for typical sewerage and treatment system configurations.

- Section 3: Quantitative Methodology; presents and summarizes established procedures for sizing equalization facilities as a function of typical or critical influent variations. Quantitative comparison is made between flow and concentration smoothing afforded by in-line and side-line equalization configurations. Methodology is presented for evaluating collection system flow variations to determine when equalization is appropriate and when collection system improvements are dictated. Procedures are outlined for using treatment plant operating data to establish effects of equalization on unit process and overall treatment plant performance.

- Section 4: Facilities Summary; contains results of the nationwide survey of equalization facilities. Data presented define the characteristics of existing equalization facilities and the nature of applications in terms of basic sewerage system and treatment system features.

* Section 5: Performance Evaluation; presents all available information collected from the literature, past studies not published, and data from operating treatment facilities that could be used to identify effects of flow equalization on individual treatment processes and overall treatment plant performance. Criteria for selecting and evaluating operating data used are summarized. Detailed data on the performance of individual plants is presented and analyzed. General operating performance of plants with equalization is compared to plants not having equalization. Theoretical effects of influent variations on unit process and treatment system performance are summarized as a guide to evaluating equalization performance.

* Section 6: Equalization Cost; unit costs of basic equalization facility types and conventional appurtenances are presented as developed from the national survey and Brown and Caldwell design files. Capital costs and costs for operation and maintenance are provided. Examples are given to illustrate cost comparison of treatment facilities designed with and without equalization.

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CONTENTS

Disclaimer ii
Foreword iii
Executive Summary iv
Figures ix
Tables xv
Acknowledgments xvii

1. Introduction 1
Background and Purpose 1
Background 1
Basic Problem Categories 2
Equalization Design Requirements 3
Purpose of the Manual 9
Scope of the Manual 9
Coverage of Subject Material 9
Information Sources Used 10
Guide to the User 10
Table of Contents 10
Executive Summary 11
Section Contents Description 11
2. Conclusions and Recommendations 12
3. Quantitative Methods 14
Determination of Equalization Applicability . 14
Facility Planning 14
Design Flow Peaking Considerations 16
Stormflow Dominance Determination 20
Analysis of Storm Dominated System 24
Equalization Sizing Methods 26
Method 1--Simple Flow Balance 27
Method 2--Simple Concentration Balance . 37
Method 3--Combined Flow and Concentration Balance 38
Method 4--Sine Wave Method 49
Method 5--Rectangular Wave Method 50
Specialized Analytical Methods 52
Measurement and Evaluation of Equalization Effects 53
Introduction 53
Performance Evaluation 53
Performance Requirements 58
References 60

4. Summary of Existing Flow Equalization Facilities 62
Equalization Facilities Survey 62
Survey Scope 62
Survey Response 86
Equalization Facilities Summary 86
Treatment Plant Characteristics 86
Equalization Facility Characteristics . . 90
Equaliation Facilities Construction Costs . . 95
Construction Costs 95
Operation and Maintenance Costs 96
5. Equalization Performance Evaluation 107
Case Histories 107
Introduction 107
Activated Sludge Plants 108
Trickling Filters 167
Oxidation Ditch Plants 173
General Performance Observations 176
Equalized Versus Unequalized Plant Performance 179
Activated Sludge Plants 179
Trickling Filter Plants 190
Effects of Equalization on Unit Process and Treatment System Performance 190
Significant Flow Equalization Benefits . 197
References 211
6. The Development of Costs for Equalization and Treatment Processes 213
Basis of Unit Cost Development 213
Construction Costs 214
Holding Tank and Basin Costs 215
Wastewater Pumping Costs 218
Aeration and Mixing Costs 218
Washdown and Solids Removal Costs 221
Additional Capital Costs 221
Operation and Maintenance Costs 221
Comparison of Equalization Costs and Treatment Plant Peaking Capacity Costs 224
References 233

FIGURES

Number		Page
1(a)	Schematic flow diagram of equalization facilities: in-line equalization	6
1(b)	Schematic flow diagram of equalization facilities: side-line equalization	6
2	Dependence of extreme flow ratios in municipal sewers on population	17
3	Peaking effects on combined sewered areas	19
4	Daily peak flow annual distribution	21
5	Daily peak flow monthly distribution	23
6	Simple flow balance schematics	28
7	Wastewater flow variation before equalization	29
8	Hydrograph for Example 4	30
9	Flow and concentration iteration schematics	40
10	Wastewater flow and BOD variation before equalization	43
11	Examples 4 and 5, flow and concentration balance	46
12	Example 6, excess volume for load equalization	48
13	Equalization volume estimation by sine wave and rectangular wave models	51
14	Distribution of log daily average influent TSS concentrations, Ypsilanti Township, Plant 1, 1974	56
15	Range difference for distributions of equal slope	56

16 Distribution of log effluent concentrations for activated sludge plants designed to meet EPA secondary treatment requirements 59

17 Flow equalization facility locations 87

18 Equalization facilities construction cost as a function of equalization capacity 100

19 Operation and maintenance cost as a function of equalization volume 105

20 Operation and maintenance cost of equalization facilities as a function of treatment plant capacity . 106

21 Distribution of weekly TSS concentrations, EPA in-house study (P/A = 1.5) 111

22 Distribution of weekly BOD_5 concentrations, EPA in-house study (P/A = 1.5) 112

23 Distribution of weekly TSS concentrations, EPA in-house study (P/A = 2.0) 113

24 Distribution of weekly BOD_5 concentrations, EPA in-house study (P/A = 2.0) 114

25 Distribution of weekly TSS concentrations, EPA in-house study (P/A = 2.5) 115

26 Distribution of weekly BOD_5 concentrations, EPA in-house study (P/A = 2.5) 116

27 Distribution of 8-day TSS concentrations, Walled Lake, Michigan/Novi, Michigan, 1974 119

28 Distribution of 8-day BOD_5 concentrations, Walled Lake, Michigan/Novi, Michigan, 1974 120

29 Distribution of TSS concentrations, Walled Lake, Michigan/Novi, Michigan, 1974 (8-day) and 1976-1977 121

30 Distribution of BOD_5 concentrations, Walled Lake, Michigan/Novi, Michigan, 1974 (8-day) and 1976-1977 122

31 Distribution of 8-day TSS loads, Walled Lake, Michigan/Novi, Michigan, 1974 123

32 Distribution of 8-day BOD_5 loads, Walled Lake, Michigan/Novi, Michigan, 1974 124

33 Equalization tank, 1 million gallon capacity, Tecumseh, Michigan 126

34 Distribution of TSS concentrations, Tecumseh, Michigan, 1970 and 1973 127

35 Distribution of BOD_5 concentrations, Tecumseh, Michigan, 1970 and 1973 128

36 Distribution of TSS concentrations, Tecumseh, Michigan, 1970 and 1976 129

37 Distribution of BOD_5 concentrations, Tecumseh, Michigan, 1970 and 1976 130

38 Distribution of TSS loads, Tecumseh, Michigan, 1970 and 1973 131

39 Distribution of BOD_5 loads, Tecumseh, Michigan, 1970 and 1973 132

40 Distribution of TSS loads, Tecumseh, Michigan, 1970 and 1976 133

41 Distribution of BOD_5 loads, Tecumseh, Michigan, 1970 and 1976 134

42 Distribution of TSS loads, Tecumseh, Michigan, 1973 and 1976 135

43 Distribution of BOD_5 loads, Tecumseh, Michigan, 1973 and 1976 136

44 Distribution of TSS concentrations, Ypsilanti Township, Michigan, 1974-1975 140

45 Distribution of BOD_5 concentrations, Ypsilanti Township, Michigan, 1974-1975 141

46 Distribution of TSS loads, Ypsilanti Township, Michigan, 1974-1975 142

47 Distribution of BOD_5 loads, Ypsilanti Township, Michigan, 1974-1975 143

48 Distribution of TSS concentrations, Ypsilanti Township, Michigan, 1976 144

49 Distribution of BOD_5 concentrations, Ypsilanti Township, Michigan, 1976 145

50 Equalization tank (background), 3 million gallon capacity, Pontiac, Michigan 146

51 Distribution of TSS concentrations, Pontiac, Michigan, East Boulevard Plant, 1973 and 1976-1977 148

52 Distribution of BOD_5 concentrations, Pontiac, Michigan, East Boulevard Plant, 1973 and 1976-1977 149

53 Distribution of TSS concentrations, Pontiac, Michigan, Auburn Plant, 1973 and 1976-1977 151

54 Distribution of BOD_5 concentrations, Pontiac, Michigan, Auburn Plant, 1973 and 1976-1977 152

55 Equalization basin, 3 million gallon capacity, Amarillo, Texas 153

56 Distribution of monthly TSS concentrations, Amarillo, Texas, 1964 and 1966 154

57 Distribution of monthly BOD_5 concentrations, Amarillo, Texas, 1964-1966 155

58 Distribution of monthly TSS loads, Amarillo, Texas, 1964 and 1966 156

59 Distribution of monthly BOD_5 loads, Amarillo, Texas, 1964 and 1966 157

60 Equalization tank, 50 million gallon capacity, Warren, Michigan 158

61 Distribution of TSS concentrations, Warren, Michigan/Renton, Washington 160

62 Distribution of BOD_5 concentrations, Warren, Michigan/Renton, Washington 161

63(a) Primary effluent BOD concentration distribution, Newark, NY 164

63(b) Primary effluent TSS concentration distribution, Newark, NY 165

64 Primary sedimentation removal efficiency distribution, Newark, NY 166

65 Distribution of TSS concentrations, Palmyra, New Jersey 168

66 Distribution of BOD_5 concentrations, Palmyra, New Jersey 169

67 Distribution of daily TSS concentrations, Midland, Michigan/Bay City, Michigan 171

68 Distribution of daily BOD_5 concentrations, Midland, Michigan/Bay City, Michigan 172

69 Distribution of monthly TSS concentrations, Arlington, Washington/Dawson, Minnesota 174

70 Distribution of monthly BOD_5 concentrations, Arlington, Washington/Dawson, Minnesota 175

71 Secondary and filter effluent TSS concentration distributions, four equalized flow plants 180

72 Secondary and filter effluent BOD concentration distributions, four equalized flow plants 181

73 Distributions of average annual effluent TSS concentrations, 31 equalized versus 46 unequalized activated sludge plants 188

74 Distributions of average annual effluent BOD concentrations, 31 equalized versus 46 unequalized activated sludge plants 189

75 Distributions of average annual TSS concentrations, 8 equalized versus 65 unequalized trickling filter plants 192

76 Distributions of average annual BOD concentrations, 8 equalized versus 65 unequalized trickling filter plants 193

77 Classification of responses to loading variations by wastewater treatment processes 199

78 Estimate on limits on soluble organics removal in activated sludge 201

79 BOD removal as a function of organic loading for activated sludge modifications 205

80 Effect of hydraulic loading on stone media trickling filter BOD removal 206

81 Effect of hydraulic loading on plastic media trickling filter BOD removal (35) 207

82 Sedimentation tank effluent TSS versus solids loading - normal hydraulic loadings 209

83 Construction cost as a function of concrete holding tank size 216

84 Construction cost as a function of concrete lined earthen basin volume 217

85 Capital cost as a function of pump station capacity 219

86 Surface aerator capital cost as a function of rated capacity 220

87 Equalization basin washdown system details 222

88 Equalization basin washdown system capital cost as a function of system capacity 223

89 Equalization facility operation and maintenance costs as a function of flow equalization facility volume 225

90 Treatment systems used in equalized vs. unequalized facilities cost comparison 227

91 Regions of cost effective equalization for cost example, treatment system 5 232

92 Regions of cost effective equalization for cost example, treatment system 9 232

TABLES

Number		Page
1	Example 2--Tabular Method	32
2	Example 3--5-Year Storm Effect	35
3	Example 3--Tabular Method, First Case	35
4	Example 3--Tabular Method, Second Case	36
5	Beginning Flow and Concentration Iterations: Example 4, In-Line Basin	45
6	Beginning Flow and Concentration Iterations: Example 5, Side-Line Basin	45
7	Equalization Facilities Summary: Plants Smaller than 1 mgd--Treatment Plant Characteristics	64
8	Equalization Facilities Summary: Plants Smaller than 1 mgd--Equalization System Characteristics	67
9	Equalization Facilities Summary: Plants Larger than 1 mgd--Treatment Plant Characteristics	71
10	Equalization Facilities Summary: Plants Larger than 1 mgd--Equalization Facility Characteristics	76
11	Plants with Equalization Information Incomplete	84
12	Construction Cost Data for Flow Equalization Facilities	97
13	Operation and Maintenance Costs for Equalization Tanks	101
14	Operation and Maintenance Costs for Equalization Basins	103
15	Treatment Facilities for Performance Evaluation	109

16 EPA Inhouse Study: Peak-to-Average (P/A) Flow and Load Ratios 110

17 Effluent Discharge Requirements for Walled Lake-Novi Plant 117

18 Mean BOD and TSS Load and Concentration Summary . . . 137

19 Effects of Equalization on Performance of Activated Sludge Plants 177

20 Effects of Equalization on Variability of Activated Sludge Plants 178

21 Activated Sludge Plant Performance 182

22 Trickling Filter Plant Performance 191

23 Summary of Alternatives in Improving Existing Treatment Plant Performance 194

24a Equalization Cost Effectiveness Example 228

24b Equalization Cost Effectiveness Example 229

24c Equalization Cost Effectiveness Example 230

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SECTION 1

INTRODUCTION

BACKGROUND AND PURPOSE

Background

Equalization as the term is applied to wastewater treatment, refers to facilities and procedures for minimizing variations in the flow rate and composition of wastewater processed at municipal treatment facilities.

Variations occur characteristically in domestic wastewater flow rate and composition as a result of cyclic activities of the human population. Additional variations are commonly imposed by a combination of (1) random and cyclic activities in the collective industrial-wastewater-generating segment of the community and (2) by storm-related effects of infiltration and inflow. In addition, the average wastewater flow rate at typical municipal treatment plants may be expected to increase by 25 to 100 percent or more over the design life of the facilities. These variations and resulting problems are accepted in wastewater treatment, and the vast majority of municipal treatment plants today routinely operate under such conditions.

Operation of wastewater treatment plants at or near constant conditions is commonly assumed to be advantageous. Improved efficiency, reliability, and control of various physical, chemical and biological treatment processes are believed possible under such conditions. Cost savings are assumed to result from elimination of excessive peak treatment capacity and from reduced periods of operation under peaking conditions. Examples supporting these assumptions are widespread in chemical process industries and in water treatment where constant operating conditions are maintained routinely and permit process optimization.

Equalization is not a new idea. Industrial applications of equalization using lagoons, in-line sewer capacity and tanks were discussed in early work by King, 1942 (1); Rudolfs, 1943 (2), 1946 (3); and Gurnham, 1955 (4), respectively. However, municipal applications of equalization received little attention until the advent of the Federal Water Pollution Control Act amendments of 1972. The Act has resulted in stricter water quality standards requiring more extensive, more sophisticated, and more reliable wastewater treatment. This in turn has contributed to significant interest in the potential of flow equalization for improving performance and reducing capital and operating costs at municipal wastewater treatment plants.

Basic Problem Categories

Equalization Definition--

For the purposes of this study, equalization will be defined as any facilities and procedures for minimizing variations in the flow through treatment plants, as long as the total flow is ultimately processed through the treatment plant. Consideration is limited to municipal wastewater treatment facilities. The term "equalization" is applied most commonly to minimizing diurnal variation at wastewater treatment plants regardless of the source of variation. Minimizing variations during storm-influenced periods only is commonly referred to as storm flow retention, or as combined sewer overflow control where variations result from the existence of combined sewers. These practices will also be considered as equalization as long as storm flows are ultimately processed by the normal treatment system, and not bypassed receiving only partial treatment before disinfection and discharge. Procedures for identifying storm flow characteristics that can be accommodated by equalization are detailed in Section 2. Temporary storage of industrial waste, or other occasional sludge discharges that would be detrimental to normal treatment, is also considered as equalization where the stored flow is ultimately processed by the normal treatment facilities. Applications of this type are common in communities with appreciable manufacturing or food processing industry for example.

Equalization Applications--

Equalization may be applied either as a means of upgrading existing facilities, or as an integral component of entirely new facilities. Planning and design considerations for the respective applications differ markedly. Upgrading of treatment plants may be required for one or more of three major reasons (5): first, to meet more stringent treatment requirements; second, to increase hydraulic and organic loading capacity; and/or third, to correct or compensate for performance problems resulting from improper plant design and/or operation. Equalization may help a plant attain higher effluent quality by:

- permitting process optimization and improving performance of existing treatment components;
- improving reliability by minimizing flow and load peaking and/or reducing or eliminating bypassing;

* reducing effects of shock loading of toxic or other upsetting influent waste components.

Increased effective hydraulic and organic loading capacity may result from allowing continued operation at constant average flow in treatment units that have reached capacity under peak flow conditions.

Equalization may help to overcome design deficiencies and reduce operational problems by:

* compensating for one or more design deficiencies more economically than correcting the deficiencies themselves;

* providing for simplified operation, thus minimizing potential difficulties from operational error.

Equalization is one of many alternatives available for application to each of these problem categories. Procedures for identifying problem areas and evaluating available alternative solutions, including equalization, are detailed in the U. S. Environmental Protection Agency's (EPA) Technology Transfer Manual on upgrading existing wastewater treatment plants (5).

Equalization may be included in new treatment facilities:

* to assist in achievement of effluent requirements; particularly where effluent requirements are strict, and where advanced and sensitive treatment processes are included;

* to minimize peaking capacity of planned treatment components;

* to permit process optimization, and to simplify operational requirements.

Application of equalization in the design of new facilities is not constrained by existing physical facilities. Accordingly, the most favorable application is permitted consistent with current knowledge of hydraulic, siting, operation, and maintenance factors.

Equalization Design Requirements

The design of equalization facilities requires evaluation and selection of a number of features:

* type and magnitude of input variations
* required volume
* facility configuration

- pumping/control mode
- type of construction
- appurtenances; aeration, mixing, odor control, cover, flushing
- cost and benefits

Design decisions must be based on the specific details and unique requirements of each individual plant. Major influencing factors will include the type and degree of treatment employed, local site conditions, and cost relative to benefits in comparison to feasible alternatives.

Quantitative procedures are available for establishing ability to equalize input variations according to their type and magnitude, and for determining required equalization volume. Criteria for establishing equalization benefits in terms of effects on unit process and treatment plant performance are presented in Section 2 along with illustrative examples. Costs of equalization construction, operation, and maintenance have been developed from an extensive survey of existing facilities and are detailed in Section 5. Equalization facility configurations, construction type, and appurtenance requirements are governed by constraints of existing or planned facilities and conventional design fundamentals.

Equalization Facility Configuration--

Equalization may be accomplished by an assortment of different kinds of facilities and procedures. The type of system employed may be dictated to a significant degree by the nature of existing collection and/or treatment facilities, by siting conditions or constraints, or by required characteristics of planned facilities.

Equalization may be accomplished in conjunction with elements of the tributary sewer system. Excess capacity in major interceptors may be used to smooth flows to the treatment plant. To justify development of the storage capacity available volume must be a significant fraction of required equalization volume. Such conditions are more likely to exist in combined sewer systems. Facilities required to take advantage of available storage capacity may range from automatic flow regulating gates with real time computer control, to manually controlled gates or inflatable dams. Variable speed pump stations in conjunction with excess interceptor capacity or wet well volume, with suitable controls may also be used to reduce flow peaking. The use of an in-sewer storage remote from the treatment plant for equalization will require regular procedures to prevent occurrence of problems resulting from accumulation of solids. Nightly drawdown of the storage system, with flushing where required, is essential to prevent excessive discharge of accumulated solids during daytime peak flow and loading periods.

Equalization capacity may be provided in the form of off-line storage tanks located at appropriate points in the collection system. This may provide economical relief for overloaded collection system components in addition to equalizing downstream flow. Location of such tanks adjacent to required pump stations minimizes duplication of facilities. Tanks located within the community remote from the treatment are generally relatively expensive due to required appurtenances to control solids accumulation, provide for fail safe operation, and prevent odor and aesthetic problems.

Currently the most common means of providing equalization is through the use of specially designed basins at sewage treatment plants. Tanks are located near the headworks to provide equalization benefits for all downstream units. Location downstream of screening and grit removal eliminates the need for handling accumulations of such materials. Location upstream of primary clarifiers provides optimum conditions for clarifier operation, but requires equipment to prevent excessive solids accumulation and to maintain aerobic conditions. Either mechanical mixing or diffused aeration, suitably designed, can satisfy both requirements. Equalization located downstream of primary clarifiers may be the most economical and troublefree application. Although constant flow benefits would not be available to the primary units, their performance is relatively insensitive to flow peaking compared with secondary or tertiary treatment components. Lower capital and operating costs, and reduced maintenance requirements for storing primary effluent without problems of solids accumulation may outweigh disadvantages of operating with normal (unequalized) primary effluent variability.

Location of equalization basins following secondary treatment may be justifiable in special circumstances. Where all or part of a plant's secondary effluent is reclaimed or processed by tertiary treatment components, particularly where influent peak-to-average flow ratios are low, equalization of this type may be practical. In such cases equalization provides protection against poor effluent quality due to upsets in the biological secondary treatment.

Equalization basins may be designed as either in-line or side-line units. In the in-line design, Figure 1(a), all flow passes through the equalization basin. This system uses the pump station to provide essentially constant flow through the plant. Since continuous pumping is required, design must be coordinated with other influent pumping requirements to eliminate costly duplication. In the side-line design, Figure 1(b), only flows greater than the daily average are diverted to the equalization basin. Depending on unequalized plant pumping requirements, this scheme may minimize additional pumping required.

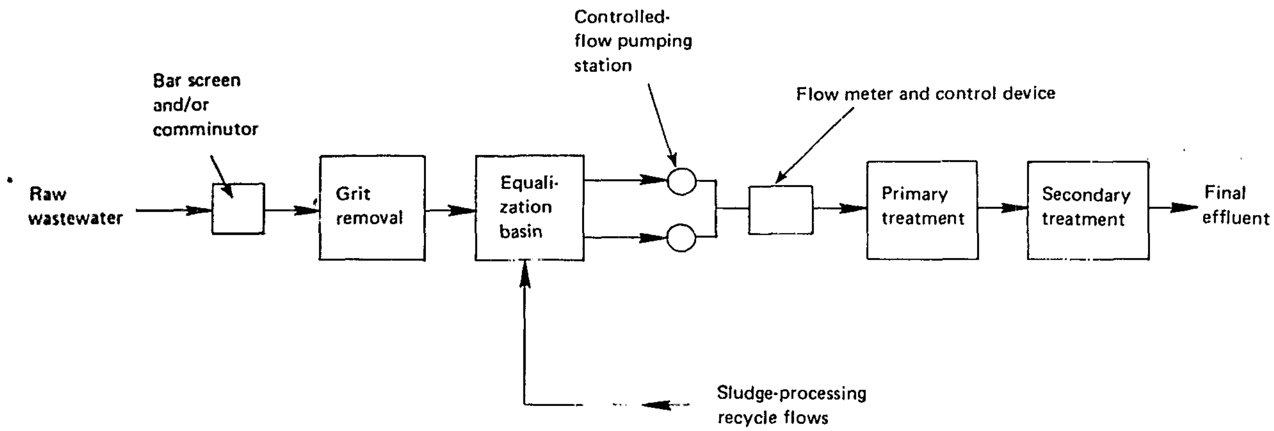

Figure 1(a) Schematic flow diagram of equalization facilities: in-line equalization

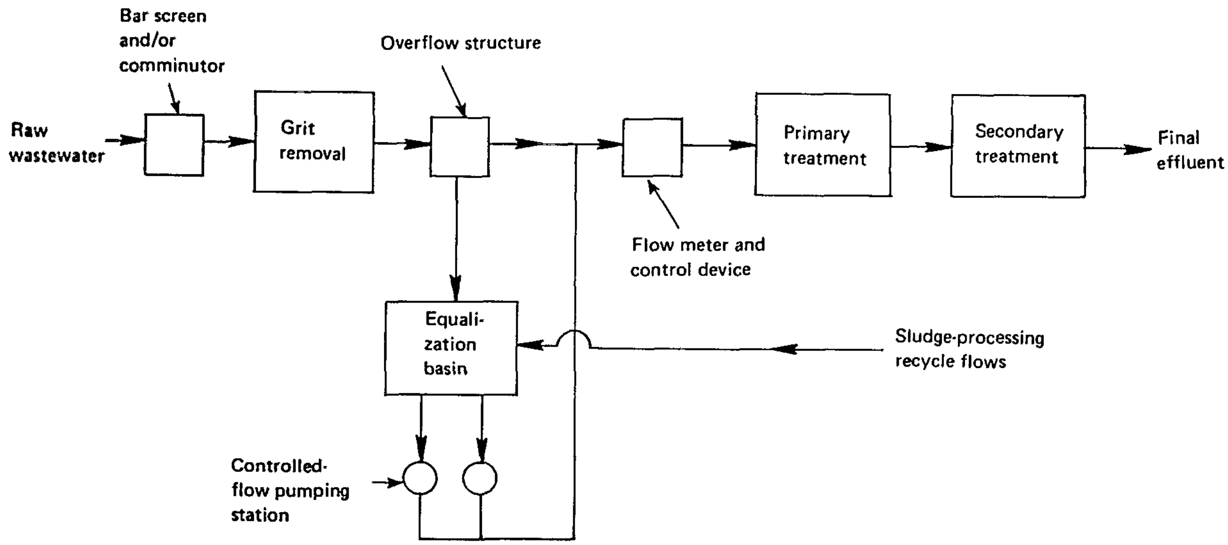

Figure 1(b) Schematic flow diagram of equalization facilities: side-line equalization

Where equalization facilities are used to provide protection against toxic or process upsetting materials, side-line facilities would be required.

Provision for variable volume in key process units may satisfy requirements for dry weather equalization. The activated sludge process is adaptable to this type of application. Significant treatment capacity reductions, as well as improved performance, may be possible when applied to package type extended aeration treatment systems.(6) Application to an oxidation ditch system has been shown to contribute to excellent overall plant performance while requiring relatively low capital expenditure.(7)

Pumping and Control Mode--

Addition of equalization to a wastewater treatment plant adds to the total head required for plant operation. The head required is equal to the sum of the maximum water surface level variation and dynamic losses through the equalization system. Additional head may be required for dewatering depending on the equalization configurations. Very few treatment plant locations afford sufficient head for operation without additional pumping.

Pumping requirements are established by plant and siting constraints, and by the equalization configuration selected. Required head may be developed either by pumping into or out of equalization. Designing equalization tanks to fill by gravity and empty by pumping permits filling even at excessive peak rates and allows installed pump capacity to be minimized. In order to maintain desired constant flow effects, effluent pumps generally require variable speed drives to accommodate day-to-day and long-term changes in influent conditions.

Filling by means of pumps, and emptying by gravity requires pump capacity to accommodate anticipated peak flows. This configuration also generally requires variable speed pumping to take full advantage of desired flow smoothing. Gravity discharge systems require regulating controls for effluent flow in addition to influent pump controls. Effective flow control requires location of a flow measuring device downstream of equalization to monitor final flows. Instrumentation and controls should be provided to maintain preselected equalized flow rate with a minimum of operator attention by automatic adjustment of pump and valve settings.

Type of Construction--

Equalization basins can be provided through the construction of new facilities or by modifying existing facilities of sufficient volume. Equalization may be implemented with relative ease in an upgrading plan that calls for the abandonment of existing tankage. Facilities that may be suitable for conversion to equalization basins include aeration tanks, clarifiers, digesters, and sludge lagoons.

New basins may be constructed of earth, concrete, or steel. Earthen basins are generally the least expensive. They can normally be constructed with side slope varying between 3:1 and 2:1 horizontal to vertical, depending on the type of lining used. Drainage facilities should be provided for ground water control to prevent embankment failure in areas of high ground water. Precaution should be taken in design to prevent erosion in large basins where a combination of aerator action and wind forces may cause the formation of large waves. It is also customary to provide a concrete pad directly under the equalization basin aerator or mixer. The top of the dikes should be wide enough to insure a stable embankment. For economy of construction, the top width of the dike should be sufficient to accommodate mechanical compaction equipment.

In-line basins should be designed to achieve complete mixing in order to maximize concentration damping. Elongated tank design enhances plug flow and should be avoided where concentration of load damping is desired. Inlet and outlet configurations should be designed to prevent short circuiting. Designs which discharge influent flow as close as possible to the basin mixers are preferred.

Compartmentation--

Design of equalization should follow established sanitary engineering practices, dividing required volume into two or more compartments or basins. This permits dewatering for maintenance and repair without interrupting service, and allows for operating flexibility. Where equalization is designed to accommodate wet weather flows, compartmented tankage allows dry weather equalization using only a portion of the facilities, helping to minimize maintenance requirements. When upgrading, existing facilities tanks being considered for abandonment should be analyzed carefully to determine possible suitability for equalization.

Aeration and Mixing--

The successful operation of both in-line and side-line basins may require mixing and aeration if placed upstream of primary clarifiers. Mixing equipment should be designed to blend the contents of the tank, and to prevent deposition of solids in the basin. To minimize mixing requirements, grit removal facilities should precede equalization basins wherever possible. Aeration is required to prevent the wastewater from becoming septic. Mixing requirements for blending municipal wastewater having a typical suspended solids concentration of approximately 200 mg/l range from 0.02 to 0.04 hp per 1,000 gallons of storage. To maintain aerobic conditions, air should be supplied at a rate of 1.25 to 2 ft^3/min per 1,000 gallons of storage.(8)

Mechanical aerators are one method of providing both mixing and aeration. The oxygen transfer capabilities of mechanical aerators operating in tap water under standard conditions vary from 3 to 4 pounds O_2 per horsepower-hour. Baffling may be necessary to insure proper mixing, particularly with a circular tank configuration. Minimum operating levels for floating aerators generally exceed 5 feet, and vary with the horsepower and design of the unit. Low-level shutoff controls should be provided to protect the unit. The horsepower requirements to prevent deposition of solids in the basin may greatly exceed the horsepower needed for blending and oxygen transfer. In such cases, it may be more economical to install mixing equipment to keep the solids in suspension and furnish the air requirements through a diffused air system, or by mounting a surface aerator blade on the mixer.

It should be cautioned that other factors, including maximum operating depth and basin configuration, affect the size, type, quantity, and placement of the aeration equipment. In all cases, the manufacturer should be consulted.

Purpose of the Manual

The purpose of this manual is to assemble and disseminate all available information on equalization applications to municipal wastewater treatment developed to date. The manual is not simply a compilation of data; rather, available information from all sources has been reviewed and evaluated and, combined with experience of the investigators, reasonable conclusions are presented. Procedures for evaluating equalization applicability and performance are presented. Information on benefits and costs of equalization have been analyzed. Recommendations concerning equalization in the planning and design of municipal sewerage systems are presented for consideration and use.

Information developed and presented in this manual rests on experience gained through municipal applications of equalization occurring almost exclusively over the last five years. Many new and recently installed equalization systems in an increasingly broad spectrum of applications will add significantly to understanding equalization uses and utility. It may well be that continuing developments in the field will require the revision of this manual in the future. Knowledge of treatment system performance related to equalization and equalization system performance is not exhaustive. However, the body of current knowledge is extensive and provides a firm basis upon which integrated treatment systems may be planned.

SCOPE OF THE MANUAL

Coverage of Subject Material

This manual presents information gathered from all sewage treatment plants having equalization facilities that could be located throughout the United States. Analysis and recommendations are based largely on this information. Analysis procedures and design principles along with case examples are presented to cover the spectrum of conditions to which equalization may be applicable. The vast diversity of individual collection system-sewage treatment plant combinations prevents exhaustive coverage of all possible conditions. Attention is concentrated on equalization facilities or systems located at treatment plants, because of the predominance of existing applications and the greatest potential effectiveness. Biological secondary treatment plants and such plants with tertiary treatment elements (most commonly nutrient removal and effluent filtration) have received the most widely distributed application of equalization. The bulk of the remaining applications are those upstream of treatment plants including in-line flow control and pump station control applications. The entire spectrum of system sizes is covered from package facilities serving limited commercial or residential developments, to facilities with major metropolitan service areas.

Information Sources Used

Information used in the preparation of this report included:

- Current project reports and data accumulated and supplied by the U. S. EPA Municipal Environmental Research Laboratory.
- A national survey to identify existing and proposed equalization facilities and to provide information on their design, cost, and performance.
- Direct communication with operating personnel at treatment facilities throughout the country.
- Private communication with investigators active in the field.
- The general literature.
- Experience of individuals involved in preparation of this manual.

GUIDE TO THE USER

Table of Contents

The table of contents provides an overview of general subject coverage provided in the manual.

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SECTION 2

CONCLUSIONS AND RECOMMENDATIONS

1. Flow equalization is seldom the only alternative for dealing with real or anticipated performance problems at a wastewater treatment plant. Care must be taken to demonstrate that flow equalization is the most effective and least expensive alternative.

2. Flow equalization benefits may be categorized as follows:

   - Reduction of peaking requirements.
   - Reduction of process overloads at existing plants under some conditions.
   - Protection against toxic upsets.
   - Potential reduction of operational problems.
   - Provides increasing benefits with increasing plant complexity.

3. Feasibility of equalizing storm flows from infiltration/inflow sources and in combined sewer systems should be assessed, as would any treatment system component, using conventional infiltration/inflow analysis procedures.

4. Where equalization is used to provide for treatment of storm flow peaking in addition to equalizing diurnal flows, sufficient treatment capacity must be provided to permit emptying storage volume.

5. Side-line and in-line equalization provide approximately equal degrees of BOD load equalization where storage volume is provided to level only diurnal flow variations.

6. In-line equalization provides greater flexibility for influent waste load equalization using storage volume in excess of the minimum required for daily flow equalization.

7. Use of side-line equalization generally will not require duplication of in-plant pumping. In-line equalization will generally require an additional pumping station.

8. Placement of equalization following primary treatment minimizes operation and maintenance, and minimizes requirements for solids removal, aeration, and odor control equipment.

9. Application of flow equalization in an activated sludge system will more than likely not reduce the soluble organics concentration of the effluent from the biological process.

10. Although design factors considerably influence the acceptable loading range of activated sludge sedimentation facilities, flow equalization may provide increased TSS and organics (particulate fraction) removals when applied to systems with peak hydraulic loading rates beyond about 1,000 gpd/ft^2.

11. The removal of soluble organics in a trickling filter would probably not be enhanced through application of flow equalization ahead of the trickling filter.

12. Efficient removal of particulate materials in a trickling filter sedimentation tank would probably not be significantly improved by equalization of upstream flow.

13. The cost of the storage volume required for equalization of the peak flows is likely to be less than the cost of the incremental treatment capacity in plants, of all sizes, where the degree of treatment exceeds simple secondary requirements.

14. Flow equalization should be considered as an alternative to additional treatment capacity wherever influent P/A ratios exceed levels of 1.3 to 1.5:1.

SECTION 3

QUANTITATIVE METHODS

DETERMINATION OF EQUALIZATION APPLICABILITY

The applicability of equalization in any wastewater treatment situation depends on its ability to function as an integral element of the most cost effective treatment system available, and as required to meet imposed discharge requirements.

Facility Planning

Equalization provides an alternative to building excess plant capacity to accommodate input peaking; it also has additional benefits derived from plant operation under more stable conditions and an ability to control intermittent plant upsetting inputs.

Conventional facility planning and design procedures formulated to assess applicability of alternatives resulting in a recommended optimum treatment scheme are as follows:

1. Identify treatment requirements (discharge permits);
2. Identify influent conditions;
3. Establish design flows for treatment;
4. Identify all feasible alternatives for meeting treatment requirements;
5. Establish cost effective treatment scheme consistent with requirements;
6. Detailed design.

Identifying influent conditions (item 2) may include conduct of a Sewer System Evaluation Survey if the infiltration/inflow analysis element of the facility plan indicates potentially excessive infiltration/inflow; or if the system is combined, resulting in overflows of combined sewage, development of a program to control the overflows will be required. The outcome of either or both of these analyses will impact item 3, the flows that have to be accommodated at the treatment plant. The applicability of equalization is determined through this process by three major factors: discharge criteria establishing the degree and reliability of treatment required; the design capacity of the treatment plant; and plant input variation and peaking characteristics.

The degree of treatment required, and resulting feasible treatment process schemes, establish the potential magnitude of cost economy available by using equalization and minimum peaking condition design. As the degree of treatment required increases, the number of treatment processes with potential cost savings from reduced design peaking increases. Thus, the applicability of equalization tends to increase with increasingly stringent discharge requirements. It is essential in considering the potential of equalization to recognize the importance of both specific effluent discharge requirements and conventional design criteria for affected treatment components. If discharge requirements are all established on a 30-day or 7-day average basis, then treatment components can be designed for 30-day or 7-day average conditions in the critical month of operation. The success of such designs depends on better-than-required performance occurring in below-average flow periods, balancing the excess discharges occurring in the above-average flow periods.

For example, the conventional activated sludge process, with requirements only for carbonaceous BOD removal, may commonly be designed for average conditions in the most critical month. In such a case, equalization is of limited benefit. However, if discharge requirements specify absolute limits on effluent components, designs must ensure acceptable performance under peak loading conditions.

For example, activated sludge processes designed for biological nitrification and for denitrification have been shown (8) to require factors of safety equal to or greater than the influent peak-to-average ratio to ensure desired performance. In such cases equalization effectively reduces required design capacity, and its applicability then depends on the relative cost of equalization volume and treatment capacity. Specifically, applicability must be determined in each individual case according to design requirements for all unit processes in the treatment scheme affected by equalization. Examples of cost comparisons for treatment schemes with and without equalization are provided in Section 5. The result of this process is identification of the most cost effective treatment scheme for meeting imposed discharge requirements.

The general range of treatment plant capacity is a major factor in determining the type of treatment process to be selected, and in establishing the unit cost of treatment components. Generally, unit costs of treatment components decrease with increasing size (economy of scale). In addition, peak-to-average flow ratios typically decrease with increasing plant size. These factors tend to reduce the economic benefit of equalization as size increases. This may be balanced off by the economy of scale factors involved in required equalization volume. The relative costs of treatment and equalization for a range of treatment plant capacities are illustrated by examples in Section 5.

The magnitude of plant input variations and peaking characteristics, and to a more limited extent, the source of those variations, is a major factor in determining the applicability of equalization. As the peak-to-average ratio (or other suitable measure input peaking characteristics) increases, the need for peaking capacity in treatment components designed without equalization increases. Thus, the advantage of equalization is greater where input peaking is greater.

Where the source of peaking variations is due to groundwater and storms, the determination of the most cost-effective overall solution to reducing impacts of peaking on the treatment facility will require evaluation of alternatives within the collection system in addition to consideration of equalization measures at the treatment plant site.

Design Flow Peaking Considerations

The flow received at a treatment facility reflects the input to, and the characteristics of, the tributary collection system. The influent flow characteristics in turn directly affect the performance efficiency of the treatment facility. Hydraulic and organic loading variations, outside the design range of the treatment units, will result in variable effluent qualities. From a national perspective, the most common extreme loading variation encountered is rainfall influenced, specifically if the tributary system has combined sewer elements. The major loading parameter of rainfall-influenced flows is hydraulic, often accompanied by discharge of inorganic solids carried into the sewer with the drainage flow. Flow variations exceeding 100 times normal non-storm influenced wastewater flows can result from storm effects, forcing storm flows to become the controlling factor in determining the loading parameter for plant design.

In this section, guidelines are developed to identify those systems that are storm-flow dominated. Flow equalization analysis techniques for wastewater system optimization will differ depending on whether wastewater-influenced variations or storm flow are the dominant design factors.

Factors Affecting Influent Flows--

The influent flow at a treatment plant is made up of wastewater discharged by served customers, infiltration of groundwater entering the collection system due to system defects, and storm inflow entering the collection system from runoff during rainfall and/or snowmelt. The location and characteristics of the discharge from customers can easily be monitored, and in most cases predicted; but the same cannot be said for infiltration and storm inflow. The impact of all these flows measured at the treatment plant is dependent on the specific characteristics of the collection system.

Sanitary Sewer Design--Conventional sanitary-only collection system design is based on sizing sewers for the estimated peak sewage flow, plus an allowance to accommodate infiltration and inflow. As the number of tributaries flowing into a specific pipe section increases, the sewage peaking factor is reduced nominally by the effect of gravity flow in the sewer. Flow peaking and minimum flow variations from varying size communities, excluding impacts of industrial or storm flows, are shown in Figure 2. Diurnal peaking flows of up to 5:1 can be expected from small systems serving 1,000 people or less, reducing to 1.5:1 for populations of 1,000,000; decreasing in proportion to increasing population.

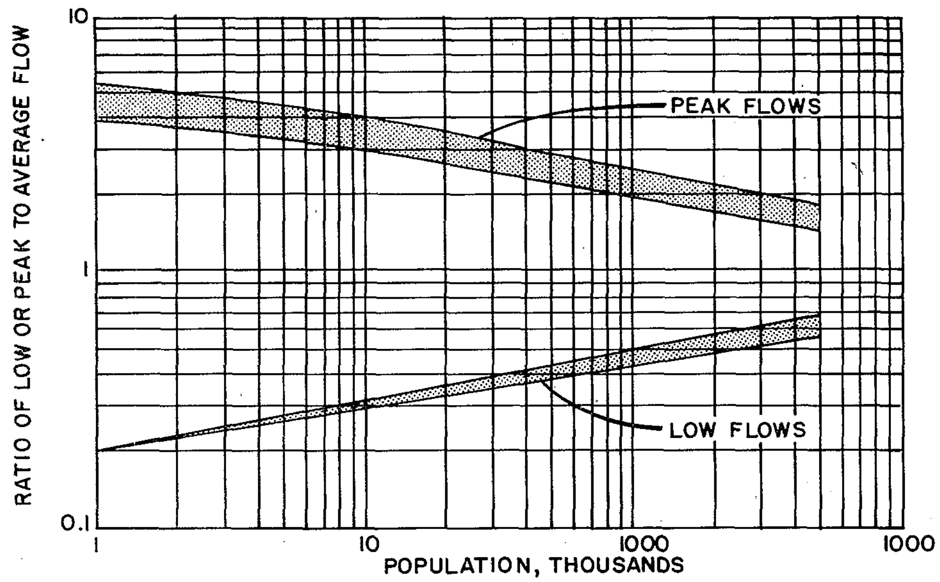

Figure 2. Dependence of extreme flow ratios in municipal sewers on population

Combined Sewer Design--Sewers designed for sewage plus storm drainage (combined sewers) are normally sized to accept all sewage flows and runoff from a storm having a specific intensity. The storm intensity is based on a storm of a specific occurrence frequency. Storm flows that exceed the design maximum are allowed to overflow at relief points throughout the collection system. In combined sewers the major capacity in the collection system is reserved for storm flows.

The hydraulic impact of combined sewers at a treatment plant is dependent on a combination of factors, namely:

1. The proportion of combined sewers within the tributary collection system;

2. Flow characteristics of the wastewater dischargers;

3. Development density;

4. Collection system size and hydraulic characteristics;

5. Rainfall patterns.

For the majority of collection systems that have tributary combined sewers the treatment plant peak flow is determined by the limits of the interceptor system transfer capacity. Flows exceeding collection system capacity overflow at relief points within the system. Because of the large flows generated by storm runoff, even limited areas of tributary combined sewers can have a dramatic impact on treatment plant flows during storm periods.

Assuming that no overflows occur a theoretical relationship can be developed to determine the impact of combined sewered areas on storm flow peaking characteristics for different values of development density, runoff characteristics of the combined sewered area and storm intensity. These factors have been combined in a simplified way in Figure 3 and assume a per capita sewage contribution of 70 gallons per day.

Figure 3 has been constructed using the following data as the base condition:

Line V1 - population density, 10/acre
Line V2 - combined sewered area percent impervious, 30 percent
Line V3 - rainfall intensity, 1.00 inch/hour

Line A, which joins the three variables V1, V2 and V3 together, expresses the relationship between flow peak-to-average ratio and percent combined sewer. For the above-assumed variables,

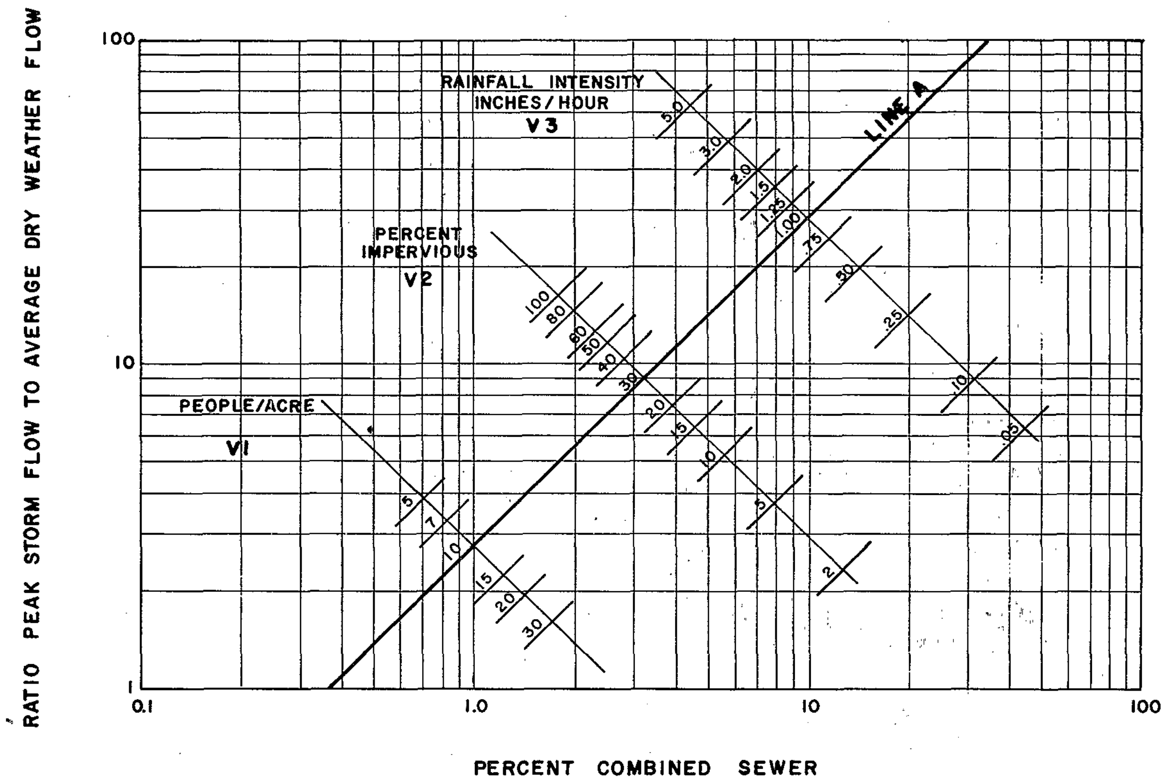

Figure 3. Peaking effects of combined sewered areas

(Assuming that no overflows occur)

this shows that with 10 percent of the tributary collection system combined, a peak flow ratio exceeding 30:1 can occur.

To use the figure for differing variables of V1, V2 and V3, use the following procedure as illustrated in the following example:

Assumptions: V1 - 7 people/acre
V2 - 20 percent
V3 - 1.5 inches/hour

Step 1: On V1 measure difference between 7 and Line A

Step 2: On V2 measure difference between 20 and Line A

Step 3: On V3 measure difference between 1.5 and Line A

Step 4: Sum the three values to determine net distance from Line A

Step 5: Draw line parallel to Line A offset by the amount determined in Step 4.

The line drawn in Step 5 is the relationship between storm flow peaking characteristics and percent combined sewer. This graph indicates that even small percentage combined sewered areas can produce storm flows far outside the hydraulic efficiency range of wastewater treatment units.

Treatment Plant Influent Flows--Influent interceptor capacity at a treatment plant is the critical peak flow controlling element of the sewerage system. Regardless of upstream conditions, flow at the plant can never exceed the influent sewer capacity. For the sanitary-only case of a system with no storm influences, designed in accordance with conventional criteria, peaking characteristics reflect the discharge to the system. For the system that is totally combined the maximum flow at the plant reflects the hydraulic limitations of the collection system network. Regardless of what is the hydraulic controlling factor, it is the flow characteristics at the plant that impact performance. In establishing the balance point for a specific system (whether it be storm flow dominated or wastewater dominated) the plant tolerance to all flow variations and not only the peaking characteristics of the plant influent has to be considered.

Stormflow Dominance Determination

The determination of whether a system is stormflow dominated requires the sequential consideration of the following:

1. Analysis of influent flow characteristics;

2. Determination of treatment plant process unit loading sensitivity.

This determination is significant, for if a system is storm dominated, then optimization of treatment requires the collection network to be included as an integral part of the analysis. The analysis emphasizes the treatment facility in the case of non-storm influenced systems.

Treatment Plant Influent Peaking Characteristics--
For any given treatment plant historic plant flow data should be analyzed, and peak hydraulic and organic loading peaking and volume characteristics identified. An example of a plot of annual hydraulic loading peaking characterisitcs for two similar size communities having measurable rainfall events of 50 times per year is shown in Figure 4. Community A has sanitary-only sewers; peak flows are contained within a small range with the exception of a few heavy storm days. Community B has tributary combined sewers; peak flows occur for all measurable rainfall events, with maximum flows being controlled

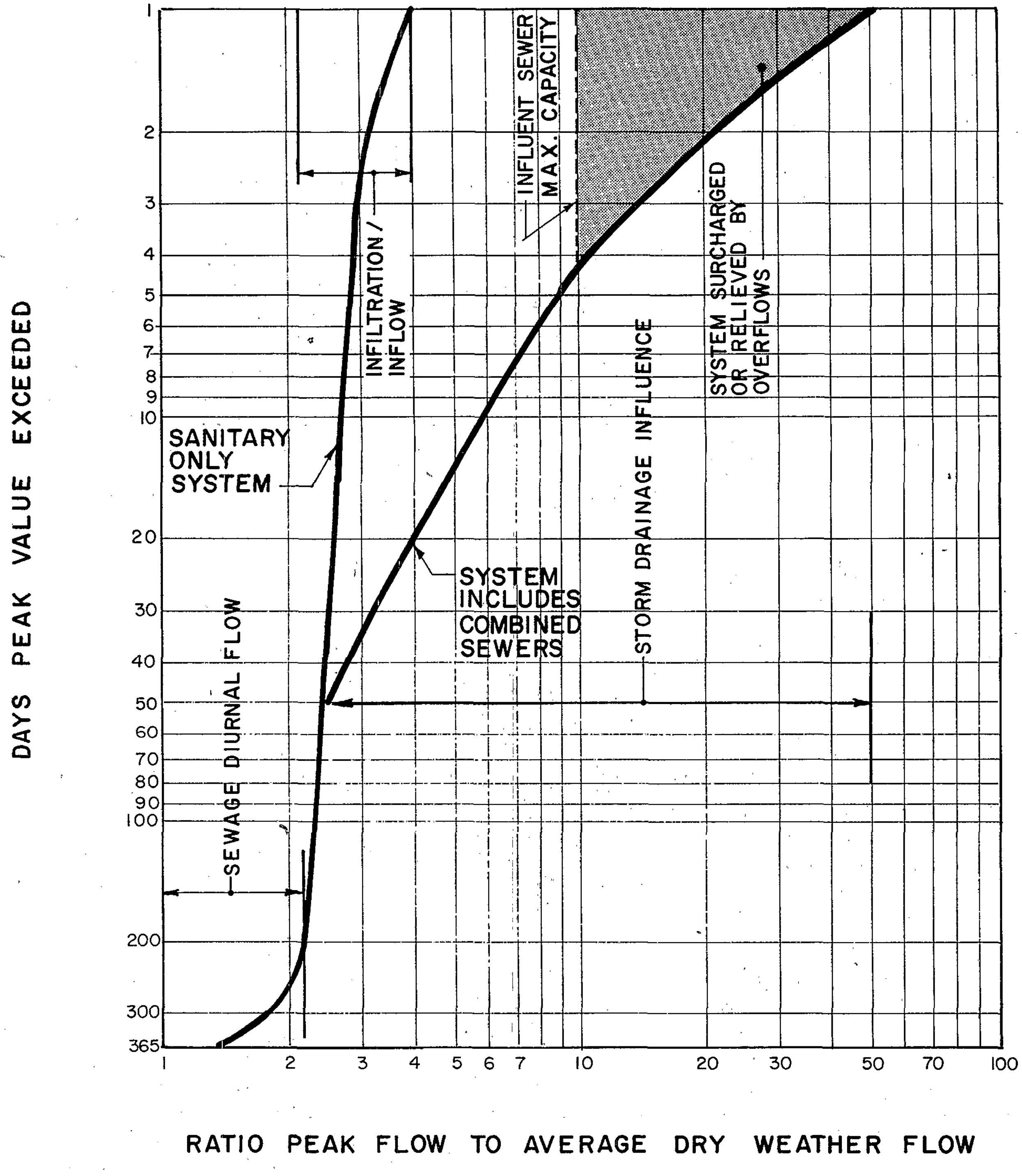

Figure 4. Daily peak flow annual distribution

by collection system limitations. Similar plots can be made for both peak organic and daily volume. The significance of the influent peaking characteristics is dependent on treatment unit sensitivity to peak loadings and the specific effluent quality requirements. Where discharge requirements are based on absolute limits, process units have to be sized to handle the peak loading. For suspended solids and biochemical oxygen demand, discharge requirements are normally based on monthly averages with higher allowances for 7-day averages and one-day values. Thus, process units can be sized for average conditions and intermittent peak loading-induced effluent deteriorations can be accepted even if they exceed the 30-day average values, but are within the one-day maximum allowance.

To evaluate the case where only average values have to be met requires a further plot by month of peak loading variation. An example for a community with seasonal infiltration/inflow for hydraulic peaking characteristics by month is shown in Figure 5. This example has plots for each month of minimum daily peak, peaks exceeded 7 times per month, 3 times per month and the monthly peak values. Similar plots can be made for organic and volume daily loadings depending on the treatment unit critical loading parameter. This data is applied to the performance characteristics of proposed treatment unit processes to determine critical loading conditions as described below.

Treatment Unit Sizing--

Process units are normally sized using average dry weather hydraulic and organic loadings. From analysis of the influent characteristics information plotted, as described above, apply the critical month loading data to the plant sized for dry weather only loads. Then check the deteriorated effluent qualities with the discharge requirements.

Case 1. If the discharge requirements can be met then the system is not stormflow dominated. The optimization of equalization basin-treatment plant sizing can proceed without further consideration of the tributary collection system as discussed under the heading of EQUALIZATION SIZING METHODS.

Case 2. If the discharge requirements cannot be met then the system may be storm dominated, requiring the following analyses.

Identify potential for operating the treatment plant on a temporary basis in differing modes. Examples are:

a. Contact stabilization for activated sludge systems;

b. Chemical addition to improve process unit performance.

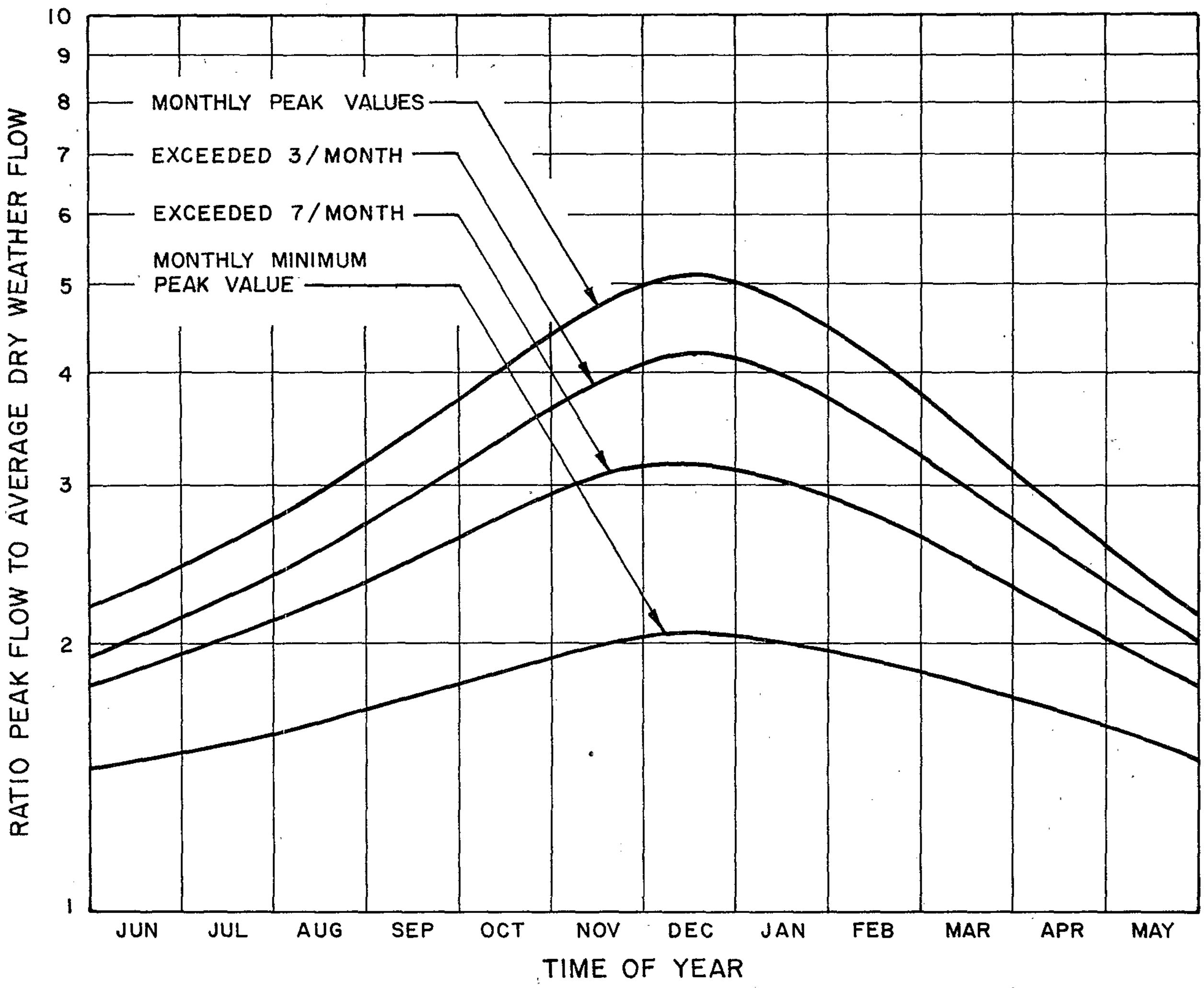

Figure 5. Daily peak flow monthly distribution

If effluent quality requirements can be met by changing the operation mode or temporary chemical addition, then the case would be considered non-storm influenced, and optimization of equalization basin-treatment plant sizing can proceed as discussed under the heading of EQUALIZATION SIZING METHODS.

If temporary operating changes are not sufficient to meet discharge standards then the system is considered storm dominated, requiring inclusion of the tributary collection system in the optimization of equalization considerations. The methodology for collection system analysis is described below.

Analysis of Storm Dominated System

For those systems that are storm dominated, optimal use of wastewater facilities requires that the analysis go beyond treatment plant/equalization basin capacity consideration, and include the collection system. The objective of including the collection system is to reduce the flows received at the treatment plant.

Literature has been published by federal and technical groups on approaches to analysis of storm-influenced and combined sewer systems. In most publications, equal emphasis is given to the hydraulic and pollutant constituents of the stormwater contribution. However, with respect to the analysis of storm flow dominated systems, it is the hydraulic contribution that is the significant parameter.

Infiltration and Storm Inflow--

An infiltration/inflow analysis is required for projects financed using EPA grants (PL 92-500, Title II, Section 201(g)(3)). In this analysis sources of infiltration and inflow are identified, quantified hydraulically, and deemed excessive or nonexcessive by economic evaluation. An infiltration/inflow source is considered excessive if costs for its removal exceed transport and treatment costs. Following conduct of the sewer system evaluation survey, grant conditions require that all economically excessive sources be removed.

When equalization is considered and found to be a cost-effective treatment component, the lower unit cost of treatment will result in shifting the treatment-sewer improvement balance so that more infiltration and inflow will be treated. The approach to be taken for analyses of infiltration and inflow in noncombined systems is covered in the EPA Handbook for Sewer System Evaluation and Rehabilitation, December 1975.(9)

For sanitary-only systems, and combined systems with no overflows, the analysis requires that the most cost-effective program be selected to address all infiltration/inflow. For systems with combined sewer overflows, the level of control is determined by a cost/benefit analysis. Specifically, EPA Program Requirements Memorandum 75-34 (10) states that controls will only be funded to a level where marginal benefits of overflow reduction exceed the marginal cost of the overflow control.

Implementation of the findings of an infiltration/inflow analysis will result in reduced plant influent flows in cases where excessive infiltration/inflow is eliminated.

Combined Sewer Overflow Control--

Where sewer systems include combined sewer service areas the process is principally the same, although somewhat different rules apply. Infiltration analysis is required, and is identical to that for separate sewers. If no overflows occur from the combined system upstream of the treatment plant, then optimization of treatment (including equalization) and sewer system improvement is as described above. If combined system overflows occur, then a separate cost effectiveness analysis for overflow elimination must be conducted to determine the degree of overflow reduction that can be justified in terms of upgrading receiving water quality and restoring beneficial uses. Details of the requirements for overflow reduction and justification for the EPA construction grant allotment are specified in EPA PRM 75-34. However, the analysis does not directly affect the process of optimizing treatment with or without equalization and sewer system improvement.

The analysis of combined sewer systems is complex, often requiring the aid of computer-based mathematical modelling to optimize system alternatives and varying control levels. Publications covering the analysis approach for combined systems are referenced in the EPA published Areawide Assessment Procedures Manual, Volume 1, Section 6.(11)

In general, where overflows occur from combined sewer areas, overflow control evaluation, including equalization considerations, can be summarized as follows: Recognizing that overflows normally occur because of the limitations of the collection system capacity for transport to the treatment plant, four major alternatives are available for correction. The object of the analysis will be to identify the least cost means of reducing overflows as a function of frequency of occurrence. The alternatives available include:

a. Source Control

1. Infiltration reduction by sewer system rehabilitation.

2. Stormwater inflow reduction by removal of stormwater connections from the sanitary sewer, i.e., removal of downspouts, illicit drainage connections, etc.

b. Storage

1. In-line storage by using existing sewer capacity or enlarging specific sections.

2. Off-line storage using surface retention basins and subgrade storage chambers.

Storage of overflow at individual or consolidated sources is a form of flow equalization. However, location of equalization facilities some distance from the treatment plant in order to accommodate storm flow peaks for a portion of the total collection generally does not satisfy requirements enabling treatment influent equalization. However, this approach should be considered where applicable to the circumstances.

The last two alternatives are indirectly related to equalization requirements in that the most economical means of control may include one or the other in combination with equalization.

The available alternatives must be evaluated to establish the means of least-cost overflow control. An appropriate level of control must then be established based on the requirements of PRM 75-34. This process will ultimately establish the amount of stormflow delivered to treatment. If applied only to reduce existing overflows, none of the alternatives will reduce existing plant flows. Remote treatment and discharge will leave plant flows unchanged. Upstream holding will increase the total quantity of wastewater to the plant without increasing existing maximum flow rates. Increasing transport capacity will increase peak treatment requirements proportionately. This last case should be recognized as virtually the same as the conventional infiltration-inflow analysis process. This would result only if the combination of increased collection system capacity and increased optimum treatment system (including equalization) capacity had been found to be less costly than all other feasible control alternatives.

The sewer system evaluation, whether for separate or combined systems, in combination with projections for expansion of the service area during the design period, establishes design flows and peaking characteristics; on which the design of all individual treatment process components will be based.

EQUALIZATION SIZING METHODS

Numerous methods have been developed that may be used to estimate the impact of equalization on normally varying wastewater flows and concentrations. Each of these methods requires some amount of data, computation, and the making of assumptions based on observations of the influent; methods differ greatly in these regards. Various methods may be used to compute required basin volume. Ability to estimate effects of equalization on wastewater characteristics varies depending on the level of detail employed in individual methods. For example, methods taking into account influent concentration or mass loading information, and assumptions concerning mixing and reaction within the equalization basin, can yield detailed information on concentration and mass loading in the effluent.

The principal methods are:

1. Simple flow balance ("mass diagram")
2. Simple concentration balance
3. Combined flow and concentration balance
4. Sine wave method
5. Rectangular wave method
6. Batch dumping
7. Random concentration

Method 1 is simplest and directly applicable to municipal wastewater treatment problems. Method 2 is oversimplified for most applications and is presented mainly as an introduction to Method 3. Method 3 is more detailed requiring greater computational effort. The method is, however, versatile and can be used for more detailed and thorough problem analysis. Methods 4 through 7 may be useful in some municipal plants, but are not expected to be as useful in this area as Methods 1 and 3. They are described briefly, accompanied by references and a summary of their applicability, advantages, and disadvantages.

Method 1--Simple Flow Balance

Simple flow balances are easily performed either graphically (mass diagram), or with a note pad and simple arithmetic. Only two inputs are required; the flow entering the system as a function of time, and either the discharge as a function of time or the schedule of basin volume regulation. The output results in the basin volume schedule if the discharge record is given, or in the discharge record if the basin volume schedule is given. These simulations may be used with any flow pattern, and easily handle diurnal variations and storm flows. The analyses may be applied to both in-line and side-line basins. This method is useful as an operating tool, since flows and volumes are easily measured.

A simple flow balance has two main drawbacks: First, no information is provided on concentrations or mass flows (mass flow being the product of concentration and flow rate). Second, data is assumed to be known exactly, not just statistically.

As in any method, care must be taken to anticipate consequences in cases where actual inputs are different from projections. For example, a typical design may provide an equalization basin volume and treatment plant capacity sized to accommodate measured and predicted domestic, commercial and industrial flows, with allowances for infiltration and inflow based on infiltration/inflow analysis. Infiltration/inflow analyses are frequently based on flow measurements taken during only moderately intense rainfall. However, flow measurements are seldom available for relatively intense storms (e.g. the 5-10 year storm). Effects of larger storms on total plant flows may be significant. Consequences on system design and performances must therefore be considered.

The basic equation for application of the simple flow balance is:

$$Q_{in}\ \Delta t = \Delta V + Q_{out}\ \Delta t \qquad (2\text{-}1)$$

where Q_{in} = flow into the system, average rate during Δt, L^3T^{-1}

Δt = time interval, T

ΔV = change in stored volume during Δt, L^3

Q_{out} = flow out of system, average rate during Δt, L^3T^{-1}

This equation is applied stepwise, or iteratively, for successive time intervals. Cumulative sums for all values up through the last step are computed and recorded. For instance, if Q_{in} and Q_{out} are given, and basin volumes are desired, a record of $\Sigma\Delta V$ must be generated. The record is comprised of the sum of ΔV values up through the time step in question. The flow balance equation (Equation 2-1) and the cumulative sum may be recorded on a graph or in a table.

The relation between the physical system and system variables for this method is shown in Figure 6. Note that for a side-line basin, the flow into the basin is $Q_{out} - Q_{in}$. (This quantity is negative when flow is being discharged from the basin.) Any consistent units may be used. For instance, if Q_{in} and Q_{out} are in millions of gallons per day, then Δt is in days and ΔV is in millions of gallons. If Q_{in} and Q_{out} are in liters per second, Δt is in seconds, and ΔV is in liters.

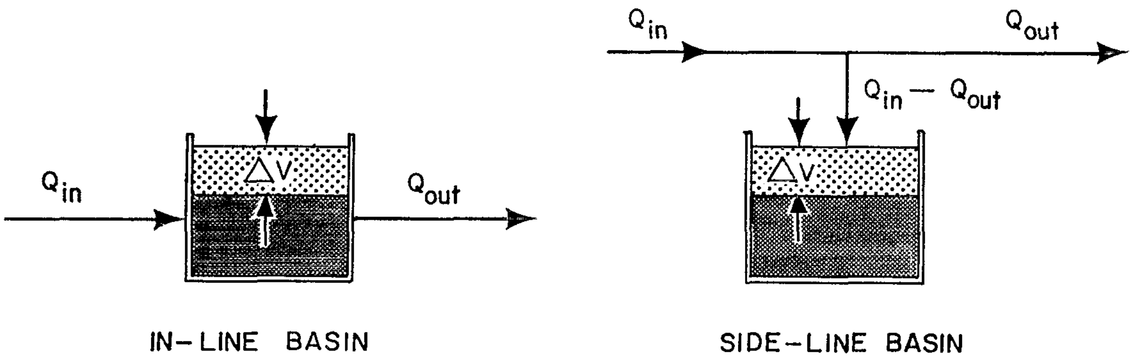

Figure 6. Simple flow balance schematics

The following three examples demonstrate use of simple flow balances, and illustrate some common phenomena in equalization.

Example 1--Graphical Solution, Diurnal Variation--

This example, drawn from the EPA Process Design Manual for Upgrading Existing Wastewater Treatment Plants, (12) describes the graphical solution for a flow balance applied to a diurnal flow variation. The flow balance illustrated in the example is used to determine the equalization volume required to exactly balance diurnal variations, producing constant effluent flow. The flow variation before equalization is shown in Figure 7. This is a typical diurnal wastewater flow pattern, with a peak-to-average ratio of 1.7 and a minimum-to-average ratio of 0.45. There is one daily peak, at about 6:00 p.m.

The average flow rate for each hour interval is used to compute the corresponding volume increment. Successive volumes are plotted to form a hydrograph or mass diagram as illustrated in Figure 7. A straight line drawn from the origin to the cumulative volume at 24 hours (dashed line, Figure 8) has a slope equal to the average flow rate over the day. In this case the average daily flow is approximately 4.3 mgd.

To equalize the diurnal varying flow, tank volume must be provided to accommodate flows in excess of the equalized flow rate. For normal diurnal variations this volume typically ranges from 10 to 20 percent of the average daily flow. The volume required for equalizing flow variations in this example is equal to the vertical distance (measured in millions of gallons) between parallel lines of slope equal to the average flow line (dashed line, Figure 8) and tangent to the extremities of the inflow mass diagram. These lines are shown as A and B on Figure 8. In this illustration, the required volume for equalization is 740,000 gallons. This volume is approximately 17 percent of the average daily flow.

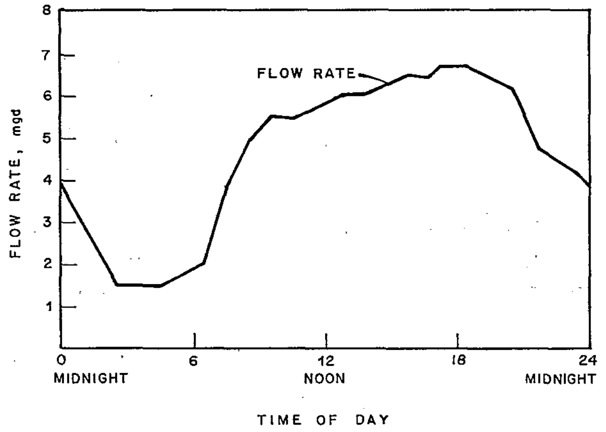

Figure 7. Wastewater flow variation before equalization

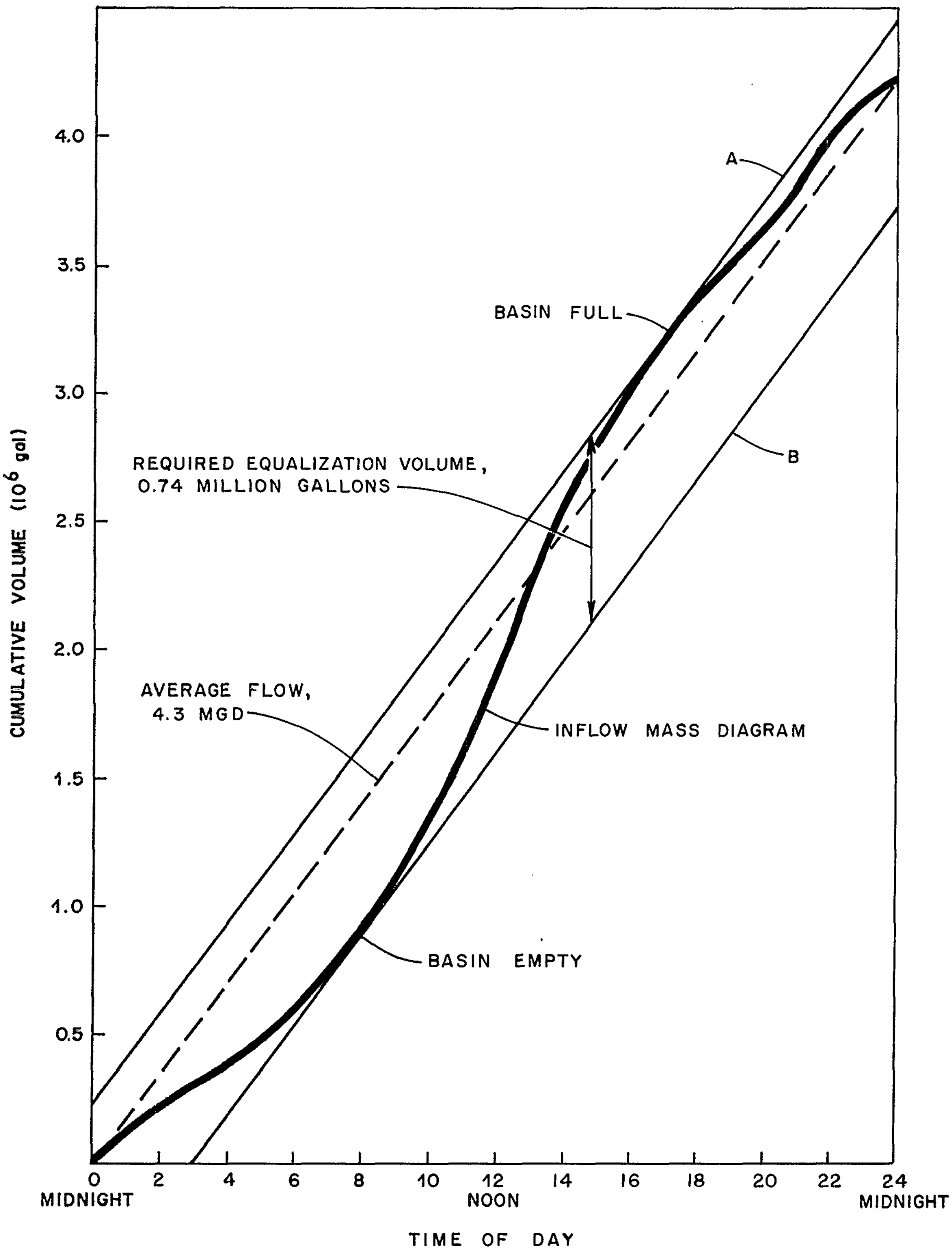

Figure 8. Hydrograph for Example 4

A physical interpretation of the hydrograph is as follows. At 8:00 a.m., the equalization basin is empty, as shown by the tangency of the inflow mass diagram with the bottom diagonal. At this point plant flow begins to exceed the average flow rate and the tank begins to fill. Accordingly, inflow mass diagram and the bottom diagonal begin to diverge at this point. At 5:00 p.m., the basin is full, as shown by the tangency of the inflow mass diagram with the top diagonal. Finally, the tank is drawn down from 5:00 p.m. to 8:00 a.m. on the following day, when the flow is below average.

Importantly, the equalization basin volume used for actual design must be greater than that obtained with the hydrograph for several reasons: to provide freeboard, minimum depth for aeration, and mixing equipment that might be used, and to provide for storm flows occurring in excess of normal diurnal variations.

Example 2--Tabular Solution, Diurnal Variation--

This example illustrates a tabular solution (Table 1) of the problem used in Example 1. Computations are summarized in Table 1. A uniform time increment (Δt) of 2 hours is used as a suitable value compatible with normal diurnal variations. Variations from one hour to the next do not greatly affect the equalization volume but variations over several hours are important.

The basin is assumed to be at a reference level of 0 at midnight. For this example, the reference level must represent at least 0.759 million gallons or the basin will not run dry before it starts to refill. The total working volume required is the maximum $\Sigma\Delta V$ minus the minimum $\Sigma\Delta V$, or 0.786 million gallons. This figure approximates the 0.74 million gallons of Example 1.

Note that the average flow estimated by the graphical method (Example 2, Figure 8) is estimated to be 4.57 mgd. This difference is due to limited accuracy of reading and plotting the graphs. Totals for $Q_{in}\Delta t$ and $Q_{out}\Delta t$ can easily be made as a check of computational accuracy; these values should correspond to the total flow over the day. The final $\Sigma\Delta V$ is very small, as required for a periodic variation. The total of ΔV values should be very small as in the example; it is not identical to the last ΔV because of round-off errors. The calculations should be carried to three significant figures, the least count corresponding to 1,000 gallons. The results, as always, cannot be more accurate than the input values--5 percent at best.

Example 3--Tabular Method, Storm Inflow--

Example 3 describes the impact of storm flow variation superimposed on the diurnal variation of Example 2. Examples

TABLE 1. EXAMPLE 2--TABULAR METHOD

Time Interval	Q_{in} (mgd)	$Q_{in}\Delta t$ (10^6 gal)	$Q_{out}\Delta t$ (10^6 gal)	ΔV (10^6 gal)	$\Sigma\Delta V$ (10^6 gal)
2400 - 200	3.0	0.250	0.381	-0.131	-0.131
200 - 400	1.6	.133	.381	- .247	- .378
400 - 600	1.6	.133	.381	- .247	- .625
600 - 800	3.0	.250	.381	- .131	- .756
800 - 1000	5.1	.425	.381	+ .044	- .712
1000 - 1200	5.6	.467	.381	.086	- .626
1200 - 1400	5.9	.492	.381	.111	- .515
1400 - 1600	6.3	.525	.381	.144	- .370
1600 - 1800	6.6	.550	.381	.169	- .201
1800 - 2000	6.5	.542	.381	.161	- .040
2000 - 2200	5.4	.450	.381	.069	+ .030
2200 - 2400	4.2	.350	.381	- .031	- .001
TOTAL	54.8	4.57	4.57	- .003	(final) - .001
AVERAGE	4.57	-	-	-	-

Q_{out} = 4.57 mgd, constant

Δt = 2 hours = 0.0833 days

$\Sigma\Delta V$ = running total of ΔV values

Working volume required = 0.030 -(-0.756) = 0.786 10^6 gal.

4 and 5, discussed under Method 3, illustrate equalization of concentrations and loads.

For this example it is assumed that the Q_{in} in Table 1 represents the maximum wastewater flow from domestic, commercial, and industrial sources, plus the highest seasonal infiltration. In addition, storm inflow and storm-related infiltration are to be considered. Assume that the effect of a 5-year storm is as shown in Table 2, and that the storm is followed by another storm, half as large, starting six hours after the initial storm ends. This example illustrates computation of Q_{out} and basin volumes to provide for flow equalization. The 5-year storm in Table 2 is representative of a separate sewer system, with no combined sewers and moderate amounts of infiltration and inflow.

This example demonstrates that additional flow could not be handled in treatment components designed for equalized flow with a capacity of 4.6 mgd, as in Example 2. The 4.6 mgd treatment capacity held constant through the day is required for the non-storm flows. Accordingly, no excess capacity is available for storm flows. Treatment capacity must exceed 4.6 mgd to permit discharge of stored storm flow.

Whether or not the storm effect in Table 2 is adequate for design depends upon regulatory policy. When a larger storm than the 5-year storm occurs, producing flows as assumed in Table 2, it is likely that sewer overflows or bypassing will occur. Such overflows or bypassing may be acceptable by virtue of the rarity of occurrence. If not, system modifications would be required. Collection system improvements to reduce infiltration and inflow or development of storage volume in the collection system would reduce storm effects on existing treatment. Alternatively, flows could be equalized at the plant or additional treatment capacity provided. In the work that follows, the 5-year storm hydrograph in Table 2 is used.

If no storage is provided, the required treatment capacity may be calculated as follows:

   6.6 mgd peak non-storm flow (Table 1)
\+ 4.2 mgd peak storm effect (Table 2)
 10.8 mgd peak flow

If equalization is located between primary sedimentation and activated sludge, the primary sedimentation tanks will need 10.8 mgd peak capacity.

If downstream process components are sized for 6.6 mgd peak flow, so that equalization is not required in dry weather, the ability of equalization to handle the storm effect can be determined. The storm produces an average flow over 20 hours of 2.1

mgd. The available capacity is 6.6 minus 4.6 mgd, or 2.0 mgd during the storm itself. With 6 hours until the next significant storm, the small remaining volume could easily be treated. In fact, the peak treatment rate could be reduced slightly, as discussed below. In that case, equalization would have to be operated during dry weather.

Treatment capacity could be sized to accommodate peak dry weather flow. Using 6.6 mgd for the maximum Q_{out}, the equalization volume required to accommodate storm flows would be about 1.03 million gallons, as derived in Table 3. Note that the storm has been assumed to occur at roughly the worst time, that is, the storm peak will coincide with the non-storm peak at about 6:00 p.m. This assumption is conservative. Also observe that the basin is empty shortly before 10:00 a.m. ($\Sigma\Delta V$ is slightly negative at 10:00 a.m.), so a subsequent storm could occur at any time thereafter, and be of any size smaller than the Table 2 storm without overloading the equalization basin treatment system. Finally, note that Q_{out} may be reduced in the last 2 hours of the storm effect without causing any difficulty in computation.

If treatment capacity is designed for the assumed peak dry weather flow and only one million gallons of storage volume is provided, it can be shown that flow equalization may still be used in dry weather. The working basin volume required for the storm (1.0 million gallons) is greater than the requirement for diurnal variation (0.8 million gallons, from Example 2). Therefore, if 1.0 million gallons are available, the flow may be fully equalized in dry weather.

A smaller maximum Q_{out} could not be used, with equalization required in dry weather, using 1.0 million gallons of storage. A greater storage volume is required is Q_{out} cannot reach 6.6 mgd.

A smaller maximum Q_{out} could be used if storage volume were increased. For instance, if Q_{out} is limited to 5.5 mgd, the solution is about 2.3 million gallons. As shown in Table 4, if the basin is empty at noon on Monday, the maximum stored volume will be 1.61 million gallons, occurring late Tuesday night during the following storm. Note that the following storm is significant in this case. Also note that the 8:00 a.m. volume is decreasing; since the storm peaks are more than 24 hours apart, the flow from following storms will eventually be treated. The tabulation was based on an empty basin at noon on Monday; but by then the basin could easily have about 0.7 million gallons. Therefore, a total working volume of about 2.3 million gallons will be needed. To verify this estimate, a pattern of preceding storms would have to be assumed, and a longer calculation would be needed. Finally, note that the basin must be in

TABLE 2. EXAMPLE 3--5-YEAR STORM EFFECT

Hours after storm begins	Storm effect flow rate (mgd)
1	0.9
2	2.0
3	2.8
4	3.5
5	4.2
6	3.8
7	3.5
8	3.4
9	3.2
10	2.7
11	2.3
12	2.1
13	1.9
14	1.6
15	1.4
16	1.1
17	.8
18	.5
19	.3
20	.2
Total	42.2
Average over 20 hr.	2.1

TABLE 3. EXAMPLE 3--TABULAR METHOD, FIRST CASE

Time interval	Q_{in} (mgd)			Q_{out} (mgd)	$Q_{in}\Delta t$ (10^6 gal)	$Q_{out}\Delta t$ (10^6 gal)	ΔV (10^6 gal)	$\Sigma\Delta V$ (10^6 gal)
	Non-storm	Storm	Total					
1200 - 1400	5.9	0.9	6.8	6.6	0.567	0.550	+0.017	0.017
1400 - 1600	6.3	2.8	9.1	6.6	.758	.550	.208	.225
1600 - 1800	6.6	4.2	10.8	6.6	.900	.550	.350	.575
1800 - 2000	6.5	3.5	10.0	6.6	.833	.550	.283	.858
2000 - 2200	5.4	3.2	8.6	6.6	.717	.550	.167	1.025
2200 - 2400	4.2	2.3	6.5	6.6	.542	.550	- .008	1.017
2400 - 0200	3.0	1.9	4.9	6.6	.408	.550	- .142	.875
0200 - 0400	1.6	1.4	3.0	6.6	.250	.550	- .300	.575
0400 - 0600	1.6	0.8	2.4	6.6	.200	.550	- .350	.225
0600 - 0800	3.0	0.3	3.3	5.0	.275	.417	- .142	.083
0800 - 1000	5.1	0	5.1	6.6	.425	.550	- .125	- .042
1000 - 1200	5.6	0	5.6	5.6	.467	.467	0	- .042
TOTAL	(54.8)	(21.3)	(76.1)	---	6.342	6.384	-0.042	-0.042
AVERAGE	4.57	1.78	6.34	---	---	---	---	---

TABLE 4. EXAMPLE 3--TABULAR METHOD, SECOND CASE[a]

Time interval	Q_{in} (mgd)			$Q_{in}\Delta t$ (10^6 gal)	$Q_{out}\Delta t$ (10^6 gal)	ΔV (10^6 gal)	$\Sigma\Delta V$ (10^6 gal)
	Non-storm	Storm	Total				
Monday:							
1200 - 1400	5.9	0.9	6.8	0.567	0.458	+0.109	0.109
1400 - 1600	6.3	2.8	9.1	.758	.458	+ .300	.408
1600 - 1800	6.6	4.2	10.8	.900	.458	+ .442	.850
1800 - 2000	6.5	3.5	10.0	.833	.458	+ .375	1.225
2000 - 2200	5.4	3.2	8.6	.717	.458	+ .259	1.483
2200 - 2400	4.2	2.3	6.5	.542	.458	+ .084	1.567
Tuesday:							
2400 - 0200	3.0	1.9	4.9	.408	.458	- .050	1.517
0200 - 0400	1.6	1.4	3.0	.250	.458	- .208	1.308
0400 - 0600	1.6	0.8	2.4	.200	.458	- .258	1.050
0600 - 0800	3.0	0.3	3.3	.275	.458	- .183	.867
0800 - 1000	5.1	0	5.1	.425	.458	- .033	.833
1000 - 1200	5.6	0	5.6	.467	.458	+ .009	.842
1200 - 1400	5.9	0	5.9	.492	.458	+ .034	.876
1400 - 1600	6.3	0.4	6.7	.558	.458	+ .100	.976
1600 - 1800	6.6	1.4	8.0	.667	.458	+ .208	1.184
1800 - 2000	6.5	2.1	8.6	.717	.458	+ .258	1.443
2000 - 2200	5.4	1.8	7.2	.600	.458	+ .142	1.584
2200 - 2400	4.2	1.6	5.8	.483	.458	+ .025	1.609
Wednesday:							
2400 - 0200	3.0	1.2	4.2	.350	.458	- .108	1.501
0200 - 0400	1.6	1.0	2.6	.217	.458	- .242	1.259
0400 - 0600	1.6	0.7	2.3	.192	.458	- .267	.993
0600 - 0800	3.0	0	3.0	.250	.458	- .208	.784
TOTAL	97.5	31.5	130.4	10.87	10.08	+0.788	0.784
AVERAGE	4.43	1.43	5.93	5.93[b]	5.50[c]	---	---

[a] Δt = 2 hours = 0.0833 days; Q_{out} = 5.5 mgd

[b] $\times \frac{12}{22} = 5.93$

[c] $\times \frac{12}{22} = 5.50$

continuous use for at least 2 days (probably more), and is not empty at the end of the calculation. That is why the total $Q_{in}\Delta t$ is greater than the total $Q_{out}\Delta t$. This alternative does not appear to be attractive, since a much smaller basin could be used with a small increase in Q_{out}.

<u>Method 2--Simple Concentration Balance</u>

This method is based on straight-forward principles, but may require extensive calculation for some applications. It is also limited to a constant flow rate, constant equalization volume, and in-line basins.

For a completely mixed basin, and a material that is unchanged during storage, the concentration balance equation is:

$$\Delta c = \frac{Q}{V}\Delta t\ (c_{in}-c) \qquad (2\text{-}2)$$

where Δc = change of concentration in basin during Δt

Q = flow rate, constant

V = basin volume, constant

Δt = time interval

c_{in} = influent concentration, average over Δt

c = concentration in basin, at beginning of Δt

Note that Q/V is the reciprocal of the basin detention time. This equation is easily modified if necessary for a material that decomposes during storage, provided that there is a good model for the rate of decomposition as a function of c. For example, if first order decay occurs, then the equation is:

$$\Delta c = \frac{Q}{V}\Delta t(c_{in}-c) - cK\Delta t \qquad (2\text{-}3)$$

where K = decay coefficient

Units of Q, V and Δt must be consistent with each other, as in Method 1. Units of Δc, c_{in}, and c must be the same, but they need not be consistent with Q and V; for instance, c can be in milligrams per liter, and V in millions of gallons. The units of K must be the reciprocal of the units for Δt.

Equations 2-2 and 2-3 may also be extended to model an equalization basin that is not completely mixed, but the equations will become more complex.

In these equations, an initial value for c must be assumed. After a few detention times, this initial value has little impact on c and its variations. Q, V, and values of c_{in} as a function of time must be supplied to perform the solution; the result is c as a function of time.

The time interval Δt should be much less than the detention time V/Q. Also, Δt should be shorter than the time-scale of variations in c_{in}.

In the case of Equation 2-2, fewer steps are needed if the following equation is used:

$$\Delta c = [1 - \exp(-\frac{Q\Delta t}{V})](c_{in}-c) \qquad (2\text{-}4)$$

where exp = the exponential function

This type of equation was developed by Reynold, Gibbon, and Attwood.(13)

Method 2 is deterministic, not stochastic; that is, it is assumed that c_{in} is exactly known at each time interval.

The previously listed equations require that Δc be much less than c. This means that Δt must be small compared to V/Q and to 1/K; hence, many steps are required for the calculations, especially for Equations 2-2 and 2-3. For a larger Δt, so that fewer steps are needed, c is appreciably different from the average concentration during Δt. A better approximation to this average concentration is $c + \Delta c/2$. By rearranging Equation 2-3, the balance equation becomes:

$$\Delta c = \frac{\frac{Q\Delta t}{V}(c_{in}-c) - cK\Delta t}{1 + \frac{\Delta t}{2}(\frac{Q}{V} + K)} \qquad (2\text{-}3a)$$

This equation is preferred to 2-3 if a relatively large Δt must be used.

Method 3--Combined Flow and Concentration Balance

This method is an extension of Methods 1 and 2 and is particularly useful for realistic applications. Any realistic pattern of flow and concentration variations may be used, and any realistic operating rule may be used for the equalization basin. For instance, with this method a basin could be analyzed that would produce a constant COD load in the outflow stream. Like Method 2, Method 3 may be adapted to reactions within the basin and to basins that are not completely mixed. The equations are fairly simple to understand. The method has one major drawback: It requires a lot of arithmetic, particularly if flows and concentrations fluctuate rapidly.

The basic equations are balances on flow and load. Equation 2-1 from Method 1 is the balance on flow. For a completely mixed in-line basin and no reactions within the basin, the load balancing equation is:

$$\underbrace{Q_{in}\, c_{in}\, \Delta t}_{\text{Mass in}} = \underbrace{Q_{out}\, c\Delta t}_{\text{Mass out}} + \underbrace{(V+\Delta V)(c+\Delta c) - Vc}_{\text{Increase in stored mass}} \qquad (2\text{-}5)$$

For reactions within the basin that are adequately represented by first-order decay, the equation is:

$$Q_{in}c_{in}\Delta t = Q_{out}c\Delta t + (V+\Delta V)(c+\Delta c) - Vc + VcK\Delta t \qquad (2\text{-}6)$$

where K = decay constant

Similar equations may be developed for side-line basins, but equations applied during basin filling will be different than those applied during basin emptying. Also, the concentration in the basin is not the same as the concentrations in Q_{in} and Q_{out} (as defined in Figure 9). This is true for a side-line basin even if mixing is complete and there are no reactions, unlike the case for Q_{out} of a simple completely mixed in-line basin. For a completely mixed basin with first-order decay, the equations during filling are:

$$Q_{in} > Q_{out} \qquad (2\text{-}7)$$

$$\Delta V = (Q_{in} - Q_{out})\Delta t \qquad (2\text{-}8)$$

$$c_{out} = c_{in} \qquad (2\text{-}9)$$

$$\underbrace{\Delta V c_{in}}_{\text{Mass into basin}} = \underbrace{(V+\Delta V)(c_B + \Delta c_B) - Vc_B}_{\text{Stored mass increase}} + \underbrace{Vc_BK\Delta t}_{\text{Decay}} \qquad (2\text{-}10)$$

where c_{out} = concentration in outlet, average over Δt

c_B = concentration in equalization basin at beginning of Δt

Δc_B = change in c_B during Δt

When the basin is neither being filled nor emptied, the equations are:

$$Q_{in} = Q_{out} \qquad (2\text{-}11)$$

$$\Delta V = 0 \qquad (2\text{-}12)$$

$$c_{out} = c_{in} \quad (2\text{-}13)$$

$$\Delta c_B = -c_B K \Delta t \quad (2\text{-}14)$$

When the basin is being emptied, ΔV is negative and the equations are:

$$Q_{in} < Q_{out} \quad (2\text{-}15)$$

$$\Delta V = (Q_{in} - Q_{out})\ \Delta t \quad (2\text{-}16)$$

$$\Delta c_B = -c_B K\ \Delta t \quad (2\text{-}17)$$

$$c_{out} = \frac{c_{in}Q_{in} + c_B(Q_{out} - Q_{in})}{Q_{out}} \quad (2\text{-}18)$$

Figure 9 shows the application of the variables for completely mixed basins.

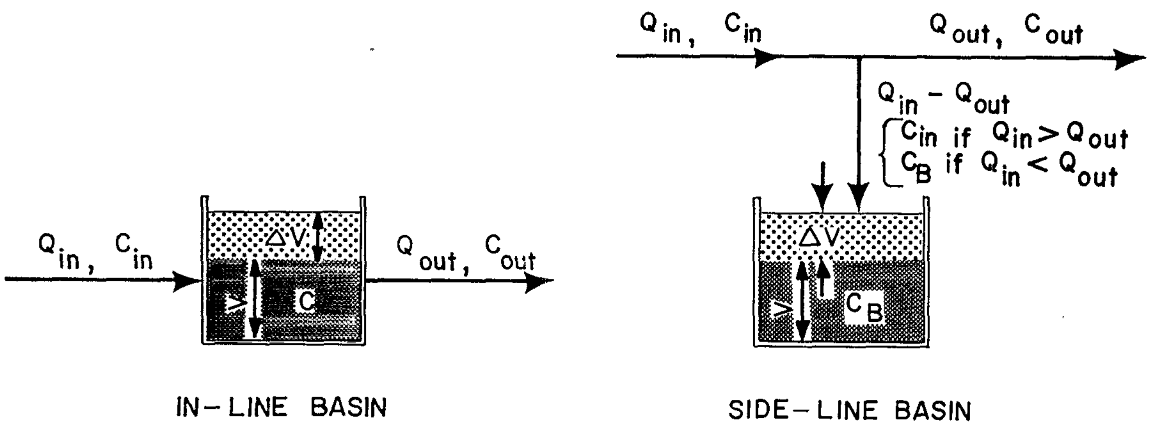

Figure 9. Flow and concentration iteration schematics

The most convenient form of the equation will depend on the items that are given, and those that are required. For instance, suppose that an in-line basin is under study and the given quantities are K, Q_{in}, c_{in}, and Q_{out}, plus initial conditions of c and V, so that the desired outputs are c and V as functions of time. Then Equation 2-1 may be written:

$$\Delta V = Q_{in}\Delta t - Q_{out}\Delta t \tag{2-19}$$

and Equation 2-6 may be written:

$$\Delta c = \frac{Q_{in}c_{in}\Delta t - c(Q_{out}\Delta t + \Delta V + VK\Delta t)}{V + \Delta V} \tag{2-20}$$

Equation 2-19 may be solved iteratively and its results will supply the V and ΔV values for each step of Equation 2-20. Each step of Equation 2-20 requires the previous Δc to generate the new c, which is required to go on.

Equation 2-20 requires that Δc be much smaller than c for much accuracy; otherwise Δc will tend to oscillate during the calculations. This oscillation may be avoided in two ways. One is to select a small Δt so that ΔV and Δc are small, which requires more steps to the calculation. If a fairly large Δt is required, better results will occur by refining the mass balance equation 2-6 by using $c + \Delta c/2$ for the average concentration during Δt, as shown in Method 2:

$$\underbrace{Q_{in}c_{in}\Delta t}_{\text{Mass in}} = \underbrace{Q_{out}(c+\tfrac{\Delta c}{2})\Delta t}_{\text{Mass out}} + \underbrace{(V+\Delta V)(c+\Delta c) - Vc}_{\text{Increased stored mass}} + \underbrace{V(c+\tfrac{\Delta c}{2})K\Delta t}_{\text{Decay}} \tag{2-21}$$

In this case, the iteration equation becomes:

$$\Delta c = \frac{Q_{in}c_{in}\Delta t - c(\Delta V + Q_{out}\Delta t + VK\Delta t)}{V + \Delta V + \frac{\Delta t}{2}(Q_{out} + VK)} \tag{2-22}$$

This equation is based on a completely mixed in-line equalization volume and first-order decay.

For a side-line basin, the necessary refinement depends on the magnitude of $K\Delta t$. If K is small, $K\Delta t$ can be small even though Δt is appreciable. Thus, refinement only applies to Equation 2-18, for concentration of the blended stream, which becomes:

$$c_{out} = \frac{c_{in}Q_{in} + (c_B + \frac{\Delta c_B}{2})(Q_{out} - Q_{in})}{Q_{out}} \qquad (2\text{-}23)$$

The other equations (Equations 2-7 through 2-17) are unchanged. Equation 2-10 may be rearranged into an iteration equation:

$$\Delta c_B = \frac{V(c_{in} - c_B) - Vc_B K\Delta t}{V + \Delta V} \qquad (2\text{-}24)$$

Thus, equations are available for side-line fully mixed basins with slow first-order decay. Equations for fast first-order decay could be developed if needed.

These Method 3 equations may be worked using units as described in Methods 1 and 2. For example, a possible set is:

V, ΔV (millions of gallons)

Q_{in}, Q_{out} (millions of gallons per day)

Δt (days)

Δc, c_{in}, c, c_{out}, c_B (milligrams per liter)

K (per day)

These units, however, yield an odd combination for load rates. The input load rate is:

$$W_{in} = Q_{in}c_{in} \qquad (2\text{-}25)$$

The output rate for an in-line basin is:

$$W_{out} = Q_{out}c \qquad (2\text{-}26)$$

The output rate for a side-line basin is:

$$W_{out} = Q_{out}\, c_{out} \qquad (2\text{-}27)$$

The units of W_{in} and W_{out} will be mgd-mg/ℓ, which must be multiplied by 8.34 (pounds per million gallons) per (milligram per liter) to be in the common units of pounds per day.

The initial conditions of c (or c_B) and V affect the calculations. If Q_{in}, c_{in}, and Q_{out} are constant or periodic (e.g. diurnally varying only), c (or c_B) and V will eventually reach periodic variation, and the initial condition of c (or c_B) will become unimportant. This will occur more rapidly as the initial value of V decreases; it also helps to start with a roughly realistic value for the concentration in the basin.

A programmable calculator is very helpful for these calculations. The examples were performed on a calculator with eight memories, "stack," and 50 program steps. The calculator's total capacity was almost a necessity.

Examples 4 and 5 illustrate the use of these equations.

Example 4--In-Line Basin--

This example uses the flow variation in Example 2 (see Method 1), a diurnal variation in input BOD concentration, a decay rate, and initial conditions to calculate the output concentration.

The input variations in flow and concentration are shown in Figure 10. This information is drawn from EPA's Process Design Manual for Upgrading Existing Wastewater Treatment Plants.(12) Note that mass loading is expressed in pounds per day, which signifies a rate, not the duration of that loading rate. Other input values are:

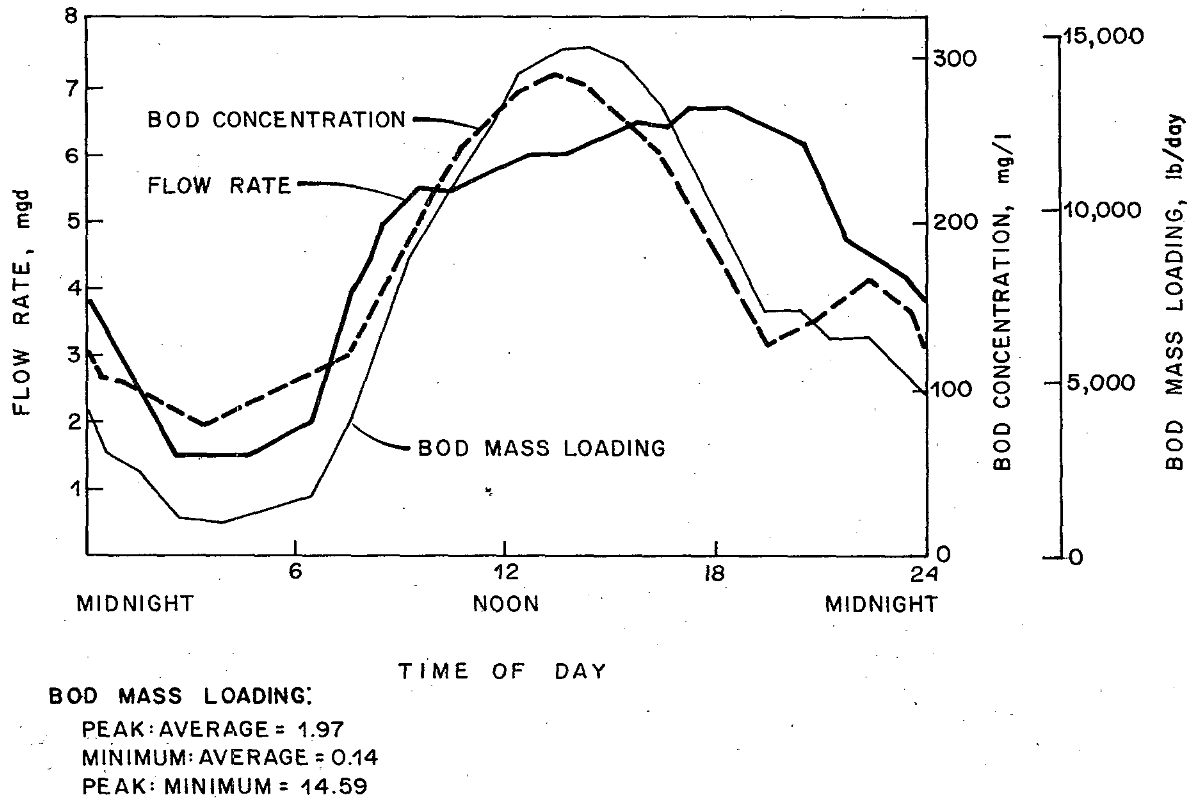

Figure 10. Wastewater flow and BOD variation before equalization

V, 0.1 million gallons at 8:00 a.m.
c, initially assumed at 150 mg/l at 8:00 a.m.
K, 0.1 per day
Q_{out}, constant, 4.57 mgd

This V of 0.1 million gallons at 8:00 a.m., coupled with the basin operation from Example 2, requires a maximum V of 0.88 million gallons at about 10:00 p.m. The K value is roughly correct for BOD in domestic sewage when the cell residence time is shorter than the critical cell residence time for biological growth. Much larger K values will apply if a biomass can develop.

The first few steps of the calculation are shown in Table 5. The calculation was begun at 8:00 a.m. to minimize the importance of the initial assumption for c; the calculation reached within one percent of its eventual steady variation after only two steps (4 hours). The calculation was carried out over three full days to verify that c was the same (within one percent) at each step from one day to the next.

The results are plotted in Figure 11 together with the results from Example 5. The in-line basin produces a delay in the peak concentration in the flow to treatment. But it does not appreciably reduce the peak, because the volume in storage, V, is small at 8:00 a.m. Thus, very little weak wastewater is in the basin to dilute the stronger mid-day wastewater. If a much larger V were maintained during the early morning hours, there would be more concentration smoothing, as illustrated below.

In-line storage reduced the average load from 7,415 to 7,185 pounds per day by virtue of BOD decomposition in the basin; a reduction of approximately 3 percent. The peak-to-average ratio for BOD mass loading was reduced from 1.92 to 1.48, and the minimum-to-average ratio was raised from 0.15 to 0.62, by in-line equalization. The overall peak-to-minimum load ratio was reduced from 13.08 to 2.40.

Example 5--Side-Line Basin--

The same conditions as in Example 4 were applied to a side-line basin. The first few lines of the calculations plus the first transition from basin filling to basin emptying are shown in Table 6. The results are plotted in Figure 11. The solution took 15 steps (more than a complete day) for the periodic output to stabilize. Through the initial 15 steps the basin concentration, c_B, is affected more than one percent by the initial value for c_B (150 mg/l). The calculation was carried out for three full days (36 steps) to verify output stability.

TABLE 5. BEGINNING FLOW AND CONCENTRATION ITERATIONS: EXAMPLE 4, IN-LINE BASIN

	Time interval			
	0800 to 1000	1000 to 1200	1200 to 1400	1400 to 1600
Q_{in} (mgd)	5.1	5.6	5.9	6.3
c_{in} (mg/l)	180	253	288	270
V (10^6 gal)	0.100	0.144	0.230	0.341
ΔV (10^6 gal)	0.044	0.086	0.111	0.144
$Q_{in}c_{in}\Delta t$ (10^6 gal-mg/l)	76.5	118.1	141.6	141.8
$Q_{out}\Delta t$ (10^6 gal)	0.381	0.381	0.381	0.381
$VK\Delta t = 0.00833V$ (10^6 gal)	0.001	0.001	0.002	0.003
$Q_{out}\Delta t+\Delta V+VK\Delta t$ (10^6 gal)	0.046	0.468	0.494	0.528
Δc (mg/l)	37.6	71.97	25.10	-12.57
c (mg/l)	150	187.5	259.6	284.7
W_{out} (lb/day)	6434	8522	10372	10611

TABLE 6. BEGINNING FLOW AND CONCENTRATION ITERATIONS: EXAMPLE 5, SIDE-LINE BASIN

	Time interval					
	0800 to 1000	1000 to 1200	1200 to 1400	1400 to 1600	2000 to 2200	2200 to 2400
Q_{in} (mgd)	5.1	5.6	5.9	6.3	5.4	4.2
c_{in} (mg/l)	180	253	288	270	140	153
V (10^6 gal)	0.100	0.144	0.230	0.341	0.815	0.884
ΔV (10^6 gal)	0.044	0.086	0.111	0.144	0.069	-0.031
c_B (mg/l)	150	158.3	192.9	222.8	210.7	203.57
Δc_B (mg/l)	8.30	34.58	29.88	12.72	-7.14	-1.70
c_{out} (mg/l)	180	253	288	270	140	157.03
Q_{out} (mgd)	4.57	4.57	4.57	4.57	4.57	4.57
W_{out} (lb/day)	6,860	9,643	10,977	10,291	5,336	5,985

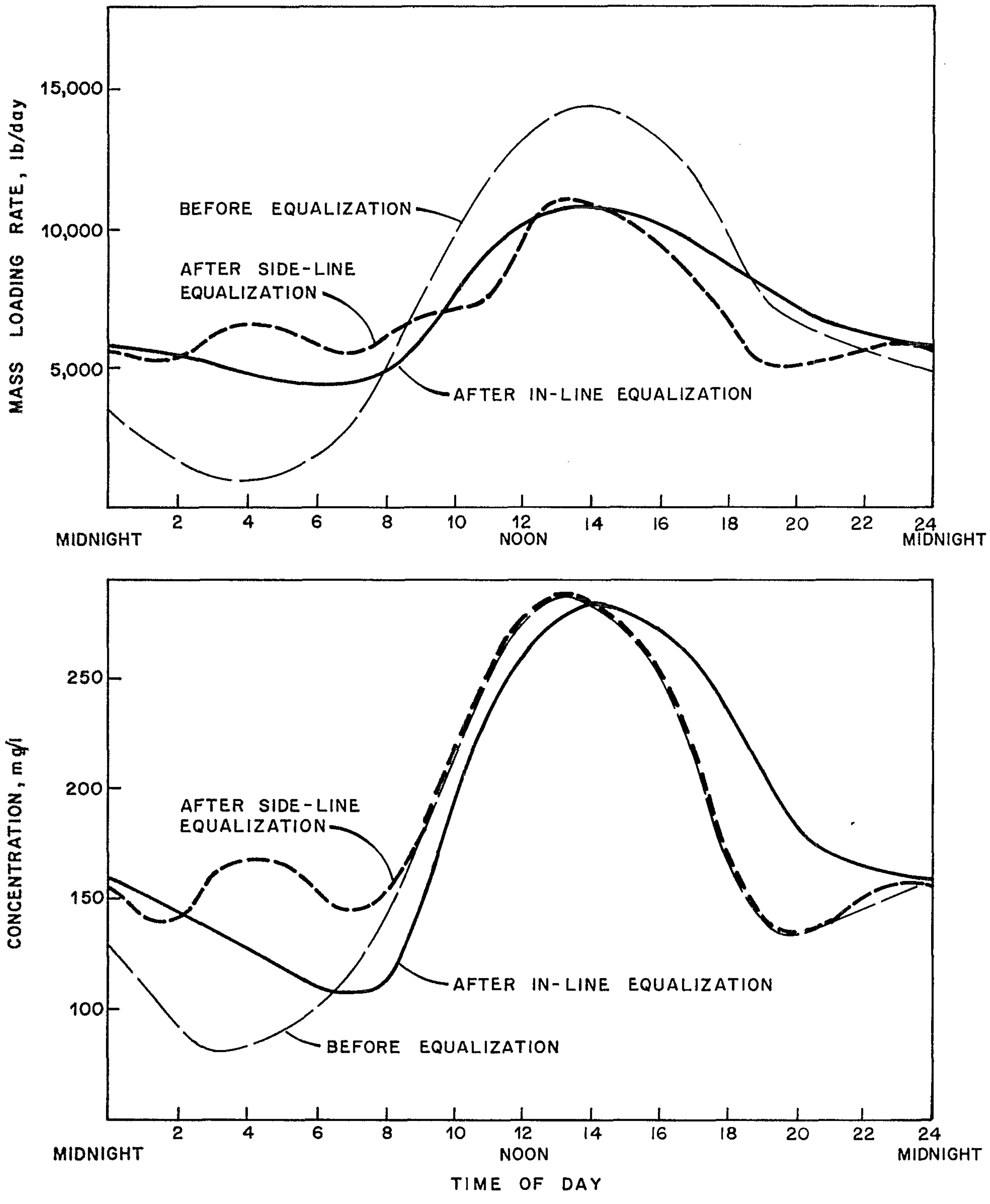

Figure 11. Examples 4 and 5, flow and concentration balance

The results have some interesting features. The side-line basin had a higher average concentration than the in-line basin because the weak early morning wastewater did not enter the side-line basin, and the decay was thus slightly increased. The equalized flow had considerable fluctuation in concentration and mass load during the night, as the equalized flow was composed of different proportions of weak, entering wastewater and strong, stored wastewater. As expected, the side-line basin was not effective at reducing the daily concentration peak, but neither was the in-line basin. For raising minimum concentrations in the early morning, the side-line basin was decidedly better. Generalizations in this regard are hazardous. It has been assumed in previous reports (14) that in-line basins provide concentration equalization, but side-line basins do not. Examples 4 and 5 show that this is not necessarily correct, and that in this case the side-line basin is more effective for raising the minimum concentration.

The side-line basin affected the peak-to-average BOD loading through flow leveling only, lowering this ratio from 1.92 to 1.53. The minimum-to-average BOD loading was raised from 0.15 to 0.74. The overall peak-to-minimum load ratio was reduced from 13.08 to 2.05. Overall, given the input assumptions of this example, the side-line equalization configuration produced load and concentration leveling performance in every respect comparable to the in-line system.

Example 6--Excess Volume for Load Equalization--

Examples 4 and 5 have used equalization volumes calculated by Method 1 procedures to just satisfy requirements for the equalization of daily diurnal variations in plant influent flow rate. Only a minimal dead volume of 0.1 x 10^6 gallons, or a total volume equal to 113 percent of required volume, was used to reflect conditions required for aeration of mixing equipment. This example is provided to illustrate the potential for more complete load and concentration leveling made possible by increasing the excess equalization volume.

The same conditions used in Example 4 were applied to an in-line basin. The basin volume used in this case included 1.6 x 10^6 gallons dead volume, or a total volume equal to 300 percent of that required for equalizing diurnal flow variations. Equalized BOD loads and concentrations were computed as in Example 4. The results are shown graphically in Figure 12, along with input variations and the output from Example 4 using only minimal dead volume. It can be seen that increasing the total storage volume from 0.88 x 10^6 gallons to 2.34 x 10^6 gallons, corresponding to 19 percent and 51 percent of the rated daily average treatment plant capacity (4.57 mgd), reduced the peak-to-average BOD load ratio from 1.92 to 1.20; compared with the reduction from 1.92 to 1.48 realized using minimal dead

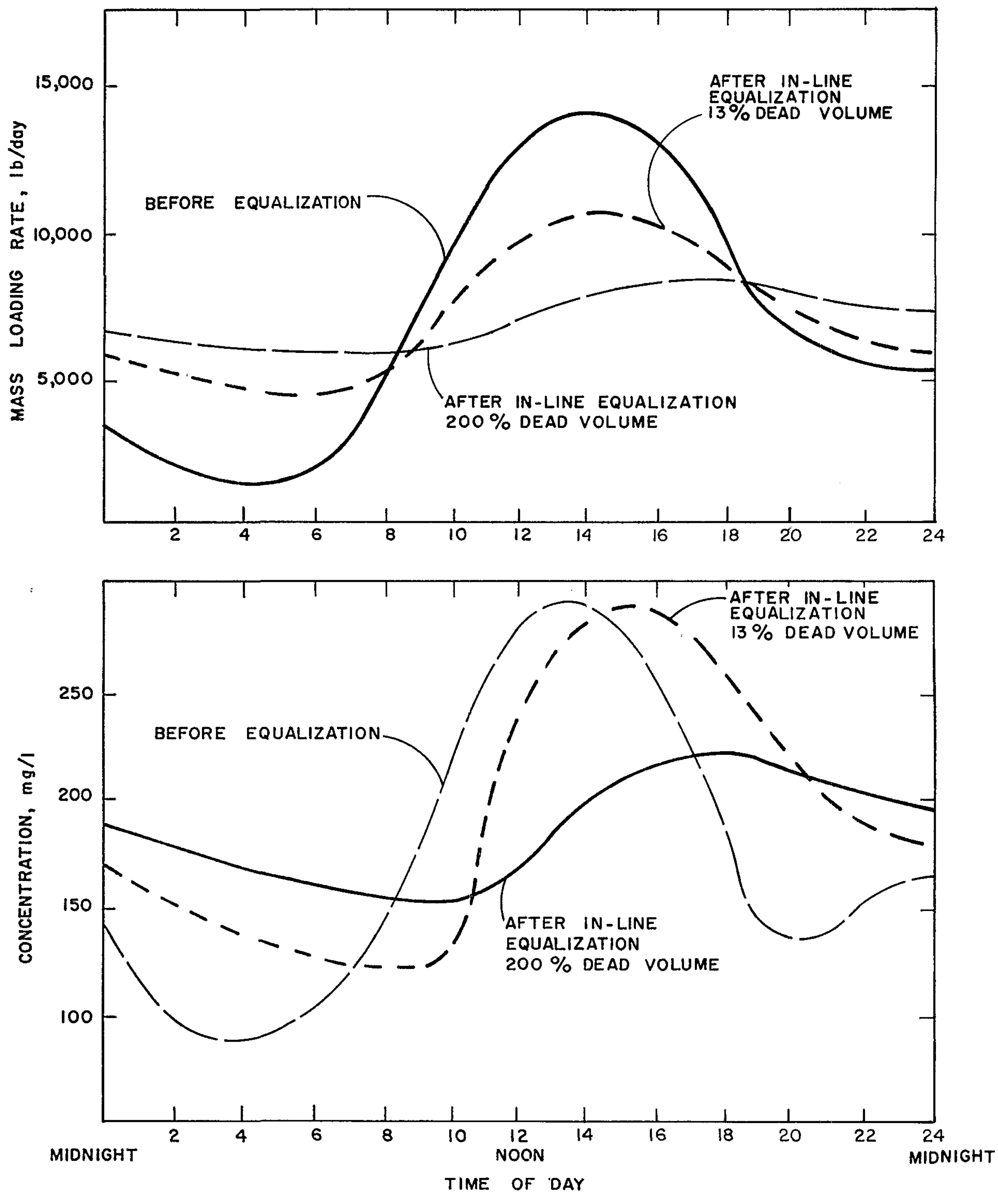

Figure 12. Example 6, excess volume for load equalization

volume. The peak-to-average BOD concentration ratio was reduced from 1.69 to 1.20, compared to the reduction from 1.69 to 1.51 realized by just meeting requirements for flow equalization. The average load reduction due to BOD decay increased to approximately 6 percent, compared to 3 percent for Example 4.

The average residence time in the larger basin (assuming well mixed conditions) is approximately one half day compared to approximately 4-1/2 hours for Example 4. The large dead volume and longer residence time would warrant serious consideration of aeration and/or mixing equipment to prevent odor problems and solids accumulation. Increased costs for the larger volume, and any additional equipment, should be justified in terms of balancing cost savings in downstream processes; such as reduced peaking capacity in biological processes designed for nitrification and denitrification.

<u>Method 4--Sine Wave Method</u>

In Method 3, stepwise equations were presented, based on conservation of volume of wastewater and on mass balance of constituents. If influent concentration, flow, and outflow are certain very simple functions, then it is not necessary to solve the equations stepwise; a direct solution is possible. Such a direct solution requires little computation and little inlet data, since simple functions are represented by very few numbers.

A direct solution has been obtained assuming that both flow and concentration may be represented by sine waves, in phase with each other and with a period of one day.(15) The flow equation used was of the form:

$$Q_{in}(t) = Q_A - (Q_P - Q_A) \sin 2\pi t \qquad (2\text{-}28)$$

where $Q_{in}(t)$ = influent flow as a function of time

Q_A = average flow

Q_P = peak influent flow

t = time, in days

From such equations, simple and rapid estimates can be made of certain equalization operations. For instance, if a constant outflow is desired, Equation 2-28, minus the constant outflow, may be integrated from $t = 0.5$ to $t = 1.0$ day, yielding the working volume of storage:

Also, the rectangular wave approach could be used for concentration and mass loading.

Generally, the rectangular wave method has similar limitations to the sine wave method. For peak-to-average input ratios less than (π-1) the rectangular wave model gave a somewhat more conservative estimate; for ratios greater than (π-1) the sine wave model becomes rapidly more conservative (see Figure 13). When a rough estimate is required and iteration is impractical, the rectangular wave or sine wave methods may be used to estimate volumes required to equalize daily diurnal variations. Nevertheless, it should be recognized that iteration is more flexible and provides reasonable accuracy.

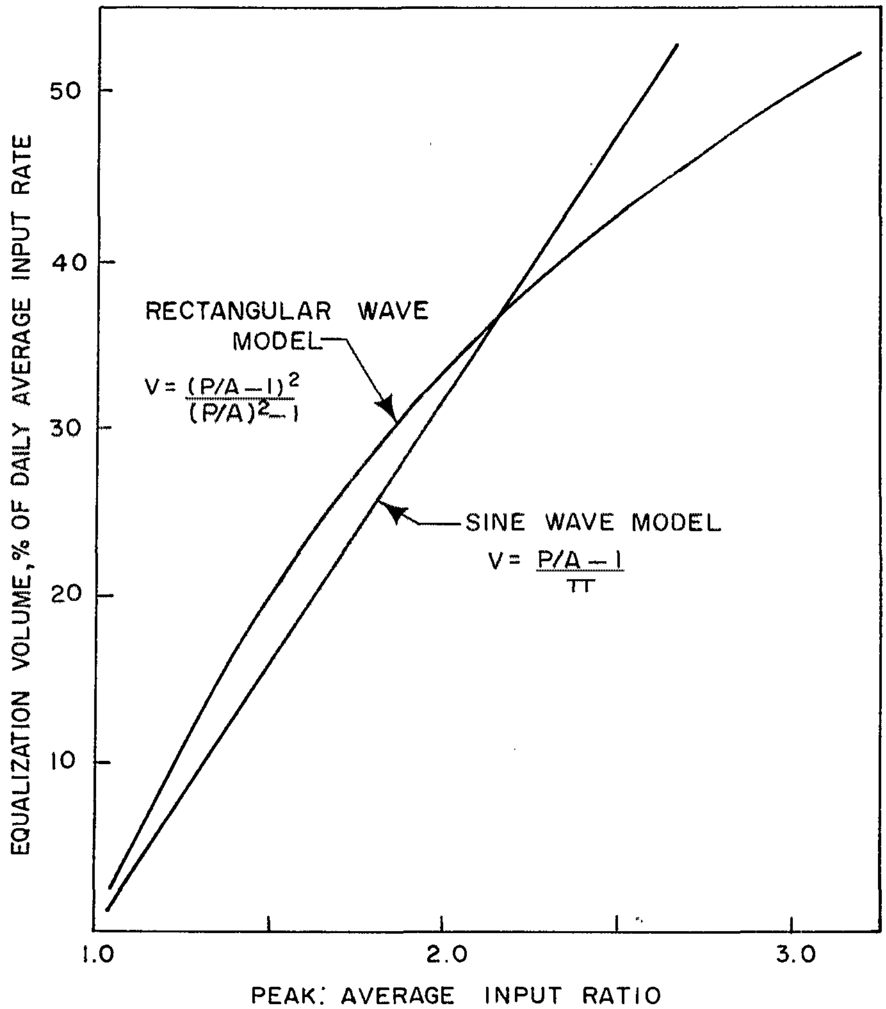

Figure 13. Equalization volume estimation by sine wave and rectangular wave models

$$V = \int_{0.5}^{1.0} (Q_{in} - Q_A)\,dt \qquad (2\text{-}29)$$

$$V = \frac{Q_P - Q_A}{\pi} \qquad (2\text{-}30)$$

Equations such as 2-30 are very easy to compute. Concentration and mass loading fluctuations as above were considered,(15) assuming completely mixed basins and first-order decay.

For comparison, Equation 2-30 was applied to the flow variation of Example 2, Method 1. Equation 2-30 yielded a working volume of 0.65 million gallons, whereas 0.79 million gallons were indicated in Example 2. Therefore, Equation 2-30 and similar equations must be used with great caution, because they may give a lower answer.

The limitation of this approach is that municipal wastewater variations do not follow simple sine waves. This was recognized in the reported work because a rough approximation was sufficient for the purpose of the example illustration. Considering that iterative methods are quite workable for diurnal variations, that the sine wave method may give lower answers, and that the sine wave method is applicable only to diurnal variations, it should be considered as approximate and used only to develop rough estimates.

Method 5--Rectangular Wave Method

The rectangular wave method is similar to the sine wave method, except that flows are approximated by rectangular waves. This method has been described (16) and applied assuming the peak-to-average flow ratio to equal the average-to-minimum ratio; which is a rough but reasonable approximation for many sewer systems. With this assumption, and constant outflow, the necessary volume is:

$$V = Q_A \frac{(x-1)^2}{(x^2-1)} \qquad (2\text{-}31)$$

where V = equalization volume, working range

Q_A = average flow

x = peak-to-average flow ratio = average-to-minimum flow ratio

This equation is easily solved. A similar equation, not much more difficult, could be developed without assuming that the peak-to-average ratio equals the average-to-minimum ratio.

Specialized Analytical Methods

Procedures described in Methods 1 through 5 are sufficient for most routine analytical needs in planning and preliminary design of equalization for typical municipal wastewater treatment applications. However, not all wastewater treatment applications lend themselves to simple routine analysis. This section briefly summarizes two additional, more sophisticated and ultimately more powerful analytical methods for analyzing specialized equalization requirements. Other methods include auto-covariance and power spectrum. Application of these methods to equalization analysis can be found in references 11, 19, 20 and 21.

Method 6--Batch Dumping Method--

For wastes produced in large numbers of batches (each batch being of equal size, but occurring randomly), a method developed by Beaudry is useful.(17) It requires no data besides the volumes of the batches and of the equalization basin, and easily computes a prediction for reduced scatter of the data due to equalization. The method, however, appears to be of limited use for municipal wastewater because the flows have periodic tendencies; they do not occur at strictly random times. Beaudry's method, or a development of it, may be useful for very small treatment systems serving, say, 20 families, where batch effects may be noticeable. Also, the common fill-and-draw lift station produces batches of wastewater, so a batch dumping method may be useful for systems with many such lift stations.

Method 7--Random Concentration Method--

This method is simple to apply, but requires considerable data and is limited in application. An autocovariance method will require more computation, but will give more accurate results from the same data, without such strict limits of application.

The random concentration method is described in detail in the Areawide Assessment Procedures Manual, Volume 1,(11) and in more simplified form elsewhere.(18, 19 and 20)

The basic basin formula is:

$$\frac{S_e}{S_i} = \frac{\Delta t}{2t} \qquad (2\text{-}32)$$

where S_e = variance of effluent concentration

S_i = variance of influent concentration

Δt = interval between samples

t = detention time of basin

This equation is easily computed, yielding a statistical result. However, it only applies to an in-line basin with constant flow and volume; the basin must be fully mixed and no significant reactions may occur in the basin. (Equations of reference 12 permit first-order decay.) To establish S_i, it is necessary to have a large number of samples collected at intervals of Δt. At least 80, and preferably 150, samples should be taken, covering at least ten times the length of the time scale of the unacceptable effluent variation. Furthermore, the data must not have any periodic tendencies, and Δt must be sufficiently long that any autocovarience effect would be negligible. If there are any appreciable periodic or autocovariant effects, Equation 2-32 will show better equalization than will actually occur. Nevertheless, Δt must be much less than t unless the equation is made more complex. The random concentration method is expected to find very little use in municipal applications.

MEASUREMENT AND EVALUATION OF EQUALIZATION EFFECTS

Introduction

In the planning and design of treatment systems including equalization, effects of equalization on the prospective treatment system must be estimated to enable the making of reasonably accurate comparisons between equalized and unequalized treatment schemes. Final comparisons will be made on the basis of the cost of alternative systems. Effects of equalization on unit process and treatment system performance must be determined to enable treatment component sizing prior to cost comparison. The critical parameter for design of individual treatment components will be a combination of influent flow rate, concentration, loading of typical BOD, and/or suspended solids. Available data projected for design conditions must be evaluated accordingly.

Data for this study were collected from treatment plants throughout the United States. Data collected consist primarily of existing plant operating records. No new performance investigations were conducted. Extensive data from previous EPA sponsored detailed studies of equalization performance were also used.

Performance Evaluation Method

In an effort to identify and assess the magnitude of equalization effects on unit process and treatment plant performance, two types of comparisons can be made: Between equalized and unequalized periods of operation at the same plant (or between equalized and unequalized parallel treatment plant sections during the same period). And between a long-term average performance of those treatment plants that have and do not have equalization facilities; possible comparisons could be based on several years of operating data. Frequency distributions or probability plots of average daily observations have been used, since the available data is generally in the form of average daily values of flow rate, concentration, etc., and to facilitate comparison of the large (annual) data sets. Performance characteristics of unit process or treatment systems may be identified by summarizing operating records in this form.

Probability Plot Characteristics--

A probability plot is a graphic presentation of the cumulative frequency distribution of observations in a data set, presenting the magnitude of individual or groups of observations as a function of their frequency of occurrence. This method has been used to summarize both influent and effluent characteristics of municipal sewage treatment plants.(22) Data have generally been found to be approximately log normally distributed.(23) In such cases the data form a straight line on log probability paper indicating that the $\log_{10}$ of individual observations are normally distributed. The well established characteristics of these distributions may conveniently be used to describe both level and variability of treatment performance. It should be recognized that these distributions apply only to completely random observations. Thus, some approximation may be involved in applying treatment plant influent and effluent characteristics that have occasionally been shown to have significant, periodic influence.(24) A plot of daily average influent suspended solids concentration for one year of operation (Figure 14) illustrates an observed log normal distribution.

The primary operating characteristics illustrated by probability plots are the level of performance and the variability of performance. They are significant because they relate to well established sewage treatment system capabilities, common effluent discharge requirements, and anticipated effects of equalizing input variations.

<u>Level of operating performance</u>--The efficiency of unit process or treatment system is interpretable from the median, mean and range of a given distribution, or set of data. The median may be seen at a glance from the midpoint or 50 percent value of the log distribution. The mean (m_e) of the data set can be computed from the median (m_d) using the relationship:

$$m_e = m_d \exp\left(2.6509\sigma^2_{\log_{10}}\right)$$

where $\sigma_{\log_{10}}$ = the standard deviation of the log distribution, determined graphically as:
$\log_{10}(x)$ @ 50% - $\log_{10}(x)$ @ 50% ± 34.15%

and where $\exp(x) = e^x$

Variability of operating performance--The variability or stability of treatment system performance on a daily average basis can be interpreted from a slope and range of comparable distributions. The slope of a distribution may be characterized by the standard deviation of the logarithms of the observations. Strictly speaking, since it is the logarithms of the observed values that are normally distributed, and not the values themselves, the standard deviation has no meaning in terms of the values; concentration for example. Nevertheless, the slope of a distribution will provide a useful tool for evaluating the variability of comparable data sets, if considered along with the range of values of the distribution covered; for example, by 80 percent (10 percent to 90 percent) or 98 percent (1 percent to 99 percent) of the observations.

Simple visual comparison of distribution slopes should be made with caution. If two distributions have identified median values their slopes will be directly comparable. In this case visual comparison is sufficient to identify differences in variability.

If median values are significantly different the respective ranges of observed values must be examined to assess relative variability in average daily performance. For example, as shown in Figure 15, two log normal secondary effluent BOD distributions having median values of 11 mg/l and 18 mg/l are parallel by visual comparison. However, variability of the two data sets differs by more than a factor of 2, with ranges of observed values encompassing 98 percent of the data (from 1 percent to 99 percent) from 5 mg/l to 23 mg/l, and from 8 mg/l to 40 mg/l.

Application to Plant Operating Records--

A wide variety of process design, operating, and environmental factors contributes to the establishment of these process performance characteristics. Although observed differences between individual data sets presented as comparable may appear to be related to equalization effects, conclusions should be drawn cautiously. Similar patterns observed to correspond to each other in unrelated data sets should increase the probability that the effects are due to equalization. General unavailability of data prevents use of more rigorous quantitative analysis.

Care must be taken in examining the presented figures in order that appropriate associations will be made between the distributions and characteristics of the physical systems they represent. For example, although distributions shown typically provide an estimate of observed median influent and effluent quality, observed median removal efficiencies cannot be deduced; yet the value computed will be reasonably close to that computed from the distribution of observed daily removal efficiencies.

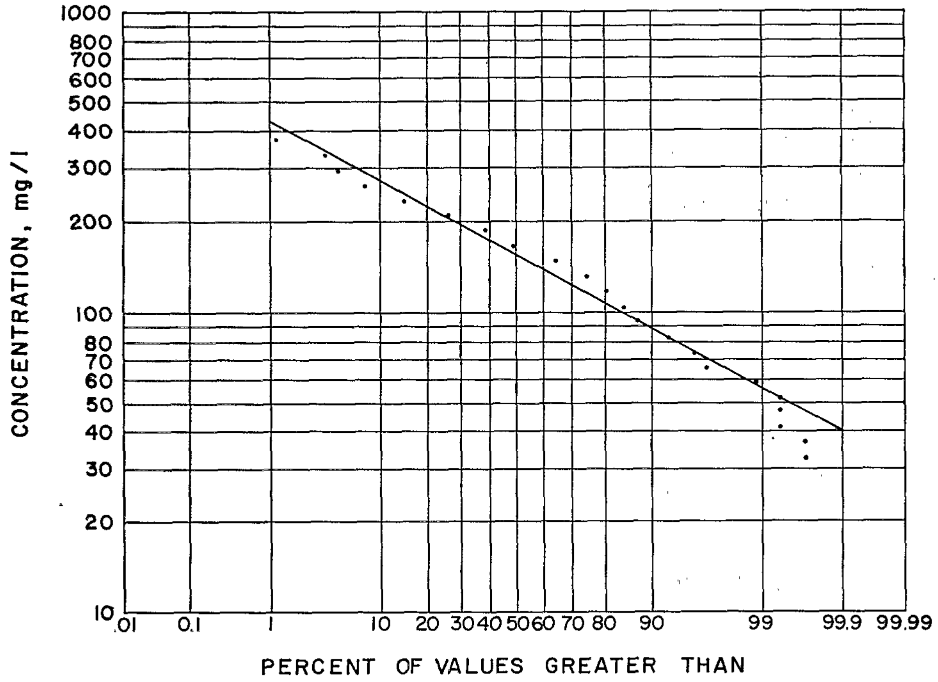

Figure 14. Distribution of log daily average influent TSS concentrations, Ypsilanti Township, Plant 1, 1974

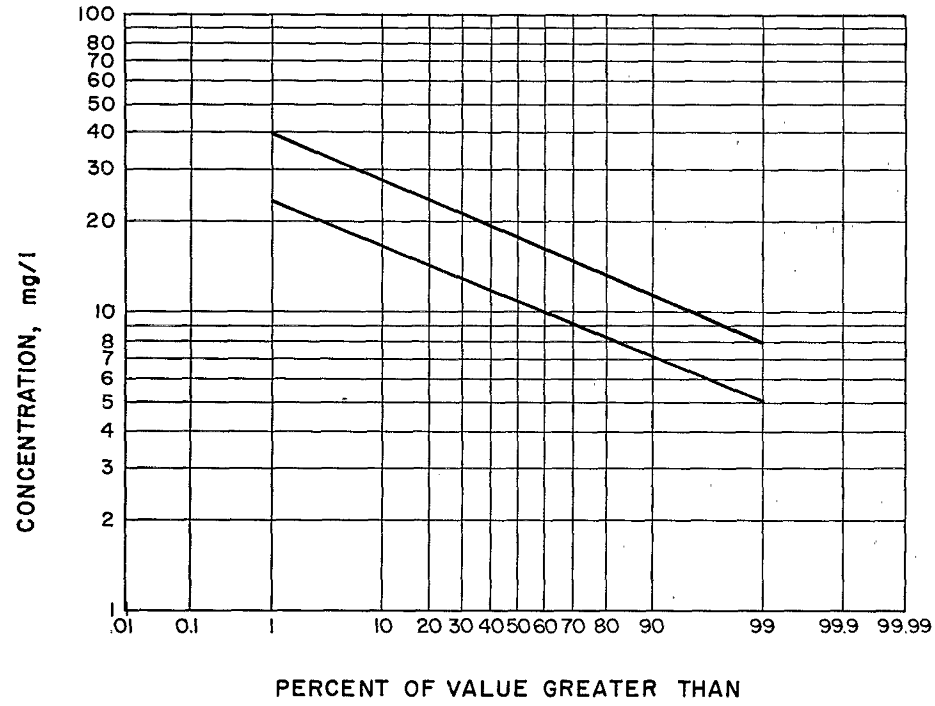

Figure 15. Range difference for distributions of equal slope

Strictly speaking the difference between influent and effluent median should be interpreted as just that; or that the median of observed effluent values was a given percentage of observed influent values.

It should be remembered that the individual observations in each distribution were daily average values, and that the range is the range of daily average values observed in the period of record used.

Influent and effluent distributions--Influent distributions are used to illustrate comparability of conditions imposed on the plants being compared, or of the different periods compared for the same plant. Effluent distributions are used to illustrate observed performance at a given level of treatment (primary, secondary, or tertiary) that can be compared between plants or different operating periods for the same plant. If performance is improved by equalization, the median level of performance would be expected to be lower than for a comparable unequalized period or plant. If stability of performance is improved by equalization, then observed effluent variability or the slope and range of the distribution should be reduced.

There is no implication that effects of equalization on plant unit processes will not result in lower effluent concentrations. Influent and effluent distributions expressed as daily average loads allow the evaluation of effects of equalization on the combined variations in concentration and flow at a given plant. Flow equalization has been shown to have a greater effect than concentration on leveling daily diurnal mass load variations. Such effects are not reflected directly in the probability plots because they are developed from daily composite average observations. Loading variations expressed on this basis will be affected by equalization insofar as daily average concentrations are affected by beneficial influence on process performance. In the case of comparing equalized treatment operations concurrently supplied from the same waste source (illustrated by observations of Ypsilanti Township and the MERL pilot plant studies), it was shown that distributions of concentration and loading observations should be in direct proportion to one another.

Comparison of average annual performance--Using probability plots the mean annual effluent BOD and TSS concentrations were computed from available data for all plants with equalization data compiled in this study. The distributions of average annual effluent composition enable the making of a general comparison and distinction between equalized plant performance and unequalized plant performance.

Performance Requirements

Performance requirements are imposed on all municipal sewage treatment plants by a combination of State and Federal regulatory requirements. The Federal (EPA) minimum requirement for meeting the definition of secondary treatment (25) is the common denominator of performance requirements. State regulatory agencies commonly impose more stringent requirements to meet the needs of local receiving waters.

Performance requirements are typically established in the form of fixed limits on amount of pollutant materials acceptable in a treated effluent. The EPA definition of secondary treatment requires that:

(a) Maximum 30-day average effluent BOD and total suspended solids (TSS) shall not exceed 30 mg/l each.

(b) Maximum 7-day average effluent BOD and TSS shall not exceed 45 mg/l each.

(c) A 30-day average minimum of 85 percent removal of influent BOD and TSS must be achieved.

The ability of a treatment plant to meet imposed performance requirements depends on many interrelated factors pertaining to its design, operation and general influent conditions. A baseline for evaluating treatment plant performance characteristics has been established by a recent statistical study for 27 activated sludge plants in the United States. (26) The plant sample was selected to provide a broad cross-section of typical well operated plants ranging in size from 0.59 mgd to 333 mgd. Considering the distribution of daily average, 7-day average and 30-day average effluent concentrations, statistical analysis was used to compute the distributions of effluent BOD and TSS required for compliance with EPA secondary treatment standards. The resulting distributions (Figure 16) are based on daily operating data for one recorded year from each of the 27 treatment plants. The significance of these distributions as they pertain to evaluating equalization benefits are as follows:

(1) Annual distributions of average daily effluent concentrations coincident with or below the calculated distributions of Figure 16 will consistently comply with secondary treatment requirements.

(2) The calculated secondary effluent concentration distributions were observed to be approximately log normally distributed.

(3) Activated sludge secondary BOD effluent performance may be expected to be slightly more stable than TSS effluent performance, as indicated by the relative slopes of the distributions. This comparison may be expressed numerically in terms of the log standard deviations: log σ_{BOD} = 0.21 and log σ_{TSS} = 0.27. For activated sludge treatment plants designed to meet EPA secondary treatment requirements, mean monthly BOD and TSS of 30 mg/l, the median annual BOD_5 concentration may be expected to be slightly higher than the mean annual TSS concentration ≈ 15 mg/l BOD versus ≈ 10 mg/l TSS.

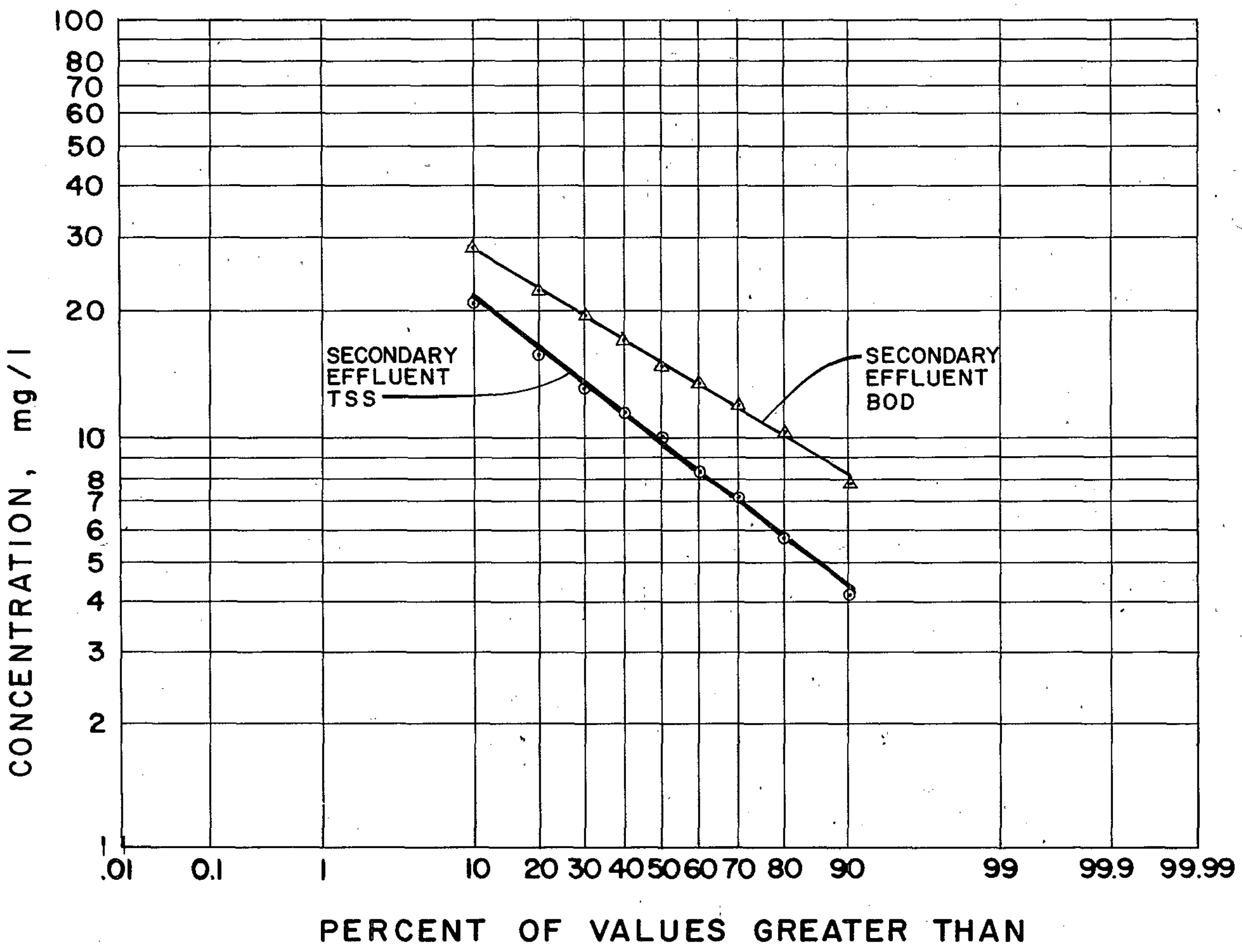

Figure 16. Distribution of log effluent concentrations for 27 activated sludge plants designed to meet EPA secondary treatment requirements

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SECTION 4

SUMMARY OF EXISTING FLOW EQUALIZATION FACILITIES

EQUALIZATION FACILITIES SURVEY

A nationwide survey was conducted to identify communities having flow equalization facilities as an integral part of their wastewater system. And to determine design details, operating practices and costs of the broadest possible range of equalization facilities. Wastewater systems with equalization facilities were located throughout the United States by contacting the ten EPA Regional Offices, and then central and district office personnel in water pollution control agencies in each of the 48 contiguous states. Some states, notably Michigan and New York, maintain a comprehensive list of all municipal treatment systems in the state coded to identify unit process components in each treatment plant. Treatment plants including equalization facilities in those states were thus readily identifiable. In other states central and district office personnel provided plant references from their records and personal contacts. Additional plants were located by references provided in direct interviews with operating personnel of wastewater systems having equalization facilities, and through articles in the current literature.

Survey Scope

The survey of existing equalization facilities was conducted by phone interviews with plant operating personnel, by distribution of written requests, by interviews with state regulatory agency engineering staffs and inspection of state maintained operating records, and by direct site visits and interviews with design engineers and plant operating personnel.

Specific information requested from each wastewater agency using flow equalization is summarized as follows:

1. Treatment and hydraulic capacity of wastewater treatment plant, including plant efficiency and discharge requirements.

2. Flow range and frequency of occurrence of plant influent flow; including minimum, average, and peak dry and wet weather flows.

3. Treatment plant type and schematic diagram.

4. Type and size of equalization structure installed.

5. Location of equalization structure within collection system or sewage treatment plant.

6. Means utilized to fill and empty equalization structure.

7. Aeration or mixing provided in equalization structure.

8. Methods used for scum and solids removal.

9. Odor control provided during operation of flow equalization facility.

10. Type and quantity of flow to equalization facility.

11. Comments or problems associated with operation of the flow equalization facility.

12. Equalization facility cost information including component costs, total construction cost, bid year and month, and yearly operating and maintenance costs.

One hundred ten municipal and sewerage agencies owning and operating 147 equalization systems responded in varying degrees of completeness to the survey. Tables 7, 8, 9, and 10 summarize the physical features and responses of each respondent for facilities smaller and larger than 1 mgd, respectively. An additional 66 equalization facilities are identified for which no detailed information was obtained. These are listed in Table 11.

Principal respondents to the survey included plant operating personnel, municipal engineering staff personnel, and consulting engineers. One or more of these sources provided information for each contact resulting in detailed information. Available time and budget limited practical survey procedures to telephone and written correspondence in the majority of cases. Reliance on these methods resulted in varying degrees of detail obtained from plant to plant. In some cases information received must be considered qualitative. This is particularly the case for plant flows reported and used in establishing peak-to-average flow ratios for typical dry and wet weather periods.

TABLE 7. EQUALIZATION FACILITIES SUMMARY: PLANTS SMALLER THAN 1 MGD--TREATMENT PLANT CHARACTERISTICS

Treatment Plant and Location[a]	Plant Size, mgd[d] Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treatment[b]	When Equalization Constr[c]	Location of Equalization Facility	Type of Structure
1. California Institute for Women Fontana, CA	0.2	1.50		Act sludge	S	AP with expansion	Following headworks	Tank conc covered
2. Deer Creek STP El Dorado, CA	0.33 (0.5)	3.0		Act sludge	S,F	WP	Following headworks	Basin earthen, lime stabilized
3. Rancho Bernardo STP San Diego, CA	1.0 (1.75)	1.66	2.57	Act sludge	S	AP with upgrading	Following headworks	PVC liner covered
4. Willits STP Willits, CA	0.64	1.56	3.75	Act sludge	S	AP with expansion	Following headworks	5 basins, 1 asphalt lined, 4 unlined
5. Northeast WWTP Oskalooska, IA	0.98 (3.3)	1.15	15.95	Trickling filter	S	AP with upgrading	Following meter pit	Basin earthen lined
6. Southwest WWTP Oskalooska, IA	0.81 (2.5)	1.4	1.96	Act sludge	S	WP	Following inlet struc	2 basins unlined
7. Boyne City WWTP Boyne City, MI	0.7 (2.3)	1.43	4.29	Act sludge	S,F	AP with upgrading	Following headworks bar screen	Basin clay lining
8. Dimondale WWTP Dimondale, MI	0.2			Act sludge	S	AP No other change	Following headworks	Tank conc
9. Essexville STP Essexville, MI	0.75 (1.87)	1.14	2.99	Trickling filter	S	AP with upgrading	Following wet well	Tank conc
10. Dawson WWTP Dawson, MN	0.26	2.0	3.29	Ox ditch	S	WP	Following headworks	Basin conc
11. Blackwood WWTP Blackwood, NJ	0.63	1.28	1.60	Act sludge	S	WP	Following headworks	Tank conc
12. Borough of Palmyra WWTP, Burlington County, NJ	0.53	1.75		Trickling filter	S	AP No other change	Following pump station	Tank conc
13. Clementon STP Clementon, NJ	0.64	1.71	1.95	Act sludge	S	AP No other change	Following headworks	Tank steel

(continued)

TABLE 7 (continued)

Treatment Plant and Location[a]	Plant Size,[d] mgd Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treat-ment[b]	When Equali-zation Constr[c]	Location of Equalization Facility	Type of Structure
14. E. Windsor WWTP E. Windsor, NJ	0.20	1.12	1.59	Act sludge	S	AP with expansion	Following pump station	Tank conc
15. Service Area 3-S N.J. Turnpike Auth Cherry Hill Twp, NJ	0.08	1.55	2.73	Trickling filter	S,F	AP with expansion	Following headworks	Tank conc
16. Service Area 4-N N.J. Turnpike Auth Mt. Laurel, NJ	0.80			Trickling filter	S	AP with expansion	Following wet well	Tank conc
17. Ramblewood STP Mt. Laurel, NJ	0.5	1.21	1.36	Act sludge	S	AP No other change	Before headworks	Tank conc
18. Stratford Sewerage Authority, Camden County, NJ	1.0	1.80	3.6	Trickling filter	S	AP with upgrading	Following headworks	Tank conc
19. Chautauqua STP Chautauqua Inst, NY	0.84 (2.1)	1.79	2.62	Act sludge	S	AP with upgrading	Following headworks	Tank conc
20. Fishkill WWTP Fishkill, NY	0.40 (0.60)	2.5		Act sludge	S,NR, F	WP	Before wet well	Pipe conc
21. Hauppauge WWTP Suffolk Co., NY	0.10 (0.29)	1.15		Act sludge	S,N,F	WP		Tank conc
22. Mohawk WWTP Colonie, NY	0.025 (0.040)			Act sludge	S	WP	Before headworks	Tank steel
23. Oakwood Knolls South S.D. Wappinger, NY	0.20 (0.20)	1.09		Act sludge	S	AP with expansion	Before wet well	Pipe conc
24. Ravenwood WWTP Colonie, NY	0.038			Act sludge	S,F	WP	Before headworks	Tank conc
25. Strathmore Ridge S.D. #8 Suffolk Co., NY	0.05 (0.05)			Rotating disc	S,F	WP	Following headworks	Tank conc covered
26. Waverly STP Waverly, NY	0.6	3.33		Act sludge	S	AP	Following headworks	Tank conc
27. Weatherford WWTP Weatherford, OK	0.52 (1.2)	1.59	1.72		S	AP No other change	2 miles upstream of WWTP	3 basins unlined

(continued)

TABLE 7 (continued)

Treatment Plant and Location[a]	Plant Size,[d] mgd Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treatment[b]	When Equalization Constr[c]	Location of Equalization Facility	Type of Structure
28. Fayette Twp McAllisterville, PA	0.13	1.31	2.29	Act sludge	S	AP	Following clarifier	Tank conc
29. Ingelside WWTP Ingelside, TX	0.50	1.27	3.18	Act sludge	S	AP	Following pump station	Tank conc
30. Woodsboro WWTP Woodsboro, TX	0.16 (0.30)	1.23	2.31	Ox ditch	S	WP	Following wet well	Basin unlined
31. Salina STP Salina, UT	0.30 (0.60)	1.5	1.5	Act sludge	S	WP	Following primary clarifiers	Basin bentonite lined
32. Shelburne Fire Dist #1 WWTP, Shelburne, VT	0.38	2.0	7.2	Act sludge	S	AP with expansion	Following headworks	Tank conc
33. Lake Samish STP Whatcom County Sewer Dist #12 Bellingham, WA	0.23	2.61		Ox pond	Intermediate	WP	Following flow splitting structure	2 basins PVC liner
34. Stevens Pass & Yodelin WWTP Stevens Pass, WA	0.062 (2.0)	1.46	1.46	Act sludge	S,F	WP	Upstream 10,000 ft from WWTP	Tank conc covered
35. NeKoosa WWTP NeKoosa, WI	0.498 (0.750)	1.13	1.29	Act sludge	S	WP	Following wet well	Basin bentonite
36. Northern Moraine STP Glenbeulah, WI	0.60 (0.60)	1.0	1.67	Act sludge	S	WP	Following pump station	Tank conc covered
37. Port Edwards WWTP Port Edwards, WI	0.56 (0.56)	1.23	1.68	Act sludge	S	WP	Following wet well	Bentonite
38. Valders WWTP Village of Valders, WI	0.15 (0.376)	1.3	2.63	Act sludge	S	WP	Following wet well	Basin asphalt

[a]STP = sewage treatment plant
WWTP = wastewater treatment plant

[b]S = secondary
F = multi-media filtration
NR = Nitrogen removal
N = Nitrification

[c]AP = After primary
WP = With primary

[d]Treatment plant capacities listed are the design capacity. Values in parens are hydraulic capacity where reported as different from treatment capacity.

TABLE 8. EQUALIZATION FACILITIES SUMMARY: PLANTS SMALLER THAN 1 MGD--EQUALIZATION SYSTEM CHARACTERISTICS

Treatment Plant and Location [a]	Size gal x 10^6	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Structure	Facility Staff Comments
1. California Institute for Women [b] Fontana, CA	0.10	50%	In-line	Diffusers	Manual	None	Gravity in Pump out	Very effective in controlling diurnal variations and thus maintaining proper control
2. Deer Creek STP El Dorado, CA	3.0	909%	In-line	Floating aerators	Manual	None	Gravity in and out	
3. Rancho Bernardo STP [b] San Diego, CA	0.20	20%	Side-line	None	None	None	Pump in Gravity out	Fabritank
4. Willits STP Willits, CA	14.8	2,313%	Side-line	None	Manual	None	Pump in Gravity out	
5. Northeast WWTP Oskalooska, IA	5.5	561%	Side-line	None	None	None	Gravity in and out	
6. Southwest WWTP Oskalooska, IA	4.1	506%	Side-line	None	None	None	Gravity in Pump out	
7. Boyne City WWTP Boyne City, MI	74.5	1,064%	In-line	Diffusers	None		Pump in Gravity out	
8. Dimondale WWTP Dimondale, MI	0.138	69%	Side-line	None	None	None	Pump out and out	Equalization facility installed when flows were larger. No longer in use.
9. Essexville STP Essexville, MI	0.6	80%	In-line	Mixers	None	Chlorine	Pump in Gravity out	Grit removal problem
10. Dawson WWTP Dawson, MN	0.26	100%	In-line	2 horiz drum rollers	Manual	None	Pump in Gravity out	Race track shape

(continued)

TABLE 8 (continued)

	Treatment Plant and Location[a]	Size gal x 10^6	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Structure	Facility Staff Comments
11.	Blackwood WWTP Blackwood, NJ	0.25	40%	In-line	Diffusers	None	None	Gravity in Pump out	
12.	Borough of Palmyra WWTP, Burlington County, NJ	0.132	25%	In-line	Diffusers	Manual	None	Pump in and out	
13.	Clementon STP Clementon, NJ	0.35	55%	In-line	Diffusers	Manual	Chlorine	Pump in and out	
14.	E. Windsor WWTP E. Windsor, NJ	0.20	100%	In-line	Diffusers	None	None	Pump in Gravity out	Grit removal has been a major problem
15.	Service Area 3-S N.J. Turnpike Auth Cherry Hill Twp, NJ	0.085	106%	In-line	Fixed aerators	Auto	None	Gravity in Pump out	
16.	Service Area 4-N N.J. Turnpike Auth Mt. Laurel, NJ	0.017	2%	Side-line	Fixed aeration	Manual	None	Gravity in Pump out	
17.	Ramblewood STP Mt. Laurel, NJ	0.102	20%	In-line	Diffusers	Manual	None	Pump in and out	Silt & grit problem on bottom. No grit removal prior to facility.
18.	Stratford Sewerage Authority, Camden County, NJ	0.297	30%	In-line	Mixers & diffusers	Manual	Chlorine	Gravity in pump out	Good shock recovery; no problems.
19.	Chautauqua STP Chautauqua Inst, NY	0.037	4%	Side-line	Mixers	Manual	None	Gravity in Pump out	
20.	Fishkill WWTP Fishkill, NY	0.025		Side-line	None	None		Gravity in and out	
21.	Hauppauge WWTP Suffolk Co., NY	0.072		In-line	Diffusers	Manual	Lime	Pump in and out	

(continued)

TABLE 8 (continued)

Treatment Plant and Location[a]	Size gal $\times 10^6$	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Structure	Facility Staff Comments
22. Mohawk WWTP Colonie, NY	0.012		In-line	Diffusers	Manual	None	Gravity in Pump out	
23. Oakwood Knolls South S.D.[c] Wappinger, NY			Side-line	Diffusers	Manual	None	Gravity in and out	Has not been used
24. Ravenwood WWTP Colonie, NY	0.024		In-line	Diffusers	Manual	None	Pump in and out	
25. Strathmore Ridge S.D. #8 Suffolk Co., NY	0.010		In-line	Diffusers	Manual	None	Gravity in Pump out	
26. Waverly STP Waverly, NY	0.12	20%	Side-line	Mixers & diffusers	Auto	None	Gravity in Pump out	Under construction
27. Weatherford WWTP Weatherford, OK	8.15	1,567%	In-line	None	None	None	Gravity in Pump out	Handles 50% residential, 50% industrial
28. Fayette Twp McAllisterville, PA	0.039	30%	In-line	Diffusers	Manual	None	Pump in Gravity out	
29. Ingelside WWTP Ingelside, TX	0.20		In-line	Diffusers	Manual	None	Pump in and out	Need a way to keep the flow regulated
30. Woodsboro WWTP Woodsboro, TX			Side-line	None	None	None	Pump in Gravity out	
31. Salina STP Salina, UT	0.5	167%	In-line	Diffusers	None	None	Gravity in Pump out	
32. Shelburne Fire Dist #1 WWTP, Shelburne, VT	0.15	39%	In-line	Diffusers & mixers	Manual	None	Pump in Gravity out	

(continued)

TABLE 8 (continued)

Treatment Plant and Location[a]	Size gal $\times 10^6$	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Structure	Facility Staff Comments
33. Lake Samish STP[b] Whatcom County Sewer Dist #12 Bellingham, WA	0.202	88%	In-line	None	Manual	None	Pump in and out	
34. Stevens Pass & Yodelin WWTP[b] Stevens Pass, WA	0.076	123%	In-line	Diffusers	Manual	None	Gravity in Pump out	
35. NeKoosa WWTP NeKoosa, WI	0.492		Side-line	None	None	None	Pump in Gravity out	Has not been used
36. Northern Moraine STP Glenbeulah, WI[d]	0.024		In-line	Diffusers	Manual	None	Pump in Gravity out	
37. Port Edwards WWTP Port Edwards, WI	0.120		Side-line	None	None	None	Pump in Gravity out	
38. Valders WWTP Village of Valders, WI	0.82		Side-line	None	Manual	None	Pump in Gravity out	Bottom should be resealed every five years.

[a] STP = sewage treatment plant
WWTP = wastewater treatment plant

[b] No ventilation system.

[c] Ventilation fan.

[d] Ventilation system: screen in top of cover.

TABLE 9. EQUALIZATION FACILITIES SUMMARY: PLANTS LARGER THAN 1 MGD--TREATMENT PLANT CHARACTERISTICS

	Treatment Plant and Location	Plant Size,[a] mgd Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treat-ment	When Equali-zation Constr	Location of Equalization Facility	Type of Structure
1.	Cypress Creek WWTP Florence, AL	10.0	1.40	5.00	Act sludge	S	WP	Following headworks	2 basins unlined
2.	5 Mile Creek WWTP 5 Mile Creek, AL	10				S			Concrete lined pond
3.	Valley Creek WWTP Birmingham and Bessemer, AL	35	1.31	4.23	Act sludge	S,N	AP with upgrading	Following headworks	Basin conc lined
4.	Central Contra Costa San Dist Walnut Creek, CA	60	1.60	4.67	Act sludge	S	AP with upgrading	Following primary sed tanks	3 ponds
5.	Chino Basin Muni Water Dist Cucamonga, CA	3.0 (3.0)	1.60	2.40	Act sludge	S	AP No other change	Following splitter structure	Basin earthen lined
6.	Laguna WWTP Santa Rosa, CA	2.5 (15)	1.60	6.00	Act sludge	S	WP	Following primary sed tanks	Basin conc lined
7.	Pismo Beach Water Reclamation Plant Pismo Beach, CA	1.2			Act sludge	S	AP No other change	Between primary sett tanks & aera-tion tanks	Basin gunite lined
8.	Redlands STP Redlands, CA	6.0 (12.0)	1.43		Trickling filter	S	AP with expansion	Following trickling filter	Basin asphalt lined
9.	Rohnert Park WWTP Rohnert Park, CA	1.7 (12.0)	1.78	3.70	Act sludge			Following headworks	Basin gunite lined
10.	Rossmore Sanitation Inc., Laguna Hills, CA	4.0	5.0	6.12	Act sludge	S	WP	Following control structure	Basin conc gunite lined
11.	Sacramento Regional WWTP, Sacramento, CA	115 (240)	1.36	10.33	Act sludge	S	WP	Following primary sed tank	3 basins, 2 conc lined, 1 earthen lined
12.	San Luis Rey WWTP Oceanside, CA	4.8	1.75	1.75	Act sludge	S	WP	Following headworks	Tank conc covered
13.	Valley Community Service Dist STP Dublin, CA	4.0 (12.0)	2.00	3.00	Act sludge	S,F	WP	Following primary treatment	Basin asph lined Tank conc
14.	Water Reclamation Plant, Livermore, CA	19.7	1.8	2.33	Act sludge & trickling filter	S,F	AP with upgrading	Following headworks	2 basins, gunite lined, earthen lined
15.	Upper Thompson WWTP Estes Park, CO	1.5 (2.0)	1.2	2.8	Act sludge	S,P	WP	Following effluent pumping	Tank conc

(continued)

TABLE 9 (continued)

	Treatment Plant and Location	Plant Size,[a] mgd Treat (Hyd)	P/A (W.W.)	P/A (W.W.)	Plant Type	Degree of Treat-ment	When Equali-zation Constr	Location of Equalization Facility	Type of Structure
16.	Broomfield WWTP Broomfield, CO	3.6	1.03	1.10	Act sludge	S	AP with upgrading	Following headworks	Basin earthen lined
17.	Clavey Road STP Highland Park, IL	17.8 (36)	1.22	6.67	Act sludge	S,F	AP with upgrading	Following distribution chamber	6 tanks conc covered
18.	Ankeny WWTP No. 3 Ankeny, IA	1.2 (3.0)	1.30	5.45	Act sludge	S	WP	Following flow splitter	Basin unlined
19.	Lawrence WWTP Lawrence, KA	18 (45)	1.22	4.44	Act sludge	S	WP	Following settling basin	Basin conc lined
20.	Manhatten WWTP Kansas City, KA	6.25 (12.5)	3.10	6.25	Act sludge	S	WP	Following primary settling basin	Basin clay lining
21.	Corrina STP Corrina, MA	1.2 (4.32)	2.42	3.60	Act sludge	S	WP	Following headworks	Tank conc
22.	Paris Utility Dist Plant, South Paris, MA	1.85 (10.9)	3.24	5.89	Act sludge	S	WP	Following headworks	Tank conc
23.	Chapaton Pumping Station, Detroit, MI		N/A	N/A	Pumping station only		WP	Before WWTP	Tank conc
24.	Dowagiac STP Dowagiac, MI	2.0 (4.0)	1.40	8.00	Act sludge	S,P	WP	Follows flow diverter	2 basins, asph lined clay lined
25.	Eaton Rapids WWTP Eaton Rapids, MI	1.2	2.0		Act sludge	S,P	AP No other change	Following grit chamber	2 tanks conc
26.	E. Lansing WWTP E. Lansing, MI	18.7 (48)	1.4	3.49	Act sludge	S,P,F	AP with upgrading	Following grit chamber	Tank conc covered
27.	Grand Haven Spring-lake WWTP, Grand Haven, MI	5.0 (7.0)	1.43	4.00	Act sludge	S,P	AP with expansion	Following headworks	Tank steel
28.	Grand Rapids WWTP Grand Rapids, MI	66 (150)	1.75	3.75	Act sludge	S,P, N,F	WP	Following primary treatment	Basin conc lined
29.	Hancock St. Pollu-tion Control Fac Saginaw, MI	32 (120)	1.13	3.64	CSO	Storm pri-mary	WP	At pump sta 5 miles up-stream of plant	Tank conc
30.	Jackson STP Jackson, MI	17	1.2	1.47	Act sludge	S,P	WP	Following primary treatment	Basin conc

(continued)

TABLE 9 (continued)

Treatment Plant and Location	Plant Size,[a] mgd Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treat-ment	When Equali-zation Constr	Location of Equalization Facility	Type of Structure
31. Lansing STP Lansing, MI	45 (50)	1.02	1.76	Act sludge	S,P	AP with expansion	Follows primary treatment	Tank conc
32. Midland WWTP Midland, MI	6.5 (13.25)	1.67	2.08	Trickling filter	S,P,F	AP with upgrading	Following flow splitter	Tank conc
33. Mt. Clemens STP Mt. Clemens, MI	4.0	1.07		Trickling filter	S		1 mile upstream of plant	Tank conc
34. Pontiac WWTP Pontiac, MI	25.5 (50)	1.4	2.5	Act sludge	S,P,F	AP with upgrading	Following primary treatment	Tank conc covered
35. Port Huron WWTP Port Huron, MI	20 (58)	2.36	3.14	Act sludge	S,P	AP with expansion	Following primary sed tanks	Tank conc
36. Southeastern Oakland Co., Red Run Drain Madison Heights, MI	63			Storm flow control			12 miles upstream of WWTP	Tank conc
37. Tecumseh WWTP Tecumseh, MI	1.4	1.31	1.31	Act sludge	S,P	AP No other mods	Following headworks	Tank conc covered
38. Trenton WWTP Trenton, MI	7.5 (17.5)	1.2	2.00	Act sludge	S,P	AP with upgrading	1/4 mile upstream of plant	Basin asph lined
39. Walled Lake Novi STP Novi, MI	2.1	1.51	3.33	Act sludge	S,P,F	WP	Following wet well	Tank conc
40. Warren STP Warren, MI	36 (50)	1.26	4.69	Act sludge	S,P,F	AP No other change	Following grit chamber	Basin conc lined, covered
41. Ypsilanti Twp. WWTP Ypsilanti, MI	4.0	1.5		Act sludge	S,P	AP No other change	Before bar rack	Tank conc covered
42. Brookhaven WWTP Brookhaven, MS	2.0 (5.0)	1.36	1.32	Act sludge	S	AP with upgrading	Following headworks	Basin earthen lined
43. Greneda WWTP Greneda, MS	3.5	1.27	2.31	Act sludge	S	AP No other change	Following headworks	Basin earthen lined
44. Shoal Creek WWTP Joplin, MO	4.5	3.03	5.39	Act sludge	S	AP No other change	Parallel to headworks	Basin clay lined
45. Warrensburg STP Warrensburg, MO	1.7 (3.4)	1.73	4.93	Ox ditch	S	WP	Following wet well	Basin conc lined
46. Reno Sparks STP Reno, NV	22			Act sludge	S,P	AP No other change	Before plant	Interceptor sewers

(continued)

TABLE 9 (continued)

	Treatment Plant and Location	Plant Size,[a] mgd Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treat-ment	When Equali-zation Constr	Location of Equalization Facility	Type of Structure
47.	Merrimack WWTP Merrimack, NH	5.0 (10.0)	1.26	1.05	Act sludge	S	WP	Following influent pump station	2 tank conc
48.	Elmwood STP Marlton, NJ	1.5	1.30		Act sludge & trickling filter	S	AP No other change	Following influent pump station	Tank steel
49.	New Providence STP New Providence, NJ	4.5 (6.0)	1.11	3.89	Trickling filter	S	WP	Following influent pump station	Tank steel
50.	Woodstream STP Marlton, NJ	1.25 (3.0)	1.31		Act sludge	S,F	AP with expansion	Following influent pump station	Tank steel
51.	Amherst STP Amherst, NY	15.0		1.88	Act sludge	S,P,F	WP	Following grit chamber	2 tanks conc
52.	Delaware STP Delaware, OH	2.5 (5-6)	1.28	2.22	Act sludge	S	AP with upgrading	Following headworks	Tank conc
53.	Hatfield Twp WWTP Colmar, PA	(4.5)	2.25	4.00	Act sludge	S,F	WP	Following raw sewage pumps	2 tanks conc
54.	Oil City STP Oil City, PA	8.0 (14)	1.33	4.13	Trickling filter	S	AP with upgrading	Following headworks	Tank conc
55.	Watertown STP Watertown, SD	4.0 (6.0)	1.22	2.00	Act sludge	S	AP with expansion	Following control house	Basin clay lined
56.	Amarillo River Rd WWTP, Amarillo, TX	12.0 (20)	1.2		Act sludge	S	WP	Following primary clarifier	Basin earthen lined
57.	Duck Creek WWTP Garland, TX	11.5 (30)	1.5	3.43	Trickling filter	S,P, C,F	AP with upgrading	Following headworks	Basin conc lined
58.	Guadalupi-Blanco River, Victoria, TX	3.0 (13.0)	1.33	4.00	Act sludge	S	WP	Following wet well	Basin unlined
59.	Midland WWTP Midland, TX	6.0 (8.0)			Trickling filter	S	WP	Following settling basin	Basin conc
60.	Odessa STP Odessa, TX	6.0 (8.5)	1.20	2.55	Act sludge	S	AP with expansion	Following primary clarifiers	2 tanks
61.	Oso WWTP Corpus Christi, TX	12.0 (15.0)		4.09	Act sludge	S	WP	Before plant	Interceptor sewer
62.	Sandy Suburban STP Sandy City, UT	1.5	1.85	2.04	Act sludge	S	WP	Following primary effluent	Basin plastic & conc lined

(continued)

TABLE 9 (continued)

	Treatment Plant and Location	Plant Size,[a] mgd Treat (Hyd)	P/A (D.W.)	P/A (W.W.)	Plant Type	Degree of Treatment	When Equalization Constr	Location of Equalization Facility	Type of Structure
63.	Lower Potomac Poll Control Plant Fairfax Co., VA	36 (68)	1.43	2.04	Advanced waste treat	S,P,C, NR,F		Following secondary chlorination	Basin PVC lining
64.	Moore Creek WWTP Charlottesville, VA	15	1.4	5.0	Advanced waste treat	S,P, N,F	WP	Following headworks	Basin
65.	Potomac Reg WWTP Woodbridge, VA	12.0 (32)				S	WP	Following secondary chlorination	Basin PVC lined
66.	Roanoke WWTP Roanoke, VA	35 (90)	1.40	3.45	Advanced Waste Treat	S,P, N,F	WP	Following primary sed basins	2 tanks conc
67.	Upper Occoquan Sewage Authority Manassas Park, VA	15.0 (30)		6.00	Act sludge	S,P,C, N,F	WP	3 locations: a) Following headworks b) Following 2nd stage recarb basin c) Before filters	a) Emerg pond b) 2 ballast ponds, concrete lined, 2 backup ponds c) Filter backwash eq tank
68.	Lacy-Olympia-Tumwater Thurston County STP Olympia, WA	14 (19.2)	2.35	2.71	Act sludge	S	AP with upgrading	Following headworks	2 basins asphalt lined
69.	Muni of Metro Seattle, West Point STP Seattle, WA	350	1.36	4.0	CSO	Primary	WP	Before plant	Interceptor sewers
70.	Ada STP Bluefield, WV	1.2	1.17	9.5	Act sludge	S	AP with expansion	2 miles upstream of STP	2 tanks conc
71.	Westside STP Bluefield, WV	2.8 (3.5)	1.79	1.43	Act sludge	S	AP with upgrading	Following lift station	2 tanks
72.	Chippewa Falls WWTP Chippewa Falls, WI	3.50	1.27			S	AP with expansion	Upstream of plant 1600 ft	Basin asphalt
73.	Reedsburg STP Reedsburg, WI	1.7			Act sludge	S	WP with upgrading	Following wet well	Basin unlined
74.	Dry Creek WWTP Cheyenne, WY	4.5 (9.0)	1.10	1.10	Act sludge & trickling filter	S	WP	Following headworks	2 basins earthen lined

[a]Treatment plant capacities listed are the design capacity. Values in parens are hydraulic capacity where reported as different from treatment capacity.

TABLE 10. EQUALIZATION FACILITIES SUMMARY: PLANTS LARGER THAN 1 MGD--EQUALIZATION FACILITY CHARACTERISTICS

Treatment Plant[f] and Location	Size gal $\times 10^6$	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
1. Cypress Creek WWTP Florence, AL	4.0	40%	Side-line	Floating aerators	Manual	None	Pump in Pump out	Need concrete lining
2. 5 Mile Creek WWTP 5 Mile Creek, AL	30	300%	Side-line	Floating aerators	Manual	None	Gravity in and out	Not operational
3. Valley Creek WWTP Birmingham and Bessemer, AL	5.0	14%	Side-line	Floating aerators	Manual	None	Gravity in Pump out	
4. Central Contra Costa San Dist Walnut Creek, CA[a]	164	273%		None			Gravity in and out	
5. Chino Basin Muni Water Dist Cucamonga, CA	1.2	40%	Side-line	Mixers	None	None	Pump in Gravity out	Solids on sides and some sedimentation on bottom
6. Laguna WWTP Santa Rosa, CA	17	680%	Side-line	None	Manual	None	Pump in Gravity out	
7. Pismo Beach Water Reclamation Plant Pismo Beach, CA	0.375	31%	Side-line	Aeration for cleaning purpose	Manual	None	Gravity in Pump out	Tourism causes wide flow variations
8. Redlands STP Redlands, CA	1.25	21%	In-line	None	Manual	None	Gravity in Pump out	
9. Rohnert Park WWTP Rohnert Park, CA	0.75	44%	In-line	Floating aerators	None	None	Gravity in Pump out	
10. Rossmore Sanitation Inc., Laguna Hills, CA	2.5	63%	In-line	Diffusers & floating aerator	Manual	None	Gravity in Pump out	

(continued)

TABLE 10 (continued)

	Treatment Plant and Location	Size gal x 10^6	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
11.	Sacramento Regional WWTP, Sacramento, CA	207	180%	Side-line	None	Water truck	None	Pump in Gravity out	
12.	San Luis Rey WWTP Oceanside, CA	1.08	23%	In-line	Diffusers	Grit chamber, no washdown	None	Gravity in and out	Plant load to half capacity. "Best part of facility is flow eq. tank."
13.	Valley Community Service Dist STP Dublin, CA	a) 2.3 b) 0.15	59%	a) All flow b) Filter backwash	None	Automatic sprinkler washdown for tank	Chlorine None	a) Grav, pump in, grav out b) Pump in, grav out	Basin normally operates as storage for primary effluent. In emergencies can operate as excess flow holding basin for raw sewage
14.	Water Reclamation Plant, Livermore, CA	a) 10 b) 18	142%	Side-line	None	Manual	Chlorine	Pump in and out	
15.	Upper Thompson WWTP Estes Park, CO	0.238	16%	In-line	None	Manual	None	Pump in Gravity out	Grease buildup is a problem.
16.	Broomfield WWTP Broomfield, CO	2	56%	In-line	Floating aerators	None	None	Gravity in Pump out	Sedimentation problems in areas not receiving proper aeration
17.	Clavey Road STP Highland Park, IL	21	118%	Side-line	None	Traveling bridge for 2 tanks. Manual wash-down for 4 tanks.	Potassium permanga-nate--odor. Sodium hy-pochlorite--disinfection & odor.	Gravity in Pump out	
18.	Ankeny WWTP No. 3 Ankeny, IA	4.0	333%	Side-line	None	None	None	Pump in Gravity out	Only maintenance is scum removal from lagoon banks

(continued)

TABLE 10 (continued)

	Treatment Plant and Location	Size gal x 10^6	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
19.	Lawrence WWTP Lawrence, KA	4.5	25%	Side-line	None	Manual	None	Gravity in and out	Has not been used as of 10/10/76
20.	Manhatten WWTP Kansas City, KA	0.25	0.04%	Side-line	None	Manual	None	Pump in Gravity out	
21.	Corrina STP Corrina, MA	0.4	33%	In-line	Diffusers	Manual	None	Gravity in Pump out	Equalization tank made into surge tank
22.	Paris Utility Dist Plant, South Paris, MA	0.21	11%	In-line	None	None	None	Gravity and pump in, pump out	Eq tanks used to neutralize alkaline beam house waste with acid tanning waste
23.	Chapaton Pumping Station, Detroit, MI [b]	28		Side-line	Diffusers	Manual	Ozone used during cleaning	Pump in Gravity out	This is a pumping station and storage facility.
24.	Dowagiac STP Dowagiac, MI	4.9	245%	Side-line	None	Manual	None	Gravity in Pump out	
25.	Eaton Rapids WWTP Eaton Rapids, MI	0.14	12%	In-line	None	None	None	Gravity in Pump out	
26.	E. Lansing WWTP[c] E. Lansing, MI	5.0	27%	In-line	Diffusers	Manual	None	Gravity in Pump out	
27.	Grand Haven Spring-lake WWTP, Grand Haven, MI	0.8	16%	Tannery waste only	None	Manual	Caustic soda	Pump in Gravity out	Tannery waste only to equal tank. Odor problems.
28.	Grand Rapids WWTP Grand Rapids, MI	10	15%	Side-line	None	Manual	None, but chlorine avail for emergencies	Gravity or pump in and out	Primary capacity is 150 mgd. Secondary capacity is 90 mgd. Design capacity is 66 mgd.

(continued)

TABLE 10 (continued)

	Treatment Plant and Location	Size gal x 10^6	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
29.	Hancock St. Pollution Control Fac Saginaw, MI	3.6	11%	Side-line	None	Automatic	Designed for sodium hypochloride. Not used yet.	Pump in Gravity out	Some structural problems, only used once, too early to tell effectiveness.
30.	Jackson STP Jackson, MI	12.5	74%	Side-line	None	Manual	None	Pump in Gravity out	
31.	Lansing STP Lansing, MI	4.0	9%	Side-line	None	Manual sludge gatherers	Emergency chlorination	Gravity in Pump out	
32.	Midland WWTP Midland, MI	3.25	50%	Side-line		Manual	Chlorine (OF condition only)	Gravity in and out	Grit is the biggest problem. Will be adding greater grit handling capacity.
33.	Mt. Clemens STP Mt. Clemens, MI	30	750%	Side-line	Floating aerators	Manual	None	Pump in and out	
34.	Pontiac WWTP[d] Pontiac, MI	3.0	12%	Side-line	None	Manual	Ozone ferrachloride	Gravity in Pump out	
35.	Port Huron WWTP Port Huron, MI	5.7	29%	Side-line	None	Automatic	Chlorine	Pump in Gravity out	Flow equalization is effective and can cut operating costs.
36.	Southeastern Oakland Co., Red Run Drain Madison Heights, MI	63	100%	Side-line	None	Auto ceiling nozzles	Sodium hypochlorite	Gravity in Pump out	Red run drain 11,000 feet long
37.	Tecumseh WWTP[d] Tecumseh, MI	1.0	71%	Side-line	Diffusers	Manual	None	Pump or gravity in & out	Flow equalization produces more consistent effluent
38.	Trenton WWTP Trenton, MI	13.5	180%	Side-line	None	Manual	None	Gravity in Pump out	Some pumping and gate problems. Keep 1 ft depth to arrest plant growth.

(continued)

TABLE 10 (continued)

Treatment Plant and Location	Size gal $\times 10^6$	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
39. Walled Lake Novi STP Novi, MI	0.315	15%	Side-line	Diffusers	None	None	Pump in Gravity out	
40. Warren STP [e] Warren, MI	50	139%	Side-line	None	Micro-strainer, low press, fire hose high press	Chlorination capability, not used	Gravity in Pump out	Basin not used for routine daily flow equalizing
41. Ypsilanti Twp.[d] WWTP Ypsilanti, MI	0.624	16%	In-line	Diffusers	None	None	Pump in and out	
42. Brookhaven WWTP Brookhaven, MS	81	4,050%	Side-line	None	None	None	Pump in Gravity out	Works well
43. Greneda WWTP Greneda, MS	72	2,057%	Side-line	None	None	None	Pump in Gravity out	Algae in pond getting into plant
44. Shoal Creek WWTP Joplin, MO	12.2	271%	Side-line	None	None	None	Gravity in and out	No problems
45. Warrensburg STP Warrensburg, MO	0.53	31%	Side-line	None	Manual	None	Gravity in and out	Equalization tank has not been used
46. Reno Sparks STP Reno, NV			Side-line	None	None	None	Gravity in and out	
47. Merrimack WWTP Merrimack, NH	0.96	19%	In-line	Floating aerators	Manual	None	Pump in Gravity out	

(continued)

TABLE 10 (continued)

Treatment Plant and Location	Size gal x 10^6	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
48. Elmwood STP Marlton, NJ	0.034	2%	Side-line	Diffusers	Manual	None	Pumped in and out	Grit buildup problem, manual valving system ineffective
49. New Providence STP New Providence, NJ	0.43	10%	In-line	None	None	None	Pump in and out Gravity out	Tank also used as pri clar. Wastewater held in eq tank pumped to another STP for treat. Hampers grease coll.
50. Woodstream STP Marlton, NJ	0.040	3%	Side-line	Diffusers	Manual	None	Pumped in and out	Equalization tank made into surge tank
51. Amherst STP Amherst, NY	1.25	8%	Side-line	None	Bridge sludge collectors manual	None	Gravity in and out	*Not constructed*
52. Delaware STP Delaware, OH	1.0	40%	Side-line	Fixed aerators	Manual	None	Pump in Gravity out	Tank valve 1/8 mile from plant, causing reduction in ability to control flow (covered trickling filters)
53. Hatfield Twp WWTP Colmar, PA	0.22	5%	In-line	Mixer	None	Chlorine	Pump in Gravity out	
54. Oil City STP Oil City, PA	0.52	7%	Side-line	None	Manual	None	Pump in, pump & gravity out	Equal. tank used 4 times in one year of operation
55. Watertown STP Watertown, SD	2	50%	Side-line	None	None	None	Gravity in Pump out	Not in operation 7/77
56. Amarillo River Rd WWTP, Amarillo, TX	3.0	25%		None	Manual	None	Pump in and out	

(continued)

TABLE 10 (continued)

Treatment Plant and Location	Size gal $\times 10^6$	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
57. Duck Creek WWTP Garland, TX	11.8	103%	In-line	Floating aerators	Manual	None	Gravity in and out	
58. Guadalupi-Blanco River, Victoria, TX	0.3		Side-line	None	None	None	Gravity in and out	
59. Midland WWTP Midland, TX	2.20		In-line	Diffuser	Manual	None	Gravity in Pump out	Pond short circuited because pipe into pond and pipe going out are on same side
60. Odessa STP Odessa, TX	2.1 0.7	47%	In-line	Floating aerators & diffusers	Manual	None	Pump in Gravity out	
61. Oso WWTP Corpus Christi, TX[d]				None	None	None	Gravity in Pump out	
62. Sandy Suburban STP Sandy City, UT	0.7	47%	Side-line	Diffusers	Manual	None	Pump in Gravity out	
63. Lower Potomac Poll Control Plant Fairfax Co., VA	14.3	40%	In-line	Mixers	Manual	None	Gravity in Pump out	
64. Moore Creek WWTP Charlottesville, VA	4.6	31%	In-line	Floating aerators			Gravity in and out	Not complete
65. Potomac Reg WWTP Woodbridge, VA	14.3	119%	In-line	Floating aerator and mixer	Manual	None	Gravity in Pump out	Under construction
66. Roanoke WWTP Roanoke, VA	30	86%	Side-line	None	Manual	None	Gravity in and out	

(continued)

TABLE 10 (continued)

	Treatment Plant and Location	Size gal $\times 10^6$	Tank Size/ Plant Size	Use of Equal Facility	Aeration System	Washdown Solids Removal	Chemicals Added	Means of Emptying Struc	Comments
67.	Upper Occoquan Sewage Authority Manassas Park, VA	a) 45 b) 1.5 3.5 c) --	333%	In-line	None	None		a) Gravity in, pump out b) Gravity in, pump out. c) Pump in and out	Plant has 100% redundancy. Ballast ponds equalize flow to AWT portion of plant. Scheduled to go on line in 1978.
68.	Lacy-Olympia-Tumwater Thurston County STP Olympia, WA	2.5	18%	Side-line	Diffusers	Semi-auto spray system	None	Gravity in Pump out	Not constructed
69.	Muni of Metro Seattle, West Point STP Seattle, WA			Side-line	None	None	None	Gravity in and out	
70.	Ada STP Bluefield, WV	0.4	33%	Side-line	Mixer	Manual	None	Gravity in and out	
71.	Westside STP Bluefield, WV			Side-line	Fixed aerators	Manual	None	Pump in/out Grav in/out	
72.	Chippewa Falls WWTP Chippewa Falls, WI	3.50		Side-line	None	Manual	None	Pump in Gravity out	Should have gutted in bottom for washdown
73.	Reedsburg STP Reedsburg, WI	2.18		Side-line	None	None	None	Pump in and out	
74.	Dry Creek WWTP Cheyenne, WY	2.51	56%	Side-line	Diffusers	None	None	Gravity in and out	

[a] No ventilation system.
[b] Ventilation using fans.
[c] Ventilation using blowers.
[d] The plant has a ventilation system.
[e] Ventilation using fans and ozonation.
[f] STP = sewage treatment plant.
WWTP = wastewater treatment plant.

TABLE 11. PLANTS WITH EQUALIZATION INFORMATION INCOMPLETE

Plant	Size (mgd)	Comments
Westside, AL		No information
Pomona, CA		No information
San Francisco, CA		Preliminary planning stage
Dover, DE		No information
Gulf Gate, FL		No information
University Shores, FL		No information
Augusta, GA	30	Step 1
Coosa River (Floyd Co.), GA	0.5	
Gainesville, GA	5	Step 1
La Grange, GA	7	Step 1
Napa City, ID		
Plano, IL		
Sterling, IL		Out for bid; Delaney w/state
Fort Wayne, IN		Surge pond, storm runoff
Indianapolis (Belmont), IN		Planning surge tank
Rockville, IN		Surge tank
Salem, IN		Surge tank
Southbend, IN		Series of surge basins around city
Bardstown, KY		Jim Stantley w/state
Harrowsburg, KY		Jim Stantley w/state
Lawrenceburg, KY		Unable to contact
Mt. Sterling, KY		Jim Stantley w/state
Richmond, KY		Jim Stantley w/state
Herculaneum, MO		
Springfield, MO		Unable to contact
Exeter, NH		No information
Freehold, NJ	0.05	No information
Greenbrier, NJ		In operation; no information
Leesburg, NJ (state prison)		No information
S. Lakewood Co., NJ		No information
Stafford, NJ		No information received
Toms River, NJ	5 (2)	Industrial
Akron, OH		Storm water surge tank
Altus, OK		Proposed; R. Beach (405)447-1950
Duncan, OK		Proposed; T. Lee (405) 252-0250
Meeker, OK		Emergency only, not yet used
Muskogee, OK		Proposed; E. Kernes (918)682-6602
Stillwater, OK		Proposed; F. Louise (415)372-0025
Coraopolis, PA		Under construction
Derry Twp., PA	5.0	Town and industry under construction
Fairview, PA	0.050	Race track in operation
Franklin, PA		Letter sent; no response
Borough of Grove City, PA		Construction not yet begun

(continued)

TABLE 11 (continued

Plant	Size (mgd)	Comments
Municipality of Hermitage, PA		Letter sent; no response
Borough of Lamoyne, PA		
Liberty Township, PA		Letter sent; no response
Lower Salford, PA	0.007	Housing development in operation
Borough of Middleboro, PA		Step 3 construction
Milford Twp., PA	0.007	High school in operation
New Milford Twp., PA	0.026	Two schools under design (1975)
Peddlers Village, Solebury Twp., PA	0.05	
Pocono Country, Monroe Co., PA	0.05	Housing development under construction
Shohola Twp., PA	0.075	Campground in operation
Oconee Co., SC	5	
Beeville, TX		S. Hunt w/state; adding 2 new plants; third to be used for excess flows; (512)358-4641
Borger, TX		No information
Ciblo Crk./Universal, TX		No information
Longview, TX		No information
Sequin, TX		No information
Tiboli, TX		S. Hunt w/state; (512)286-3313
N. Bonneville, WA		No information

Survey Response

Survey results provided information on individual flow equalization projects located in 26 of the 48 contiguous states. All major geographical sections of the United States are represented. Figure 17 shows the location of all treatment plants identified with flow equalization facilities.

EQUALIZATION FACILITIES SUMMARY

Treatment Plant Characteristics

Applicability of equalization, in terms of the range and degree of potential benefits, is dependent in varying degrees on the particular characteristics of the specific treatment plant.

The type of treatment used is generally related to how sensitive plant performance is to influent variations and peaking characteristics. For example, activated sludge systems, including the secondary clarifier, are typically more sensitive to flow peaking than comparable fixed film biological treatment systems. This is largely due to differences in settling characteristics of the respective biological solids and may be partially compensated for by greater operational flexibility in typical sludge systems.

The degree of treatment, in combination with the specific processes used, establishes the magnitude of potential cost savings by using equalization to minimize peaking capacity. Potential cost savings increase as the number of unit processes downstream of equalization benefitting from reduced peak flows and/or loadings increases. Thus, a treatment plant incorporating activated sludge secondary treatment, designed for nitrification, chemical addition for phosphorus removal, and effluent filtration, will find equalization of significantly greater potential benefit than a comparably sized conventional activated sludge plant with no additional treatment requirements.

The timing of equalization construction with respect to treatment plant construction may be significant in terms of potential design applicability and flexibility. Equalization additions to existing plants have design constraints imposed by the facilities in operation. The addition of equalization to existing facilities may increase effective plant capacity and/or reduce or eliminate existing operational problems.

Treatment Plant Type--

The distribution of basic treatment types at plants currently using equalization is as follows:

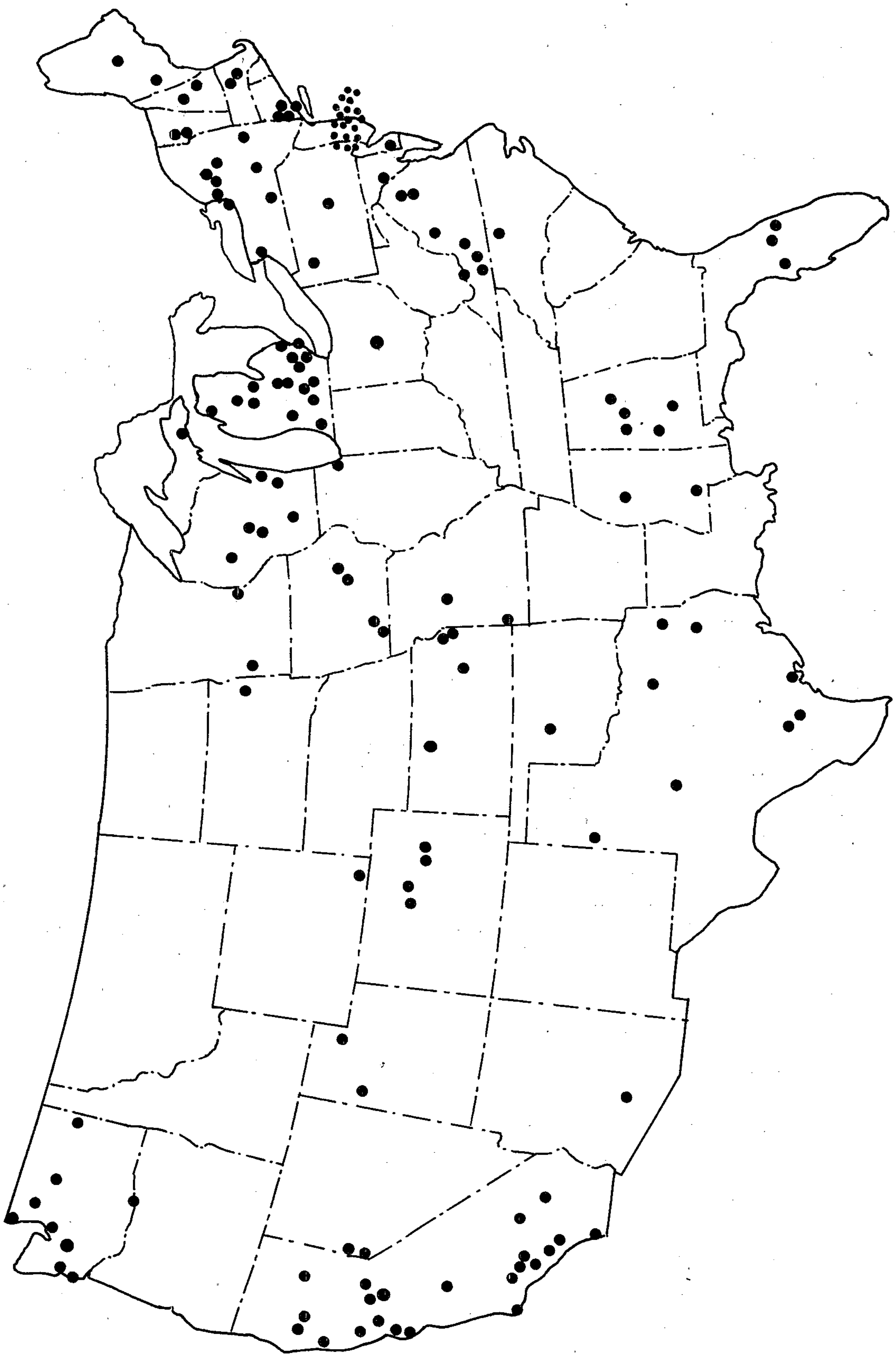

Figure 17. Flow equalization facility locations

Activated sludge	80
Oxidation ditch, activated sludge	3
Activated sludge with advanced waste treatment*	3
Activated sludge with trickling filters	3
Trickling filters	13
Rotating disc	1
Oxidation pond	1
Pump station	1
Combined sewer overflow control, primary	3

The distribution is similar for both large and small plants, with approximately 80 percent of equalization systems accompanying activated sludge plants. This is due in part to the predominance of activated sludge plants throughout the country. The smaller number of equalization installations at trickling filter plants does not necessarily lower potential for benefits at such plants. On the other hand, the small number of applications at advanced waste treatment plants is primarily a reflection of a small number of plants of this type currently in use. Detailed assessment of costs and benefits must be made in the specific context of each local sewerage system.

The pump station and combined sewer overflow control applications are a special case as far as this report is concerned. These equalization applications exist almost exclusively to accommodate peak storm flows. Many other such systems exist throughout the country. However, the focus of this report is on non-storm equalization, accordingly no effort was made to include all such systems in the survey.

Degree of Treatment--

Survey response indicates that equalization currently used at treatment plants, with degrees of treatment ranging from primary to advanced waste treatment, is distributed as follows:

Primary	2
Primary with chemical coagulation	1
Secondary	69
Secondary with phosphorus removal	11
Secondary with nitrification	3
Secondary with effluent filtration	11
Secondary with phosphorus removal and effluent filtration	6
Secondary with nitrification, phosphorus removal and effluent filtration	3

*Any additional unit processes following secondary treatment for removal of BOD, suspended solids, nutrients, etc.

Secondary with phosphorus removal, effluent filtration, and carbon adsorption	1
Secondary with nitrification, phosphorus removal, effluent filtration and carbon adsorption	1
Secondary with nitrogen and phosphorus removal, effluent filtration and carbon adsorption	1

Equalization systems in use at secondary treatment plants account for 64 percent (69 of 108) of the facilities currently in use. Equalization at secondary treatment plants, including between one and four additional tertiary processes (nutrient removal, effluent filtration, carbon adsorption), account for nearly all of the remaining systems identified; 37 of 108 or 34 percent. Treatment plants with degrees of treatment higher than secondary accounted for 22 percent of equalized plants smaller than 1 mgd compared to 42 percent of plants larger than 1 mgd. The distribution of equalized flow plants between secondary and higher treatment, and between large and small plants, is in reasonable correspondence with the proportions of the respective categories in existence.

Timing of Equalization Construction--

Survey response indicates that approximately half of equalization facilities currently in use were built as an integral part of a new treatment plant. The other half were added to existing facilities. The distribution of construction timing is as follows:

With treatment plant (WP)	51
After treatment plant with no other plant changes (APm)	15
After treatment plant, as part of plant upgrading (APu)	22
After treatment plant, as part of plant expansion (APe)	16

The distribution of construction timings is virtually the same for both large ($>$ 1 mgd) and small ($<$ 1 mgd) treatment plants.

Approximately 70 percent of new treatment plants including equalization, in both large and small plant categories, are designed for secondary treatment only. The large proportion of plants being in this category is due largely to wet weather peaking. Only 4 of 34 plants in this group report wet weather peak to average flow ratios less than 3:1.

It is of interest to note the relatively small number of treatment plants (15) to which equalization facilities have been added as the only plant modification.

Equalization Facility Characteristics

The basic features of an equalization system that define its characteristics with respect to treatment plant performance and overall cost include:

- Volume
- Type
- Location in treatment system
- Physical details
- Appurtenances

Equalization Volume--

Equalization volume is conveniently expressed as a percentage of the design treatment capacity. Volumes of equalization systems at 35 treatment plants smaller than 1 mgd capacity range from 5 percent to 2,300 percent. The average volume of 29 systems with volumes less than 200 percent is 54 percent. Twenty-five of the 34 volumes are encompassed by a band ranging from 15 to 70 percent at 1.0 mgd. The band increases in width with decreasing plant capacity to from 15 to 130 percent at 0.1 mgd. Three plants with equalization volumes less than 10 percent use either influent interceptor, or influent pump station wet well volume for partial flow equalization.

Volumes of equalization systems at 68 treatment plants larger than 1 mgd capacity ranged from 5 percent to 4,000 percent. The average volume of 63 systems with volumes less than 500 percent is 69 percent. Considering the 56 systems with volumes less than 150 percent, the average equalization volume is 45 percent. Three out of four treatment plants larger than 1 mgd and having equalization facilities use equalization volumes ranging from 10 to 100 percent of design treatment capacity. Equalization volumes used in the majority of equalized flow plants are significantly larger than minimum volumes required for equalization of daily diurnal variations. Response to the survey indicates that in a great proportion of the cases where larger equalization volumes are used, the specific purpose is to accommodate peak flows occurring during storm periods. A significant excess volume is also used at plants expecting occasional spills of toxic or other process upsetting materials.

Type and Location of Equalization--

Of the flow equalization facilities surveyed, approximately 39 percent have in-line equalization systems processing all flow entering the treatment plant. An additional 56 percent of the facilities have side-line equalization systems processing excess (diurnal and storm/sanitary) flow over a set flow rate. Approximately 5 percent of the systems surveyed use up-stream equalizing; incorporating storage capacity in the collection system with suitable controls, with variable speed pumping equipment as required to equalize influent flow to the treatment plant.

Depending on the specific characteristics and requirements of individual treatment plants, the type of equalization system and its location within the treatment plant may have substantial effects on overall system cost and operability. Available storage capacity in oversized interceptor sewers may enable significant flow smoothing at minimal capital expense. Pumping requirements of in-line equalization are generally greater than for side-line systems. However, in-line systems provide greater flexibility for concentration and load smoothing, and more positive protection against shock load effects. Side-line systems provide the potential lowest cost in-plant equalization capability, and may enable use of existing structures that would otherwise be abandoned.

Of the 95 agencies responding to the survey, 91 of the 130 flow equalization facilities pumped either in or out of the equalization structure. Only 14 installations pumped both in and out. Gravity flow operation (both in and out) of the equalization structure was reported in only 25 installations. The distribution of those facilities which pumped flow into the equalization structure versus those which used gravity was approximately 50 percent. Similarly, emptying the equalization structure was evenly divided between pumping and gravity systems.

The location of the equalization structure within a collection system or sewage treatment plant has a significant impact on the type of flow equalization facility components desired or required. Of the 95 responding agencies (a total of 130 structures) the distribution of facilities by location is as follows:

In or adjacent to the collection system	8
Upstream of the sewage treatment plant headworks	18
Downstream of the headworks or influent structure	92
Downstream of primary treatment	20
No information provided	1

Equalization at treatment plants smaller than 1 mgd capacity is most commonly located at the headworks, receiving raw sewage (31 of 37 plants in this size range were of this type). A major reason for this apparent preference is that a large proportion of smaller plants are designed as integrated package units, and not compatible with an intermediate equalization stage. Many such plants are also designed without primary sedimentation.

Approximately 56 percent (36 of 64) of equalization systems at treatment plants greater than 1 mgd capacity are located before primary treatment. An additional 30 percent (19 of 64) systems are located following primary treatment. Four systems are located following secondary treatment to minimize peaking in tertiary treatment components.

Physical Characteristics and Appurtenances--

Requirements for design of an equalization system, including the type of structure and major equipment features, are highly dependent on the details of the specific local situation.

The type of structure chosen depends on costs of construction associated with required volume, site constraints, whether construction will be at or below grade, and the type of mechanical equipment to be installed. Existing structures that would otherwise be abandoned or made available for collection system capacity must be evaluated in terms of compatibility with equalization needs, and in comparison to new facilities.

Major equipment requirements--such as aeration and/or mixing, solids removal, covering, and odor control--depend largely on the characteristics of the wastewater to be stored and on site requirements. Equalization storage of raw sewage generally requires solids removal or mixing equipment to sufficiently prevent solids deposition. Experience with equalization of municipal wastewater following primary sedimentation indicates that solids accumulation is very slight, and that solids removal or mixing for solids suspension is not necessary.

Requirements for aeration of wastewater in temporary storage depend on particular characteristics of the incoming wastes, length of storage, sensitivity of subsequent biological treatment, and sensitivity to potential odors of the surrounding area. Typical raw or settled municipal wastewater, unseeded by active biological populations, may be stored in equalization systems, with residence times of up to a half day, without creating difficulties. However, the longer the residence time, the more potential the system will have for generating odors, floating sludge, and contributing to operational problems in the following biological treatment process.

Requirements for covering and odor control depend on waste characteristics, desired equalization conditions (including whether or not aeration is provided), and the sensitivity of the local environment. Treatment plants close to, or upwind of residential areas will require closer attention to the problem of odors.

Type of structure--Survey response indicated the range of conventional tank and basin types currently used to provide required equalization volume. The distribution of types of equalization as evidenced in 130 structures is summarized below:

Concrete pipe	8
Asphalt-lined earthen basin	9
Concrete-lined earthen basin	18
PVC-lined earthen basin	4
Clay-lined earthen basin	6
Gravel-lined or soil stabilized earthen basin	2
Unlined earthen basin	26
Open concrete tank	40
Covered concrete tank	12
Steel tank	5
No information provided	17

Structure types at 38 plants with treatment capacity less than 1 mgd include 24 tanks (63 percent) and 14 basins (37 percent). A basin is distinguished from a tank by the use of an earth work for sides and bottom, as opposed to structural walls and floor. Structure types at 70 plants with treatment capacity larger than 1 mgd include 39 basins (56 percent) and 31 tanks (44 percent). Overall the distribution between basins and tanks was nearly equal; including approximately 50 percent basins, 44 percent tanks, with the remaining 6 percent consisting of pipe and wet well volume in the collection system. Approximately 40 percent of the basin systems are reported to be unlined. An additional 40 percent are equipped with a pavement lining of either asphalt or concrete; the remaining 20 percent have flexible plastic or rubber, clay, or stabilized soil lining. It is interesting to note that several agencies with unlined earthen basins indicated that they would prefer lined basins to minimize operation and maintenance requirements.

Aeration, mixing, and solids removal--Overall, mixing or aeration is provided as an integral part of the flow equalization system in only approximately 50 percent of the facilities surveyed. The distribution of methods, or combination of methods used, is as follows:

No mixing or aeration systems	66
Mechanical mixing	14
Air diffuser system	24
Surface aerators	18
Combination of air diffusers and surface aerators	1
Combination of mixer and air diffusers	3

Combination of mixer and surface aerators	1
No information provided	3

Washdown or solids removal facilities in most of the installations surveyed are of the manually-operated type; the majority of these use fire hydrants and hoses. The general distribution of washdown facilities for the 130 equalization structures is as follows:

No washdown or solids removal systems	38
Manual system	74
Automated system	11
No information provided	7

Survey response concerning aeration, mixing and solids removal equipment at plants with treatment capacity less than 1 mgd reflects the influence of the equalization facility's location before primary treatment. Twenty-two of 31 plants with equalization before primary treatment provide aeration for combined requirements of oxygenation, and to prevent solids settling. None of the plants smaller than 1 mgd are equipped with mechanical aeration or mixing. Only two of the 31 plants have mechanical solids removal equipment, but 21 of the 31 have a manual system for basin flushing and cleaning. Seven plants with equalization prior to primary treatment have no provision for either aeration or solids removal. These plants are predominantly in the southwest and use lagoon type facilities for equalization. Such systems function essentially as oxidation ponds, and, therefore, can provide successful trouble-free operation.

Survey response for plants with treatment capacity greater than 1 mgd provides comparison of equipment requirements for equalization facilities located before and after primary sedimentation. Mechanical solids removal equipment is used in less than 10 percent of all these facilities, regardless of location. However, approximately 75 percent of all systems, both before and after primary sedimentation, are provided with manually operated flushing or washdown equipment.

Aeration and mixing equipment is used in 67 percent (24 of 36) of the systems located before primary settling. Only 16 percent (3 of 19) of the systems located after primary settling use aeration. This is consistent with requirements in light of differences between settled and unsettled sewage, and illustrates potential cost savings for equalization systems located after primary sedimentation.

Odor control, covers--The vast majority of flow equalization facilities currently in use are not fitted with any type of odor control system. Of those having this feature, by far the larger percentage used some type of chlorination system. The exact distribution by type is as follows:

No odor control system	101
Chlorination system	22
Ozonation system	2
No information provided	5

Three of the 24 equalization odor control systems are at treatment plants smaller than 1 mgd; less than 10 percent of the plants in this group. The remaining 21 odor control systems accompany equalization at approximately 30 percent of the treatment plants larger than 1 mgd. The higher proportion at larger treatment plants is due to a combination of circumstances, including more sensitivity to odor potential in larger communities, increasing potential for significant odors parallel with increasing plant capacity, and a greater flexibility in the design of larger facilities due to economies of scale.

As noted previously, 12 equalization systems (5 at plants less than 1 mgd, 7 at plants greater than 1 mgd) are covered, and may also be considered as having odor control. Only three equalization systems currently in use are equipped with covers and chemical odor control capability.

EQUALIZATION FACILITIES CONSTRUCTION COSTS

Typically, detailed breakdown of construction costs by equipment element incorporated in the various flow equalization facilities is not available for the majority of installations responding to the survey. In addition, costs associated with equipment common to other areas of the wastewater treatment plant are generally unavailable, or were impossible to separate from the cost records of treatment plant and flow equalization facility installation. Included in this category are items such as pumping stations, electrical equipment and controls, ventilation systems, yard piping, and chlorination equipment.

Construction Costs

No apparent correlation was noted between construction cost figures received and the level of complexity of the individual facilities. Based on this observation, it would seem that the construction cost of an equalization facility is not solely related to the size and type of equipment included. Local influences are more likely to play a significant role in determining such costs. Some of the more significant factors that must be addressed when evaluating a given site or facility include the following:

1. Location of the equalization facility in relation to other unit processes or, alternatively, location within the collection system.

2. Shared use of equipment and piping with unit processes at the treatment plant.

3. Site conditions.

4. Labor and material costs and availability.

5. Competitiveness of construction and supplier market.

6. Time allotted for construction.

7. The existence of the equalization facility as an addition to existing facilities or as an integral part of a new treatment plant.

Table 12 summarizes construction cost data for existing equalization facilities. To make these data comparable, all construction cost information has been trended to a common cost level--an Engineering News Record (ENR) construction cost index of 2600. The construction costs listed in Table 12 do not include any allowance for engineering, legal, or administrative costs. Cost information for this analysis was obtained from 72 of the 95 responding agencies. Figure 18 presents a log-log regressional plot of the construction cost data.

Operation and Maintenance Costs

Operation and maintenance (O&M) costs for wastewater treatment facilities can be separated typically into labor, materials, energy, and chemical components. Cost data for O&M provided by 34 of the 95 responding agencies were, for the most part, incomplete and inconclusive. By far the great majority of agencies surveyed had no breakdown of O&M costs for their flow equalization facilities, and the information that was received was not generally classified by individual O&M components. Tables 13 and 14 summarize available data for equalization tanks and basins.

The most prevalent response related to the yearly O&M cost was "minimal." It was assumed that "minimal" cost was in relation to the yearly O&M cost of the associated sewage treatment plant. Of course, if plant O&M costs were high, the amount spent on O&M for the flow equalization facility could be substantial. But because of the inconclusive nature of this type of response, the agencies classifying such costs as "minimal" were considered as non-responsive.

TABLE 12. CONSTRUCTION COST DATA FOR FLOW EQUALIZATION FACILITIES

Plant Name	Ref No	Equal Fac Vol (10^6 gal)	Year of Constr or Bid	Constr Cost (10^3 $)	ENR 2600 Cost (10^3 $)	Type (tank) (basin)
Capacity <1 mgd	Tables 7 & 8					
Fontana, CA	1	0.1				T
Deer Creek STP, CA	2	3.0	1974-76			B
Rancho Bernardo Pt., CA	3	0.2	1972	3	5	B
Willits, CA	4	16.0	1977	153	209	B
Northeast/Oskaloosa, IA	5	4.07	1972			B
Southwest/Oskaloosa, IA	6	4.07	1973	34	46	B
Boyne City, MI	7	74.5	1976	300	411	B
Dimondale, MI	8	0.138	1974	58	74	T
Essexville, MI	9	0.60	1969-70	250	512	T
Dawson STP, MN	10	0.26	1972	100	165	B
Blackwood, NJ	11	0.25	1970	113	213	T
Borough of Palmyra, NJ	12	0.132	1973	150	171	T
Clementon, NJ	13	0.35	1973	90	124	T
Windsor, NJ	14	2.0	1974	355	486	T
NJ Turnpike Auth. Service Area 3-S, NJ	15	0.085	1972	129	190	T
NJ Turnpike Auth. Service Area 4-N, NJ	16	0.017	1973	21	29	T
Ramblewood, NJ	17	0.102	1968-69			T
Stratford, NJ	18	0.297	1973	340	465,800	T
Chautauqua, NJ	19	0.0371	1976	20		T
Waverly STP, NY	26	0.12	1976	75	77	T
Weatherford, OK	27	8.15	1968	69		B
McAlisterville, PA	28	0.039	1973-75			T
Salina, UT	31	0.5	1976			B
Shelburne, VT	32	0.15	1977	77	79	T
Bellingham, WA	33	0.202	1976	720	850	B
Stevens Pass, WA	34	0.076	1976	150	162	T
Capacity >1 mgd	Tables 9 & 10					
Cypress Creek STP, AL	1	2.0	1970-72	150	282	B
5 Mile Creek STP, AL	2	30	1975-77	1,000	1,180	B
Valley Creek STP, AL	3	5.0	1973-76	700	959	B
Central Contra Costa San Dist, CA	4	164.0				B

(continued)

TABLE 12 (continued)

Plant Name	Ref No	Equal Fac Vol (10^6 gal)	Year of Constr or Bid	Constr Cost (10^3 $)	ENR 2600 Cost (10^3 $)	Type (tank) (basin)
Chino Basin Muni W.D., CA	5	1.2	1977	200	206	B
Santa Rosa, CA	6	17.0	1966-67	138	351	B
Pismo Beach STP, CA	7	0.38	1975			B
Redlands, CA	8	1.25	1972			B
Rohnert Park, CA	9	0.75	1970-71	495	930	B
Rossmoor San Inc., CA	10	2.5	1971-75	188	354	B
Sacramento, CA	11	207.0	1976	959	1,035	B
San Luis Rey TP, CA	12	1.08	1973-75	408	560	T
Dublin, CA	13	2.3	1972	180	266	B
Livermore, CA	14	20.0	1974	491	634	B
Estes Park, CO	15	0.238	1975	120	154	T
Broomfield, CO	16		1975			B
Highland Park, IL	17	21.0	1973	8,700	12,876	T
Ankeny STP, IA	18	4	1974-75	125	161	B
Lawrence STP, KA	19	6.7	1975-77	380	448	B
Manhattan, KA	20	0.25	1976	252	267	B
Corrina, ME	21	0.40	1969-71			T
South Paris, ME	22	0.21	1973	190	262	T
Detroit, MI	23	28.0	1968	5,875	15,747	T
Dowagiac, MI	24	4.9	1977	310	365	B
Eaton Rapids, MI	25	0.07				T
E. Lansing, MI	26	5.0	1976	1,935	2,864	T
Grand Haven-Spring Lake, MI	27	0.80	1973			T
Grand Rapids, MI	28	10.0	1972	674	1,267	B
Saginaw, MI	29	3.6	1975-77	7,167	8,457	T
Jackson, MI	30	12.5	1972	750	1,410	B
Lansing, MI	31	4.0	1966	73	204	T
Midland, MI	32	3.25	1971-72			T
Mt. Clemens, MI	33	30.0	1974	4,560	5,882	T
Pontiac, MI	34	3.0	1974-75			T
Port Huron, MI	35	5.7	1972-73	2,500	3,700	T
Southeastern Oakland Co., MI	36	63.0	1972	25,000	47,000	T
Tecumseh, MI	37	1.0	1969-72	525	1,076	T
Trenton, MI	38	13.5	1971			B
Walled Lake-Novi, MI	39	0.315	1971-72	150	247	T
Warren, MI	40	50.0	1969	5,102	10,580	B
Ypsilanti, MI	41	0.624				T
Brookhaven, MS	42	81.0	1965			B
Greneda, MS	43	72.0	1965	314	839	B
Joplin, MO	44	12.2	1972-73	80	132	B
Warrenburg, MO	45	0.53	1977			B
Reno Sparks, NV	46					B
Merrimack, NH	47	0.956	1969-70	197	444	T

(continued)

TABLE 12 (continued)

Plant Name	Ref No	Equal Fac Vol (10^6 gal)	Year of Constr or Bid	Constr Cost (10^3 \$)	ENR 2600 Cost (10^3 \$)	Type (tank) (basin)
Elmwood STP, NJ	48	0.258	1973	144	197	T
New Providence, NJ	49	0.426	1971	162	334	T
Woodstream STP, NJ	50	0.299	1976	72	77	T
Amherst, NY	51	0.623	1976-77	1,310	1,545	T
Delaware, OH	52	1.0	1973	32		T
Colmar, PA	53	0.22	1972			T
Oil City, PA	54	0.52	1976	131	179	T
Watertown, SD	55	10.0		142	154	B
Amarillo, TX	56	3.0	1965-68	25	65	B
Garland, TX	57	11.8	1974-76	840	1,243	B
Odessa, TX	60	2.80	1976	800		T
Sandy City, UT	62	0.70	1961			B
Lower Potomac Poll Cont, VA	63	14.3	1974	540	696	B
Moore Creek STP, VA	64	4.6	1976	750	810	B
Potomac Reg. STP, VA	65	14.3	1976	750	810	B
Roanoke, VA	66	30	1973	2,193	3,005	T
Upper Occoquan Sewer Auth, VA	67	10	1974	530	683	B
Lacy-Olympia-Tumwater, WA	68	2.5		1,600		B
Seattle, WA	69					B
Ada STP, WV	70	0.4	1974-75	850	1,164	T
Westside STP, WV	71		1978-79	2.0 M	2.16 M	T
Cheyenne, WY	74	1.35	1975-76	201	260	B

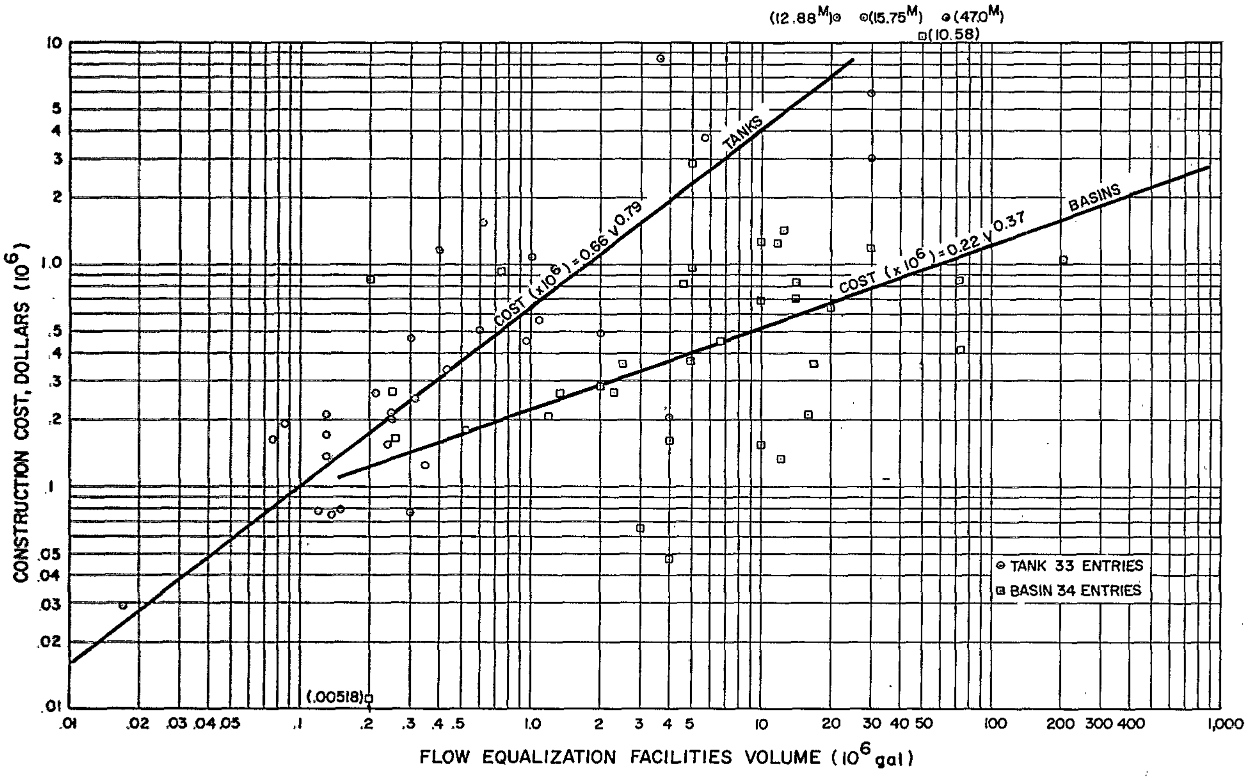

Figure 18. Equalization facilities construction cost as a function of equalization capacity

TABLE 13. OPERATION AND MAINTENANCE COSTS FOR EQUALIZATION TANKS

Plant name	Table ref no.	Type[a]	Equalization facility vol., mg	Annual O&M cost, $
Capacity < 1 mgd	Tables 7 & 8			
C.I.W., CA	1	A	0.100	
Dimondale, MI	8	B	0.138	Minimal
Essexville, MI	9	B	0.600	500-1,000
Blackwood, NJ	11	A	0.248	
Palmyra, NJ	12	A	0.132	
Clementon Sew Auth Plant, NJ	13	A	0.350	16,300
E. Windsor, NJ	14	A	0.200	4,000
Cherry Hill, NJ	15	A	0.085	Minimal
Mt. Laurel, NJ	16	A	0.017	Minimal
Ramblewood, NJ	17	A	0.102	
Stratford, Sew Auth, NJ	18	A	0.297	
Chautaugua, NY	19	A	0.036	
Waverly, NY	26	A	0.110	
Shelburne, VT	32	A	0.150	
Stevens Pass, WA	34	A	0.076	
Capacity > 1 mgd	Tables 9 & 10			
San Luis Rey TP, CA	12	A	1.160	
Valley Comm., CA	13	A	0.150	Minimal
Upper Thompson, CO	15	A	0.238	Minimal
Clavey Road, IL	17	B	21.000	15,000
Lawrence, KA	19	B	5.000	5,000
Manhattan, KA	20	A	2.900	4,000
Corrina Sew Dist Plt, MA	21	A	0.400	14,500
Paris Utility Dist, MA:	22			
Equal tank			0.210	27,000
Ind waste holding tank			0.130	11,800
Chapaton P.S., MI	23	B	28.000	200,000
Eaton Rapids, MI	25	A	0.140	
E. Lansing, MI	26	A	5.000	29,000
Grand Haven/Spring Lake, MI	27	A	0.800	2,500
Hancock St. P.S., MI	29	B	3.600	45,000
Lansing, MI	31	B	4.000	Minimal
Midland, MI	32	A	3.250	
Mt. Clemens, MI	33	B	30.000	
Pontiac, MI	34	A	3.000	

(continued)

TABLE 13 (continued)

Plant name	Table ref no.	Type[a]	Equalization facility vol., mg	Annual O&M cost, $
Capacity > 1 mgd, continued				
Port Huron, MI	35	A	6.000	4,186
Southeastern Oakland Co., MI	36	B	63.000	800,000
Tecumseh, MI	37	A	1.000	Minimal
Walled Lake-Novi, MI	39	A	0.315	
Ypsilanti, MI	41	A	0.624	
Merrimack, NH	47	A	0.952	17,200
Elmwood, NJ	48	A	0.258	
New Providence, NJ	49	A	0.426	Minimal
Woodstream, NJ	50	A	0.299	
Amherst, NY	51	A	1.250	56,000
Delaware, OH	52	A	1.000	Minimal
Hatfield Twp., PA	53	A	0.220	2,500
Oil City, PA	54	B	0.476	Not yet used
Odessa, TX	60	A	2.800	Minimal
Roanoke, VA	66	B	22.000	Minimal
Ada STP, WV	70	B	0.400	
Westside STP, WV	71	A		
Ely, MN		A	0.055	

[a] A = Equalization facility for diurnal flows (in-line and side-line).
B = Equalization facility for storm flows.

TABLE 14. OPERATIONS AND MAINTENANCE COSTS FOR EQUALIZATION BASINS

Plant name	Ref no.	Type[a]	Equalization facility vol., mg	Annual O&M cost , $
Capacity < 1 mgd	Tables 7 & 8			
Deer Creek, CA	2	A	3.000	
Rancho Bernardo	3	A	0.200	Minimal
Willits, CA	4	B	16.000	800
Northeast Oskaloosa, IA	5	B	4.070	Minimal
Southwest Oskaloosa, IA	6	B	4.070	Minimal
Boyne City, MI	7	A	57.000	
Dawson, MN	10	A	0.260	13,414
Weatherford, OK	27	A	8.150	2,252
Salina, UT	31	A	0.500	
Lake Samish, WA	33	A	8.020	1,095
Capacity > 1 mgd	Tables 9 & 10			
Cypress Creek, AL	1	B	4.000	5,000
5 Mile Creek, AL	2	B	30.000	
Valley Creek, AL	3	B	5.000	10,000
Central Contra Costa, CA	4	B	164.000	20,000
Chino Basin, CA	5	A	1.200	Minimal
Laguna, CA	6	B	17.000	Minimal
Pismo Beach, CA	7	A	0.375	
Redlands, CA	8	A	1.250	
Rohnert Park, CA	9	A	0.750	Minimal
Rossmoor San Inc., CA	10	A	2.500	33,435
Sacramento, CA	11	A	222.000	
Valley Comm., CA	13	A	2.300	1,872
Livermore, CA	14	B	20.000	Minimal
Broomfield, CO	16	A	2.000	
Ankeny, IA	18	B	4.000	3,000
Dowagiac, MI	24	B	4.900	Under const
Grand Rapids, MI	28	A	10.000	14,728
Jackson, MI	30	A	12.500	
Trenton, MI	38	B	13.500	Minimal
Warren, MI	40	A	50.000	
Brookhaven, MS	42	A	81.000	Minimal
Greneda, MS	43	B	72.000	Minimal
Joplin, MO	44	A	14.660	Minimal
Warrensburg, MO	45	A	0.530	Minimal
Watertown, SD	55	B	2.000	Minimal
Amarillo, TX	56	A	3.000	
Duck Creek, TX	57	A	11.800	32,500
Sandy Suburban, UT	62	A	0.700	

(continued)

TABLE 14 (continued)

Plant name	Ref no	Type[a]	Equalization facility vol., mg	Annual O&M cost, $
Capacity > 1 mgd, continued				
Lower Potomac, VA	63	A	14.00	50,000
Moore Creek, VA	64	A	4.60	25,000
Potomac Regional, VA	65	A	14.30	20,000
Upper Occaquan, VA	67	A	45 & 10	Minimal
Lacy-Olympia-Tumwater, WA	68	B	2.50	Not const
Dry Creek, WY	74	A	2.50	3,500

[a]A = Equalization facility for diurnal flows (in-line and side-line).
B = Equalization facility for storm flows.

A log-log regressional analysis of O&M cost data is shown in Figure 19. An attempt was made in the analysis to develop cost curves for both in-line facilities (used for either all or excess daily flow) and side-line installations (used primarily for storm inflow). No apparent correlation was observed between type of use and complexity or size of the equalization structure.

The O&M cost data listed in Tables 13 and 14 were also compared on the basis of size of the associated treatment plants. A log-log regressional analysis of these data is shown in Figure 20. Although these data varied greatly, a more reasonable pattern emerged that indicates that O&M costs associated with equalization facilities are more closely related to the size of the wastewater treatment plant than to the size of the equalization facility installed.

A flow equalization facility must obviously produce some additional O&M costs for a wastewater agency because of the additional equipment and controls involved in such an operation. Recorded observations at several plants previously studied by others, and survey data received during this study indicate that only a negligible amount of operator time is required on a day-to-day basis for routine operation and maintenance procedures for both in-line and side-line installations. Major cost factors were primarily for repair of equipment and control systems.

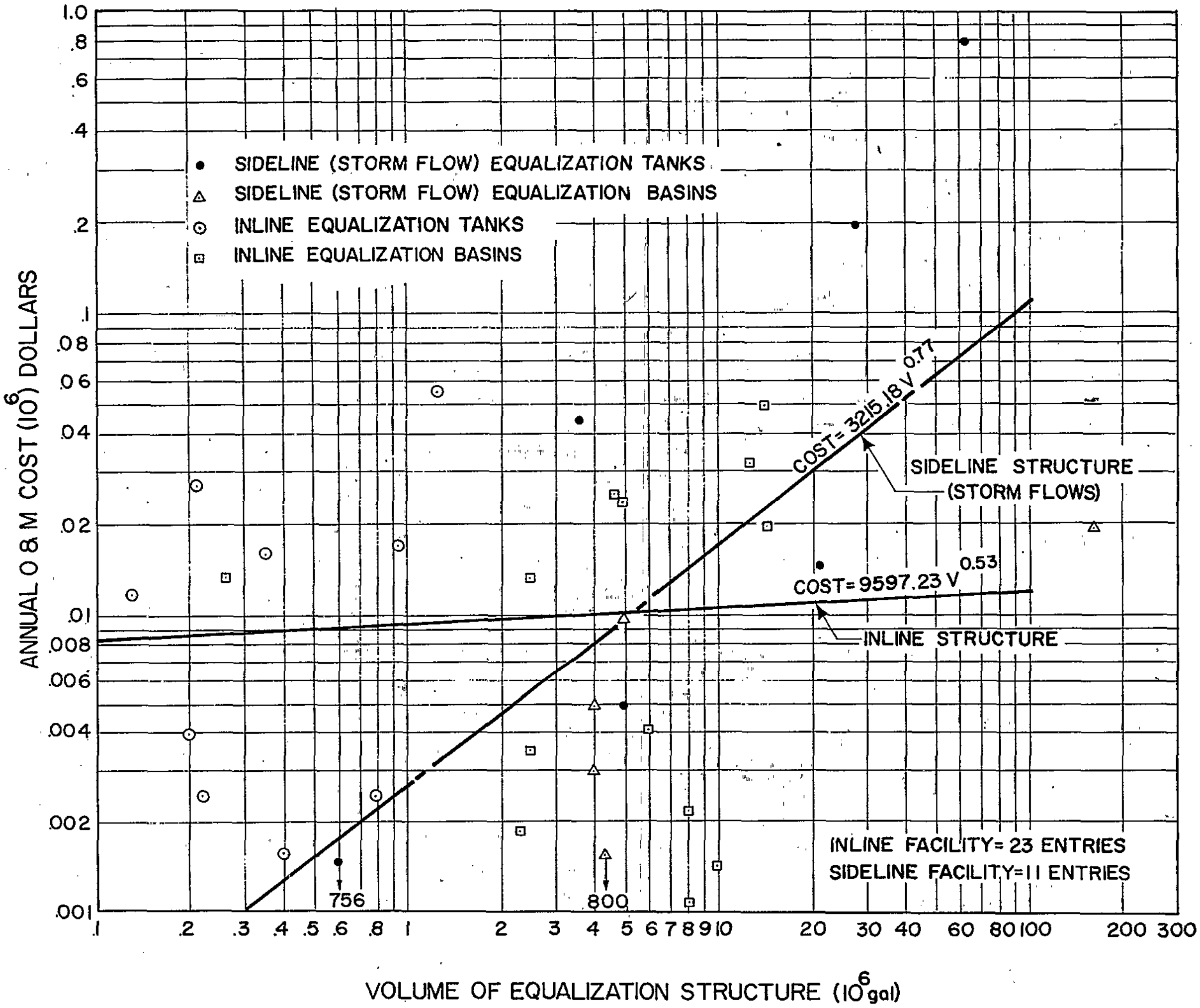

Figure 19. Operation and maintenance cost as a function of equalization volume

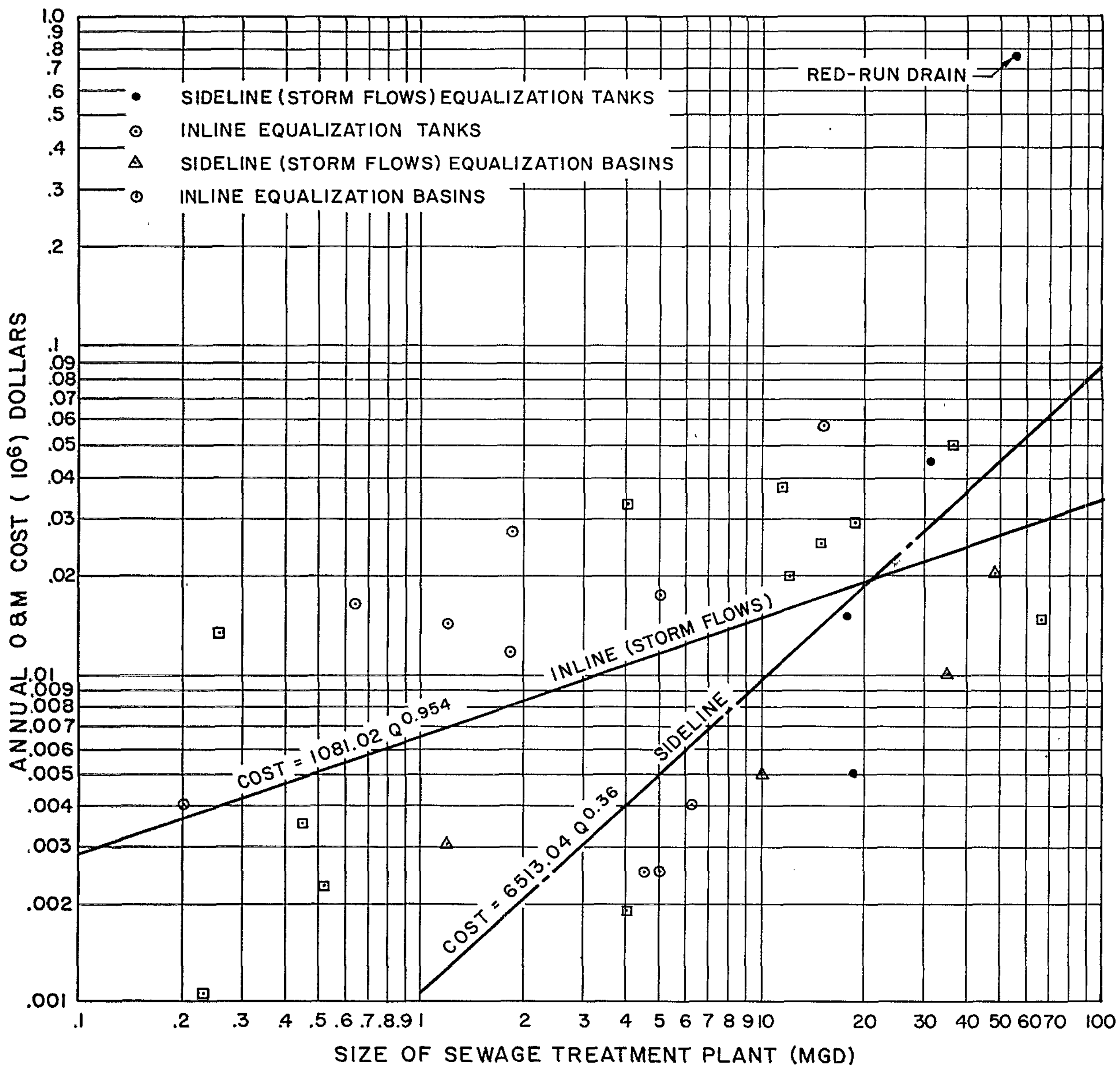

Figure 20. Operation and maintenance cost of equalization facilities as a function of treatment plant capacity

SECTION 5

EQUALIZATION PERFORMANCE EVALUATION

CASE HISTORIES

Introduction

Increased treatment process stability and improved performance are presumed benefits of equalization frequently cited in favor of including it as a treatment component. Treatment process design relationships and extensive qualitative reasoning generally support these assumptions. But current understanding of the complex interactions between treatment process components under typically variable diurnal loading conditions is not sufficient to quantitatively predict the benefits of equalization. Nevertheless, efforts to define its cost effectiveness require quantitative information. Recent EPA sponsored equalization studies and treatment plant operating records obtained in this study have, therefore, been analyzed to determine quantitative effects of equalization on both performance level and day-to-day variability of conventional wastewater treatment facilities.

To evaluate effects of equalization on treatment process and plant performance using only existing plant operating records, a careful selection of the different types of data is essential. A broad range of influent, environmental and operational factors affect treatment plant performance. To minimize the influence of extraneous variables, treatment plants should be selected for analysis primarily if equalization has been added to an existing plant as the only significant physical or operational change during the period of interest. In such cases existing plant operating records can be analyzed for the years preceding and following the time equalization began. Significant effects of equalization on average daily performance may be identified, giving appropriate consideration to differences in influent conditions of the respective periods of operations. Very few suitable treatment plants are in existence, and fewer yet have operating records adequate for thorough analysis. In a few cases, where data are available, two years of equalized flow performance data are presented in separate comparisons. Before and after data is supplemented with data from additional treatment plants (with and without equalization) having detailed operating records available for analysis. The performance analysis includes treatment facilities in a range of sizes, treatment types, and types of equalization facilities (Table 15).

Performance characteristics of plants studied are established by means of probability plots, as described in Section 2. Data are analyzed uniformly for plant influent and secondary effluent; where available, primary and tertiary effluent analyses are included. Typically, operating records reflect daily average flows and concentrations, with concentrations composited proportional to flow. One year (365 consecutive days) of operating data is used where possible to provide a description of typical operating conditions. Performance characteristics are established for each plant on the basis of BOD and TSS concentrations. Characteristics are also examined for some plants in terms of BOD and TSS loadings to illustrate the significance of differences between flow and concentration distributions over a typical annual cycle.

In most cases, logarithmic transformations are found to normalize both influent and effluent distributions. Comparisons must be made in terms of the logarithmic standard deviations and coefficients of variation in spite of the lack of physical significance. In an effort to best describe observations of plant performance characteristics and effects of equalization, graphic presentations are used, permitting the reader to supplement comparisons discussed. Effects on the performance level are reflected by median and mean values of respective data sets. Effects on day-to-day process variability are shown by the slopes of the respective distributions.

The treatment plants for which data are analyzed have widely varying characteristics, so comparison of plant performance should be made with due caution. In spite of the variety of characteristics among plants, and the wide range of design and operating factors other than equalization (or the lack of it), general patterns may be found that can be attributed to equalization effects. The annual performance characteristics required for compliance with secondary treatment standards (Figure 16), as described in Section 2, provide a generalized basis for evaluating performance of individual plants.

Activated Sludge Plants

EPA MERL Pilot Plant--

An in-house study of the effects of input variations on activated sludge treatment was conducted by the EPA at the Municipal Environmental Research Laboratory, Cincinnati, Ohio. Facilities consisted of two independent, parallel, 20 gpm

TABLE 15. TREATMENT FACILITIES FOR PERFORMANCE EVALUATION

Plant type and location	Design capacity	Equalization type	Operating data
Activated sludge:			
EPA Pilot Plant, Cincinnati, OH	2 @ 20 gpm	Constant flow	Side-by-side
Walled Lake-Novi, MI	0.7 mgd	Side-line	Equalized
Tecumseh, MI	1.4 mgd	In-line	Before/after
Ypsilanti Twp, MI	3.7 mgd, 3.8 mgd	In-line unequalized	Side-by-side
Pontiac, MI	8.5 mgd	Side-line	Equalized
Amarillo, TX	9.5 mgd	Side-line	Before/after
Warren, MI	35 mgd	Emergency	Unequalized
Renton, WA	29 mgd	Influent regulation	Part. equalized
Newark, NY	1.8 mgd	In-line	Unequalized/ equalized
Trickling filter:			
Palmyra, NJ	0.53 mgd	In-line	Before/after
Midland, MI	6.5 mgd	Side-line	Equalized
Bay City, MI	12 mgd	---	Unequalized
Oxidation ditch:			
Dawson, MN	0.26 mgd	Variable A.S. vol.	Equalized
Arlington, WA	2.1 mgd	---	Unequalized

capacity activated sludge treatment trains. Flow to one plant was maintained at a constant 20 gpm, and the other was maintained at a daily average flow of 20 gpm, with diurnal peak to average variations of 1.5, 2.0 and 2.5:1 imposed during consecutive study periods of approximately 3 months each. Since flow to the plants was provided by pumping directly from the raw sewage source, no concentration equalization was provided in the constant flow system; as shown by values (Table 16) determined from diurnal sampling during each of the peak-to-average flow periods. Unit process loading rates were maintained as follows: primary clarifier overflow rate = 1,200 gpd/ft^2; aeration tank loading ≈ 35 lb BOD/1,000 ft^3; secondary clarifier overflow rate = 650 gpd/ft^2.

TABLE 16. EPA INHOUSE STUDY: PEAK-TO-AVERAGE (P/A) FLOW AND LOAD RATIOS

P/A Flow	Date	P/A Load (COD) Constant Flow		P/A Load (COD) Varying Flow	
1.5	Feb '74	1.6	1.7	2.1	1.85
1.5	Apr '74	1.8		1.6	
2.0	Jul '74	1.7	1.55	2.4	2.1
2.0	Oct '74	1.4		2.2	
2.5	Jan '75	1.4	(TSS)	2.5	(TSS)
2.5	Jan '75	1.3		2.4	
2.5	Feb '75	1.7		2.3	
2.5	Apr '75	1.8		2.7	

Influent, primary effluent, and secondary effluent TSS and BOD concentration distributions of the parallel constant and diurnally varying flow pilot plants are shown in Figures 21 and 22 for a peak-to-average flow ratio of 1.5; Figure 23 and 24 for a peak-to-average flow ratio of 2.0; and Figures 25 and 26 for a peak-to-average flow ratio of 2.5. Differences between distributions of primary effluent TSS and BOD concentrations and loads at all three levels of peak-to-average flow were slight. In addition removals observed were not typical of conventional experience. Accordingly, little emphasis is placed on this information, and more detailed comparison will not be made.

Distributions of secondary effluent BOD and TSS at a peak-to-average ratio of 1.5 were virtually the same for constant and varying flow conditions. The pattern of similarity between TSS and BOD distributions was observed to continue in the other

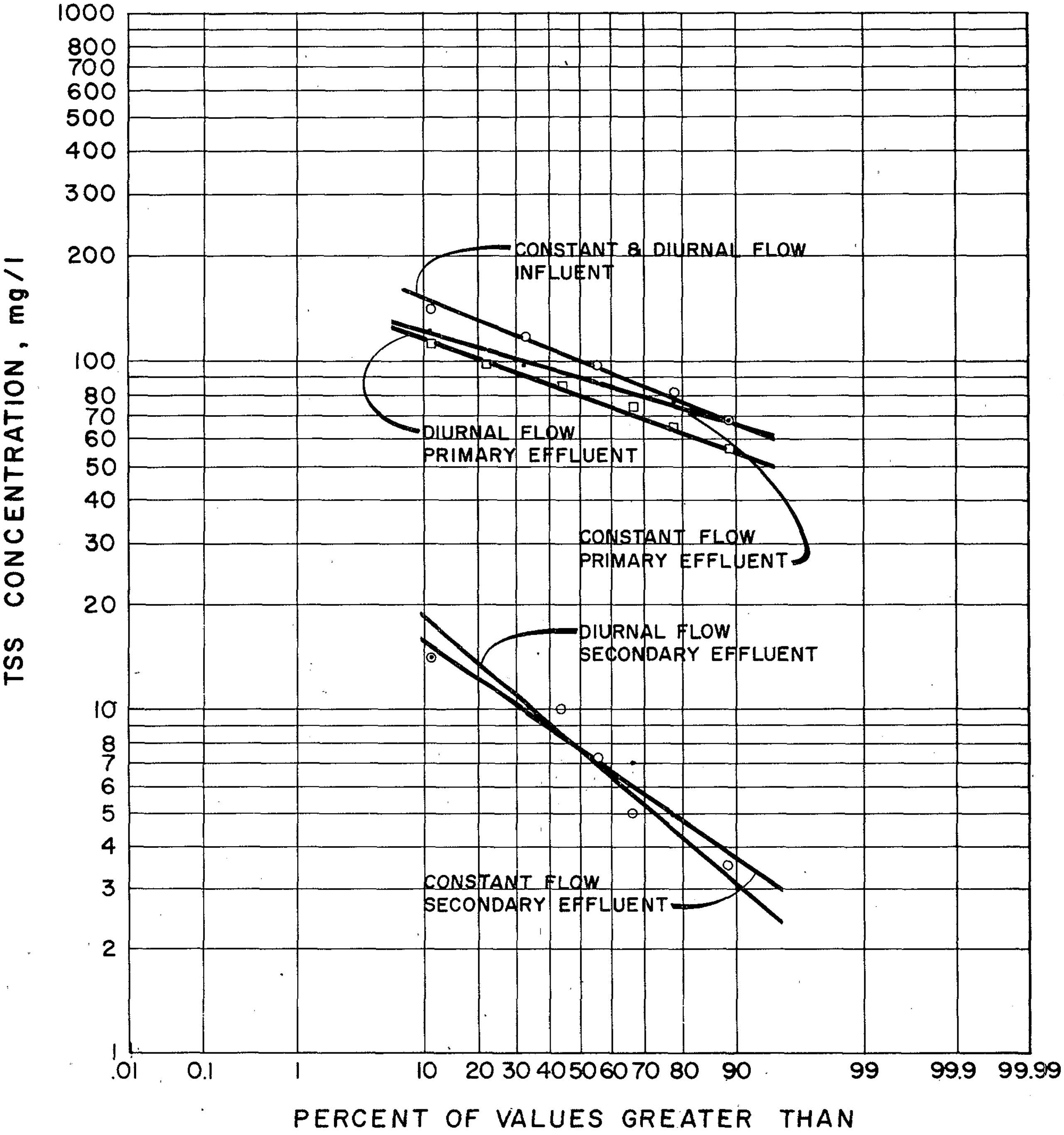

Figure 21. Distribution of weekly TSS concentrations, EPA in-house study (P/A = 1.5)

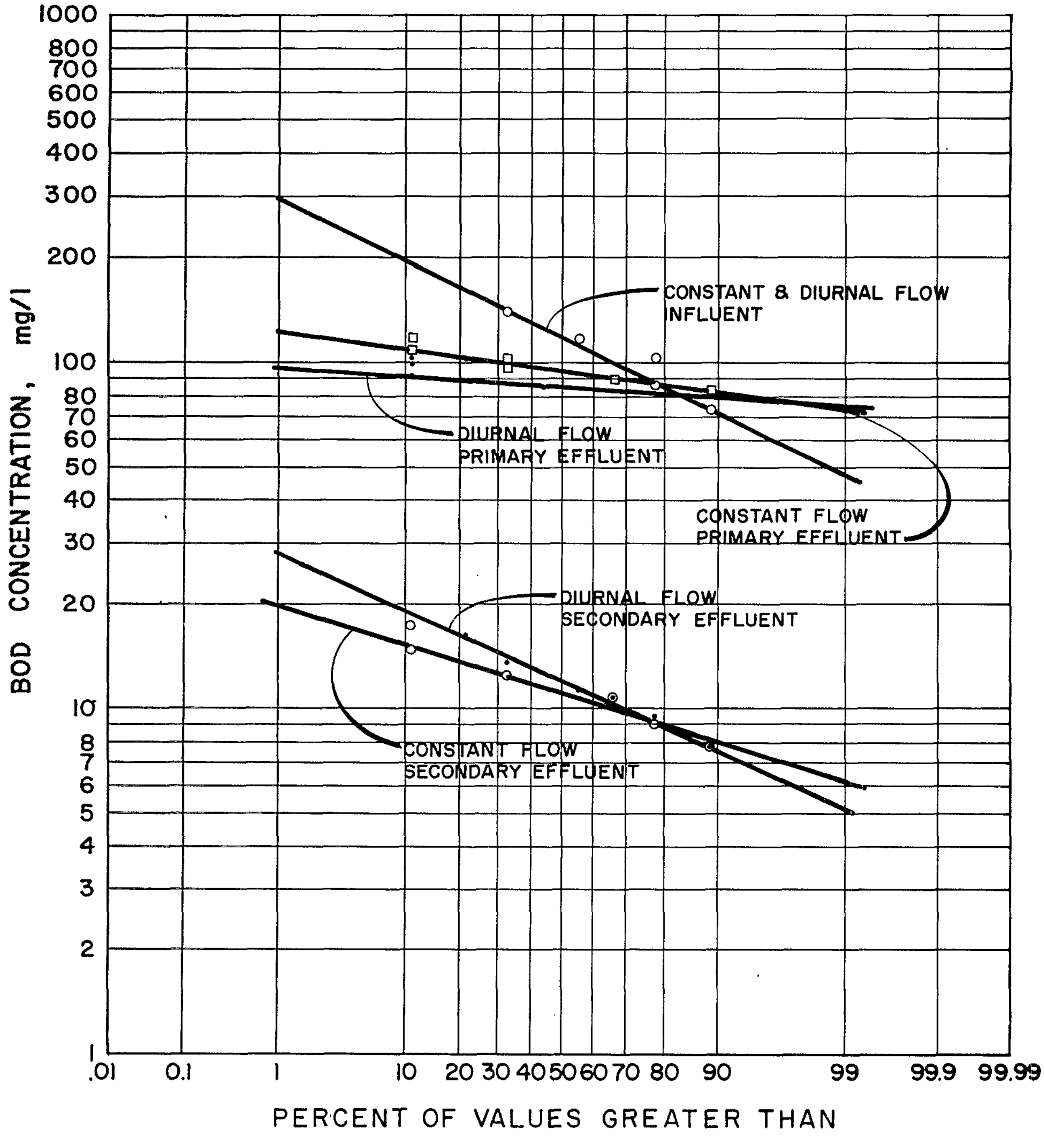

Figure 22. Distribution of weekly BOD_5 concentrations, EPA in-house study (P/A = 1.5)

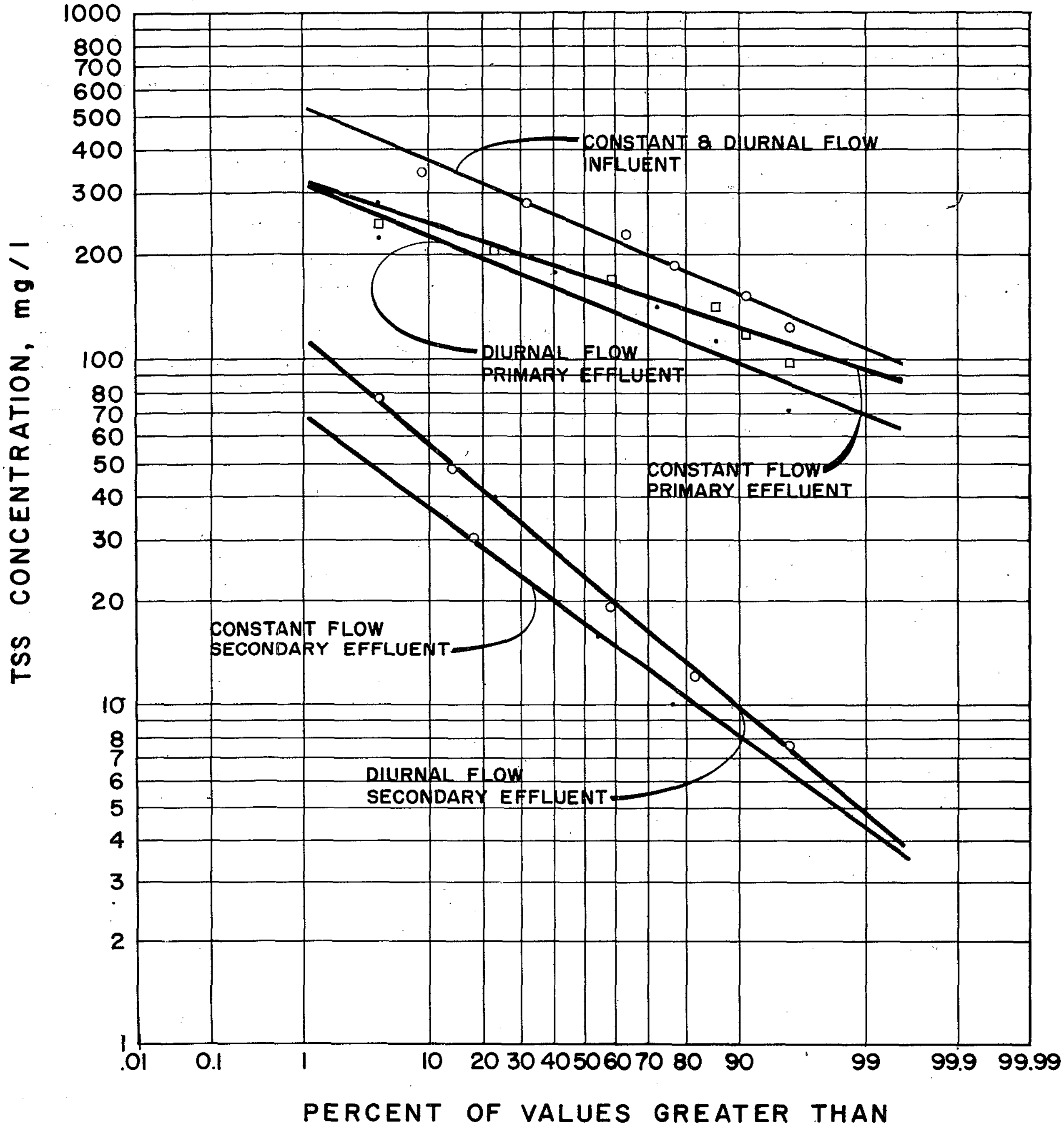

Figure 23. Distribution of weekly TSS concentrations, EPA in-house study (P/A = 2.0)

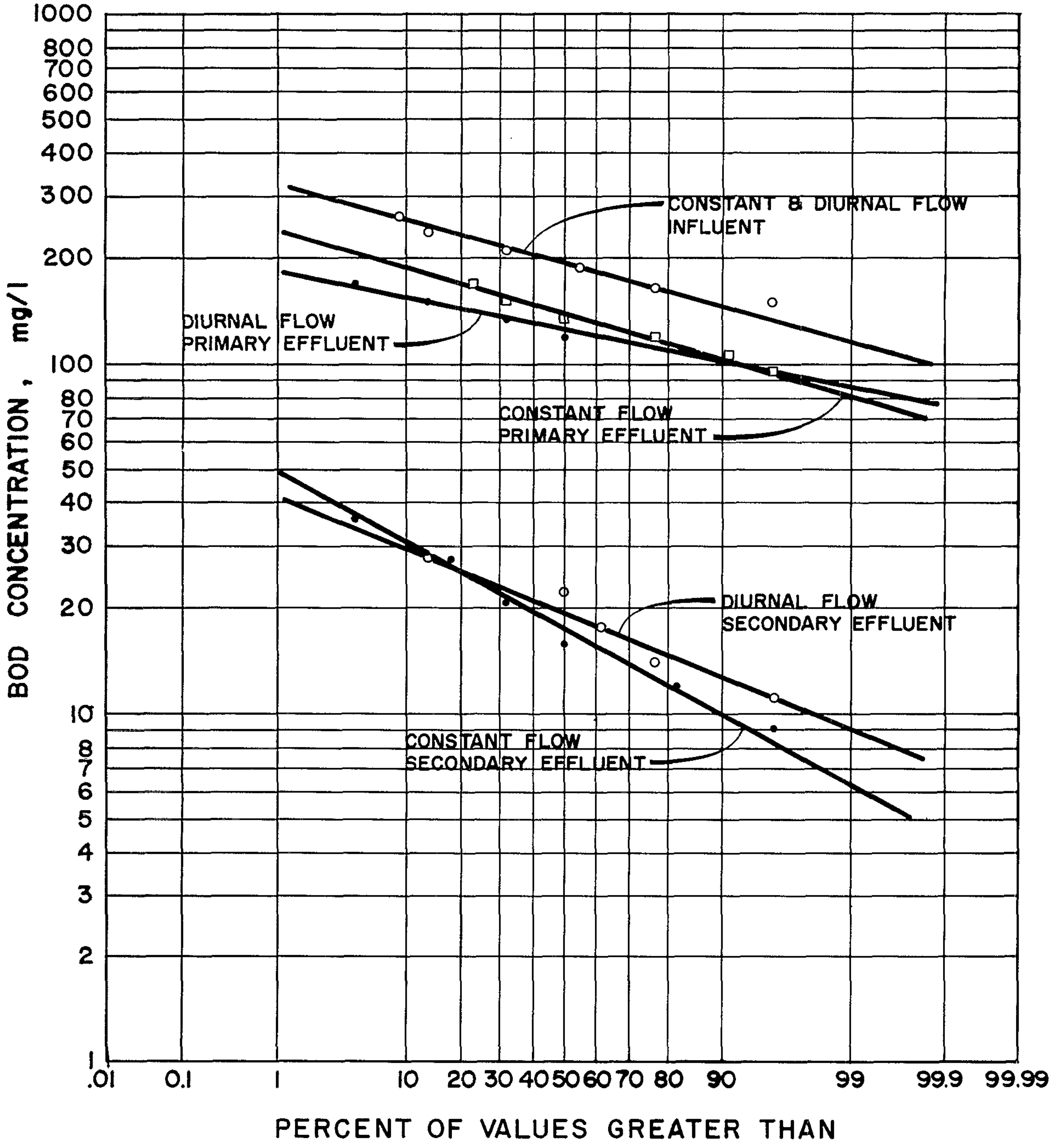

Figure 24. Distribution of weekly BOD_5 concentrations, EPA in-house study (P/A = 2.0)

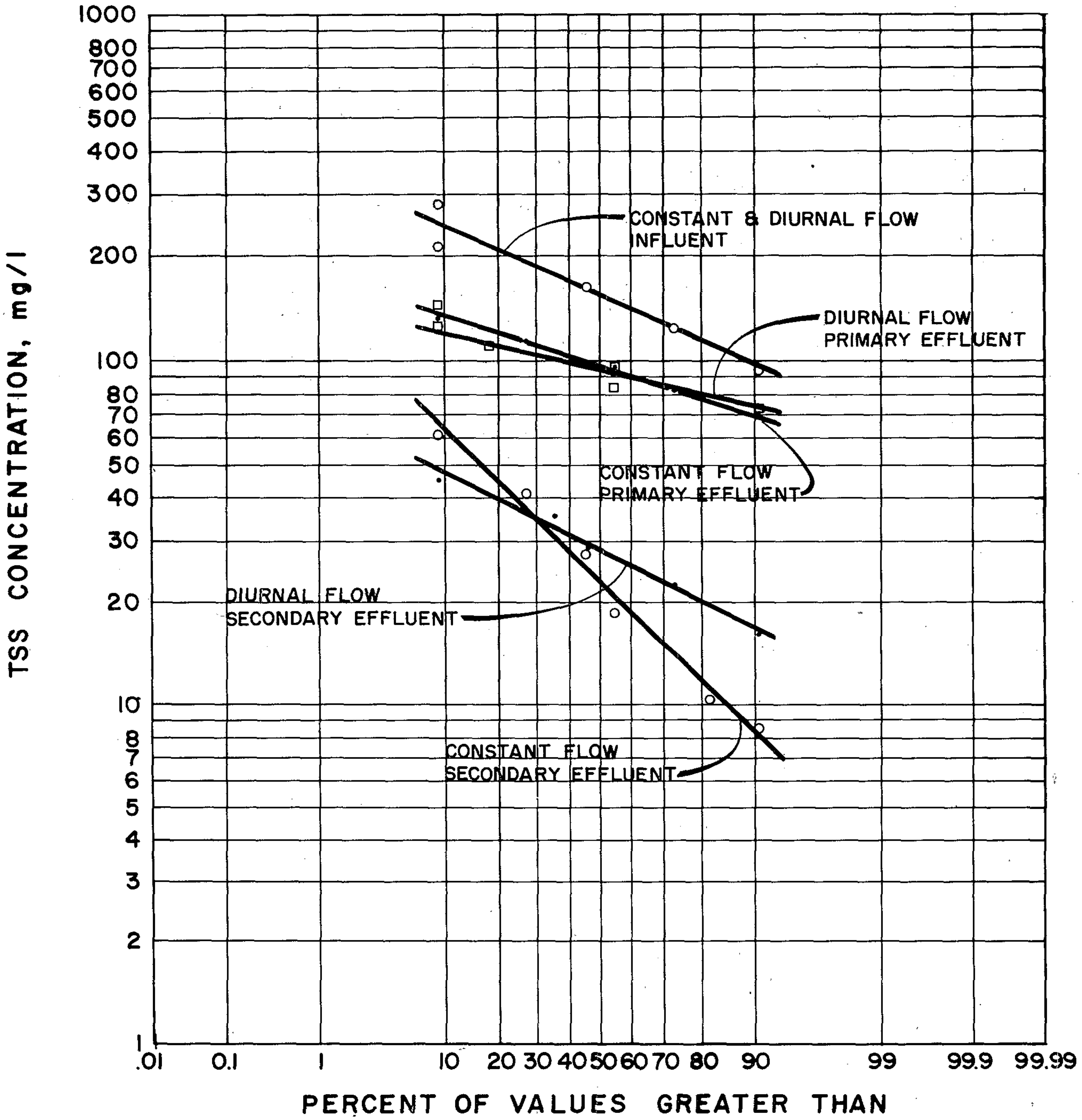

Figure 25. Distribution of weekly TSS concentrations, EPA in-house study (P/A = 2.5)

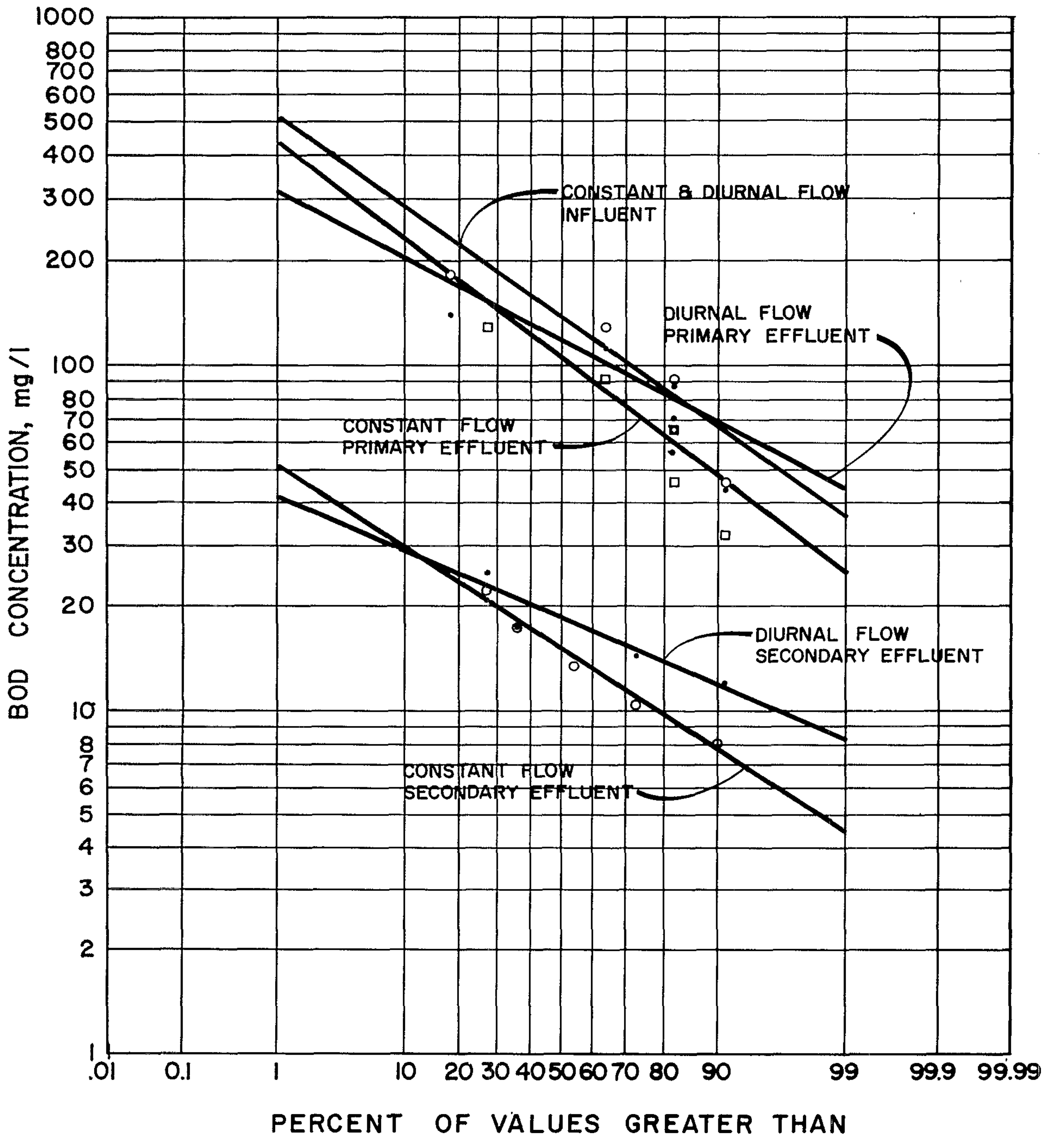

Figure 26. Distribution of weekly BOD_5 concentrations, EPA in-house study (P/A = 2.5)

two peak-to-average ratio periods. As the peak-to-average ratio was increased, the difference between distributions for constant and varying flow conditions is observed to increase. At a peak-to-average ratio of 2.0, performance under constant flow was better in terms of both TSS and BOD, with differences in mean values of approximately 10 percent and 25 percent respectively. Variability of BOD performance was slightly greater for constant flow conditions. But variability of TSS performance was slightly less for constant flow. At the peak-to-average ratio of 2.5, TSS and BOD performances were again both better under constant flow conditions, with differences between mean effluent concentrations each approximately 20 percent. The variability of BOD and TSS distributions was observed to be greater under constant flow conditions during this period.

The data from this study suggest that the increasing effectiveness of equalization for improving process performance runs parallel to the increase of influent variability. It is not possible to distinguish, however, between the relative importance of simple flow equalization and a somewhat lower degree of influent load equalization provided (Table 15).

Walled Lake-Novi, Michigan--

A study of flow equalization at the Walled Lake-Novi, Michigan, wastewater treatment plant (27) was conducted from January 1974 to February 1975. The facilities consist of a 0.7 mgd activated sludge "package" plant and effluent filters, with a 0.34 million gallon sideline equalization tank. The plant has stringent effluent discharge requirements (Table 17). These discharge requirements dictated conservative design: secondary clarifier overflow rates = 360 gpd/ft^2; effluent filtration rates = 1 gpm/ft^2. Complete details of plant design and operation are given by Foess et al. (27) The aerated equalization tank is operated to maintain essentially constant flow through the plant on a daily basis. Average plant flow is estimated at the beginning of each day. Controls then provide for either pumping excess raw wastewater from the influent wet well to the equalization tank, or for allowing stored wastewater to flow back into the secondary process during deficient flow periods. The plant does not have a primary clarifier.

TABLE 17. EFFLUENT DISCHARGE REQUIREMENTS FOR WALLED LAKE-NOVI PLANT

Parameter (mg/l)	30-day average	7-day average	Daily maximum
BOD			10
TSS	10	15	
NH_3-N			2
Total-P	20% of influent		

During the study period the plant was operated continuously according to routine equalized flow procedures. For one week (October 28 to November 3, 1974), the equalization tank was not used, so that plant performance could be observed under unequalized flow conditions. The peak-to-average flow ratio under typical dry weather conditions in 1977 is approximately 1.5.

Influent, secondary effluent and filtered final effluent TSS and BOD concentration and load distributions for equalized and unequalized flow periods during the 1974 study are shown in Figures 27 through 30. Differences between the respective concentration and load distributions are attributable to differences in the distribution of average daily flows experienced between the equalized and unequalized periods studied. The influent loading distributions (Figures 31 and 32) indicate the comparability of loading conditions experienced in the respective periods.

Comparison of distributions from equalized and unequalized flow periods shows that some differences occurred both between the level of performance and degree of performance variability. The variability in all distributions of the one-week unequalized period may be artificially low compared to the rest of the one-year equalized period. One week of consecutive average daily concentration measurements benefits from the natural homogeneity of conditions experienced in relatively short periods of time; effects of the full range of typical annual influent conditions are simply not represented.

Distributions of influent and effluent BOD (Figure 28) indicate that secondary effluent in the equalized flow period was both better and less variable than in the unequalized period, and that filtered final effluent was better but more variable. It may be noted that influent BOD loading conditions were somewhat lower and less variable during the equalized flow period. However, because of the very light loading rates imposed on this plant, the differences observed in influent conditions should have had slight, if any, effect on overall performance.

Influent and effluent TSS concentration distributions (Figure 27) indicate that, contrary to observed BOD performance, secondary effluent in the equalized flow period was both of poorer quality and more variable than in the unequalized flow period. Filtered final effluent, on the other hand, was both better and less variable under equalized conditions. Influent TSS loading under equalized conditions was somewhat higher, but had similar variability to that under unequalized conditions. By virtue of the differences observed between respective influent distributions, conclusions about the effect of equalization as opposed to other process variables are difficult to draw. The uniformly better quality of filtered final effluent observed

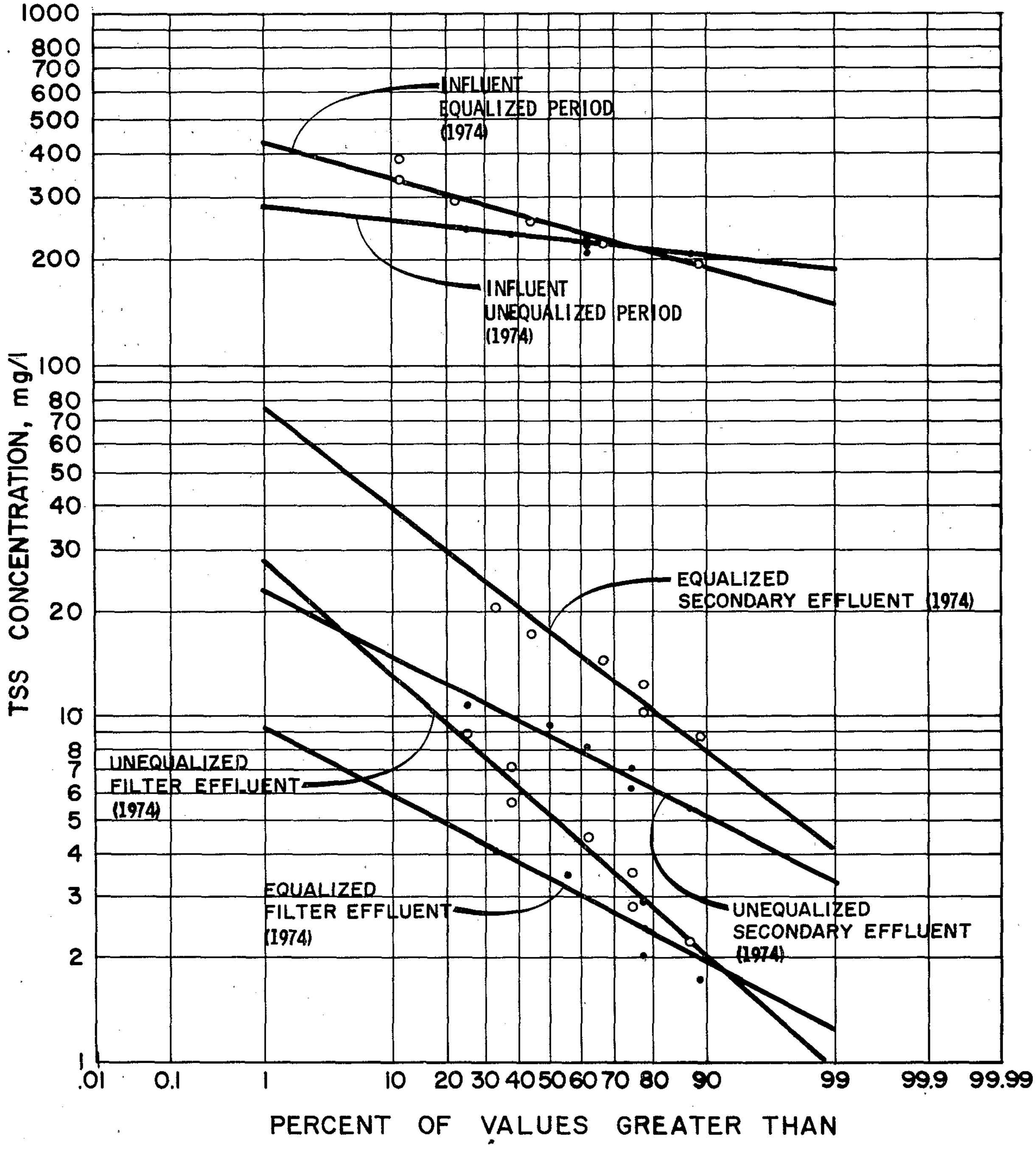

Figure 27. Distribution of 8-day TSS concentrations, Walled Lake, Michigan/Novi, Michigan, 1974

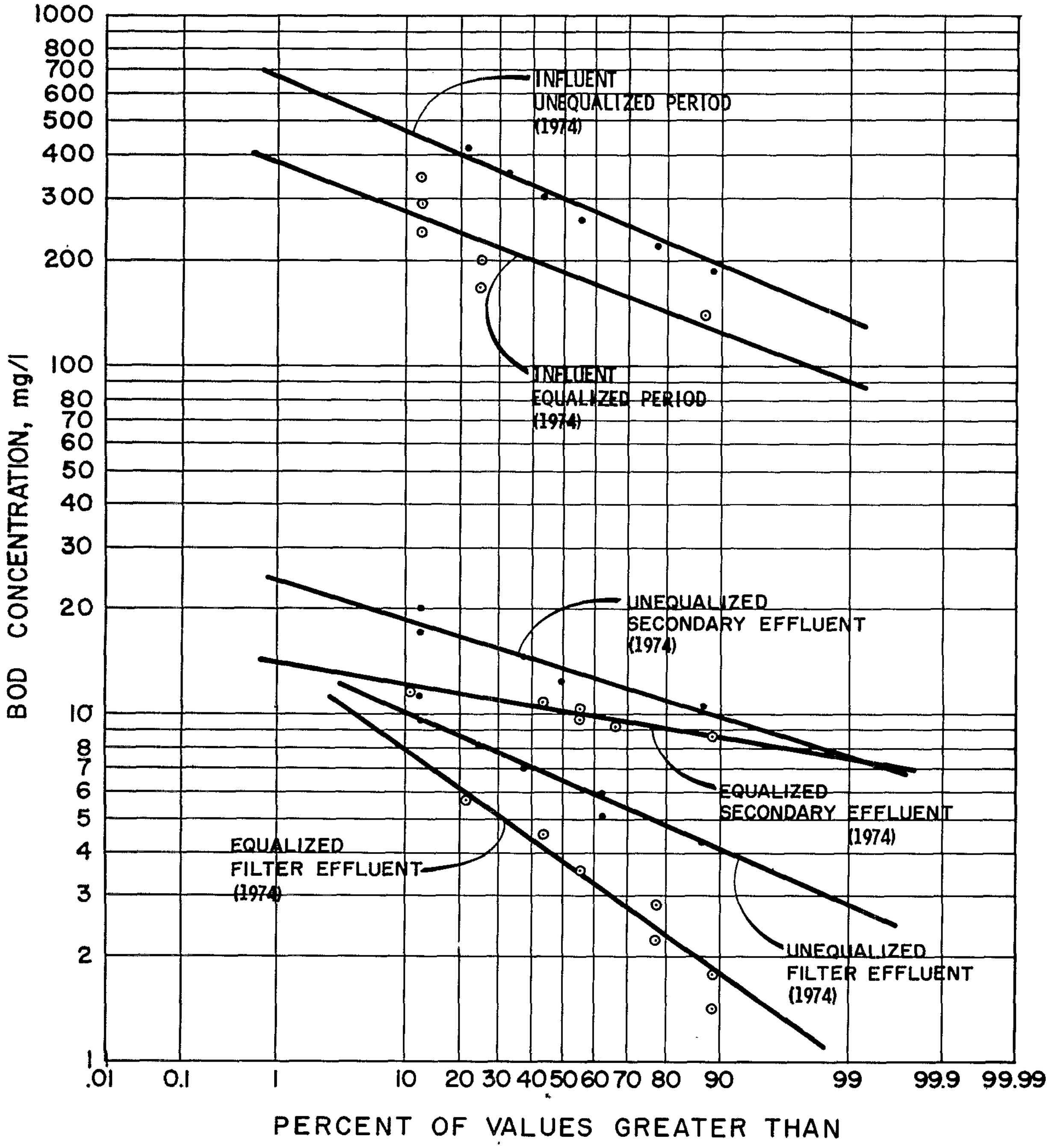

Figure 28. Distribution of 8-day BOD_5 concentrations, Walled Lake, Michigan/Novi, Michigan, 1974

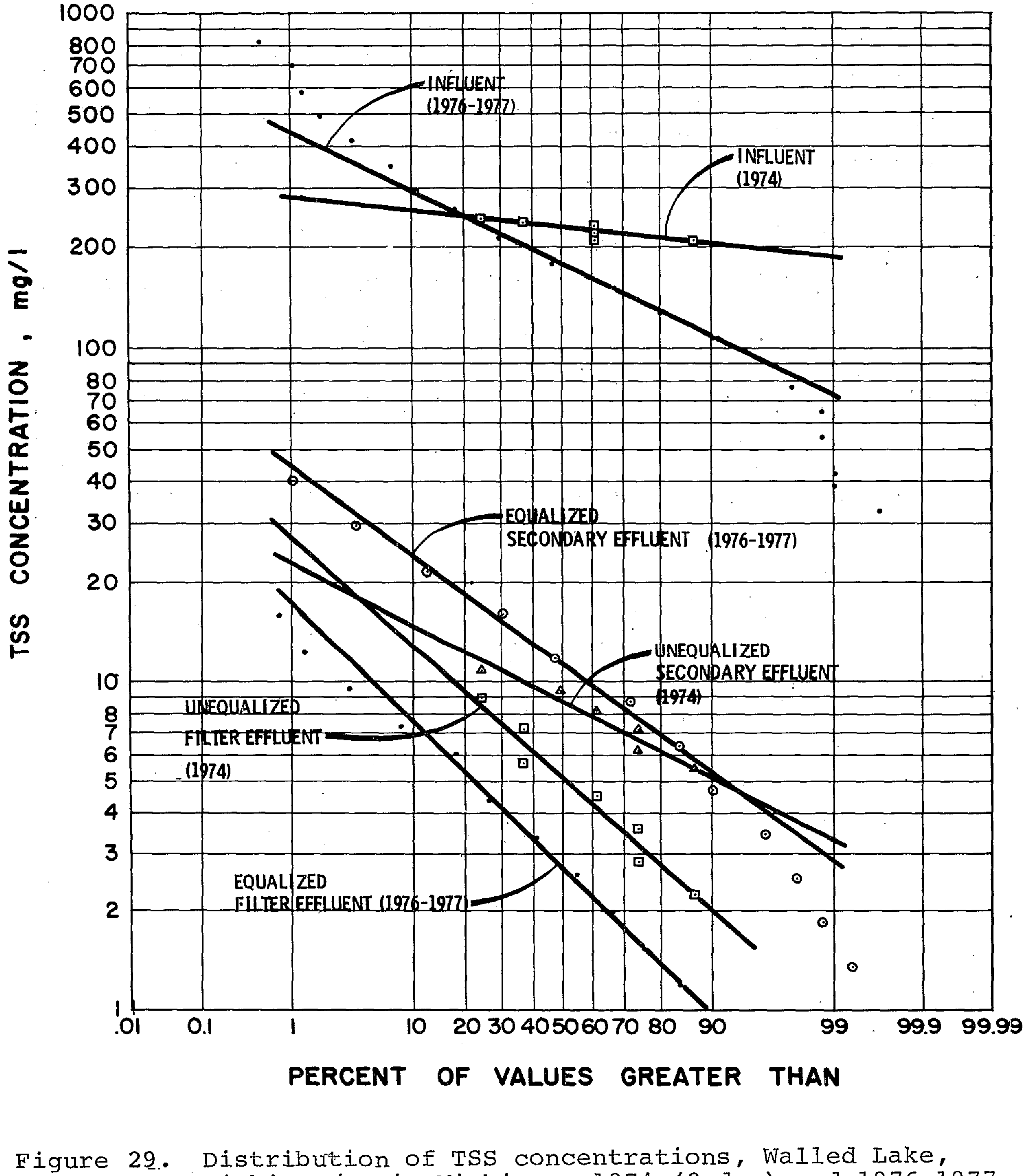

Figure 29. Distribution of TSS concentrations, Walled Lake, Michigan/Novi, Michigan, 1974 (8-day) and 1976-1977

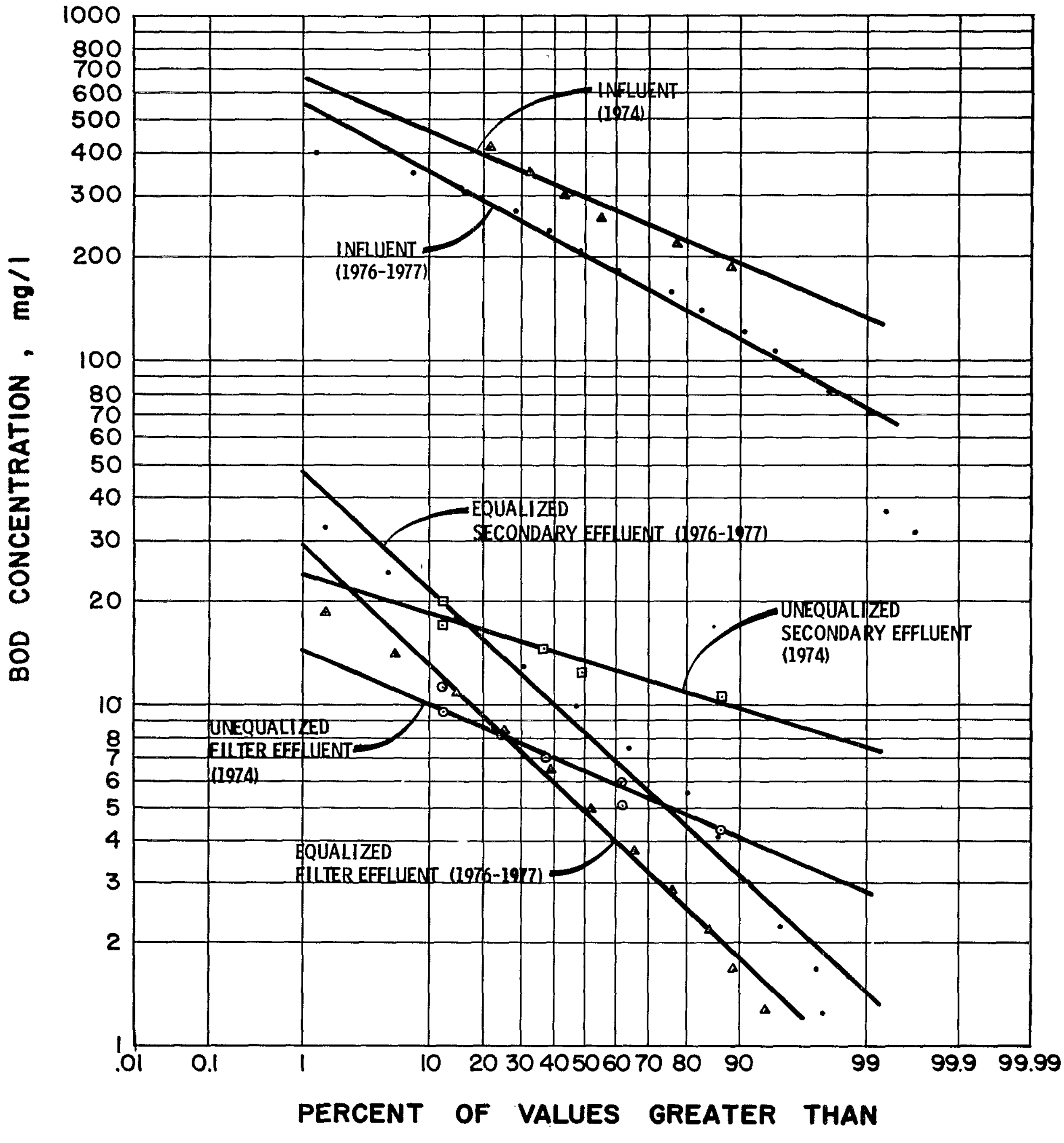

Figure 30. Distribution of BOD_5 concentrations, Walled Lake, Michigan/Novi, Michigan, 1974 (8-day) and 1976-1977

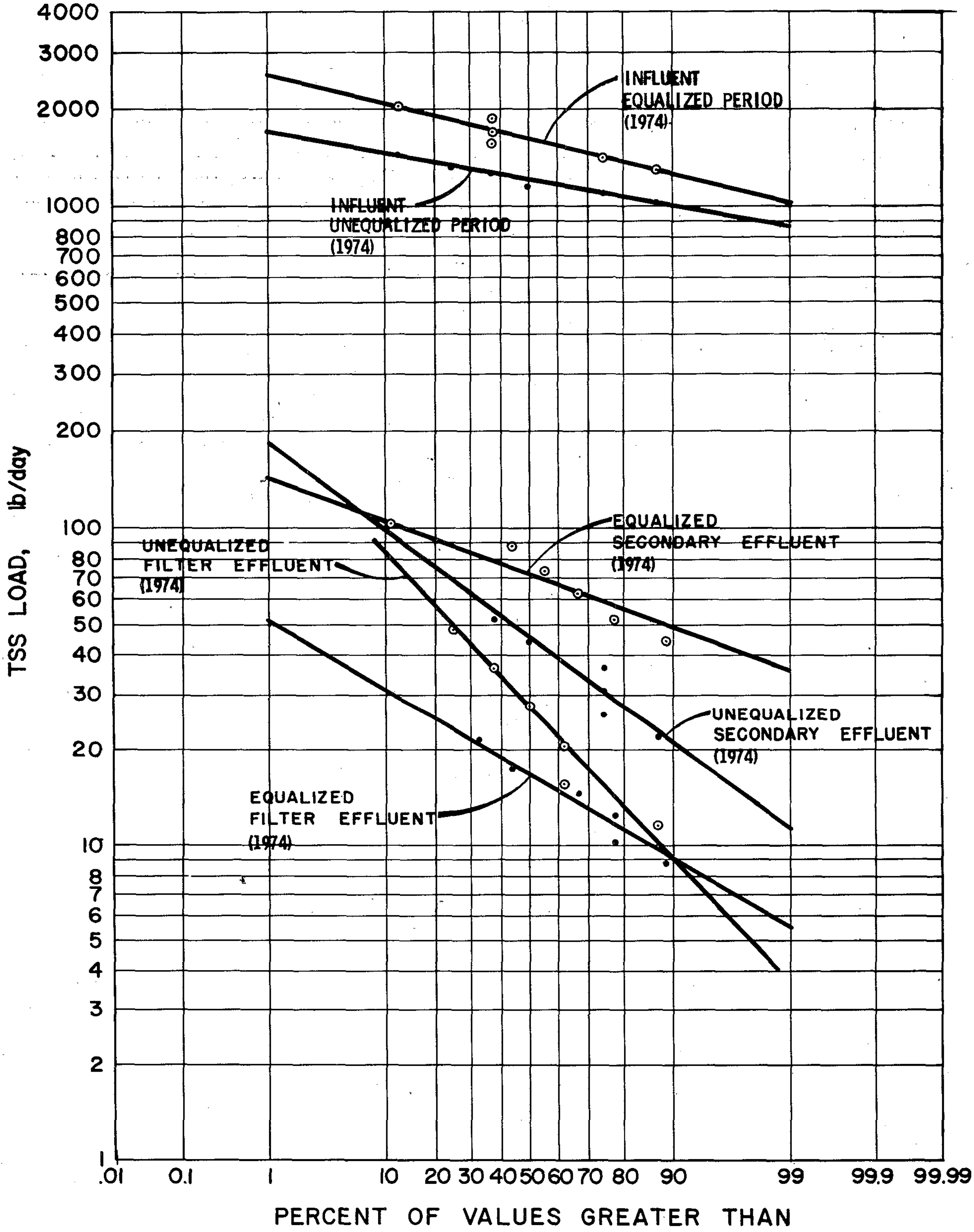

Figure 31. Distribution of 8-day TSS loads, Walled Lake, Michigan/Novi, Michigan, 1974

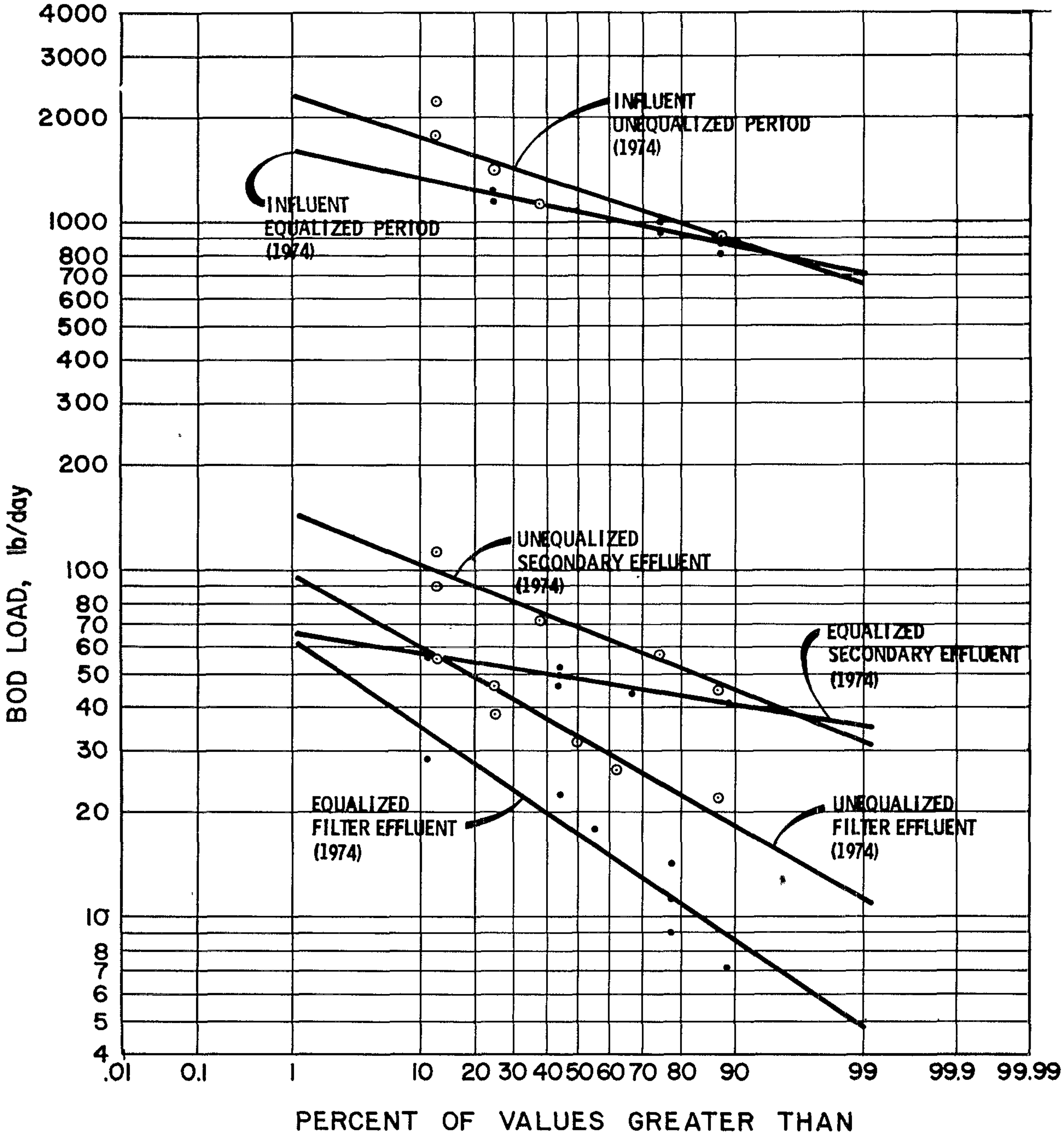

Figure 32. Distribution of 8-day BOD_5 loads, Walled Lake, Michigan/Novi, Michigan, 1974

during equalized operation suggests that equalization is a positive influence on effluent filter performance. These observations correspond closely to those reported in the original study.

Distribution of equalized influent and effluent TSS and BOD concentrations for the year from August 1976 to July 1977 (Figures 29 and 30) are compared to the unequalized distributions from the 1974 study (Figures 27 and 28). The distributions of influent TSS and BOD concentrations for 1976-1977 are comparable to those of the equalized 1974 period, and correspondingly lower and somewhat more variable than those of the 1974 equalized flow period.

The 1976-1977 distribution of secondary effluent TSS shows improved performance compared to the equalized 1974 period, but still slightly poorer performance than the corresponding unequalized period. The median 1976-1977 filter effluent TSS performance is approximately 3 mg/l, with 80 percent of effluent concentrations between 1 mg/l and 7 mg/l. Corresponding 1974 equalized and unequalized filter effluent medians are 3.5 mg/l and 5 mg/l, with 80 percent ranges of 1 mg/l to 6 mg/l and 1 mg/l to 13 mg/l, respectively.

The 1976-1977 distribution of effluent BOD concentrations shows significantly better but more variable performance than in the corresponding 1974 periods. Effluent filter BOD performance during 1976-1977 is not quite as good as during the 1974 equalized period, but still substantially better than during the unequalized period. This is in reasonable agreement with patterns observed for TSS performance.

Tecumseh, Michigan--

In 1972 a 1 mgd equalization/emergency storage basin (Figure 33) was added to the Tecumseh, Michigan sewage treatment plant. No other physical or significant operational changes were made at this plant during the period from 1970 through the present. The facilities consist of a 1.4 mgd contact stabilization, activated sludge process with conventional headworks, and primary and secondary clarifiers. The peak-to-average flow ratio during typical dry weather conditions is about 1.3. Average dry weather flow in 1976 was approximately 1.1 mgd.

The equalization basin is divided into two equal 500,000 gallon compartments. One compartment is normally kept empty to provide emergency bypass storage from the headworks in the event of industrial spill detection. Oils and plating wastes from local manufacturing industry periodically contributed to severe plant upsets before the equalization addition. The second compartment is routinely used for equalization of daily flow variations. Effluent from the primary clarifier flows by gravity to the equalization basin. Flow to the activated sludge process is

Figure 33. Equalization tank, 1 million gallon capacity, Tecumseh, Michigan

pumped from storage at the estimated average daily flow preset by the operator and maintained by wet well level pump controls. Mixing and aeration are provided in the equalization tank by a coarse-bubble, diffused air system.

Influent and effluent distributions of TSS and BOD concentrations and loads observed during the year before equalization (1970) and two years following equalization are shown in Figures 34 through 43. Comparison of influent loadings for the three years (Figures 38 through 41) show that loadings in equalized periods are significantly higher than in the unequalized period. Mean values of the various distributions (Table 18) illustrate the differences observed in influents and effluents during the periods of record. In general, differences between loading distributions for the periods compared were similar to those observed between concentration distributions.

The distribution of effluent BOD concentrations was significantly lower in the two equalized flow periods than in the period before equalization. In 1973, the equalized effluent BOD

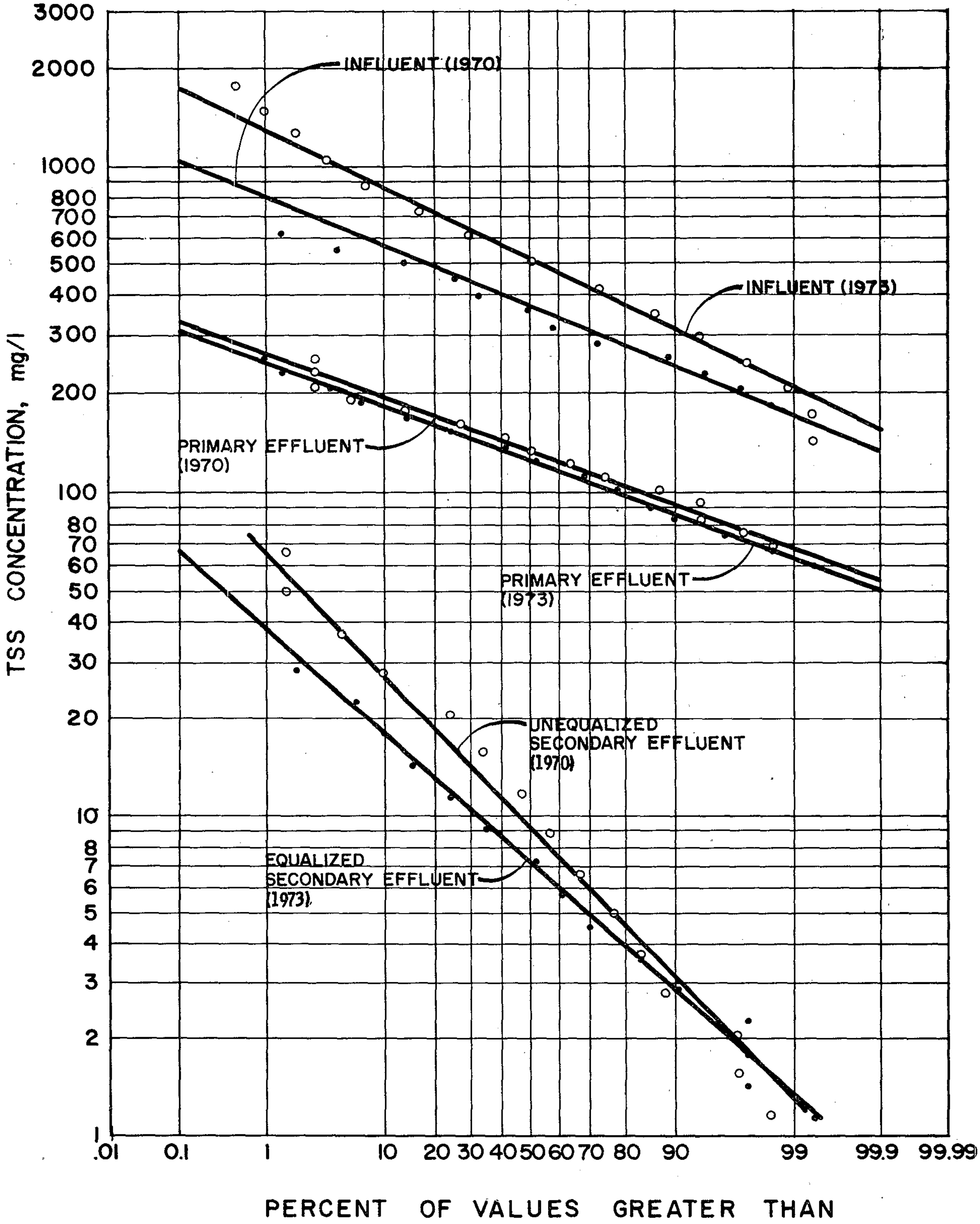

Figure 34. Distribution of TSS concentrations, Tecumseh, Michigan, 1970 and 1973

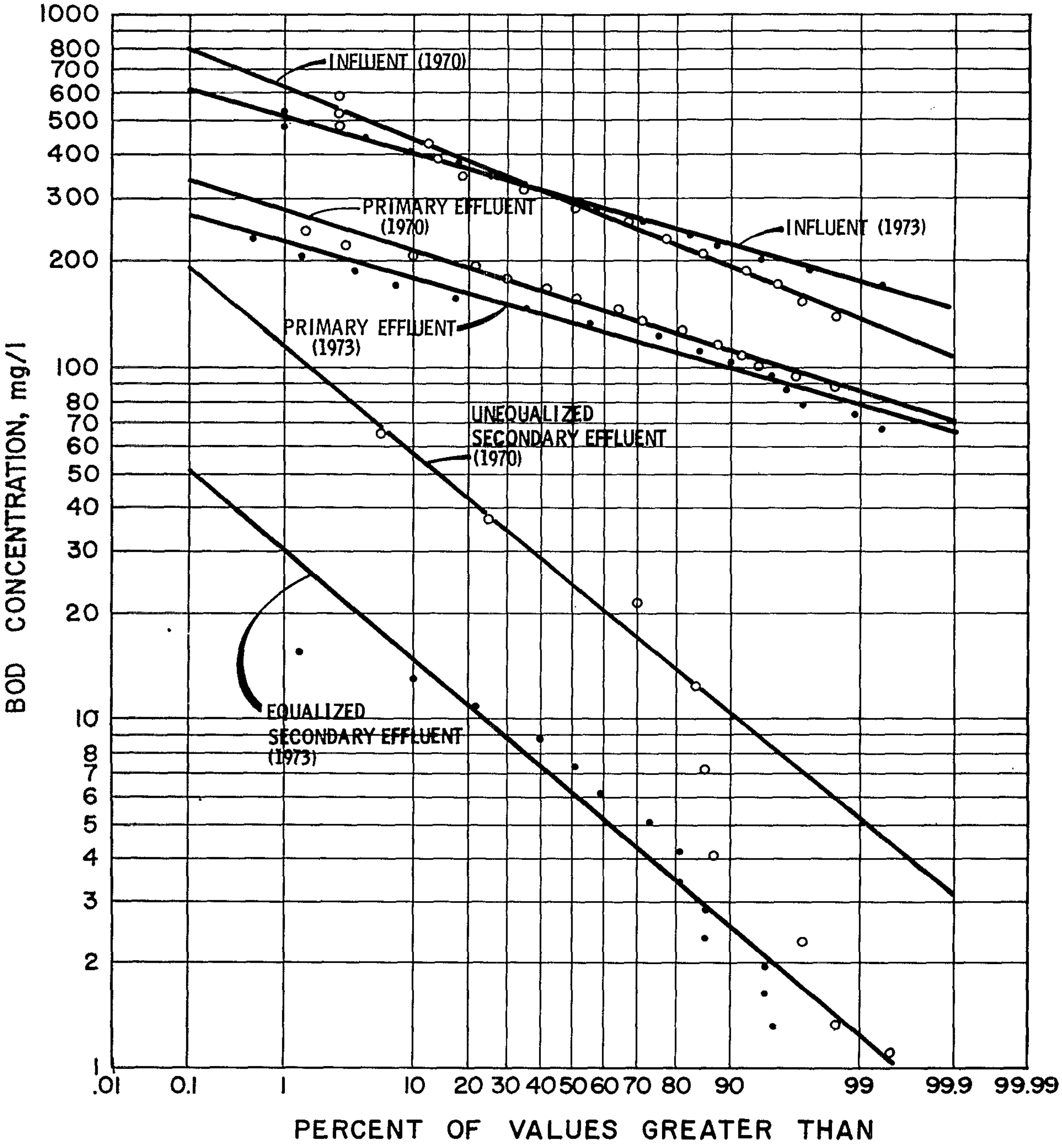

Figure 35. Distribution of BOD_5 concentrations, Tecumseh, Michigan, 1970 and 1973

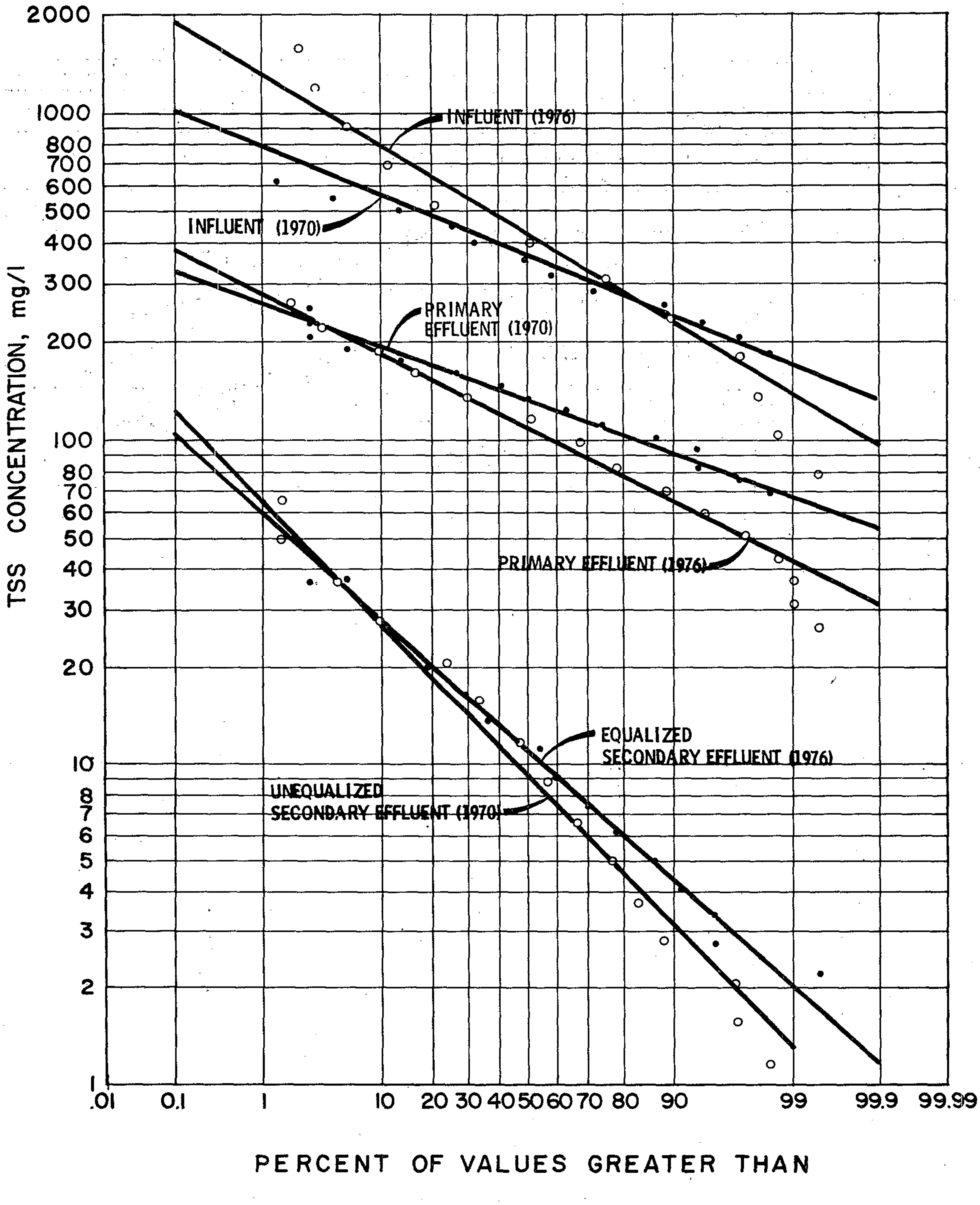

Figure 36. Distribution of TSS concentrations, Tecumseh, Michigan, 1970 and 1976

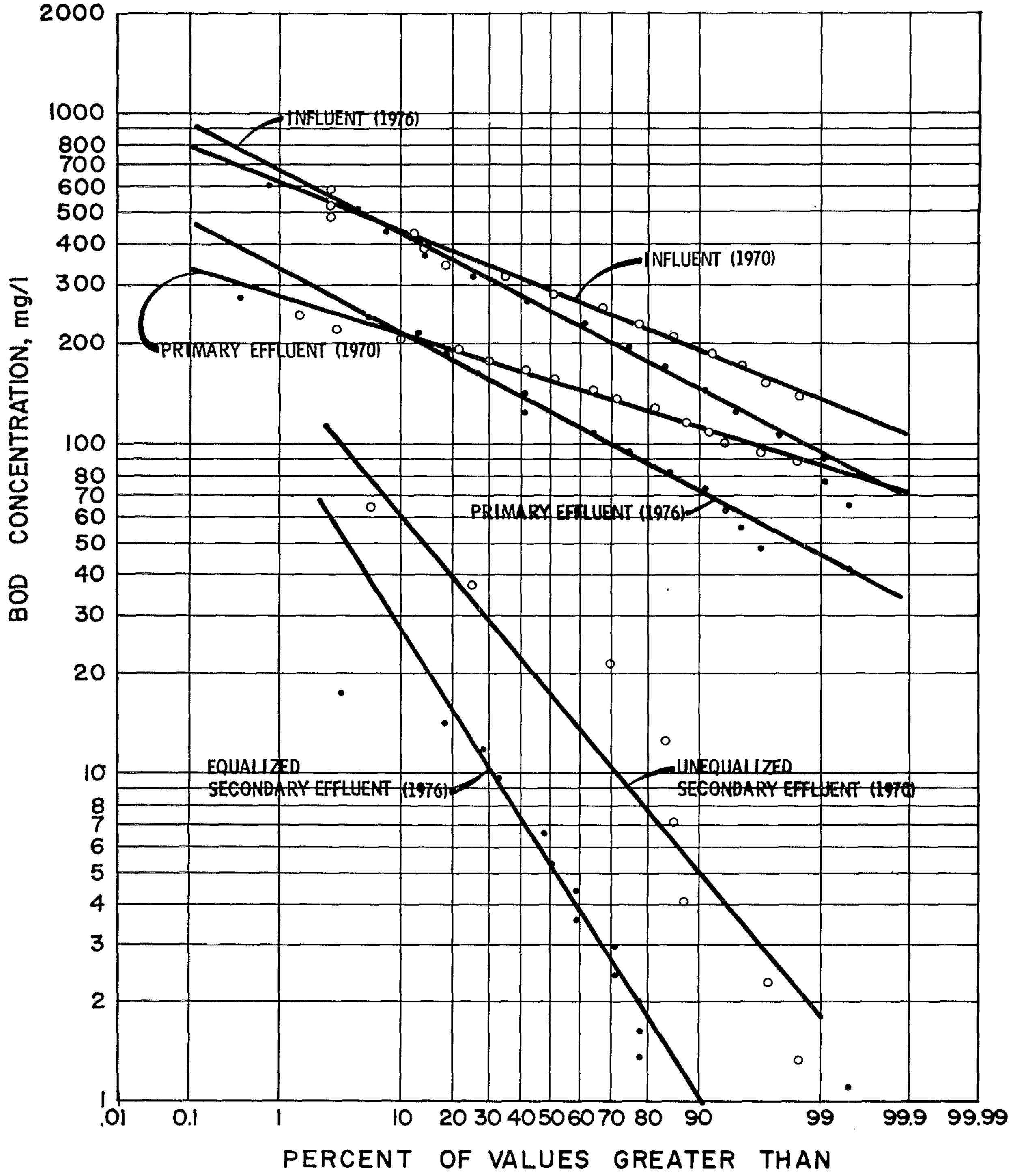

Figure 37. Distribution of BOD_5 concentrations, Tecumseh, Michigan, 1970 and 1976

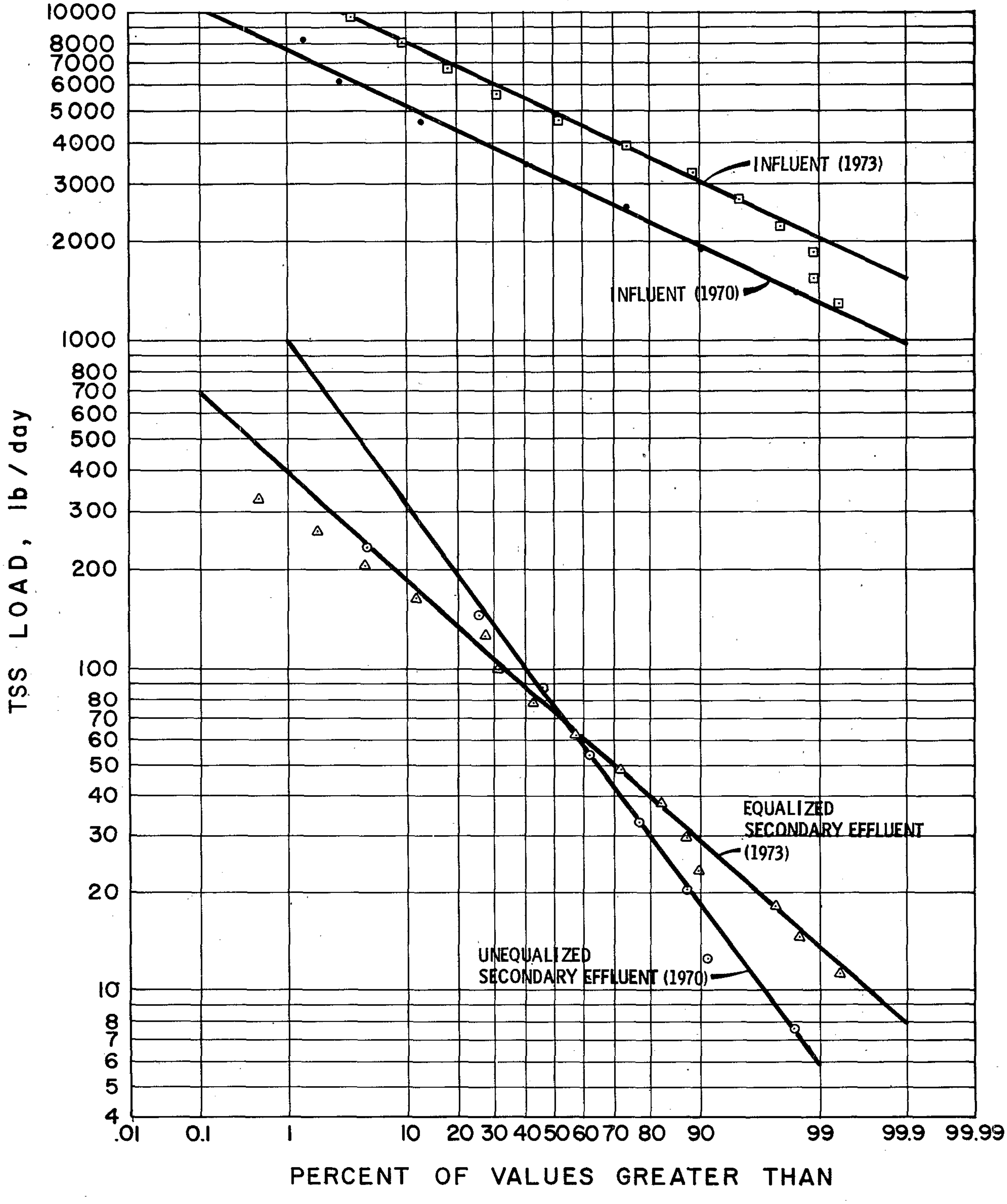

Figure 38. Distribution of TSS loads, Tecumseh, Michigan, 1970 and 1973

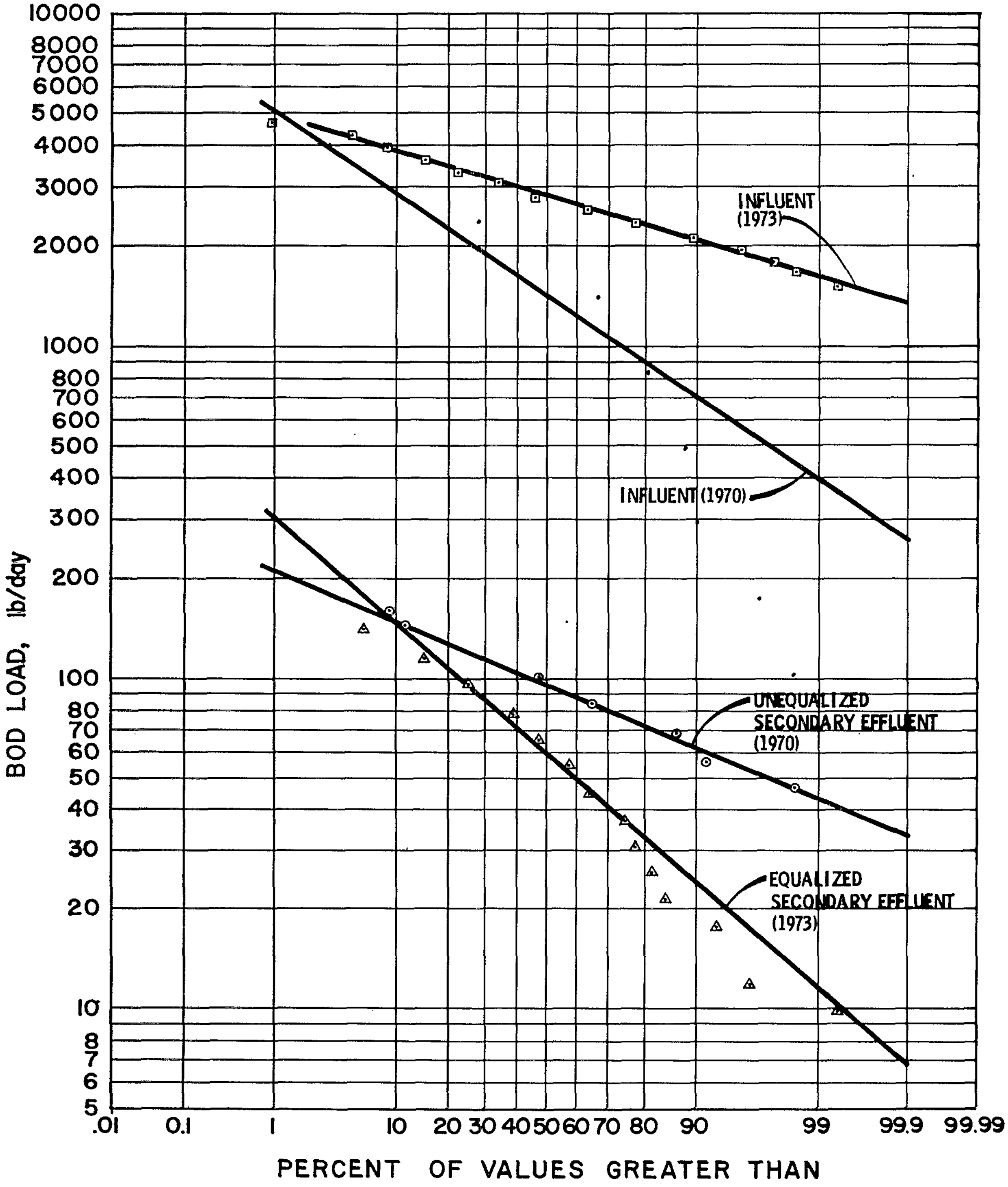

Figure 39. Distribution of BOD_5 loads, Tecumseh, Michigan, 1970 and 1973

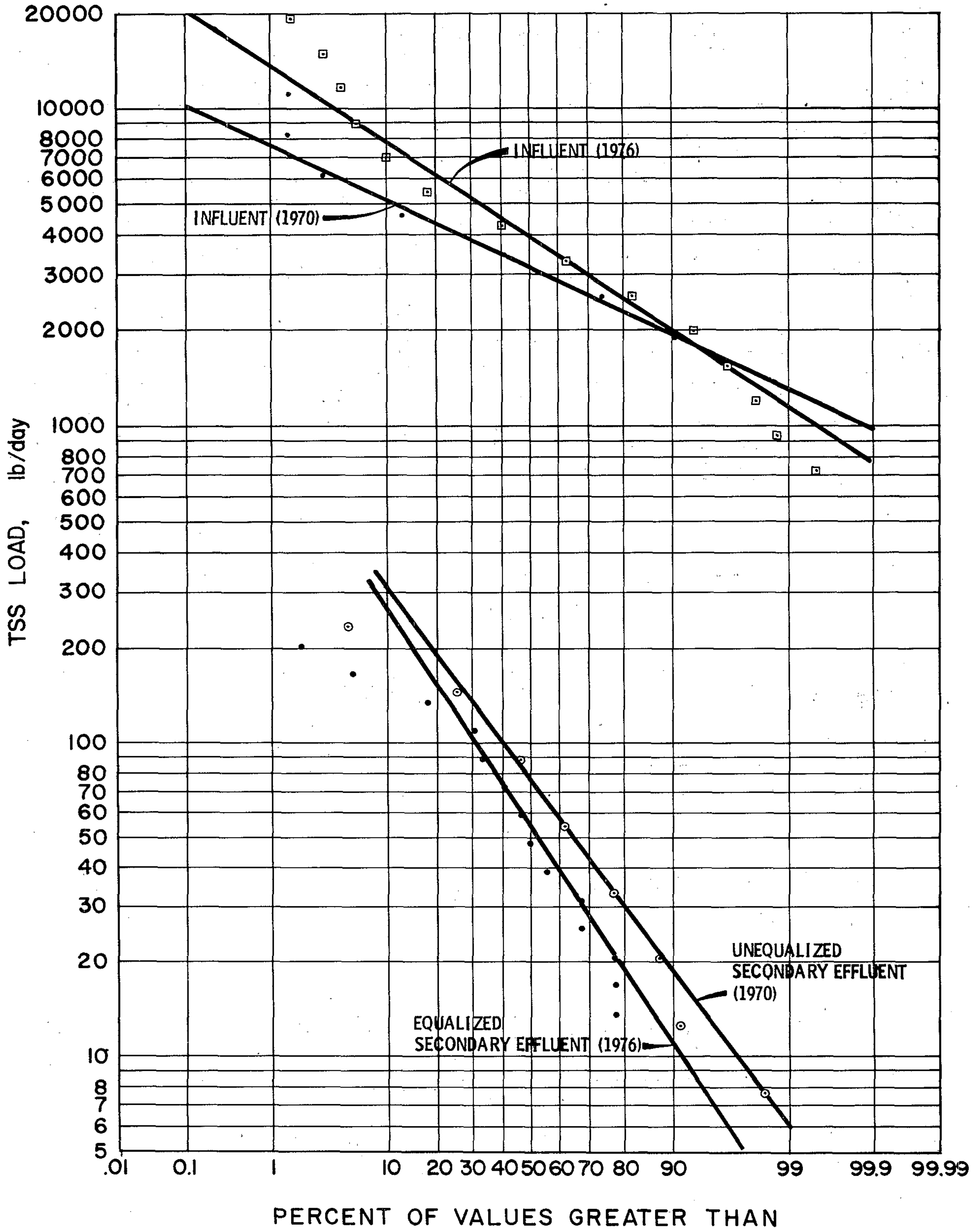

Figure 40. Distribution of TSS loads, Tecumseh, Michigan, 1970 and 1976

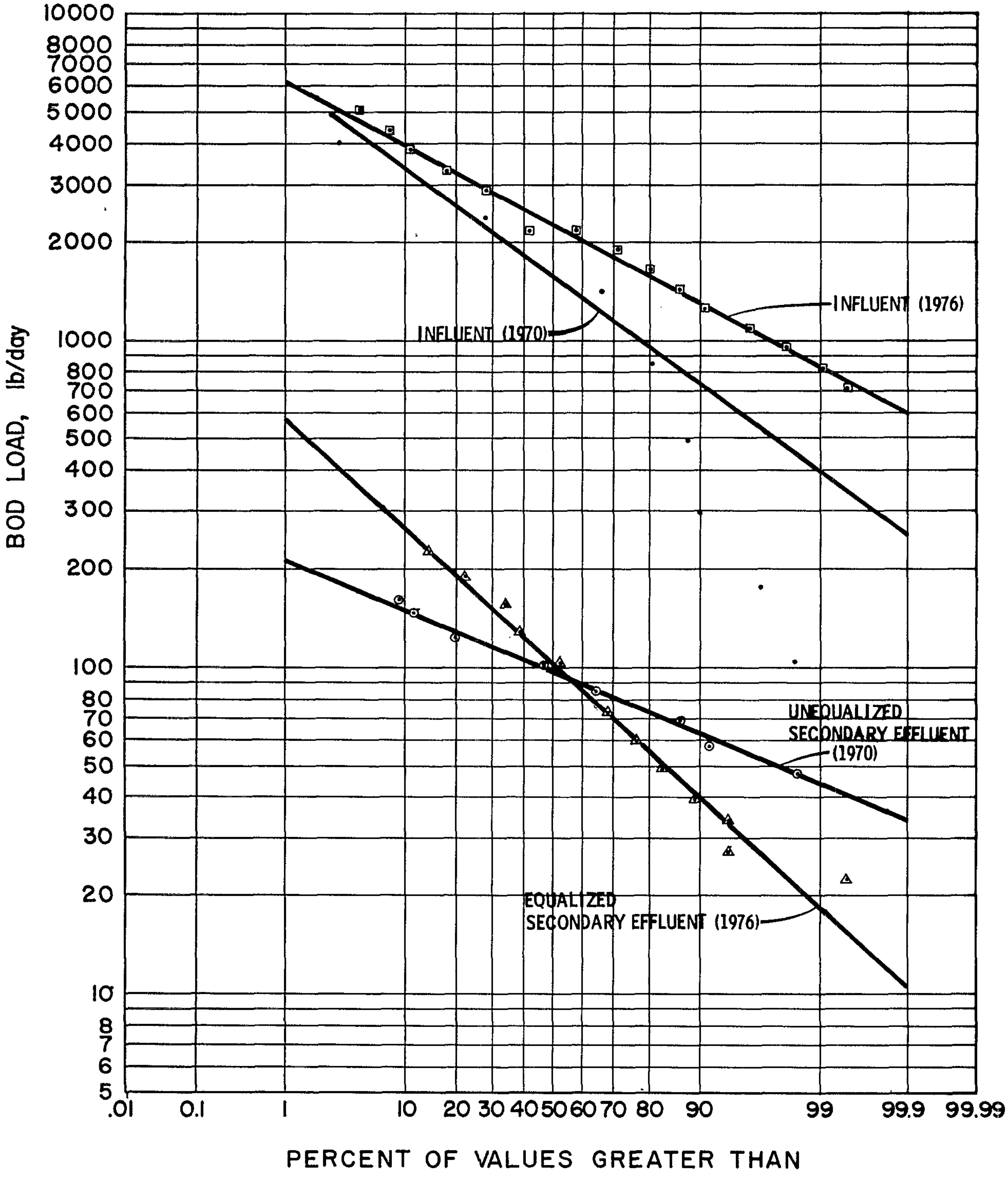

Figure 41. Distribution of BOD_5 loads, Tecumseh, Michigan, 1970 and 1976

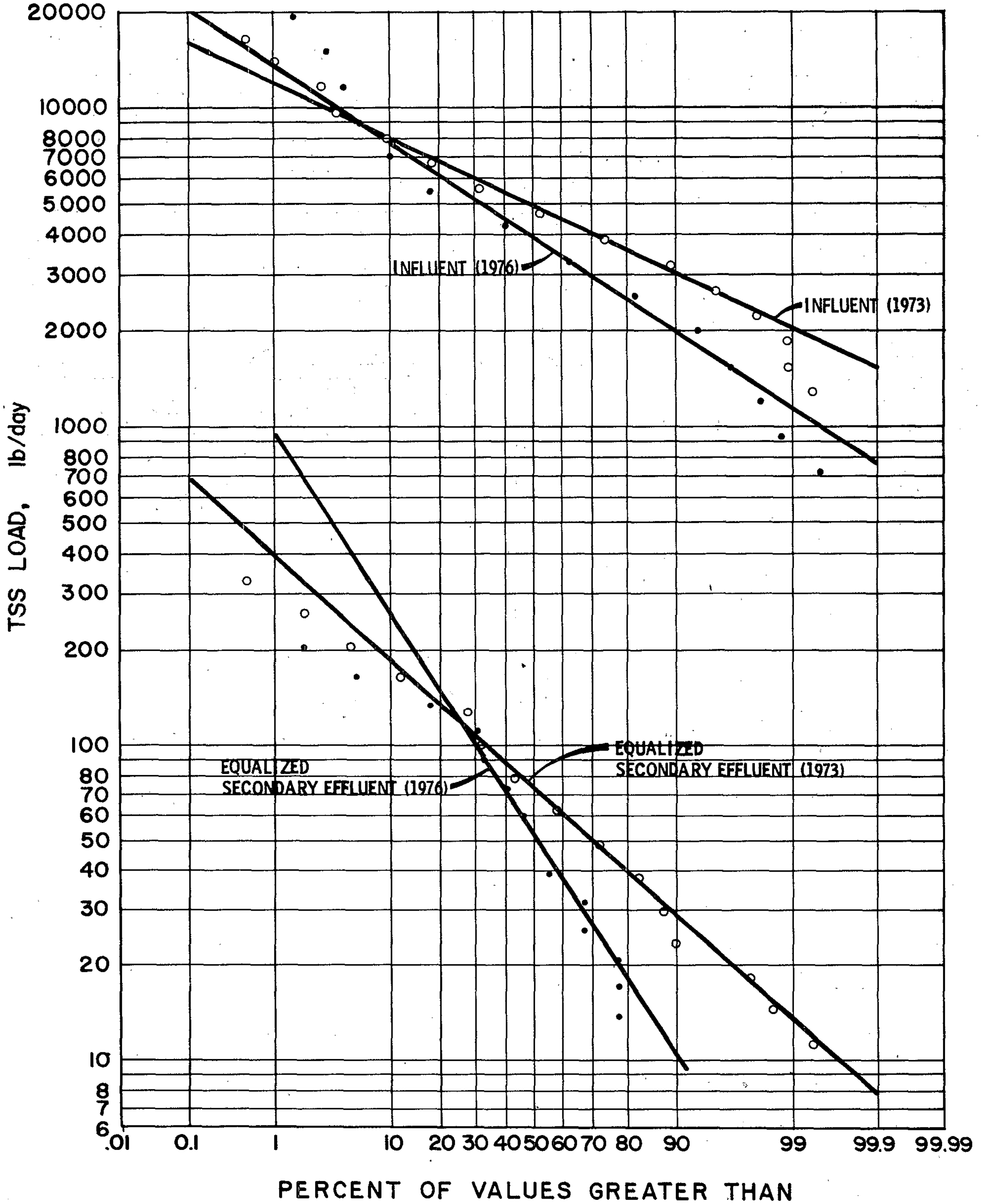

Figure 42. Distribution of TSS loads, Tecumseh, Michigan, 1973 and 1976

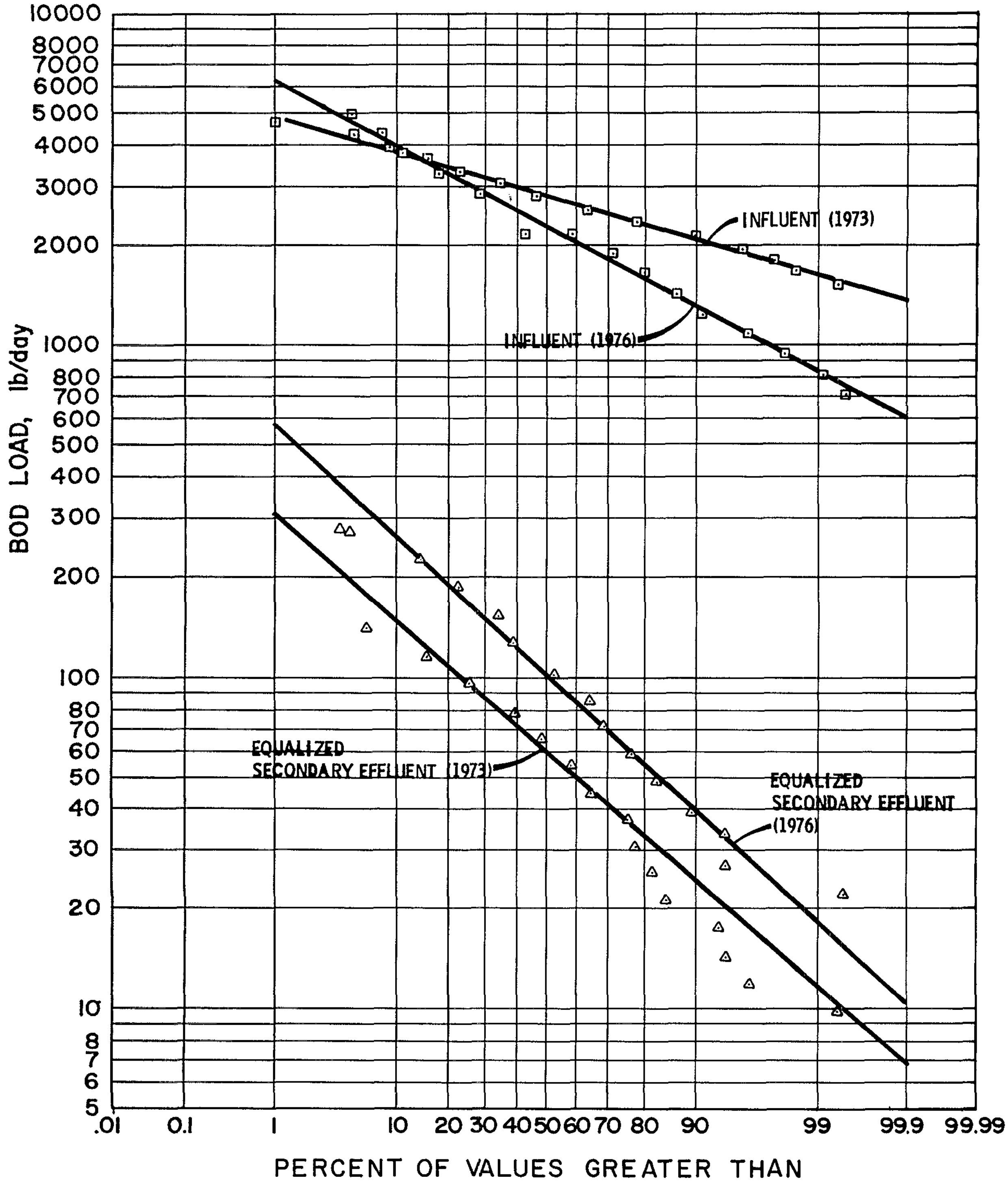

Figure 43. Distribution of BOD_5 loads, Tecumseh, Michigan, 1973 and 1976

TABLE 18. MEAN BOD AND TSS LOAD AND CONCENTRATION SUMMARY

Item	1970	1973	1976
Mean BOD load, lb/day:			
Plant influent	1,829	2,923	2,489
Plant effluent	98	76	134
Mean TSS load, lb/day:			
Plant influent	3,376	5,351	4,505
Plant effluent	136	94	115
Mean BOD conc., mg/l:			
Plant effluent	28	8	12
Mean TSS conc., mg/l:			
Plant effluent	13	9	14

performance was less variable than before equalization. However, although performance in 1976 was still significantly better than before equalization, it was slightly more variable.

The distributions of effluent TSS concentrations for the three years examined does not show a consistent pattern. In the equalized 1973 period the mean effluent TSS concentration was lower and slightly less variable than before equalization. However, in 1976 the mean effluent TSS concentration was nearly the same although slightly more variable than in the year before equalization.

No consistent relationship was apparent between effluent quality and influent loading. When loadings were lowest before equalization, effluent quality was poorest. However, during the equalized flow periods effluent quality was generally best during the period of higher loading. Although this suggests a simple inverse relationship between loading and performance, it is generally accepted that the opposite relation should exist for plants such as Tecumseh operating at loadings approaching the design capacity. Overall, with the exception of 1976 TSS concentrations, effluent performance appears to have benefitted from equalization. This is in agreement with observations of plant operating personnel.

Ypsilanti Township, Michigan--

A side-by-side comparison of equalized and unequalized activated sludge treatment performance (28) was conducted from June 1974 to July 1975. The Ypsilanti Township sewage treatment facilities consist of two parallel, but independent, activated sludge plants on a common site. Separate interceptors serve the two plants. Although influent sewage to the two plants is not identical, characteristics are similar, and local factors affecting sewage composition and influent variations are common to both. Plant No. 1 is equipped with an in-line equalization basin for diurnal flow smoothing; Plant No. 2 has no flow equalization. Plant No. 1 has a capacity of 3.7 mgd rated at a secondary clarifier overflow rate of 800 gpd/ft^2, and has no primary clarifier. Ferric chloride is added to the aeration tank effluent for phosphorus removal.

Average daily flows to both plants under typical operating conditions in 1974-1975 were about 4.0 mgd. Hourly average flows ranged from 1.5 to 6.0 mgd, with raw sewage peak-to-average ratios of about 1.5 under typical dry weather conditions. During the study period, average influent BOD was approximately 200 mg/l, with average hourly concentrations ranging from 50 to 400 mg/l. Influent wastewater to both plants contains about 25 percent industrial wastewater.

The in-line equalization system consists of two converted anaerobic digesters providing a total capacity of 624,000 gallons. The tanks are aerated to prevent solids deposition and to maintain aerobic conditions in the untreated wastewater. Flow is pumped to the tanks from the plant influent pump station. Constant gravity flow to the downstream treatment processes is maintained by automatic control of the equalization tank discharge valve.

During the experimental period, the unequalized Plant No. 2 was not altered from its normal operating routine. Data from this plant provide a basis for comparing performance characteristics for constant and varying flow conditions. Plant No. 1 was observed under three modes of operation: (1) equalized flow at approximately design capacity, maintained for all but five weeks of the June 1974 to July 1975 study period; (2) equalized flow for hydraulically stressed conditions in the secondary clarifier for brief periods between March 17 and April 17, and from June 3 to July 3, 1975; and (3) unequalized flow conditions were maintained from May 14 to June 14, 1975.

Operating data from the two plants for the calendar year 1976 are presented to provide an additional comparison of equalized and unequalized performance.

Influent and effluent BOD and TSS concentrations and load distributions from the 1974-1975 study of equalized Plant No. 1 and unequalized Plant No. 2 are shown in Figures 44 through 47. The obvious similarity between distributions of BOD and TSS concentration and BOD and TSS load results from the concurrence of operating periods with nearly identical influent wastewater components. Due to the similarity of the distributions only the concentration figures will be discussed. Influent and effluent BOD and TSS concentration distributions for 1976 are shown in Figures 48 and 49.

The distribution of effluent BOD from equalized Plant No. 1 was lower than the unequalized distribution; approximately 7 mg/l difference in median values. The annual means can be calculated as 15.8 mg/l and 23.3 mg/l respectively. Variability of the equalized performance was slightly greater than unequalized performance; approximately 20 percent difference in log standard deviations. The observed difference between TSS performances was approximately 20 percent in log standard deviations. The observed difference between TSS performance levels of equalized and unequalized plants was negligible; both population means approximately 17 mg/l. In this case, performance variability of the equalized plants was less than for the unequalized plant; approximately 27 percent difference in log standard deviations. Considering the comparability of the plant loading conditions over the study period, and the concurrent period of observation, equalization appears to have contributed to improved plant performance.

Observations of Plant No. 1 performance during alternating consecutive 31-day unequalized, equalized, and unequalized operating periods (from April 13 to July 14, 1975) should be considered along with the comparison of Plant No. 1 and No. 2 above. The performance summary (28) indicates that little difference was observed between equalized and unequalized flow periods. If anything, performance during the equalized flow period was judged to be slightly better. However, differences in flow and loading conditions during the respective periods were large enough to have had as much or more influence on performance as the existence or absence of flow equalization.

Comparing concentration distributions for the 1974-1975 study (Figures 44 and 45) and for 1976 (Figures 48 and 49) reveals that plant performance was remarkably similar during the two periods. Almost no difference can be seen between effluent TSS distributions of the two plants during the two periods. Median annual performance is approximately 13 to 15 mg/l; each with approximately the same level of variability.

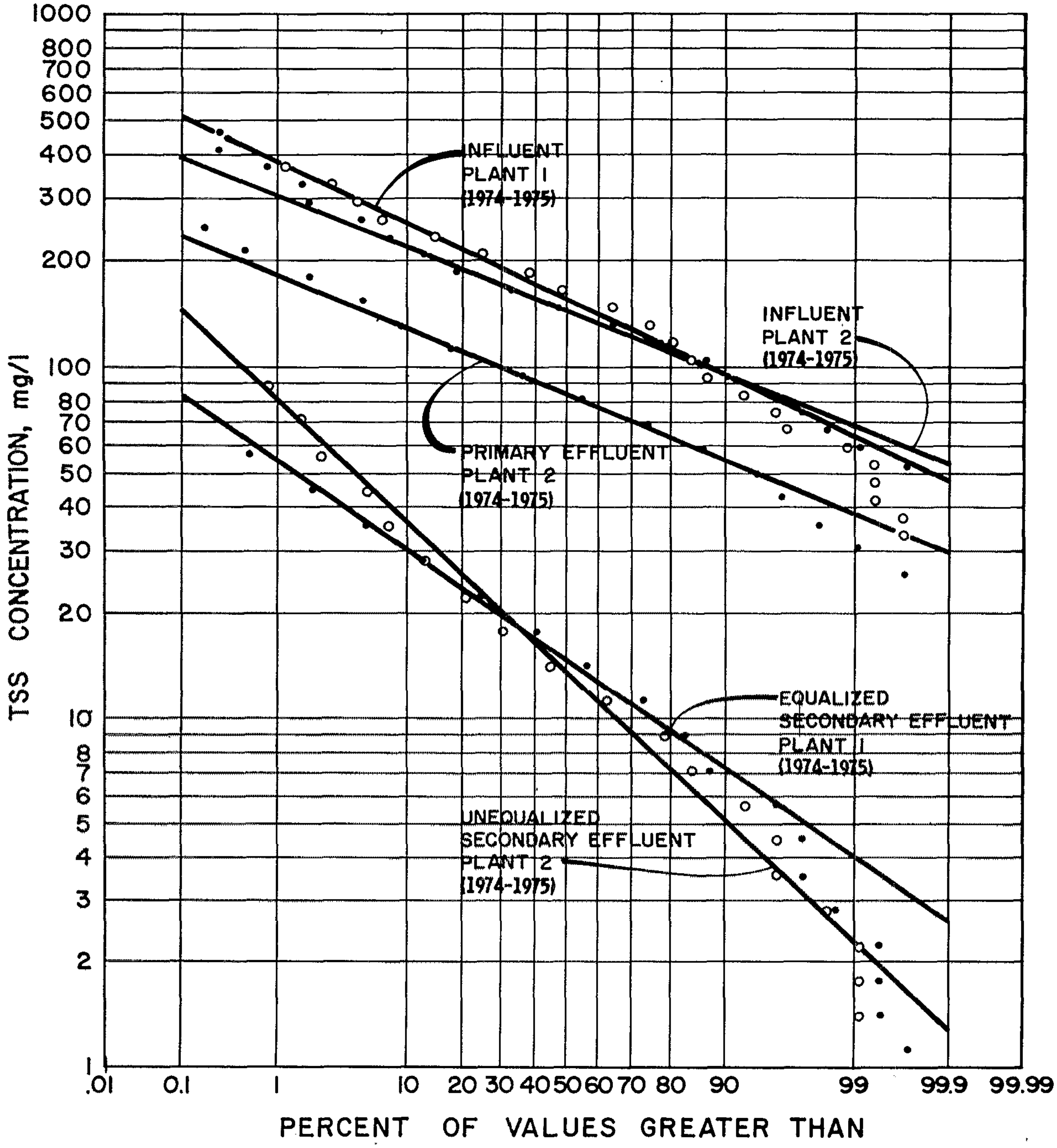

Figure 44. Distribution of TSS concentrations, Ypsilanti Township, Michigan, 1974-1975

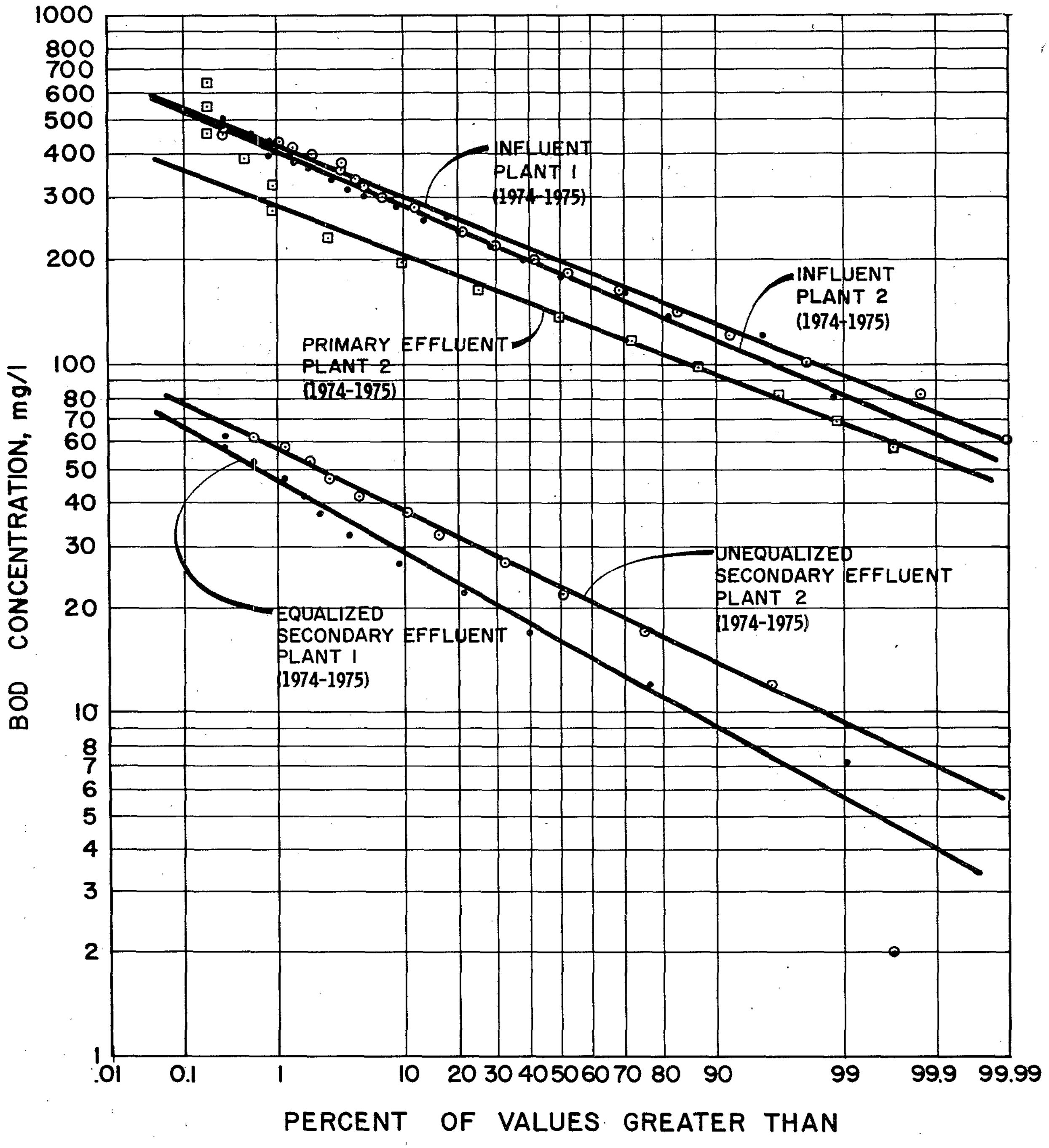

Figure 45. Distribution of BOD_5 concentrations, Ypsilanti Township, Michigan, 1974-1975

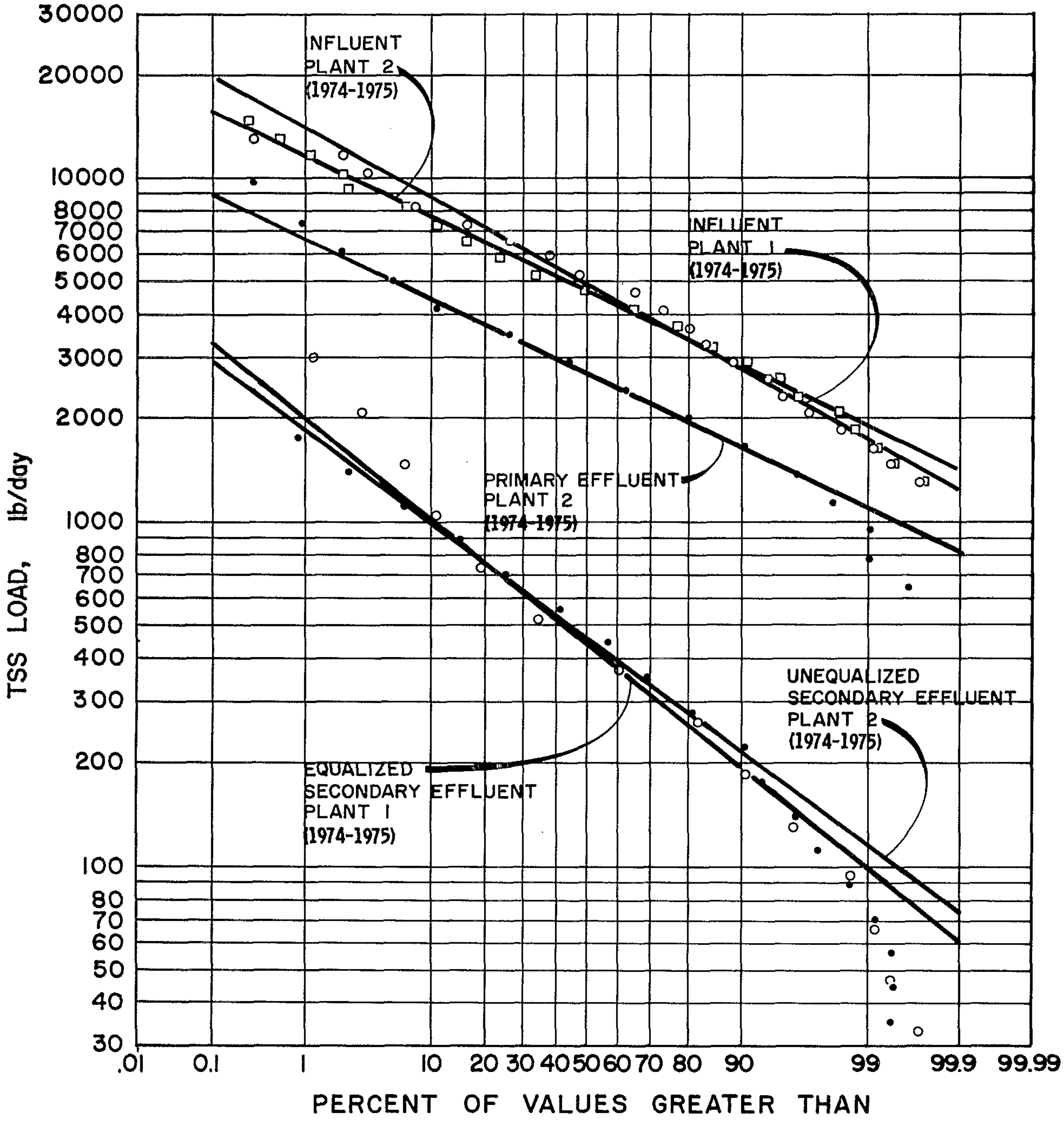

Figure 46. Distribution of TSS loads, Ypsilanti Township, Michigan, 1974-1975

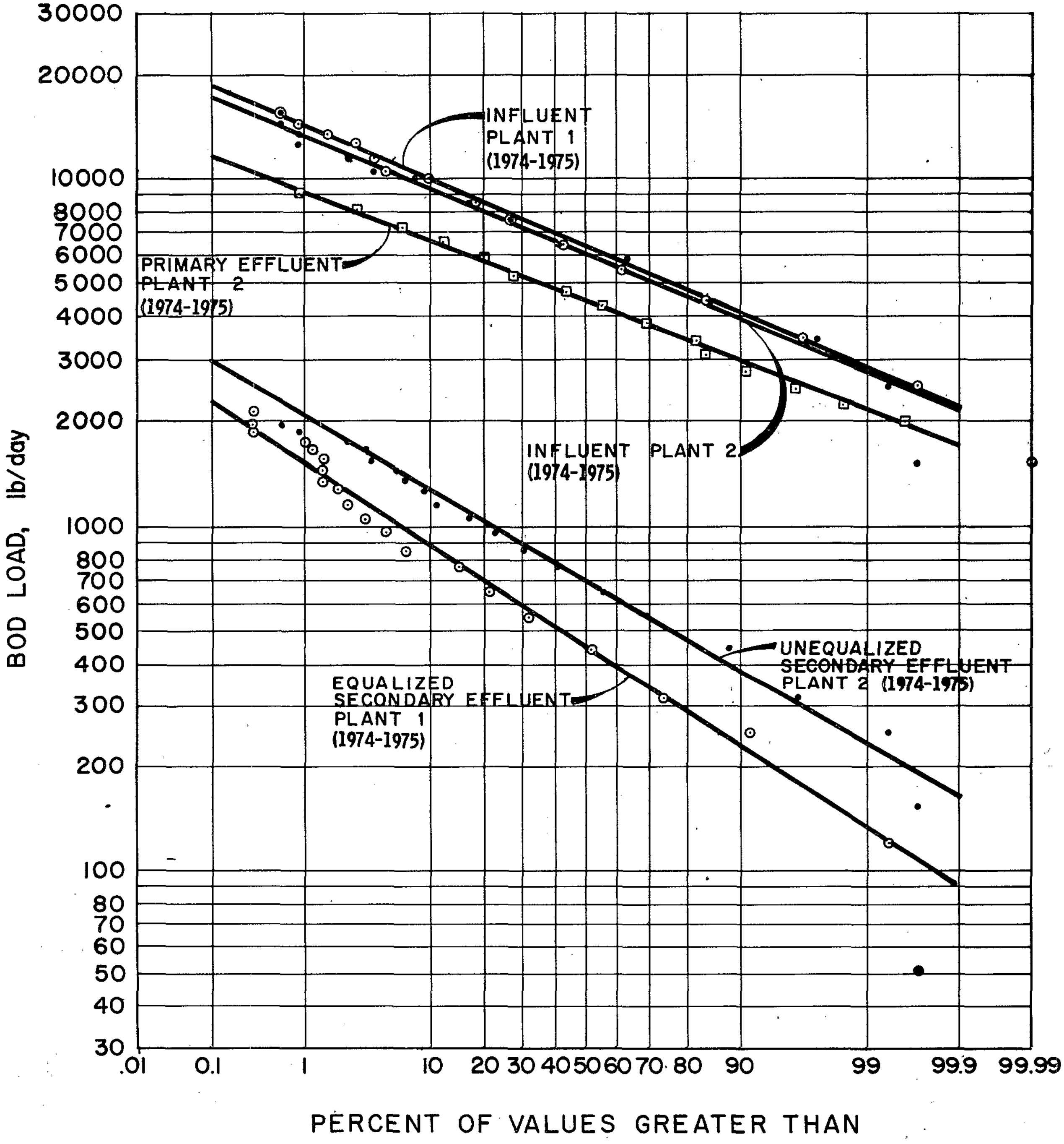

Figure 47. Distribution of BOD_5 loads, Ypsilanti Township, Michigan, 1974-1975

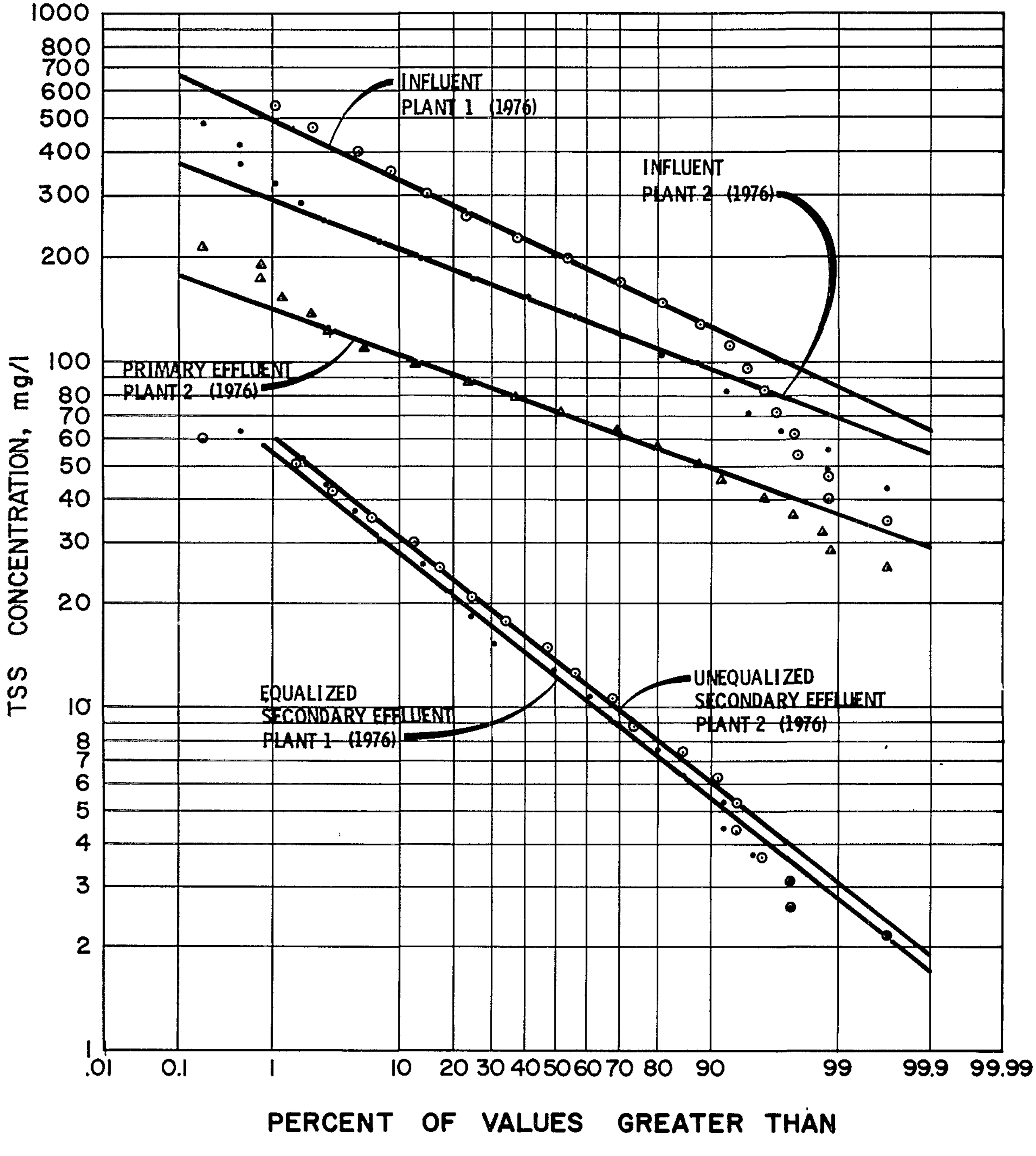

Figure 48. Distribution of TSS concentrations, Ypsilanti Township, Michigan, 1976

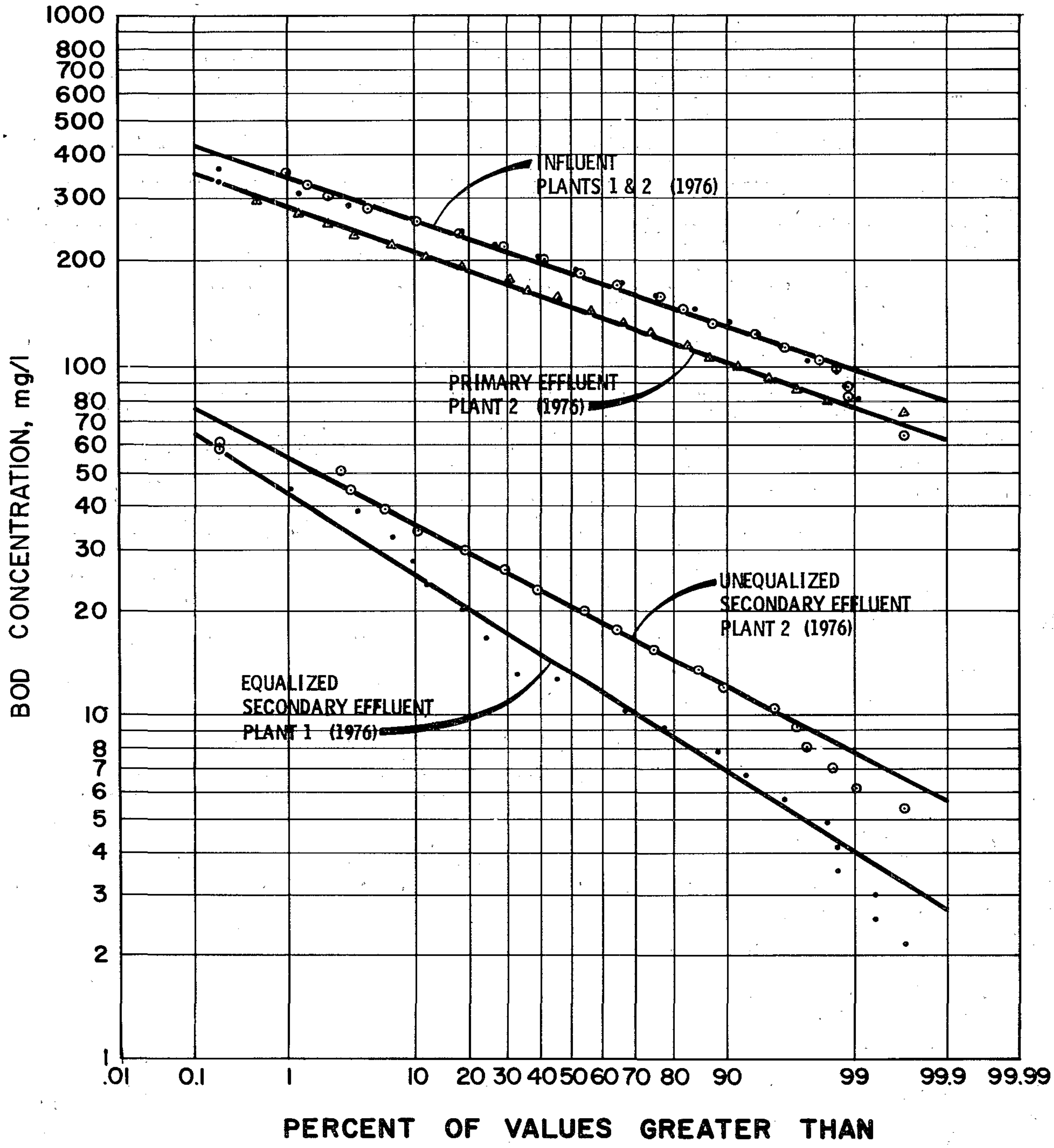

Figure 49. Distribution of BOD_5 concentrations, Ypsilanti Township, Michigan, 1976

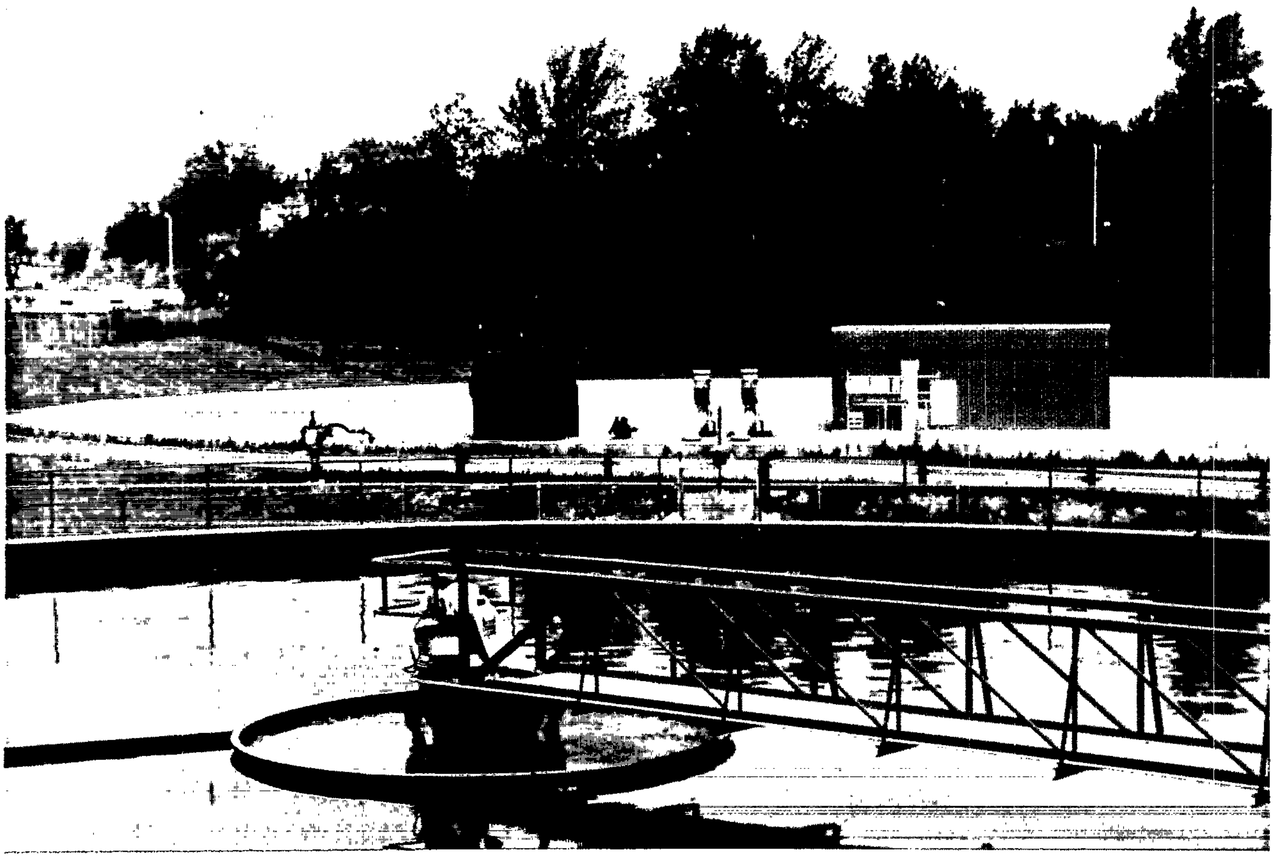

Figure 50. Equalization tank (background), 3 million gallon capacity, Pontiac, Michigan

Pontiac, Michigan--

Two activated sludge plants, East Boulevard and Auburn, treat wastewater from the City of Pontiac. Flow in a single major interceptor passes the East Boulevard plant, where the desired quantity is diverted for treatment. The remainder is conveyed to the Auburn plant for treatment. The existing East Boulevard activated sludge facilities have been in operation for more than 30 years. A 3 million gallon side-line equalization tank (Figure 50) was added in 1975 to limit peak daily flows to the effluent filters, to stabilize operating conditions for phosphorus removal, and to provide some protection against toxic spills from extensive local manufacturing industry. No other changes were made in the plant, but the capacity of the Auburn plant was expanded at the same time, reducing flows to East Boulevard. This reduced process loading at both plants, so that performance characteristics before and after equalization are not directly comparable. For example, average BOD loading to the East Boulevard plant was approximately 28 percent lower in 1976 than in 1973 (Figure 51).

The East Boulevard plant has a capacity of about 10 mgd. Average dry weather flow to the plant in 1977 was maintained between 8 and 9 mgd. The facilities consist of primary clarification followed by equalization, activated sludge, and secondary clarification. Ferric chloride and polymer are added to aeration tank effluent for phosphorus removal. Final effluent is piped to the Auburn plant for effluent filtration and disinfection.

The Auburn plant is also a conventional activated sludge plant with primary and secondary clarification. It does not have equalization facilities of its own. However, flows to the plant are partially equalized by operation of the East Boulevard and Auburn plants in series, equalization at East Boulevard, and the location of the Auburn plant at the downstream end of the system. Effluent filters and chlorine contact facilities at the Auburn plant treat the combined flow from both plants. Total average dry weather flow to the two plants in 1977 was about 20 mgd, with a peak-to-average ratio typically 1.3 to 1.4. Wet weather flows may exceed 50 mgd.

During typical dry weather flow operating periods, primary effluent is diverted to the equalization tank when flows at the East Boulevard and Auburn plants exceed 8 and 12 mgd, respectively. Stored wastewater is pumped to secondary treatment in deficit flow periods. The stored wastewater is not mixed or aerated.

Annual distributions of average daily influent and effluent TSS and BOD concentrations at the East Boulevard plant are shown in Figures 51 and 52. Data from 1973 are representative of operation prior to the addition of equalization at East Boulevard and extensive additions to the Auburn plant downstream. Data from September 1976 to August 1977 cover the year immediately following commencement of equalization at East Boulevard.

The secondary effluent TSS distribution in the equalized period indicates better and slightly less variable performance than in the unequalized period. However, lighter influent and primary effluent loadings in the equalized period could easily have resulted in the observed secondary effluent differences. Comparison of secondary effluent BOD distributions (Figure 52) reveals very little difference in performance between the equalized and unequalized periods. In fact, the secondary process performance in the unequalized period could be considered superior because of the substantially higher loadings during that period.

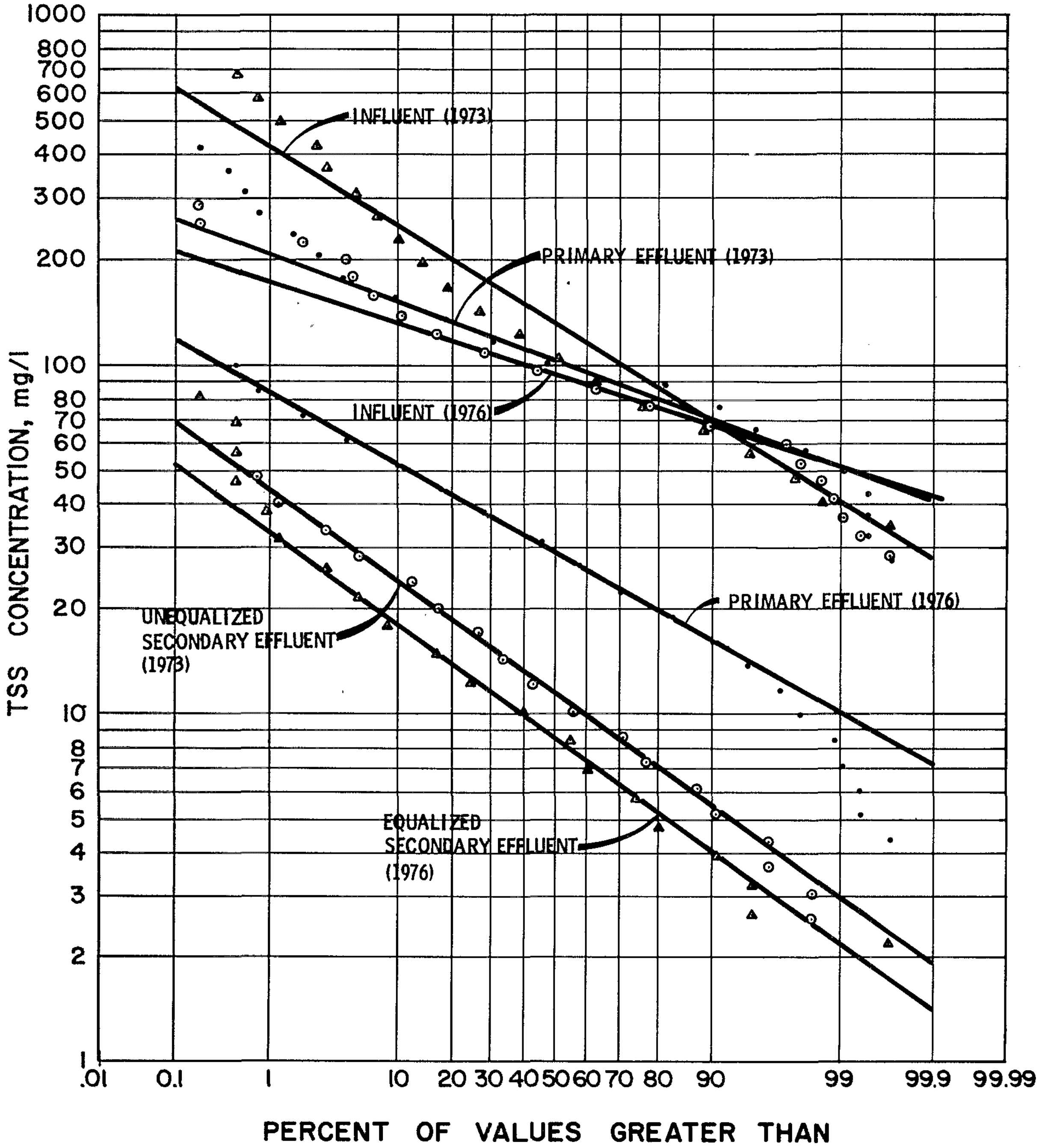

Figure 51. Distribution of TSS concentrations, Pontiac, Michigan, East Boulevard Plant, 1973 and 1976-1977

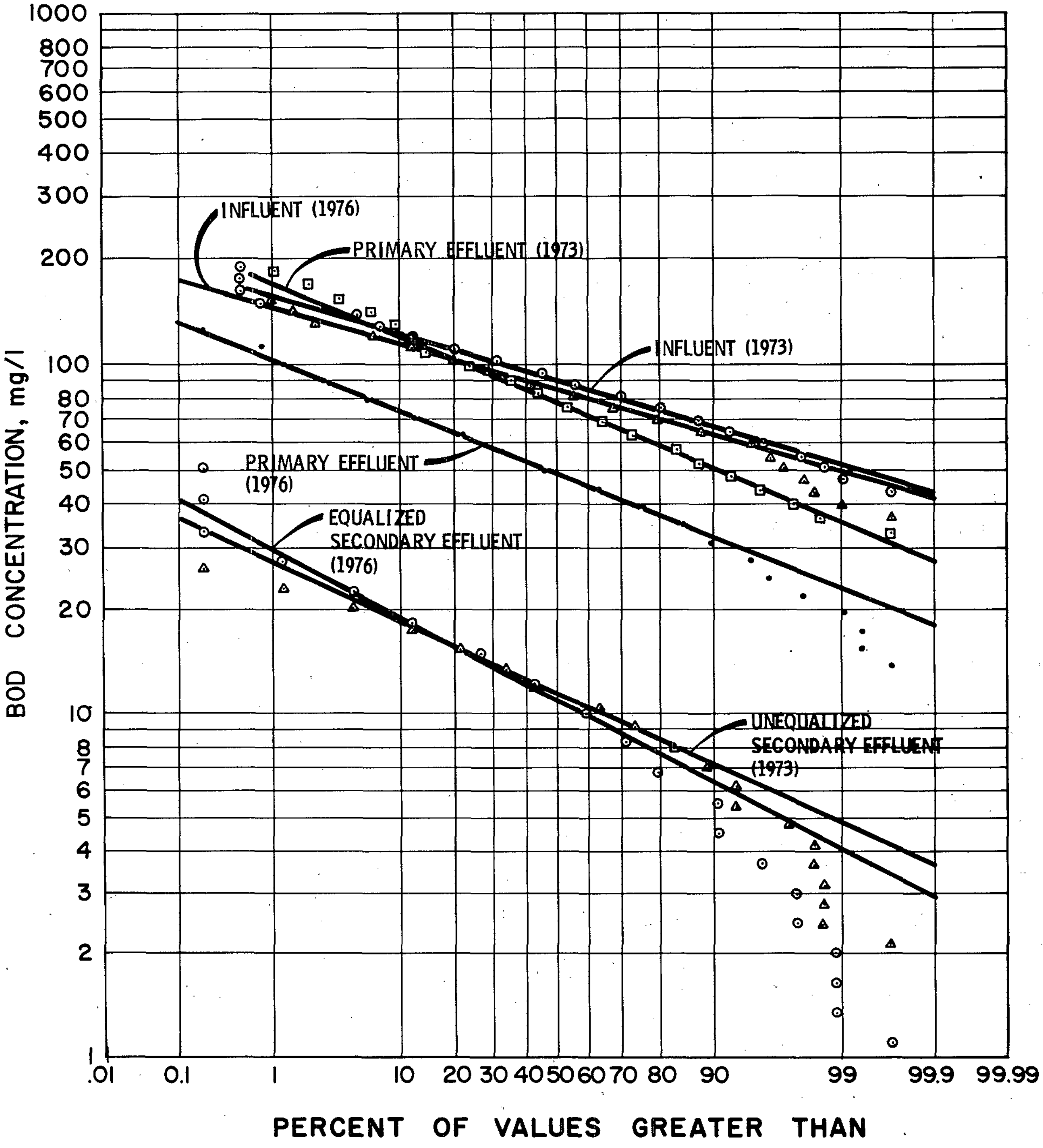

Figure 52. Distribution of BOD_5 concentrations, Pontiac, Michigan, East Boulevard Plant, 1973 and 1976-1977

Annual distributions of average daily influent and effluent TSS and BOD concentrations at the Auburn plant are shown in Figures 53 and 54. The 1973 and 1976-1977 data used correspond to those used to characterize operation prior to and following equalization use at the East Boulevard plant. The major improvements in primary and secondary TSS and BOD effluent distributions are attributed to individual unit process loading reductions resulting from expansion of the Auburn plant; including primary clarifier, activated sludge, and secondary clarifier capacity.

Comparison of Auburn and East Boulevard plant performances for the 1976-1977 period does not permit any conclusions to be drawn with respect to the significance of equalization. Secondary effluent BOD performance of the Auburn plant is significantly better than that of East Boulevard. Performance of the two plants, with respect to BOD, in 1973 was virtually identical.

Amarillo, Texas--

A 3 million gallon equalization lagoon (Figure 55) was added to the Amarillo River Road treatment plant in September 1965 to reduce peak flows, which were creating significant solids loss from the secondary clarifiers almost daily. At that time, the plant capacity was 7.5 mgd, but it was receiving an average dry weather flow of 10.5 mgd, with afternoon peak flows of 17 mgd. Secondary clarifier overflow rates during peak flow periods were approximately 1,100 gpd/ft^2. Consistent effluent quality at this plant must be maintained because the reclaimed wastewater is subsequently used for cooling water (after additional treatment) by a steam-electric power generating plant and an oil refinery. The excess effluent is discharged to a dry stream bed. No other significant changes were made in the plant at that time, but some operational modifications were introduced. Details of operating conditions and changes have been reported in the literature.(29)

The River Road treatment facilities consist mainly of primary clarifiers, plug flow activated sludge, and secondary clarifiers. Chlorinated effluent is discharged to two 9 million gallon effluent storage ponds before reuse or discharge. The 3 million gallon equalization lagoon receives primary effluent that is pumped to storage during excess flow periods. Flow is returned by gravity to the influent channel of the primary clarifiers during deficit flow periods, with the desired rate maintained by an operator preset, automatic flow control valve.

Distributions of influent and effluent BOD and TSS concentrations and loads observed during corresponding 3-month periods (August-October) before (1964) and after (1966) instituting

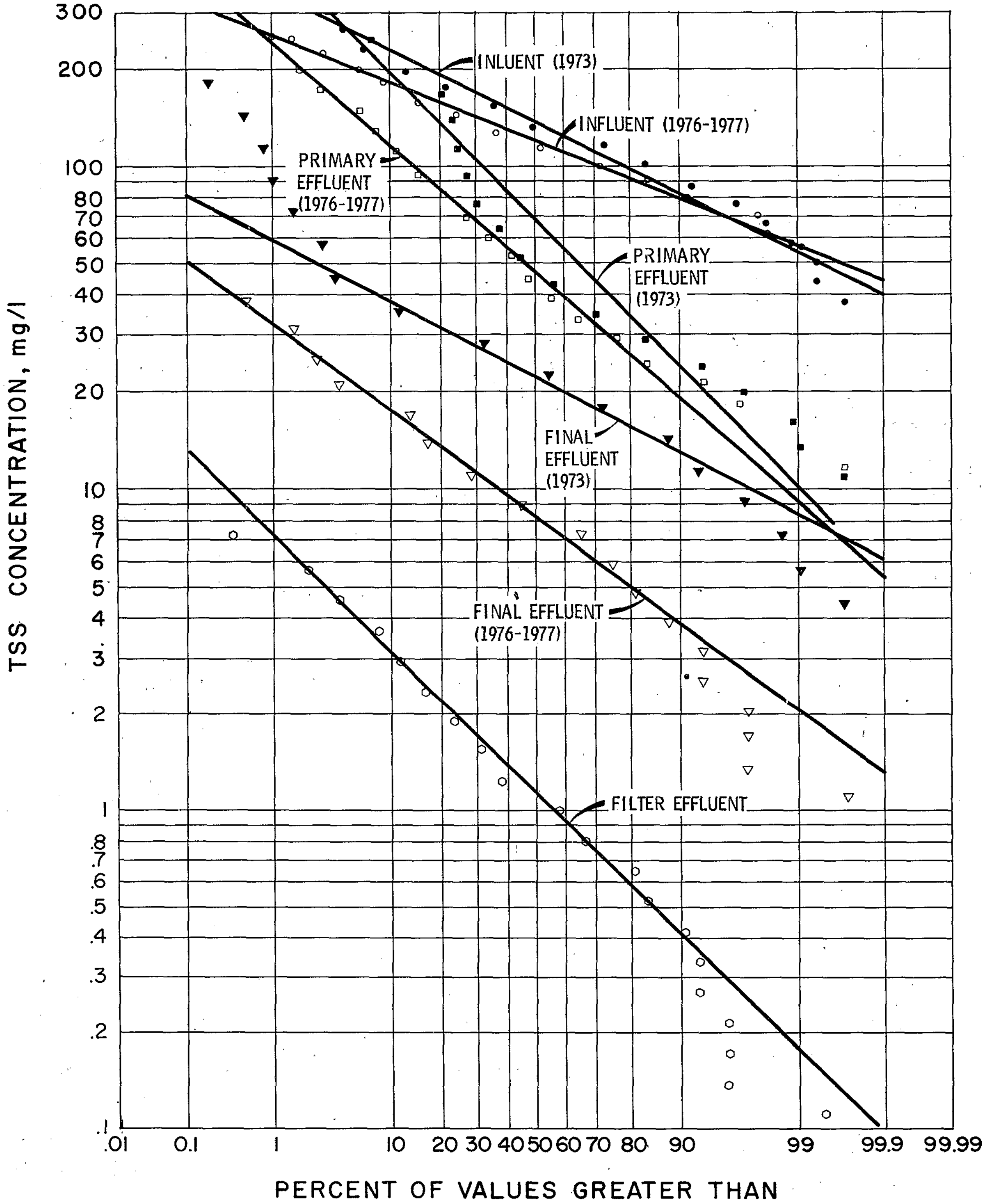

Figure 53. Distribution of TSS concentrations, Pontiac, Michigan, Auburn Plant, 1973 and 1976-1977

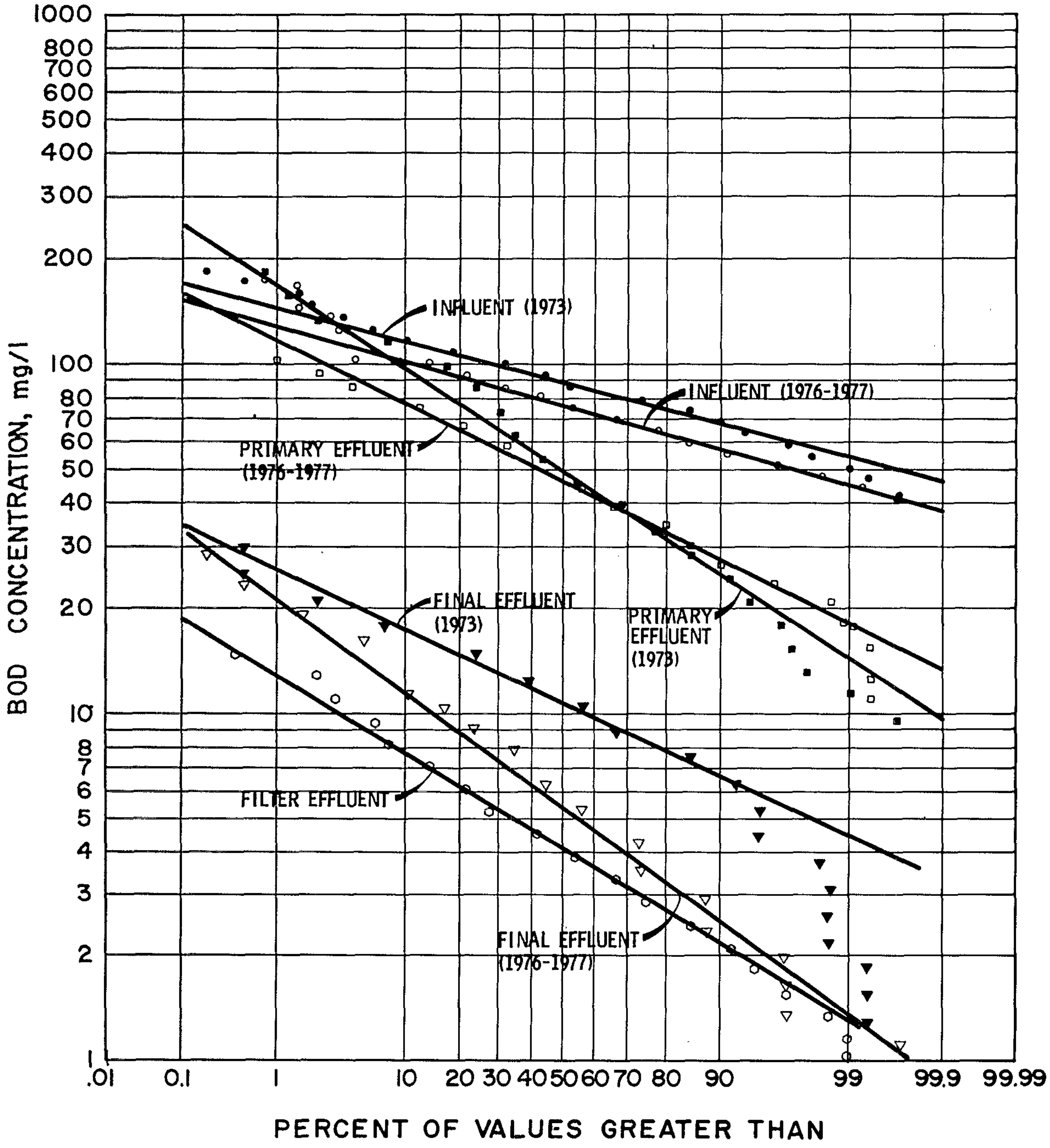

Figure 54. Distribution of BOD_5 concentrations, Pontiac, Michigan, Auburn Plant, 1973 and 1976-1977

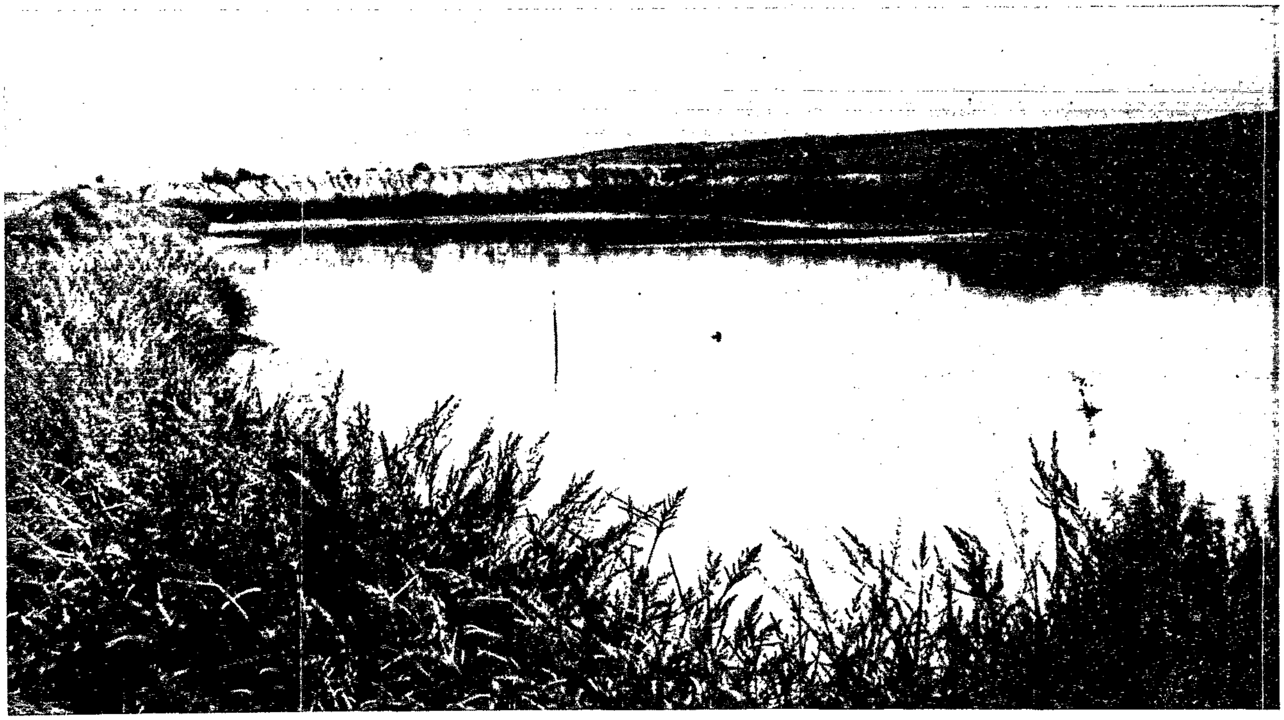

Figure 55. Equalization basin, 3 million gallon capacity, Amarillo, Texas

equalization are shown in Figures 56 through 59. The concentration and loading distributions of both BOD and TSS show similar patterns, attributed to the similarity of influent flow distributions in the equalized and unequalized periods examined.

The distribution of effluent BOD concentrations was observed to be lower and less variable during the equalized flow period, with the difference in distribution means being approximately 8 mg/l (or 20 percent), and approximately 20 percent difference in the log standard deviations. Very slight difference was observed between distributions of equalized and unequalized TSS concentrations; equalized performance was approximately 5 percent lower but more variable. These observations correspond generally to recollections of plant operating personnel. Although significant deterioration of effluent TSS quality was reported almost daily, (29) plant records reflected by Figure 56 indicate relatively small improvement. It is possible that effluent sampling procedures permitted some bias in favor of better effluent quality. Overall reported performance and performance characteristics described herein indicate a slight but positive influence of equalization on effluent quality.

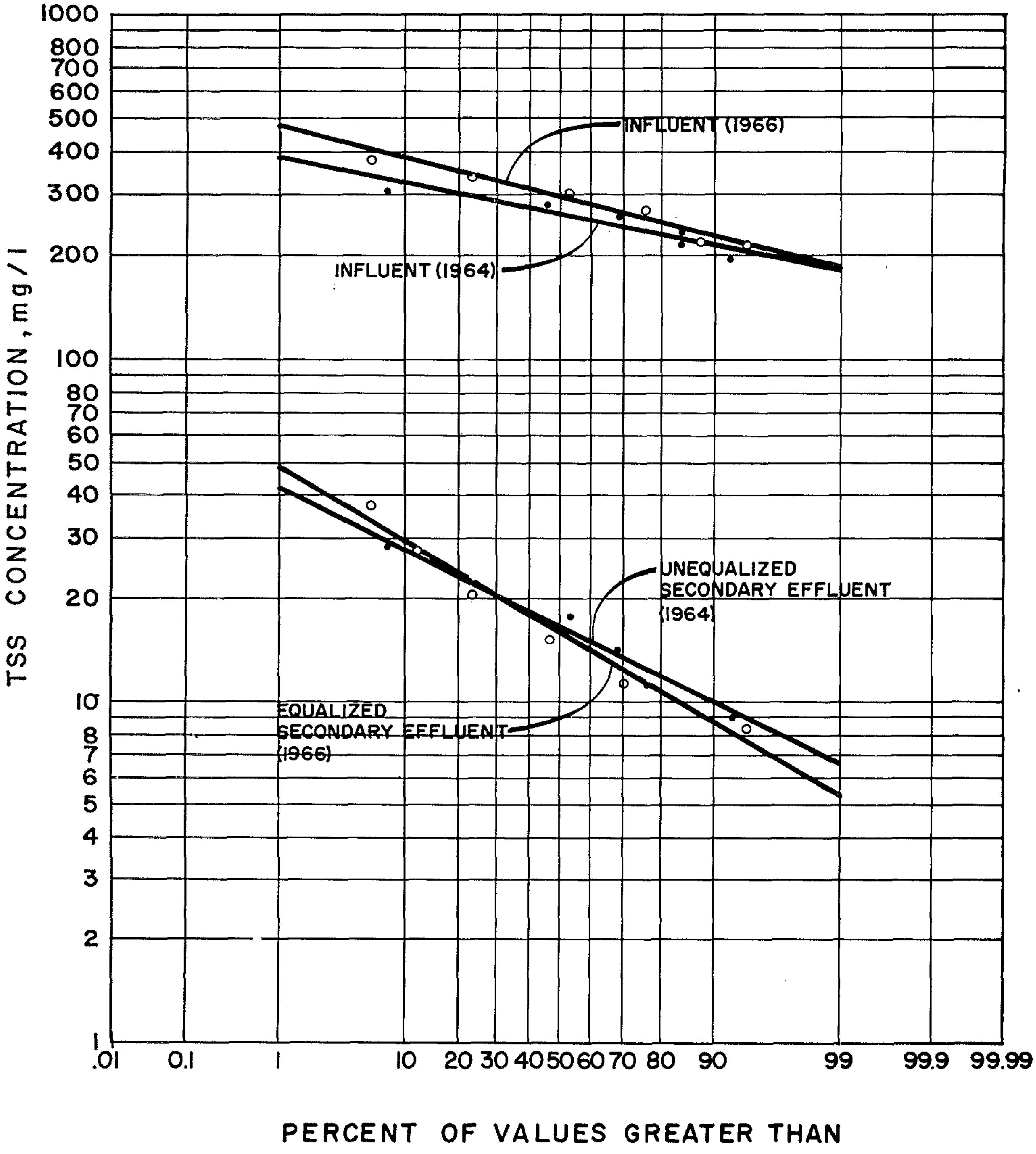

Figure 56. Distribution of monthly TSS concentrations, Amarillo, Texas, 1964 and 1966

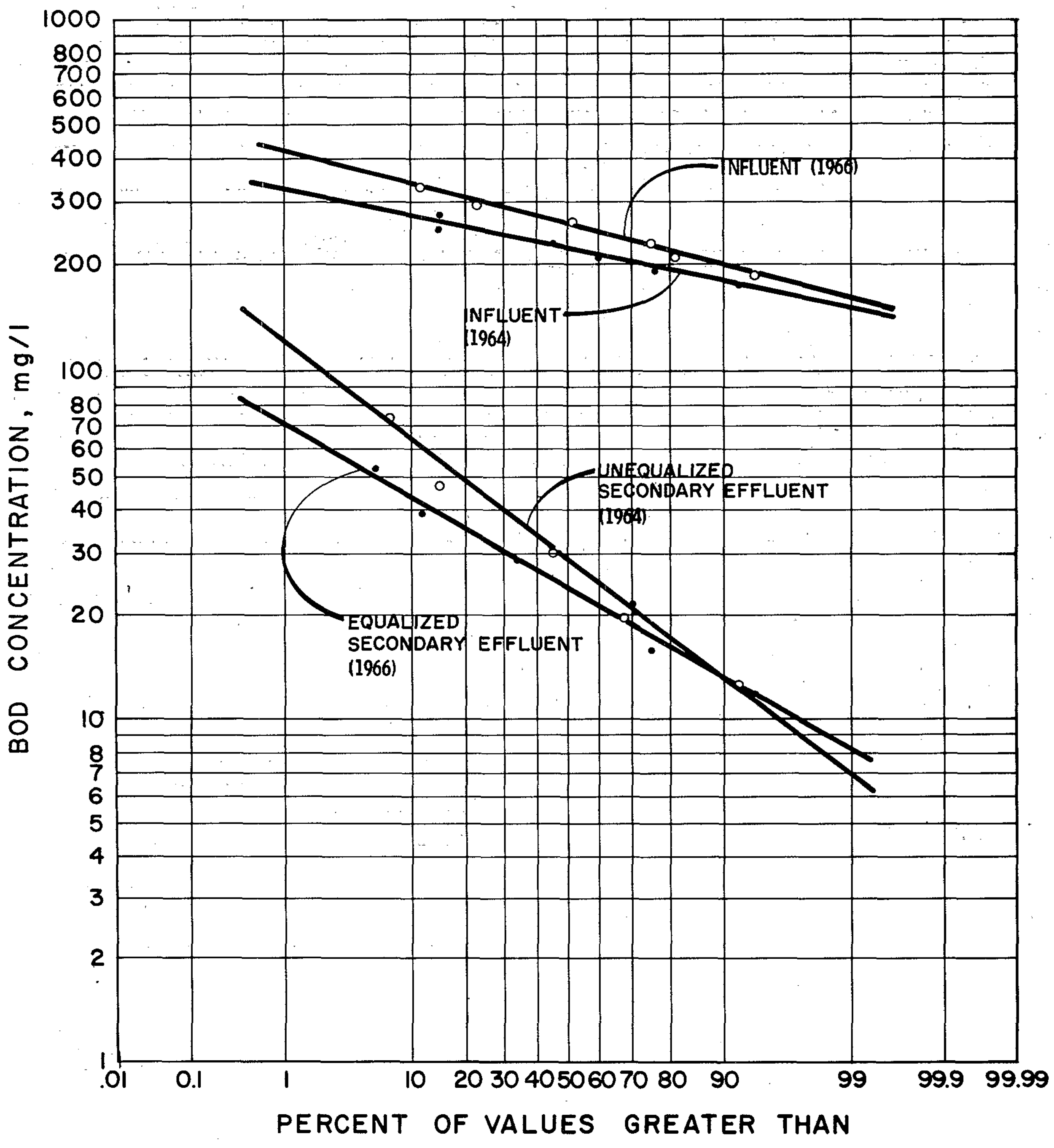

Figure 57. Distribution of monthly BOD_5 concentrations, Amarillo, Texas, 1964-1966

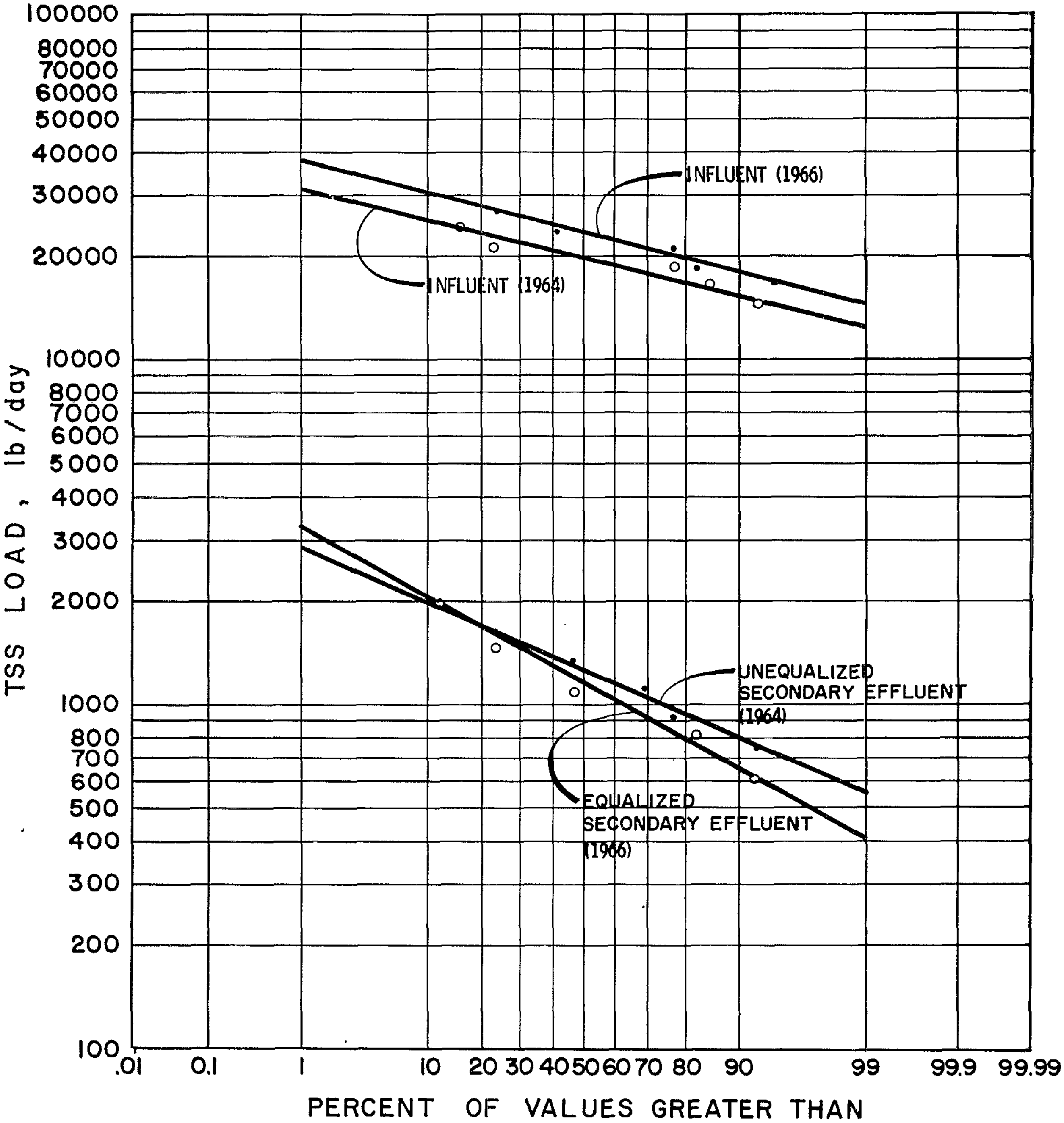

Figure 58. Distribution of monthly TSS loads, Amarillo, Texas, 1964 and 1966

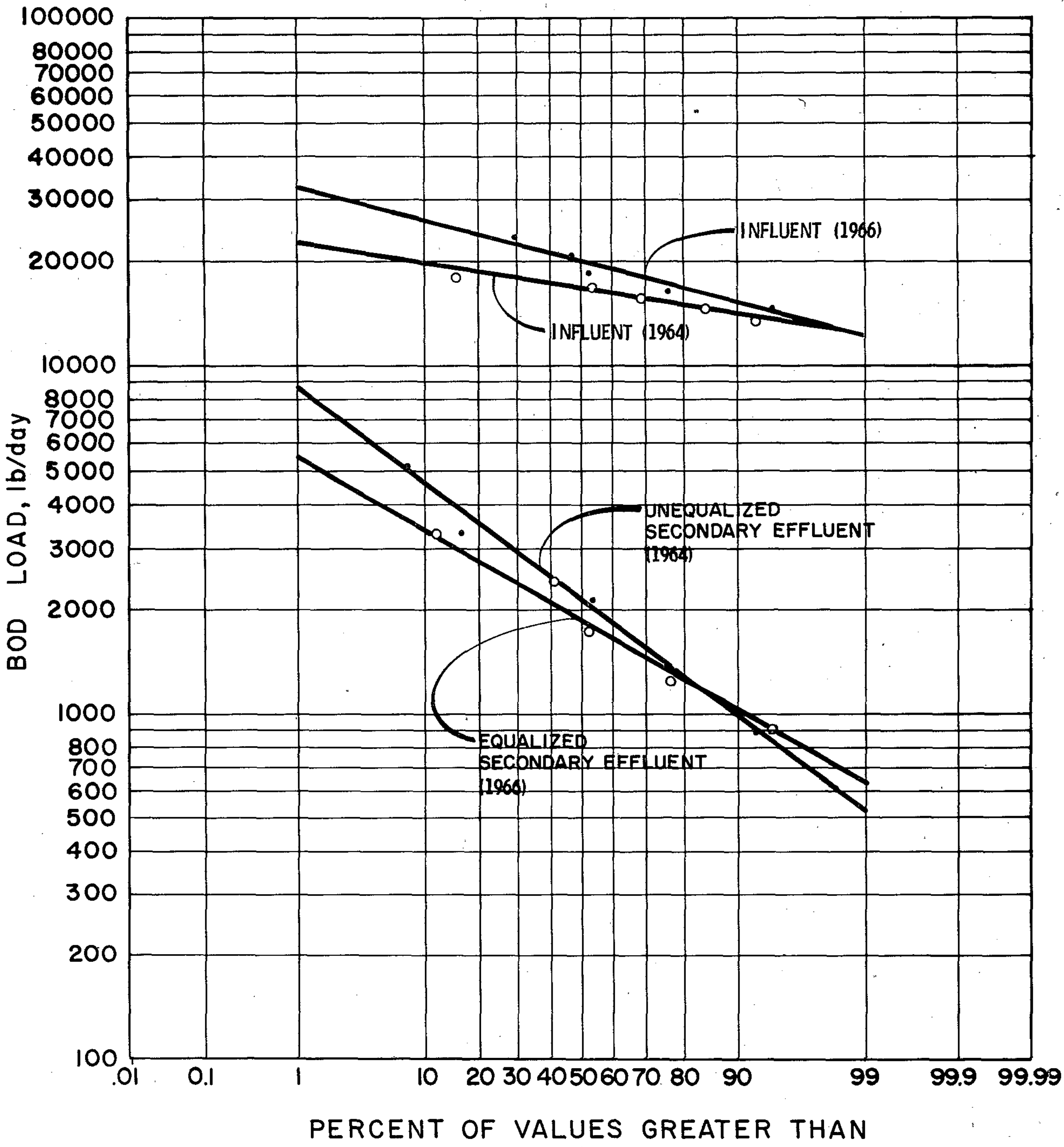

Figure 59. Distribution of monthly BOD_5 loads, Amarillo, Texas, 1964 and 1966

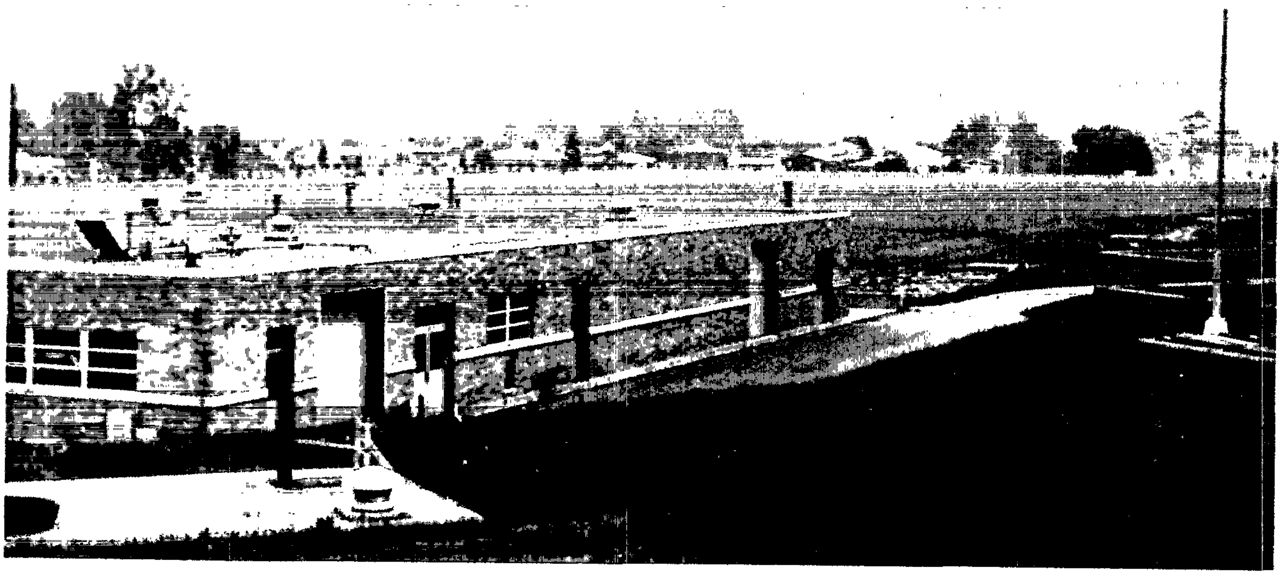

Figure 60. Equalization Tank, 50 million gallon capacity, (center, behind building, underground), Warren, Michigan

Warren, Michigan--

The City of Warren, Michigan has a large activated sludge treatment plant serving a major urban and industrial center in southeastern Michigan adjacent to Detroit. The treatment plant has a design treatment capacity of 36 mgd, hydraulic capacity of 80 mgd, and pumping capacity of 150 mgd. Flows as high as 50 mgd during storm periods can be treated successfully. Average dry weather flow in 1977 was 32 mgd, with peak-to-average ratio about 1.2:1. Nearly half of the waste flow is of industrial origin.

Facilities consist of conventional headworks, two parallel (essentially identical) trains of primary clarifiers, diffused air activated sludge, and secondary clarifiers; all of which comprise "east" and "west" plants, and effluent rapid sand filters prior to chlorination and discharge. The plant also has a 50 million gallon equalization/emergency storage tank (Figure 60) that is divided into 7 and 43 million gallon compartments, and covered. The tank is currently used for temporary storage of storm flows and influent waste when industrial spills are detected. The tank can be filled by gravity directly from the headworks or following primary sedimentation. Storm flows in excess of 80 mgd are automatically diverted to the tank. The plant is also equipped with continuous automatic cyanide monitoring for industrial spill detection. Flow during spills is diverted, treated, and returned to the plant so as to avoid process upsets. The plant is currently being provided with computerized control of pumping and valving that will permit use of the retention tank to equalize daily variations in plant influent flow when completed.

Influent and effluent TSS and BOD concentration distribution observed for the Warren plant in the calendar year 1976 are shown in Figures 61 and 62. The plant can be seen to provide consistent high quality performance. The somewhat high secondary effluent TSS concentrations are not critical because of the subsequent effluent filtration.

Renton, Washington--

The metropolitan area of Seattle, Washington is served in part by a large activated sludge secondary treatment plant located at Renton, Washington. This plant, from the headworks through to the secondary clarifiers, has many similarities to the Warren, Michigan plant. The Renton plant has a design treatment capacity of 36 mgd, hydraulic capacity of 96 mgd, and pumping capacity of 194 mgd. The plant is designed for treating peak dry weather flows of 72 mgd; flows of this magnitude averaged over 24 hours have been treated successfully in recent months. Typical peak-to-average flow ratios during dry weather are about 1.3 to 1.4:1; average daily flows of 50-60 mgd have been experienced for several days at a time during wet weather. Slightly less than half of the waste loading is of industrial origin.

Facilities at Renton consist of conventional headworks followed by parallel arrangement of primary clarifiers, aeration tanks and secondary clarifiers, similar to facilities at Warren. The activated sludge system is designed to permit operation ranging from plug flow to contact stabilization with a wide range of step aeration, feeding, and solids return options in between. The plant is not equipped with equalization facilities. The influent interceptor, however, is designed for an ultimate hydraulic capacity of 375 mgd, and therefore has excess volume that can be backed up behind the influent pump station during peak wet weather flows. Available volume at current wet weather flow is between 4 and 6 million gallons. This volume is used only during excessive flow conditions because of the problems involved with solids deposition during storage. Very heavy solids discharge occurs as storage volume is reduced to normal flow levels, creating excessive solids loading and oxygen demand conditions.

Influent and effluent TSS and BOD concentration distributions observed for the Renton plant for the calendar year 1976 are presented in Figures 61 and 62, along with data for the Warren plant. The Renton plant provides consistent high quality performance. Comparison of Renton and Warren performance shows that the degree of secondary effluent variability is similar for

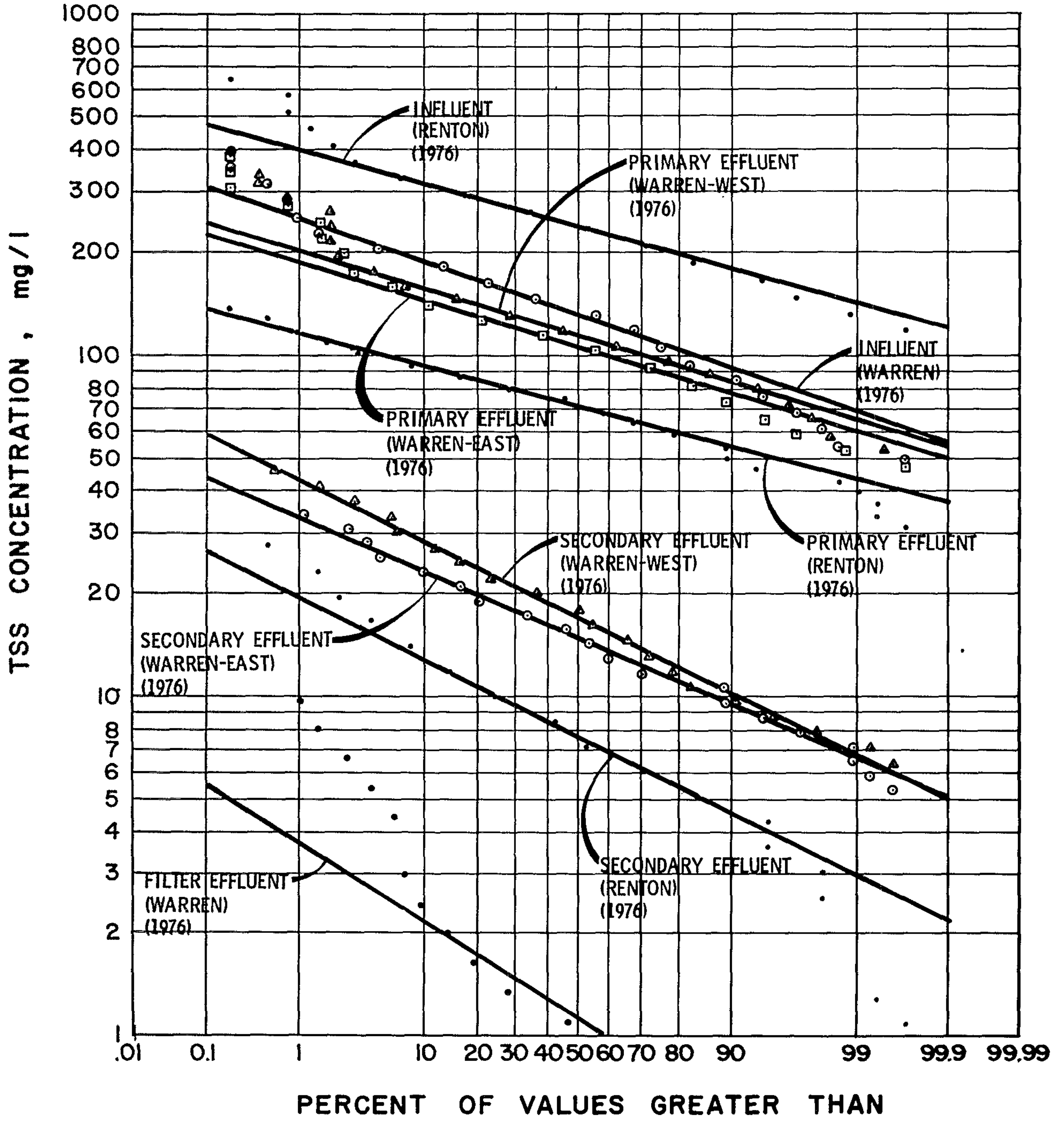

Figure 61. Distribution of TSS concentrations, Warren, Michigan/ Renton, Washington

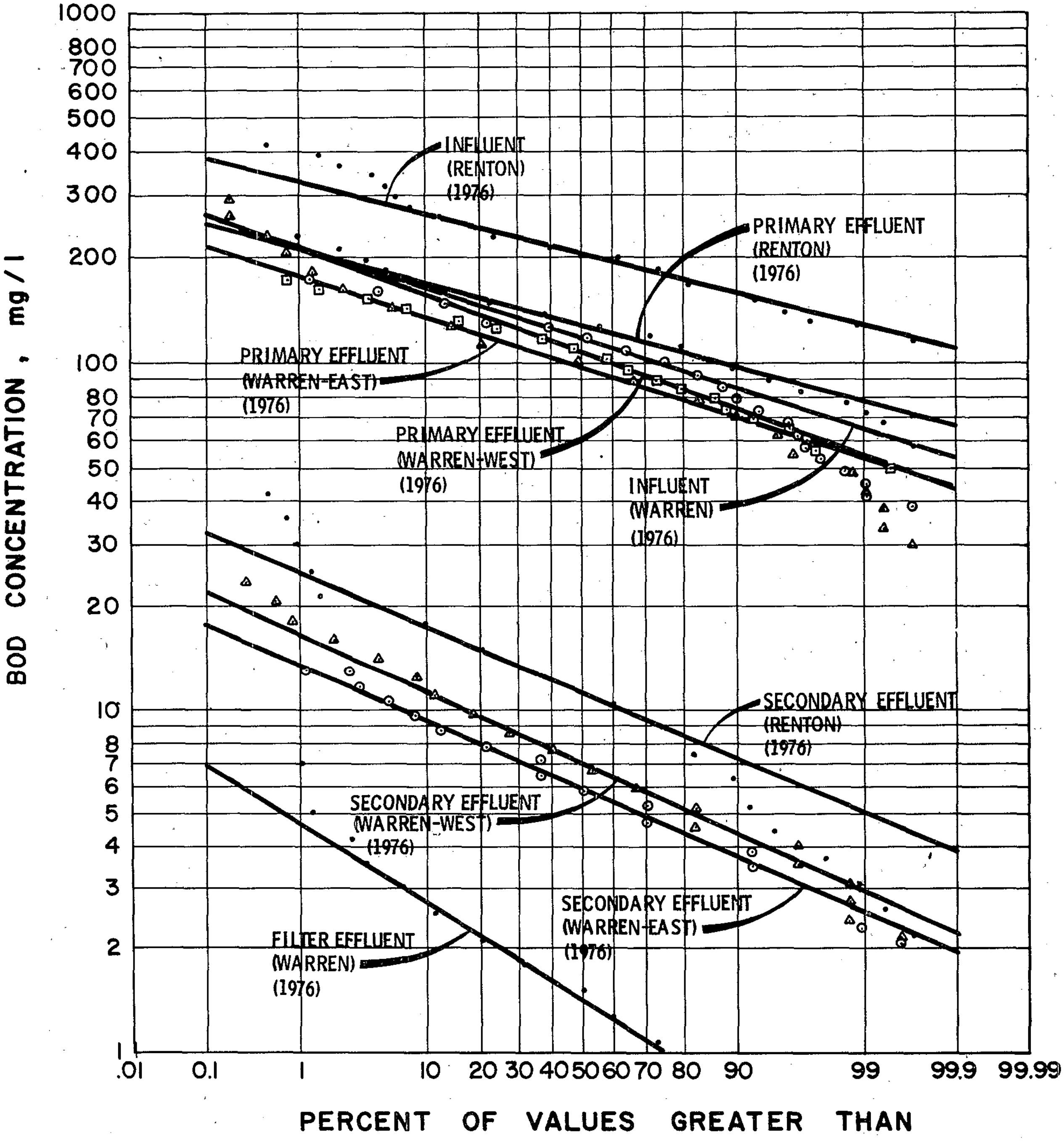

Figure 62. Distribution of BOD_5 concentrations, Warren, Michigan/ Renton, Washington

the two plants. Average effluent TSS at Renton is significantly lower than at Warren; the reverse is true for effluent BOD.

Several factors may contribute to these observed differences. Operation of the Renton plant is focused specifically on maintaining low effluent TSS. This objective is used to establish desirable operating conditions in the activated sludge system, and adjustments are continuously made in order to maintain optimum settling conditions in the secondary clarifiers. The Warren plant, on the other hand, has effluent filters to maintain desired effluent TSS, allowing aeration conditions to be optimized for BOD removal. In addition, the Warren plant uses alum addition to achieve required phosphorus removal. The use of alum undoubtedly contributes to the observed differences in secondary effluent quality.

Newark, New York--

The sewage treatment plant at Newark, New York was used for a full-scale study of simulated flow equalization sponsored by the EPA from March to July, 1971.(30) Newark has a conventional activated sludge plant with an average flow (1971) of approximately 1.8 mgd, with a peak-to-average ratio of about 1.4 under typical dry weather conditions.

Treatment facilities consist of conventional headworks followed by parallel sets of primary clarifiers, aeration tanks, and secondary clarifiers. The plant is not equipped with equalization facilities. For the purposes of the study all plant flow was processed through half of the parallel treatment units, effectively doubling all process loadings. This was done in an effort to generate operating conditions that would result in effluent performance varying with flow rate. Under the high loading study conditions the primary clarifier overflow rate was approximately 1,000 gpd/ft^2 at average flow of 1.8 mgd, or 1,700 gpd/ft^2 at peak flows of 3.0 mgd during unequalized flow conditions. The plant was operated for two consecutive periods: the first with flows unequalized to establish plant performance, with diurnal variation at the elevated flow rates; the second with flows equalized, using the second aeration tank to establish plant performance under essentially constant flow conditions.

Data generated in the study are somewhat controversial, and difficult to interpret clearly. Bulking problems were experienced during the equalized Phase 2 period, so that comparing equalized and unequalized secondary performance is not possible. Evaluation of equalization effects on primary sedimentation using the study data is complicated by several factors: significant increases in <u>all</u> waste constituents, including BOD, COD, TSS, VSS, etc., were observed across the equalization tank; samples taken were not composited proportional to flow; influent flows during the unequalized period were 15 to 20 percent higher than in the equalized period. Due to the limitations of the data, the original investigators relied primarily on the two 2-day equalized and unequalized flow sampling periods, during which bi-hourly sampling was conducted. Conclusions based on these data are that primary sedimentation TSS efficiency was 59 percent during equalized flow, as opposed to 23 percent during unequalized flow. Also the coefficient of the variation of diurnal concentrations was approximately 50 percent less during equalized flow.

Primary sedimentation performance data from the study have been reviewed and interpreted independently. Probability plots of the daily average TSS and BOD data, adjusted by appropriate factors to approximate flow weighted composite concentrations, are presented in Figure 63. Distributions of TSS and BOD removal percentages calculated from the same adjusted data are presented in Figure 64. Distributions of influent and primary effluent BOD concentrations (Figure 63) show that little difference was observed between equalized and unequalized periods. This corresponds to original study conclusions.

Distributions of plant influent and primary effluent TSS concentrations (Figure 63) show that primary effluent concentrations during unequalized flow were consistently below those during the equalized period. However, primary sedimentation performance during the equalized flow period should be determined with respect to primary clarifier influent concentrations (equalization tank effluent concentrations). As shown, TSS levels in the equalization tank were substantially higher than in the plant influent, which resulted in the observed higher removals across the primary tank during equalized flow. The distributions of equalized and unequalized flow TSS removal percentages (Figure 64) show mean TSS removal to be approximately 40 percent for the 6-week equalized flow period, and approximately 35 percent for the corresponding unequalized flow period.

Differences in day-to-day primary effluent variability observed between equalized and unequalized periods can be seen in the TSS distributions of Figure 63. The broader range of daily average primary effluent TSS concentrations observed during the unequalized period appears to result largely from the broader range of influent concentrations.

The conclusion of this analysis is that a modest improvement in primary sedimentation TSS removal was observed during equalized flow conditions. However, differences in influent conditions between the two periods, and the undefined increases in major waste constituents across the equalization tank prevent development of general conclusions with respect to equalization effects on primary clarifier performance.

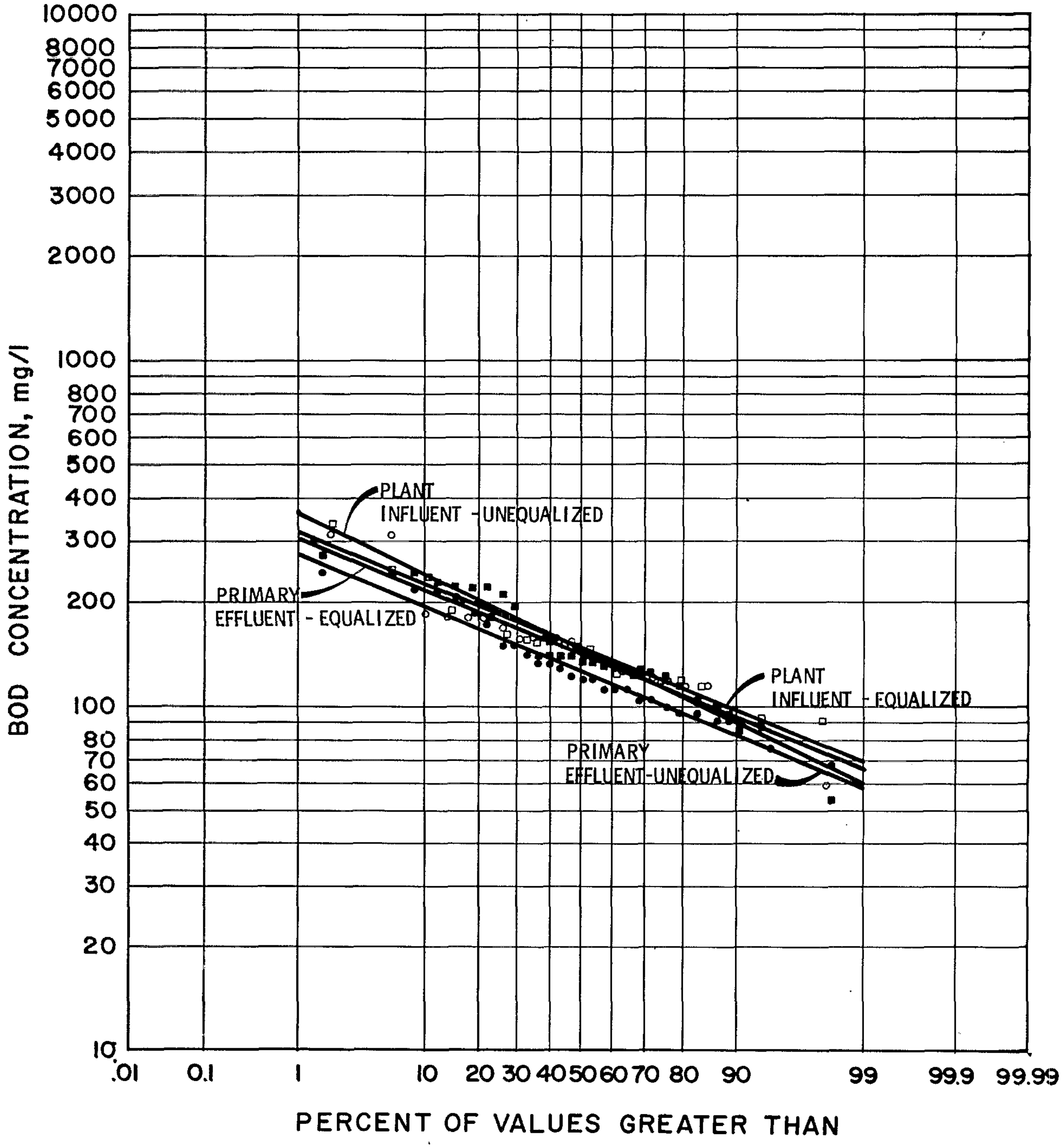

Figure 63(a). Primary effluent BOD concentration distribution, Newark, NY

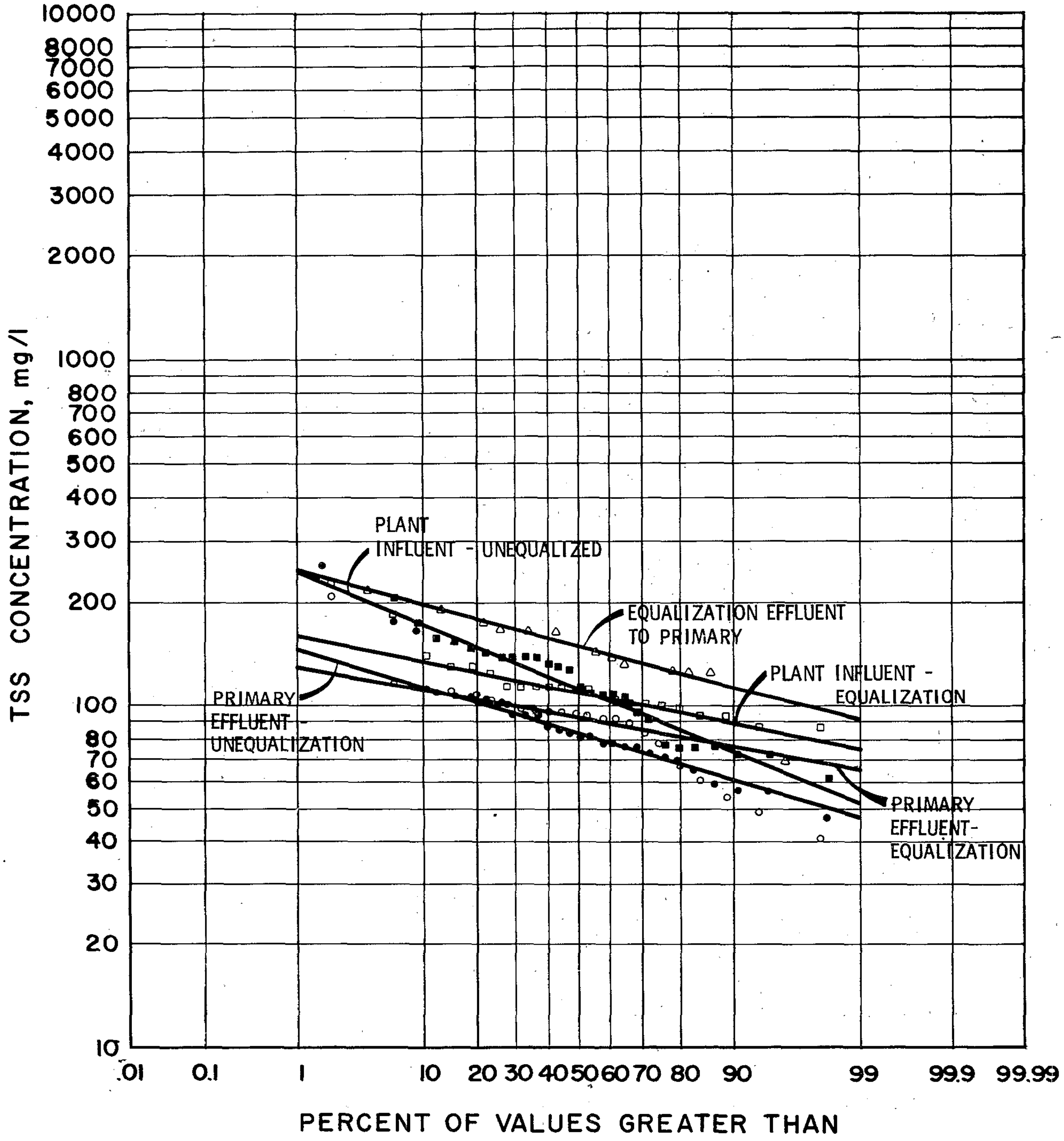

Figure 63(b). Primary effluent TSS concentration distribution, Newark, NY

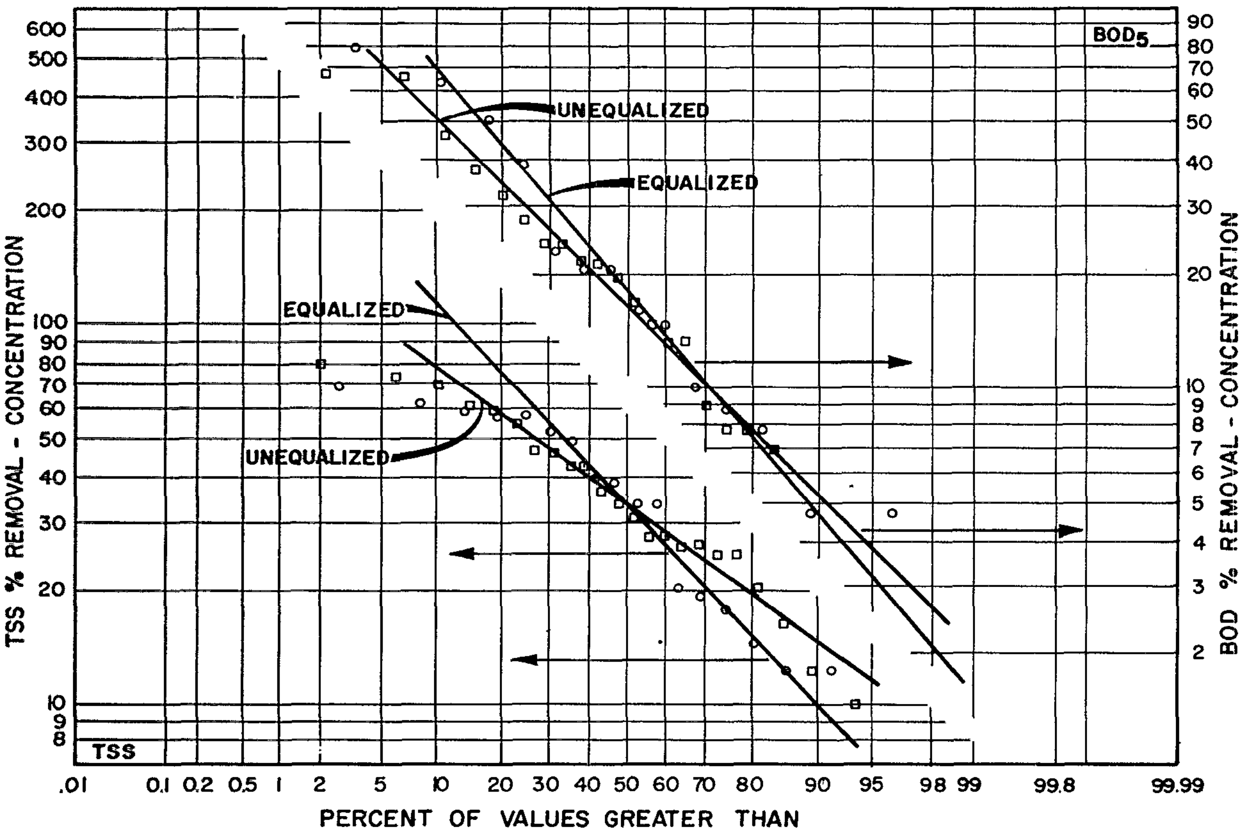

Figure 64. Primary sedimentation removal efficiency distribution, Newark, NY

Trickling Filters

Performance data for a recent year of operation from three trickling filter secondary treatment plants are analyzed in this section. The plant at Palmyra, New Jersey is the only trickling filter plant located during the equalization survey that had equalization added as the only plan modification at the time of construction. The plant at Midland, Michigan is the only additional trickling filter plant with equalization located during the survey for which reasonably complete operating records were obtained. Performance data from the unequalized trickling filter plant at nearby Bay City, Michigan are used to provide comparison to Midland data.

The performance data in this section are presented as available information of this type, but not necessarily representative of performance at typical trickling filter plants.

Palmyra, New Jersey--

The Borough of Palmyra, New Jersey is served by an approximately 30-year old standard rate trickling filter secondary treatment plant with design capacity of 530,000 gallons per day. The facilities consist of an influent pump station followed by an in-line aerated equalization tank, primary clarifiers, standard rate trickling filter, secondary clarifiers, and chlorination prior to discharge.

The equalization tank, with a capacity of 130,000 gallons, was added to the plant in 1975 to accommodate an anticipated flow increase of 200,000 to 250,000 gpd from new tributary residential developments. Current (1977) flows during typical dry weather periods average approximately 0.4 mgd, with peak flows of 0.6 to 0.7 mgd.

Only secondary effluent data are available for analysis. Unequalized data from the calendar year 1974 and equalized data for equalized operation from the year 1976 are summarized in Figures 65 and 66.

The annual distributions of secondary effluent TSS (Figure 65) show that effluent in the equalized year is substantially better than in the unequalized year. The equalized mean effluent TSS (50.6 mg/l) is 35 percent lower than the unequalized mean effluent (78.5 mg/l). Equalized effluent TSS is also significantly less variable. In the equalized year effluent TSS is between 26 and 70 mg/l for 80 percent of the time, compared to a range of 24 and 160 mg/l for 80 percent of the unequalized period.

The annual distributions of secondary effluent BOD (Figure 66) show that equalized BOD performance is better than in the

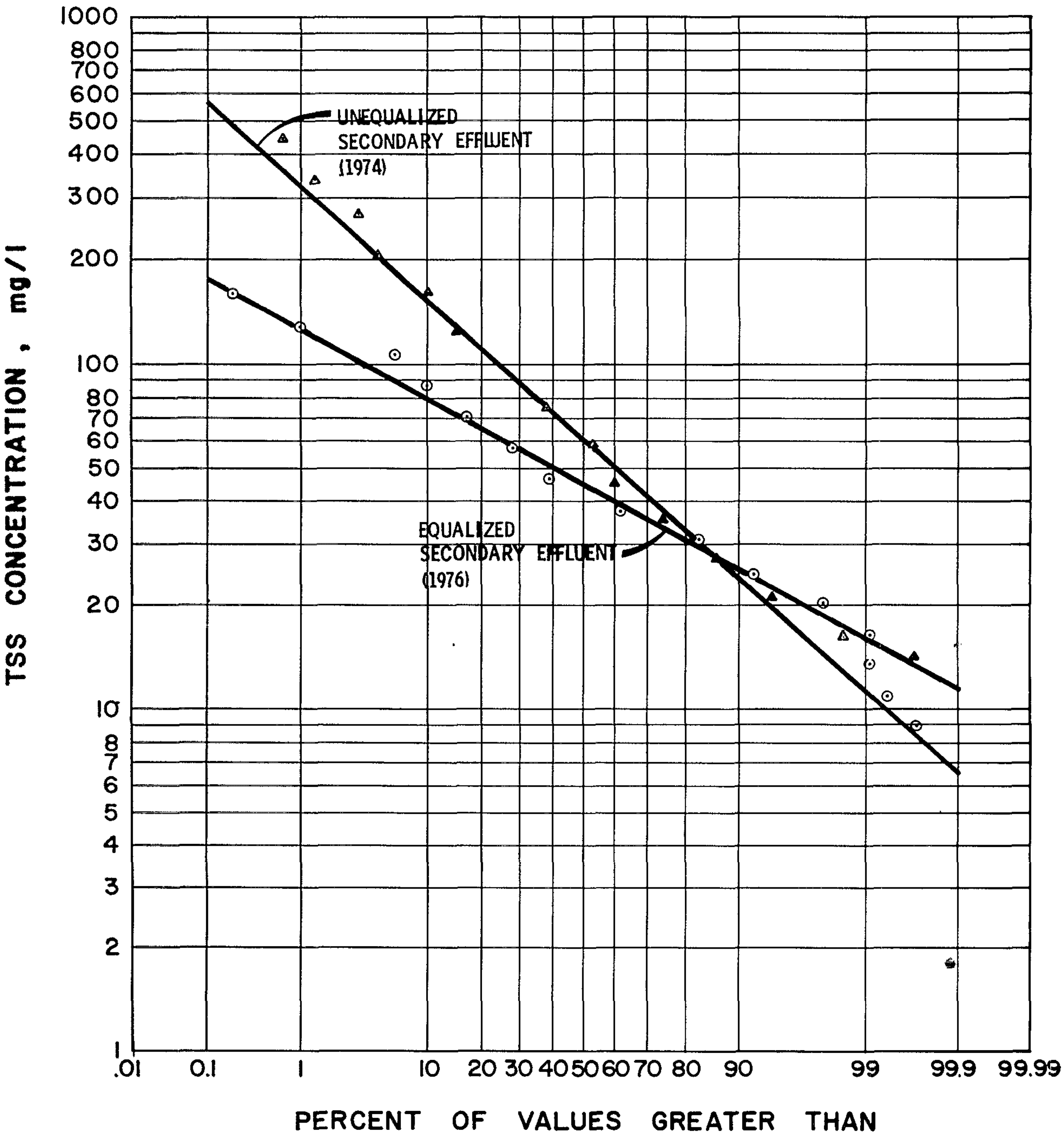

Figure 65. Distribution of TSS concentrations, Palmyra, New Jersey

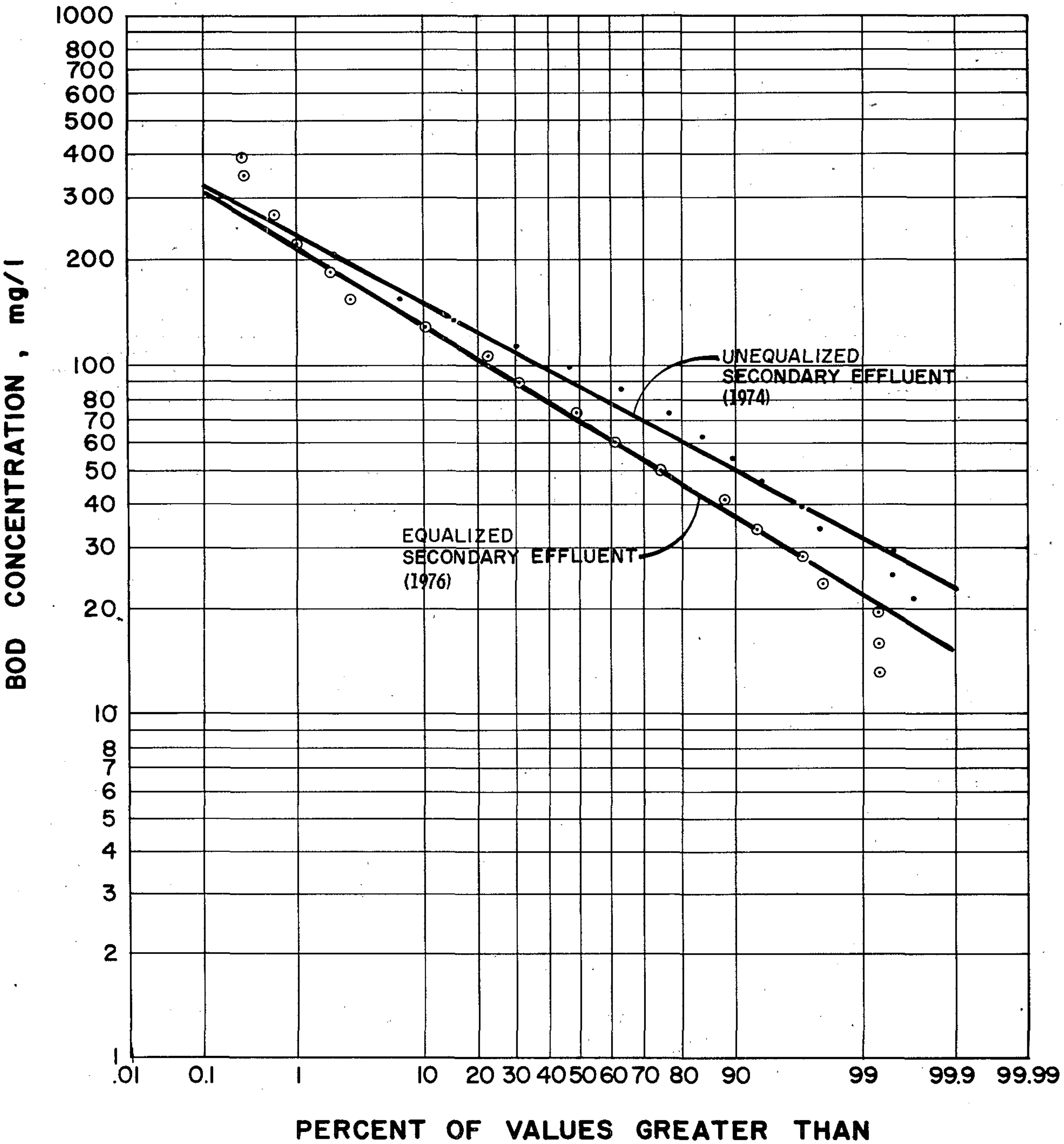

Figure 66. Distribution of BOD_5 concentrations, Palmyra, New Jersey

unequalized year. The equalized mean effluent BOD (76 mg/l) is 18 percent lower than the unequalized mean effluent (93 mg/l). Secondary effluent variability is approximately the same for both periods; with 80 percent of effluent BOD's between 35 mg/l and 130 mg/l in the equalized year, and between 40 and 150 mg/l in the unequalized year.

Equalization appears to have significantly improved treatment performance of the Palmyra facilities.

Midland, Michigan--

Treatment facilities at Midland consist of two-stage, high rate, plastic media trickling filters; primary and intermediate clarification; chemical flocculation before secondary clarification; and effluent filtration. The plant has a treatment capacity of about 6.5 mgd, with hydraulic capacity about 13 mgd. It is equipped with a 3.25 million gallon equalization tank. Daily average dry weather flows are approximately 6 mgd, with peak-to-average flow ratio ranging from 1.5 to 1.65. Average daily flows during wet weather are typically 8 to 8.5 mgd, with peak wet weather flows occurring in the range of 10 to 12.5 mgd. The equalization tank is used to smooth daily diurnal flow variations, and to supplement treatment capacity during wet weather conditions. A summary of operating conditions has been recently reported in the literature.(31)

Equalization was added to the existing secondary treatment plant in 1972. Significant additional plant modifications were made concurrently, including chemical addition for phosphorus removal.

Bay City, Michigan--

Treatment facilities at Bay City are similar to those at Midland, but they have no equalization. Treatment processes include primary and secondary clarification, and single-stage, standard-rate, plastic-media trickling filters. Ferric chloride is added to second stage filter effluent for phosphorus removal. Average daily flows are about 12 mgd, with a typical peak-to-average flow ratio of approximately 1.3.

Distributions of influent and effluent BOD and TSS concentrations for the equalized 6.5 mgd trickling filter plant at Midland, and the unequalized 12 mgd trickling filter plant at Bay City (Figures 67 and 68), are presented primarily as background information. The distributions provide an example of secondary effluent quality observed over one-year periods at the respective plants. The similarity of the influent and effluent distributions may be attributed at least in part to the similarity of physical characteristics of the two plants, and to the similarity of environmental factors. Their geographical proximity contributes to waste variations and influences process performance.

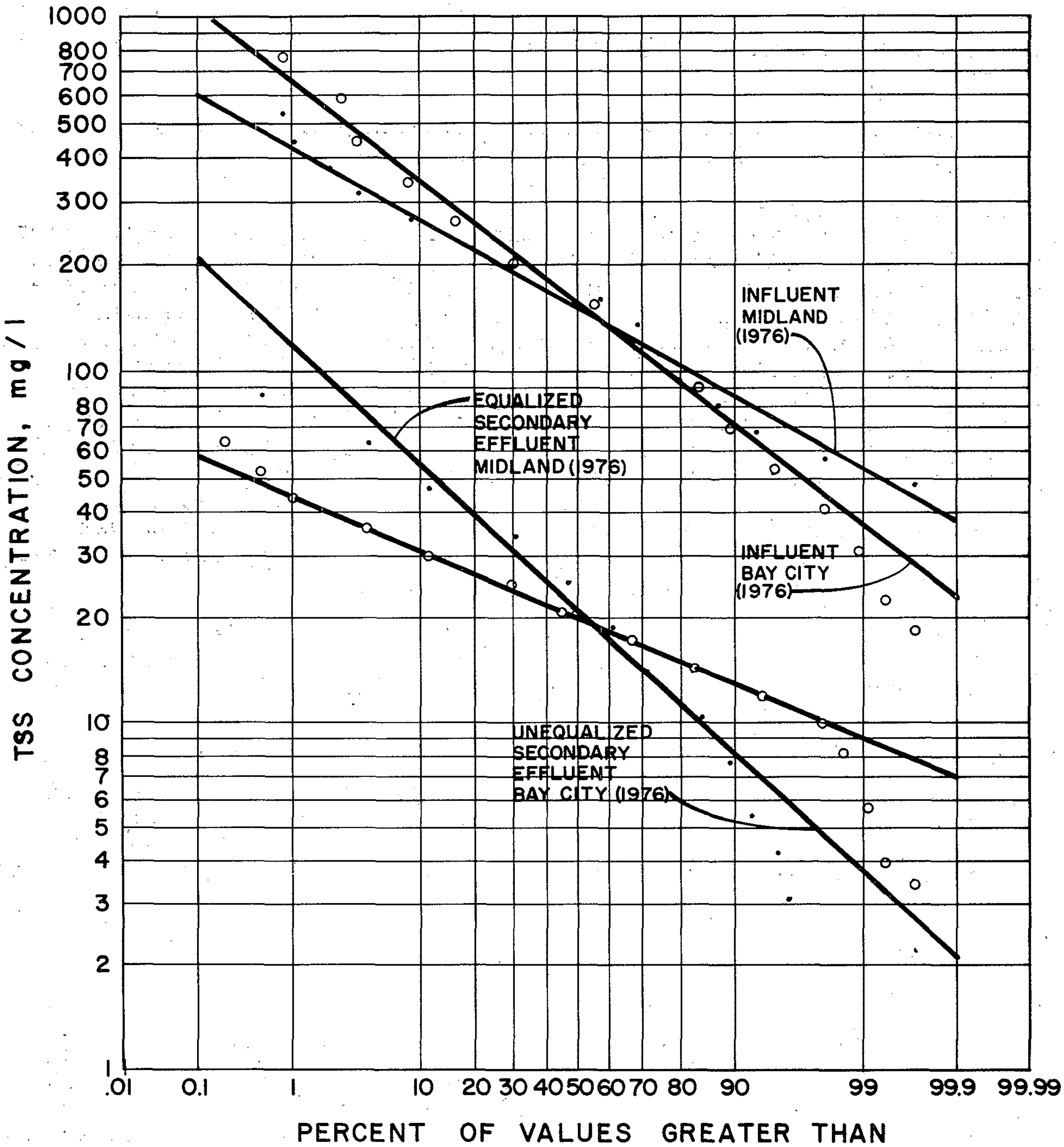

Figure 67. Distribution of daily TSS concentrations, Midland, Michigan/Bay City, Michigan

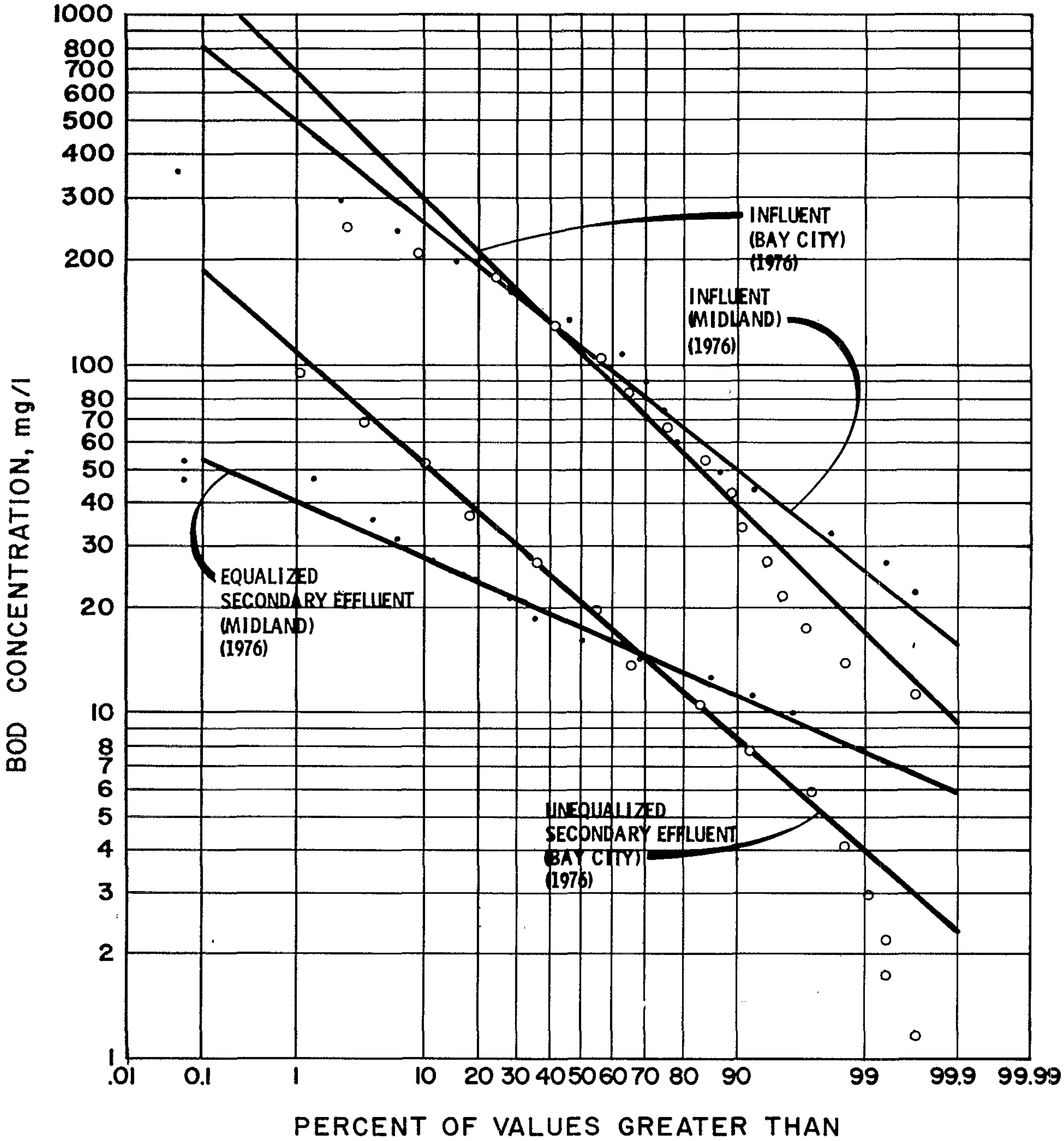

Figure 68. Distribution of daily BOD_5 concentrations, Midland, Michigan/Bay City, Michigan

Oxidation Ditch Plants

Oxidation ditch activated sludge treatment plants have received widespread application recently. Typically, applications in predominantly rural areas are in plants of no greater capacity than 5 mgd. A recent study (32) has surveyed oxidation use, performance and costs, providing detailed reference information. Another EPA sponsored study at Dawson, Minnesota (33) provides information on the operation of an oxidation ditch treatment plant with facilities for flow equalization. Performance data from this study are compared to data from an unequalized plant of similar size at Arlington, Washington.

Dawson, Minnesota--

Dawson, Minnesota is served by a 260,000 gallon per day secondary treatment plant, with an oxidation ditch and activated sludge facilities. The plant is designed to meet effluent requirements of 5 mg/l TSS and BOD, and 0.1 mg/l ammonia nitrogen.

Following screening, raw wastewater is pumped directly to the "oxidation ditch" aeration channel. The aeration channel is designed to provide 83,000 gallons of equalized storage by varying the mixed liquor depth between a low level of 3 feet and a high level of 4 feet. At design flows the mixed liquor depths will correspond to aeration times of 17.7 and 25.4 hours. Use of storage volume during initial operation is reported to have reduced peak flows by approximately 31 percent.(34) Flows averaged approximately 160,000 gallons per day. Following the oxidation ditch, the Dawson plant has two secondary clarifiers in series. Design overflow rates are 580 and 400 gpd/ft^2, respectively. The plant has facilities for chemical addition and flocculation between two clarifiers to insure desired solids removal. Chemical addition has not been necessary during the initial operation.

Performance data for the Dawson plant developed in an EPA sponsored study (33) are summarized in Figures 69 and 70. Distributions of influent and final secondary clarifier effluent TSS (Figure 60) indicate that the average performance is good, but that a relatively high degree of effluent variability is observed. Distribution of influent and secondary clarifier effluent BOD (Figure 70) indicates excellent BOD performance, and stable overall performance.

Arlington, Washington--

Arlington, Washington is served by an unequalized oxidation ditch, activated sludge, secondary treatment plant. Design capacity is 1.5 mgd; current flows (1977) average approximately 400,000 gpd. The Arlington facilities consist of conventional headworks followed by the oxidation ditch aeration channel and secondary clarifier.

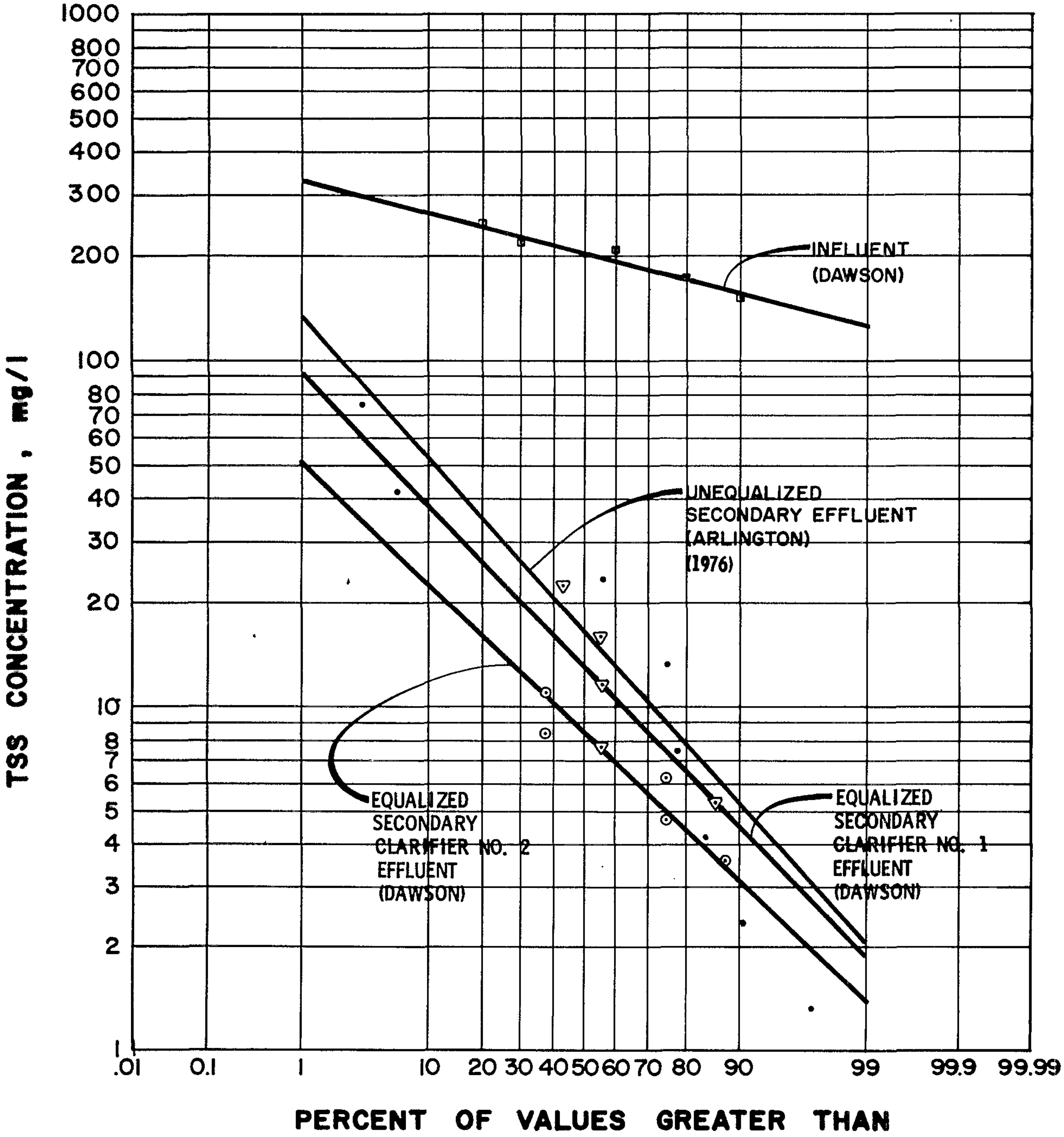

Figure 69. Distribution of monthly TSS concentrations, Arlington, Washington/Dawson, Minnesota

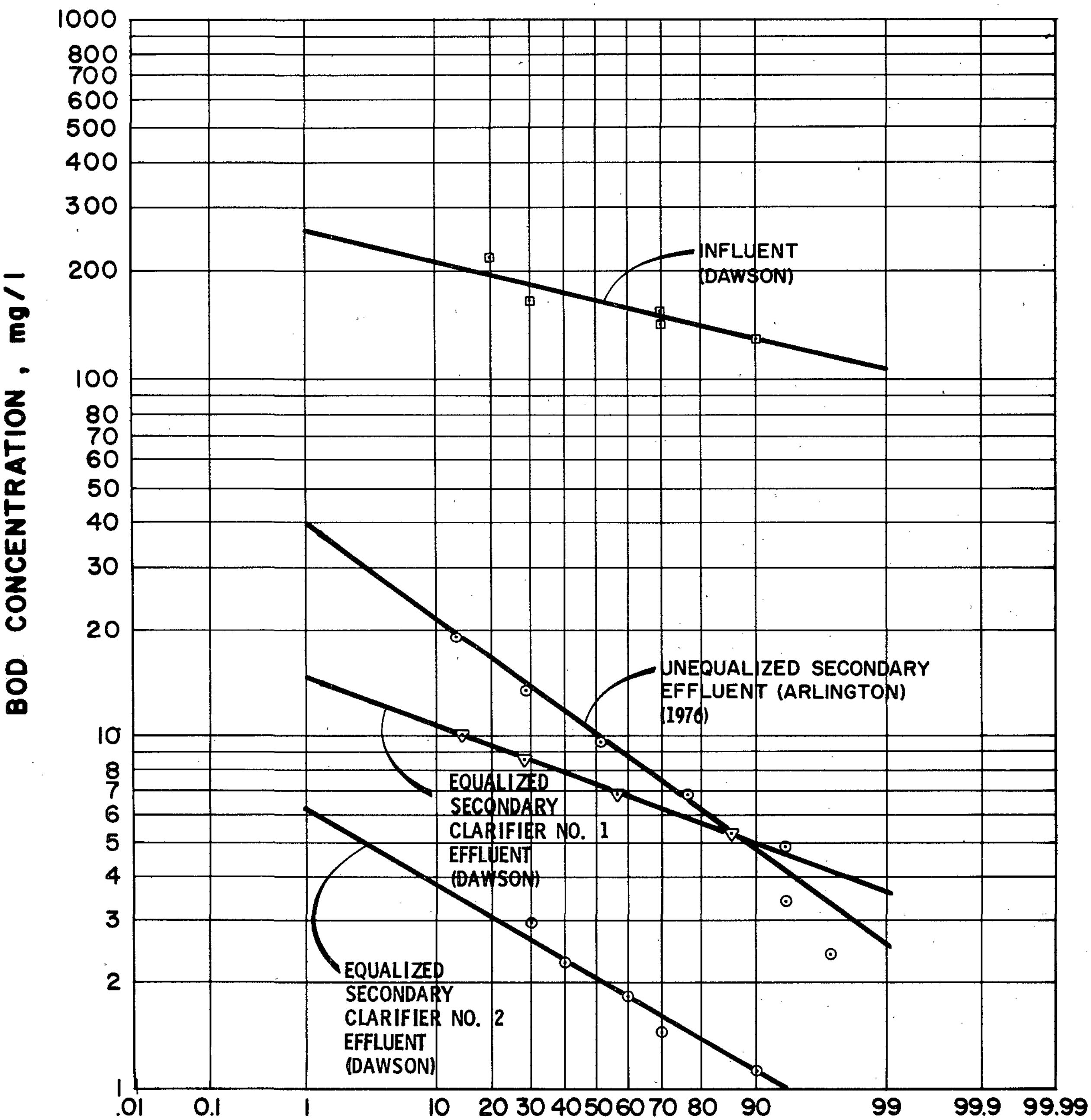

Figure 70. Distribution of monthly BOD_5 concentrations, Arlington, Washington/Dawson, Minnesota

Annual distributions of secondary effluent TSS and BOD concentrations for 1976 are summarized in Figures 60 and 70, along with corresponding distributions for Dawson, Minnesota. The distribution of effluent TSS was similar to that observed for Dawson, with an annual mean of 19.1 mg/l and variability slightly higher than at Dawson. The observed mean effluent BOD (Figure 70) is approximately 9 mg/l, again with the level of variability typical of other activated sludge plants although somewhat higher than the high quality Dawson effluent.

General Performance Observations

Inspection of influent and effluent distributions (Figures 21 through 70) reveals some general performance patterns.

Primary Sedimentation--

At the primary treatment level, mean BOD and TSS concentrations and overall removals were better under equalized than unequalized conditions. As with secondary effluent, the pattern of change in effluent variability was not consistent. Slightly less variability was observed in equalized primary effluent concentrations in 1973; but in 1976 slightly more variability occurred in the equalized primary effluent.

Secondary Processes--

Performance of the five full-scale activated sludge plants was better almost across-the-board under equalized conditions (Table 19). Greater differences were observed in BOD than in TSS performance under equalized and unequalized conditions. Also, differences in equalized and unequalized performances tended to be greater for the smaller plants. Performance variability (Table 20), on the other hand, showed no consistent pattern. A slight overall tendency was indicated toward less variability of effluent BOD under equalized conditions. And a similarly slight overall tendency toward greater variability of effluent TSS was apparent under equalized conditions. The relatively slight and inconsistent correspondence between influent variability under equalized conditions and variability of suspended solids concentrations in secondary clarifier effluent is not surprising. But as described in detail in Appendix B, the influence of the biological process directly preceding the secondary clarifier is far greater than the influence of plant influent. Although typical plant influent TSS concentrations may range from 200 to 300 mg/l, secondary clarifier influent TSS concentrations range from 1,500 to 4,000 mg/l. In addition, settling characteristics are strongly influenced by conditions in the biological process preceding the clarifier.

Effluent Filtration--

Mean effluent BOD and TSS concentrations of filtered secondary effluent at Walled Lake-Novi were significantly better

TABLE 19. EFFECTS OF EQUALIZATION ON PERFORMANCE OF ACTIVATED SLUDGE PLANTS

Plant name and study period	Secondary effluent, mean concentration (mg/l)		
	Unequalized	Equalized	% Change
Tecumseh, 70/73			
BOD	28.4	7.7	-73
TSS	12.8	9.1	-29
Tecumseh, 70/76			
BOD	28.4	11.9	-58
TSS	12.8	14.4	+13
Walled Lake-Novi, 74			
BOD	13.7	6.8	-50
TSS	21.5	9.5	-56
Walled Lake-Novi, 74/76-77			
BOD	14.2	11.1	-22
TSS	9.5	13.6	+43
Ypsilanti Township, 74/75			
BOD	24.7	17.6	-29
TSS	17.0	18.0	+6
Ypsilanti Township, 76			
BOD	22.8	15.0	-34
TSS	17.0	15.1	-11
Amarillo			
BOD	34.4	26.7	-22
TSS	17.9	17.8	--
Pontiac, 73/76-77			
BOD	12.3	12.0	--
TSS	13.6	10.1	-26
Average change, BOD	--	--	-36
Average change, TSS	--	--	-8

TABLE 20. EFFECTS OF EQUALIZATION ON VARIABILITY OF ACTIVATED SLUDGE PLANTS

Plant name and study period	Secondary effluent, $\log_{10}$ standard deviation		
	Unequalized	Equalized	% Change
Tecumseh, 70/73			
BOD	0.43	0.30	-30
TSS	0.36	.31	-14
Tecumseh, 70/76			
BOD	.43	.56	+30
TSS	.36	.32	-11
Walled Lake-Novi, 74			
BOD	.10	.06	-40
TSS	.18	.28	+56
Walled Lake-Novi, 74/76-77			
BOD	.83	.33	-60
TSS	.97	.25	-74
Ypsilanti Township, 74-75			
BOD	.17	.20	+18
TSS	.33	.24	-27
Ypsilanti Township, 76			
BOD	.18	.22	+22
TSS	.28	.28	--
Amarillo			
BOD	.26	.20	-23
TSS	.17	.21	+24
Pontiac, 73/76-77			
BOD	.16	.18	+13
TSS	.25	.25	--
Average change, BOD	--	--	-9
Average change, TSS	--	--	-6

under equalized conditions. The variability of filter effluent BOD concentrations was substantially greater under equalized than unequalized conditions. In fact, during both equalized and unequalized periods filter effluent variability was greater than the clarifier effluent feed. This result may perhaps be due to biological activity in the filters. The variability of filter effluent TSS concentrations was less under equalized conditions.

Secondary effluent and final filter effluent distributions for equalized flow plants are summarized in Figures 71 and 72.

As the peak-to-average flow ratio was increased, the mean effluent BOD and TSS concentrations of the EPA MERL pilot study of activated sludge systems under constant flow conditions was observed to improve compared to the variable flow systems. No consistent pattern of influence on effluent variability was observed.

EQUALIZED VERSUS UNEQUALIZED PLANT PERFORMANCE

Average annual performances of equalized and unequalized sewage treatment plants are compared in this section. Only a small portion of equalized plants could be analyzed directly, assessing the influence of equalization on individual plant performance. However, comparison of average annual performance of equalized and unequalized plants, at comparable levels of treatment, provides an alternative means of assessing the effectiveness of equalization. Performance data for 43 flow equalized facilities are compared to data for unequalized periods at 16 equalized flow plants and 35 plants with no equalization.

Activated Sludge Plants

Average annual influent and effluent BOD and TSS concentrations for 31 equalized and 46 unequalized activated sludge plants are summarized in Table 21 and Figures 73 and 74. Differences in mean annual effluent concentrations between the equalized and unequalized plants in the sample are insignificant. At the primary treatment level, the range of BOD and TSS performance for unequalized plants is actually somewhat better than for plants with equalization prior to primary treatment. At the secondary treatment level the case is just the opposite. In both cases it must be recognized that the broad range of design, operating, and environmental factors contributing to performance characteristics at each treatment plant may well have more significance than would flow equalization. Accordingly, generalizations concerning the effects of equalization on plant performance using the data in this section should be made with caution.

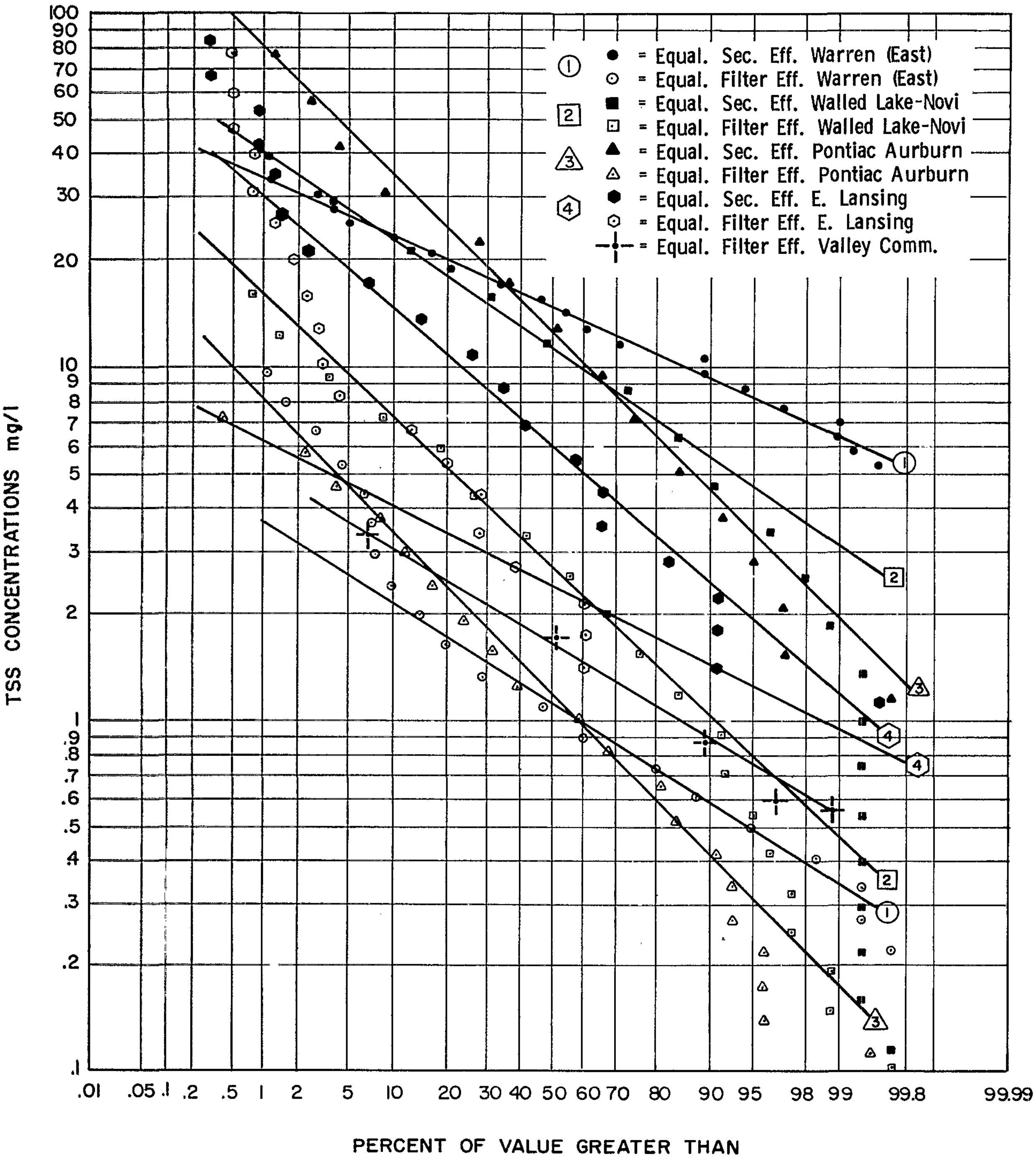

Figure 71. Secondary and filter effluent TSS concentration distributions, flour flow equalized plants.

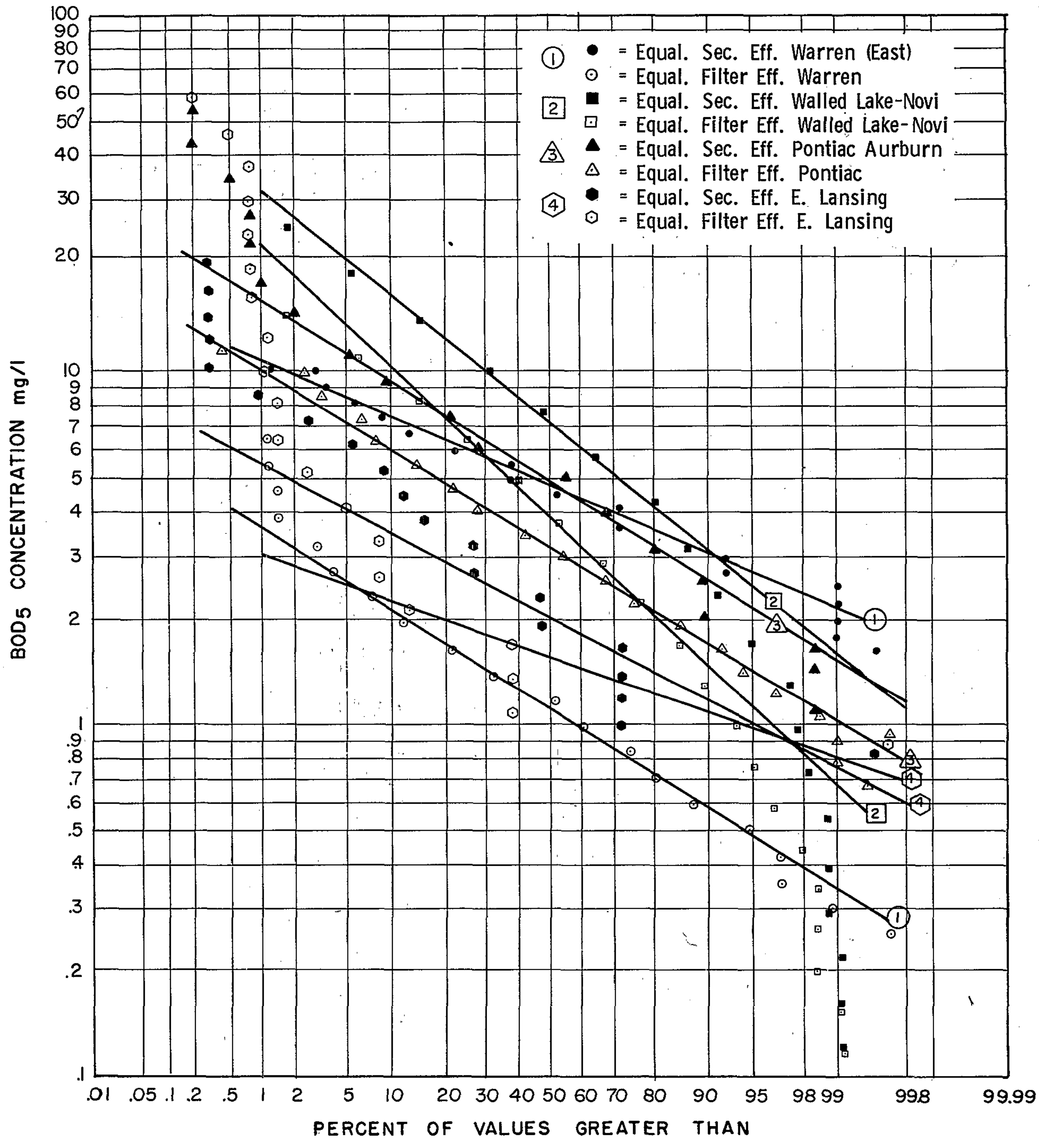

Figure 72. Secondary and filter effluent BOD concentration distributions, four flow equalized plants

TABLE 21. ACTIVATED SLUDGE PLANT PERFORMANCE

Plant	Influent					Primary Effluent					
		BOD_5		TSS		BOD_5			TSS		
	Freq of Data[a]	Conc mg/l	# of Data Points	Conc mg/l	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
<u>Equalized</u>											
Rossmoor, CA	M	213	9	227	9						
Valley Community Services District, CA	D	298	419	297	422	163	45	420	93	69	422
Bloomsfield, CO	D	202	100	175	100						
Freeport, IL	W					120		8	136		8
East Lansing, MI (north side)	D	142	365	117	365	73	49	324	93	21	324
East Lansing, MI (south side)	D	142	365	117	365	70	51	333	94	20	333
Grand Rapids, MI	D	114	335	118	355	90	21	342	63	47	357
Jackson, MI	D	91	364	149	360	97	--	364	147	1	362
Lansing, MI	D	162	366	311	366	125	23	366	187	40	366
Pontiac, MI (Auburn Plant)		80	12	122	12	49		12	55		12
Pontiac, MI (E. Blvd. Plant)		82	12	96	12	48		12	96		12
Port Huron, MI	D	53	338	85	366	73	--	322	119	--	365
Tecumseh, MI (1973)	D	303	207	562	187	129	57	176	138	75	192
Tecumseh, MI (1976)	D	273	241	479	241	135	51	231	118	75	231
Trenton, MI	D	318	366	417	366	279	12	366	246	41	366
Walled Lake/Novi, MI (1974)	D	263	8	258	8						
Walled Lake/Novi, MI (1976)	D	165	328	185	303						
Warren, MI	D	113	364	129	366	100	12	364	109	16	363
Ypsilanti Township, MI (1974-1975)	D	200	346	170	357						
Ypsilanti Township, MI (1976)	D	189	363	208	362						
Clementon, NJ	M										
Marlton, NJ (Elmwood STP #2)	D										

(continued)

TABLE 21 (continued)

Plant		Influent				Primary Effluent					
		BOD_5		TSS		BOD_5			TSS		
	Freq of Data[a]	Conc mg/l	# of Data Points	Conc mg/l	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
Equalized											
Ramblewood, NJ	D										
Woodstream, NJ	W										
Dept. of Environmental Conservation, NY	M										
Fishkill, NY		151	12	176	12						
Newark, NY		212	23	142	23	176	17	24	97	32	23
Wappinger, NY		150	40	162	40	92		40	55		40
Hatfield Township, PA	M	101	21	169	19	87	14	20	88	48	17
Amarillo, TX	M	263	15	293	15						
Austin, TX	M										
Odessa, TX	M										

[a]M = monthly
D = Daily
W = weekly

(continued)

TABLE 21 (continued)

	Secondary Effluent						Filter Effluent					
	BOD_5			TSS			BOD_5			TSS		
Plant	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
<u>Equalized</u>												
Rossmoor, CA	13	94	9	11	95	9						
Valley Community Services District, CA	10	97	76	11	96	140	3.2	99	400	2.2	99	422
Bloomsfield, CO	38	81	101	32	82	100						
Freeport, IL	20	83	8	26	81	8						
East Lansing, MI (north side)	3	98	324	7	94	324	2	99	331	4	91	333
East Lansing, MI (south side)	5	96	364	11	91	364	2	99	331	4	91	333
Grand Rapids, MI	19	83	348	22	81	362						
Jackson, MI	4	96	366	9	94	366						
Lansing, MI	15	91	366	28	91	366						
Pontiac, MI (Auburn Plant)	6		12	9		12	4.4		12	1.4		12
Pontiac, MI (E. Blvd. Plant)	11		12	10		12						
Port Huron, MI	10	81	328	11	87	364						
Tecumseh, MI (1973)	8	97	212	9	98	189						
Tecumseh, MI (1976)	12	96	236	14	97	241						
Trenton, MI	18	94	366	33	92	366						
Walled Lake/Novi, MI (1974)	10	96	8	18	9	8	4.3	98	8	4.2	98	8
Walled Lake/Novi, MI (1976)	12	93	358	12	94	359	4	98	327	3	98	326
Warren, MI	7	94	361	16	88	361	1.6	99	356	1.4	99	357
Ypsilanti Township, MI (1974-1975)	17	92	335	19	89	371						
Ypsilanti Township, MI (1976)							14	93	359	14	93	358
Clementon, NJ	20		12	19		12						
Marlton, NJ (Elmwood STP #2)	10		64	23		126						

(continued)

TABLE 21 (continued)

	Secondary Effluent						Filter Effluent					
	BOD_5			TSS			BOD_5			TSS		
Plant	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
<u>Equalized</u>												
Ramblewood, NJ	22		87	80		97						
Woodstream, NJ	11		41	42		66						
Dept. of Environmental Conservation, NY	44	4		58	4							
Fishkill, NY							1.6		12	1.4		12
Newark, NY												
Wappinger, NY	12		40	8		40	12		40	8		40
Hatfield Township, PA	24	76	20	56	67	20	8.0	92	20	8.7	95	21
Amarillo, TX	26	90	15	17	94	15						
Austin, TX	11		12	13		12						
Odessa, Tx	13		12	32		12						

(continued)

TABLE 21 (continued)

Plant	Secondary Effluent						Filter Effluent					
	BOD_5			TSS			BOD_5			TSS		
	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
Unequalized												
Lodi, CA	15		365	21		365						
District 26, L.A., CA				12		365						
District 32, L.A., CA (1973)	7		365	8		365						
District 32, L.A., CA (1976)	3		365	2		365						
Hyperion, L.A., CA	8		365	6		365						
Long Beach, L.A., CA (1973)	5		365	9		365						
Long Beach, L.A., CA (1976)	8		365	5		365						
Los Coyotes, L.A., CA (1973)	9		365	12		365						
Los Coyotes, L.A., CA (1976)	8		365	14		365						
Pomona, L.A., CA	11		365	7		365						
San Jose Creek, L.A., CA (1973)	9		365	8		365						
San Jose Creek, L.A., CA (1976)	4		365	2		365						
Valley Selling Basin, L.A., CA	10		365	23		365						
Whittier Narrows, L.A., CA				12		365						
Palo Alto, CA	18		365	24		365						
Rossmoor, CA	48		24	53		26						
Sacramento, CA	10		365	15		365						
San Jose-Santa Clara, CA	26		365	33		365						
Calumet, Chicago, IL	19		365	18		365						
Hanover Park, Chicago, IL	10		365	10		365						
Hazelcrest, Chicago, IL	16		365	23		365						

(continued)

TABLE 21 (continued)

Plant	Secondary Effluent						Filter Effluent					
	BOD_5			TSS			BOD_5			TSS		
	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
<u>Unequalized</u>												
Northside, Chicago, IL	13		365	16		365						
Indianapolis #1, IN	28		365	25		365						
Indianapolis #2, IN	17		365	12		365						
Pontiac, MI (Auburn Plant)	9	89	24	20	85	24						
Pontiac, MI (E. Blvd. Plant)	12	86	20	14	87	20						
Port Huron, MI	10		365	11		365						
Tecumseh, MI	28	91	62	13	97	60						
Walled Lake/Novi, MI	14	94	7	9.5	96	7	7.0	97	7	6.9	97	7
Ypsilanti Township, MI (1974)	24		12	19		12						
Ypsilanti Township, MI (1976)	25			17								
Lincoln, NB	66		365	81		365						
Newark, NJ												
Philadelphia, PA	63		365	77		365						
Amarillo, TX	32	85	11	17	94	11						
Leon Creek, San Antonio, TX	9		365	17		365						
Rolling Road, San Antonio, TX	11		365	11		365						
Salado Creek, San Antonio, TX	6		365	12		365						
Pullman, WA (1972)	28		36	56		36						
Pullman, WA (1974)	45		28	52		32						
Pullman, WA (1975)	25		32	4		32						
Renton, WA (1974)	5		365	7		365						
Renton, WA (1976)	11		12	8		12						
East Milwaukee, WI	16		365	15		365						
West Milwaukee, WI	14		365	21		365						
Winnipeg-Manitoba, Canada	31		365	26		365						

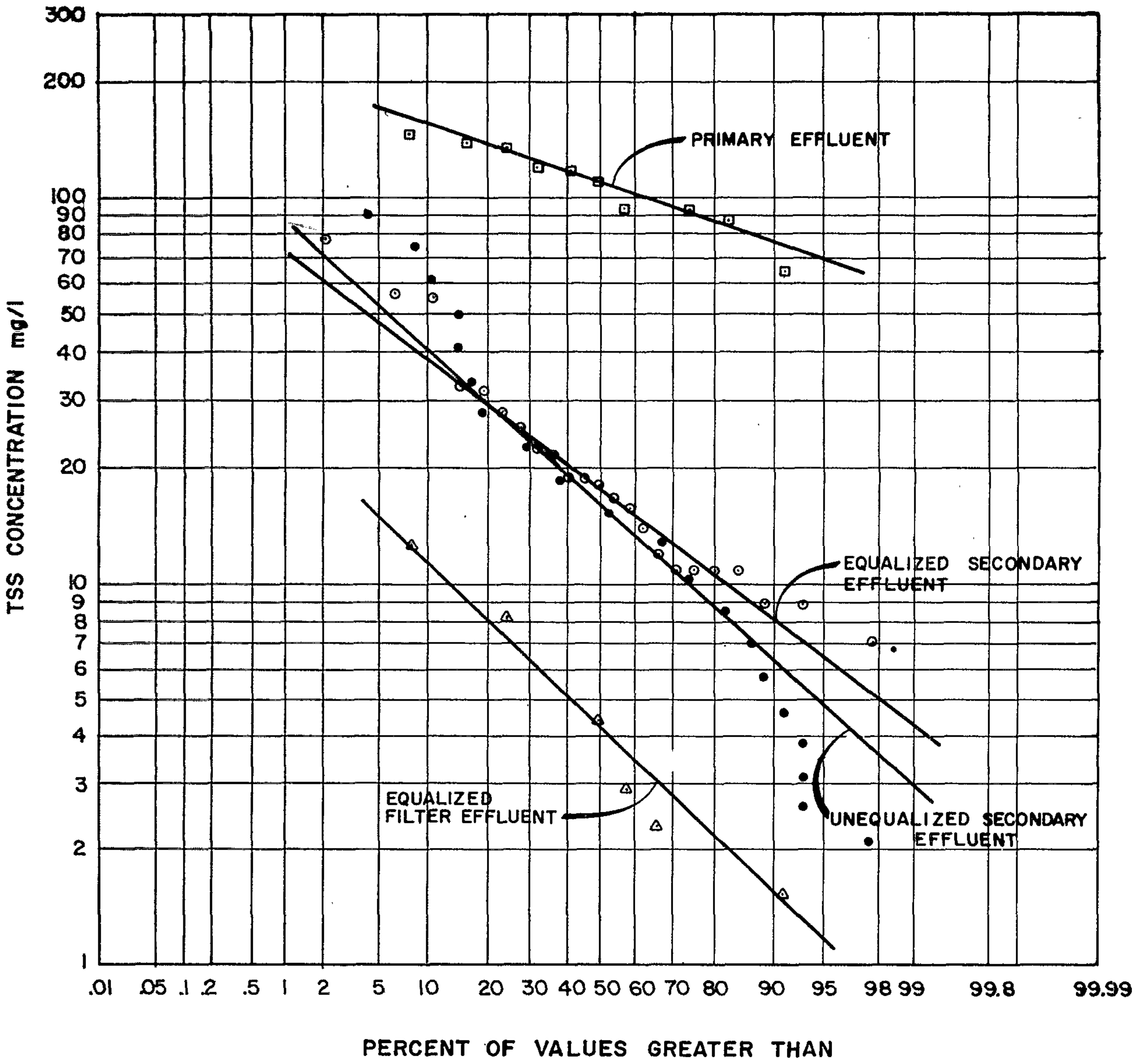

Figure 73. Distributions of average annual effluent TSS concentrations, 31 equalized versus 46 unequalized activated sludge plants

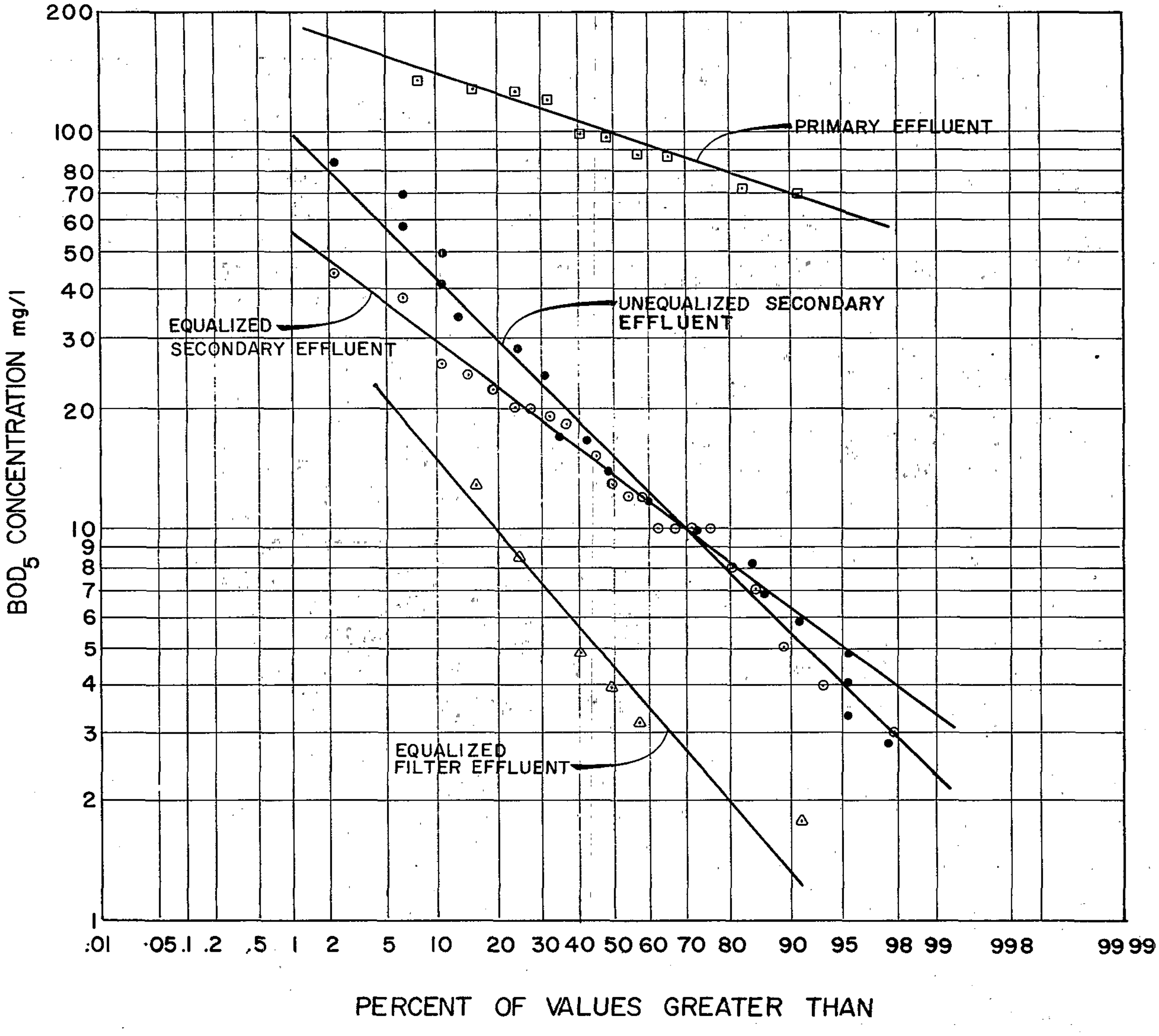

Figure 74. Distributions of average annual effluent BOD concentrations, 31 equalized versus 46 unequalized activated sludge plants

Trickling Filter Plants

Average annual influent and effluent BOD and TSS concentrations for 8 trickling filter plants with equalization, and 65 unequalized trickling filter plants, are summarized in Table 22 and Figures 75 and 76. Data on unequalized plant performance was obtained from an EPA MERL report, "Upgrading Trickling Filters", 430/9-78-004, June 1978.

Comparison of secondary effluent distributions for the two groups shows that the range of effluent BOD and TSS concentrations is about the same. The differences between mean values and slopes of the distribution appear to be a result of having such a small sample of equalized plants.

EFFECTS OF EQUALIZATION ON UNIT PROCESS AND TREATMENT SYSTEM PERFORMANCE

The theoretical benefits of flow equalization vary depending upon whether the plant is proposed (new) or existing. Flow equalization at a new plant is advantageous in that unit processes can be designed for constant, average rates of flow and damped variations in input organics. Cost/benefit ratios for such a situation are analyzed in Section 5.

For existing plants where treatment performance is to be improved or treatment capacity increased, analyzing flow equalization is more complicated. Factors such as the cost and efficiency of correcting "bottle-necks" (i.e., insufficient aeration capacity, limitation in hydraulics, etc.), design deficiencies (i.e., poor clarifier hydraulics, poor sludge settleability due to excessive floc shear in aeration, etc.), and site conditions (land limitation) all play a role in the selection of the preferred alternative remedy for a given problem.

Table 23 summarizes a number of performance problems, and alternative solutions for each situation. The purpose of Table 23 is to illustrate that flow equalization is <u>one</u> possible remedy for a number of treatment plant performance problems. This is not meant to be an exhaustive index of alternatives, but an indication of other potential solutions that must be considered. Most often routine monitoring data required for monthly NPDES compliance reports will not be sufficient to fully define the nature of a given problem. For example, if the effluent from an activated sludge plant is high in TSS (Table 23, Symptom II), it is first necessary to identify <u>why</u> the removal of suspended material is not adequate. Is the system hydraulically overloaded at peak flows? Are the sludge particles heavy and well flocculated, or are they light and diffuse? Is there some other difficulty? Details of this type must be provided before benefits of equalization can be assessed.

TABLE 22. TRICKLING FILTER PLANT PERFORMANCE

Plant		Influent				Primary Effluent						Secondary Effluent					
		BOD_5		TSS		BOD_5			TSS			BOD_5			TSS		
	Freq of Data[a]	Conc mg/l	# of Data Points	Conc mg/l	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points	Conc mg/l	% Rem	# of Data Points
Equalized																	
Essexville, MI	D	78	242	95	253	30	68	241	26	73	253	9	88	245	10	89	253
Mapleshade, MI	W											77		40	115		49
Midland, MI	D	136	365	167	359	72	47	345	45	73	332	19	86	332	21	87	329
Marlton, NJ (Elmwood STP #1)												12		70	32		134
New Providence, NJ	D											8		363	12		362
Palmyra, NY	D											82		366	50		366
Ingleside, TX	M											6		12	30		12
Midland, TX	M											22		12	24		12
Unequalized																	
Bay City, MI	D	148	351	183	355	105	29	341	244		355	19	87	353	27	85	355
Palmyra, NJ												95		20	99		20

[a]D = daily
W = weekly
M = monthly

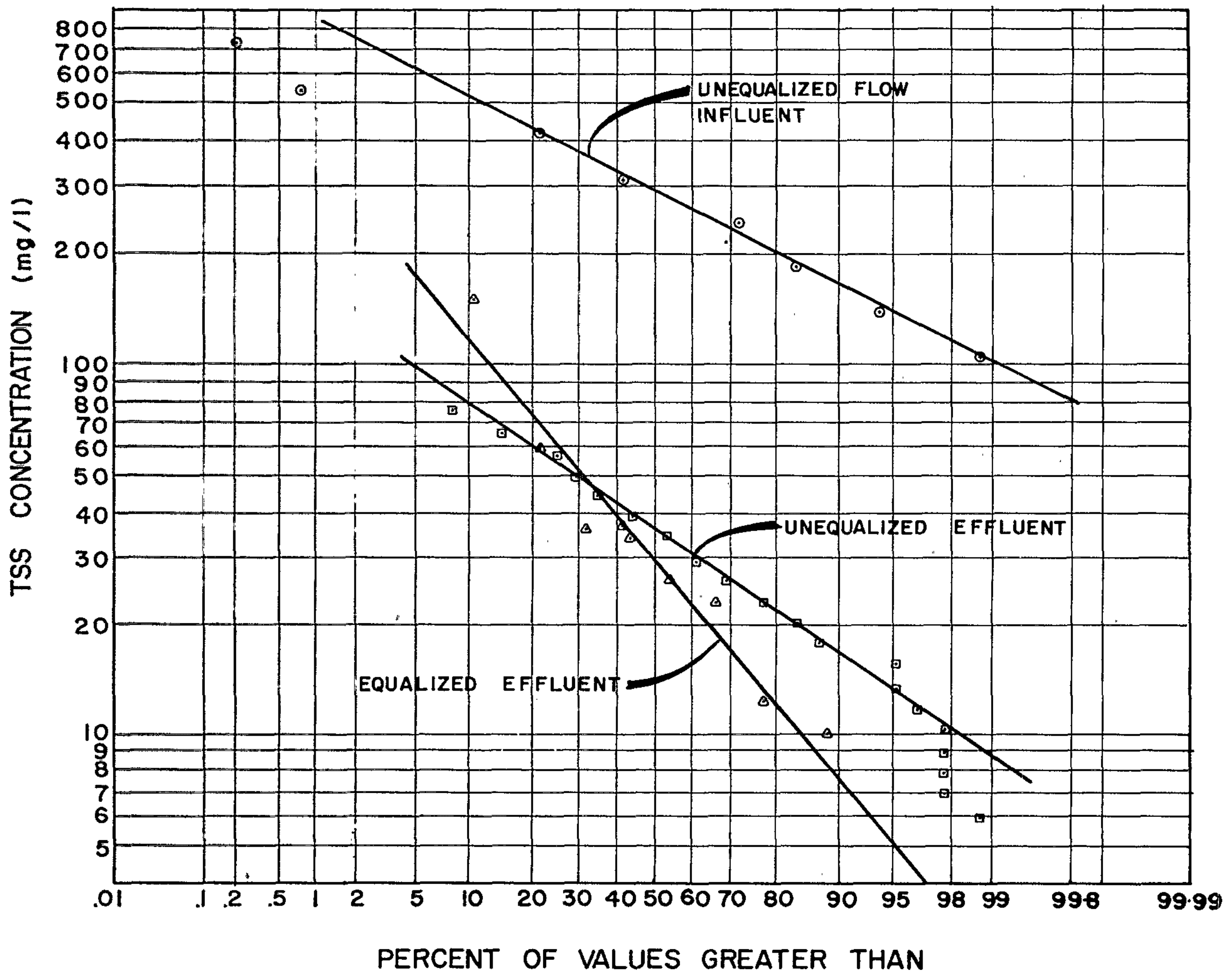

Figure 75. Distributions of average annual TSS concentrations, 8 equalized versus 65 unequalized trickling filter plants

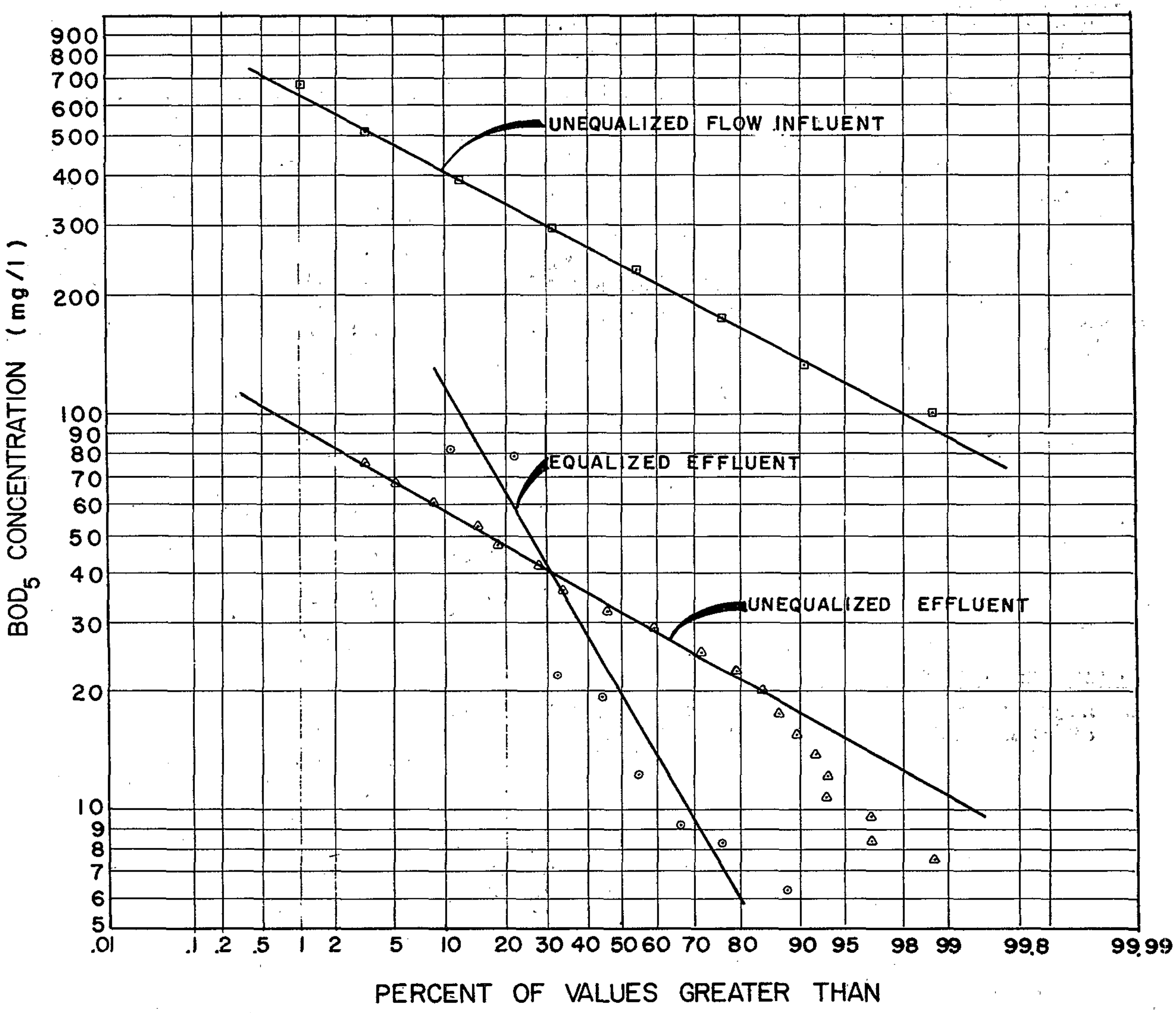

Figure 76. Distributions of average annual BOD concentrations, 8 equalized versus 65 unequalized trickling filter plants

TABLE 23. SUMMARY OF ALTERNATIVES IN IMPROVING EXISTING TREATMENT PLANT PERFORMANCE

Symptom	Possible Cause(s)	Possible Remedies
PRIMARY CLARIFIER		
I. Carryover of settleable solids at peak flows.	1. Hydraulic overload of clarifier.	a. Flow equalization ahead of primary clarifier. b. Add additional primary clarifiers. c. Add chemicals, particularly at peak flows. d. Design downstream processes to accommodate solids carryover.
	2. Clarifier design deficiency - hydraulic	a. Modify inlet hydraulics (e.g. extend or shorten skirt in center-feed circular clarifier; provide inlet baffles in rectangular clarifier). b. Modify outlet hydraulics (e.g. add weir length; move weirs). c. Add additional clarifiers. d. Distribute flow equally to multiple clarifiers. e. Provide wind screen to reduce effect of wind action.
ACTIVATED SLUDGE SYSTEM		
II. High secondary effluent TSS	1. Hydraulic overload of clarifier @ peak flows.	a. Flow equalization to constant flow. b. Add additional clarifiers. c. Add chemicals, particularly at peak flow.
	2. Poor sludge settleability; sludge is "light" or diffuse.	a. Add more air if DO<2 mg/l. b. Add chemicals. c. Reduce energy intensity of aeration (contributes to floc breakup). d. Adjust F/M and MCRT for improved SVI. e. Check nutrient balance (N and P). f. Flow equalization.
	3. Sludge blanket rises up and overflows at peak conditions.	a. Flow equalization. b. Pace RAS rate with feed rate. c. Install sludge blanket controls; pace RAS with sludge blanket.

(continued)

TABLE 23 (continued)

Symptom	Possible Cause(s)	Possible Remedies
		d. Analyze sludge settleability and densification routinely; manually set RAS rates.
		f. Adjust F/M and MCRT.
	4. Shock loads of toxic materials.	a. Flow equalization.
		b. Add emergency storage basin (if shocks can be detected and diverted).
		c. If activated sludge system is plug flow, modify to complete mix (depending upon amount and concentration of toxicant).
		d. Add powdered activated carbon to aeration basin.
		e. Add "roughing" biofilter upstream from activated sludge.
	5. Clarifier design deficiency - hydraulic	a. (See I.2.)
	6. Clarifier design deficiency - sludge withdrawal	a. Modify sludge scraper (e.g. add suction withdrawal).
		b. Modify sludge hopper.
III. High secondary effluent particulate BOD	See Causes and Remedies under "II. High Secondary effluent TSS".	
IV. High secondary effluent soluble BOD	1. High average daily soluble BOD loading.	a. Increase contact time by adding additional aeration volume.
		b. Decrease F/M by increasing MLVSS.
		c. If aeration system is "completely mixed", add baffles to change residence time distribution.
		d. Add additional oxygenation capacity if DO is low.
		e. Add "roughing" biofilter upstream from activated sludge.
	2. High peak soluble BOD loading.	a. (See IV.1.)
		b. Flow equalization (volume required for smoothing peak BOD loading would probably be greater than that required for flow smoothing).

(continued)

TABLE 23 (continued)

Symptom	Possible Cause(s)	Possible Remedies
	3. Shock loadings of toxic materials.	a. (See II.4.)
	4. Shock loadings of slowly degradable materials.	a. (See II.4.) b. Increase MLVSS. c. Increase contact time by adding additional aeration volume.
TRICKLING FILTER SYSTEM		
V. High secondary effluent TSS	1. Hydraulic overload of clarifier at peak flows.	a. (See II.1.) b. Add tube settlers.
	2. Clarifier design deficiency - hydraulic	a. (See I.2.) b. Add tube settlers.
VI. High secondary effluent particulate BOD	See Causes and Remedies under "V. High secondary effluent TSS".	
VII. High secondary effluent soluble BOD	1. High soluble BOD loading.	a. If existing biofilter is rock, change to plastic media. b. Increase media depth. c. Add additional biofilter units. d. Add aeration basin downstream from biofilter (upstream from clarifier). e. Add or increase biofilter recirculation. f. Provide positive, continuous ventilation of biofilter.
	2. High peak soluble BOD loading.	a. (See VII.1.)
	3. Shock loadings of toxic materials.	a. Flow equalization. b. Add emergency storage basins if shocks can be detected or anticipated. c. (See VII.1.)
	4. Shock loading of poorly degradable materials.	a. (See VII.3.)

This report is not intended to provide information on the broad range of alternative solutions to existing plant performance problems. Information on alternative upgrading techniques, for example, are provided in the USEPA Design Manual on Upgrading Existing Wastewater Treatment Plants.(35)

Significant Flow Equalization Benefits

This section is intended to identify those circumstances which separately, or combined, represent the most likely situations in which flow equalization will be useful and appropriate. The categories have been developed from analysis of data on existing flow equalization applications, and evaluation of benefits theoretically attainable from typical wastewater treatment unit processes operating under equalized flow conditions.

Flow equalization benefits may be categorized as follows:

- reduction of peaking requirements;
- reduction of process overloads at existing plants under some conditions;
- protection against toxic upsets;
- potential reduction of operational problems;
- provides increasing benefits with increasing treatment plant complexity; and
- reduction of plant recycle impacts from intermittent side-streams such as batch sludge dewatering.

Peaking Requirements--

The term "peaking" in this context is intended to cover peak hydraulic flow as well as peak mass loading of organics. Planning and design methodology for reducing peaking conditions by providing flow equalization is given in Section 2. Costs of providing peaking capacity in wastewater treatment components are compared to costs of providing flow equalization in Section 5.

Wastewater treatment processes are affected by peaks in different ways. Effects vary from little or no change in process performance to a complete loss of treatment capability. In most wastewater treatment applications, unit processes are installed consecutively to form the overall process train. The effect of peaking on effluent quality must therefore be evaluated on the aggregate of unit processes in a given system because of the interactive effects of individual unit processes on each other. Interactive effects of several unit processes can range from a dampening or elimination of upstream process deterioration by downstream unit processes to complete system failure. However, designing and operating unit processes in a wastewater treatment system to handle highly variable loadings, and the potential "cascading" effect of process deterioration in upstream processes can be expensive. Therein lies the potential cost savings associated with flow equalization as it may be applied to dampen loading variations.

The effects of peak loadings on wastewater treatment fall into three general categories:

1. No effect
2. Gradual deterioration
3. "Threshold" effect

These three categories are described schematically in Figure 77. In the first two categories, flow equalization does not provide benefits in terms of process performance improvements. This is because in category 1 (no effect) peak loadings do not influence process performance. In category 2 (gradual deterioration) the effects of process performance deterioration at peak loadings are averaged out by increased process performance at loadings below average conditions, with no net effect on average daily performance. When a wastewater treatment unit process responds to peak loadings according to the general category (threshold effects) process performance at peak loadings deteriorates significantly, and average daily performance is adversely affected; sometimes significantly. In terms of process performance, maximum benefits are obtained from the application of flow equalization in situations where wastewater systems or unit processes respond to peaking conditions according to category 3, threshold effects.

Pretreatment and Primary Sedimentation--Pretreatment facilities include prechlorination, bar screens, influent pumping, preaeration, grit handling, and flow measurement. These processes are conventionally sized for peak flow. Accordingly, reduction in peak flow will reduce pretreatment processes cost. However, if equalization facilities are located upstream of pretreatment, operating problems may occur and excessive labor costs may be required for process maintenance.

Primary sedimentation is intended to remove settleable and floatable solids which include various amounts of suspended solids, BOD, organic and ammonia nitrogen, and other constituents. In typical raw municipal wastewater, the settleable solids contain approximately 35 percent of the total BOD_5; this percentage can vary considerably, depending on the prescribed industrial wastes and other factors. Primary sedimentation

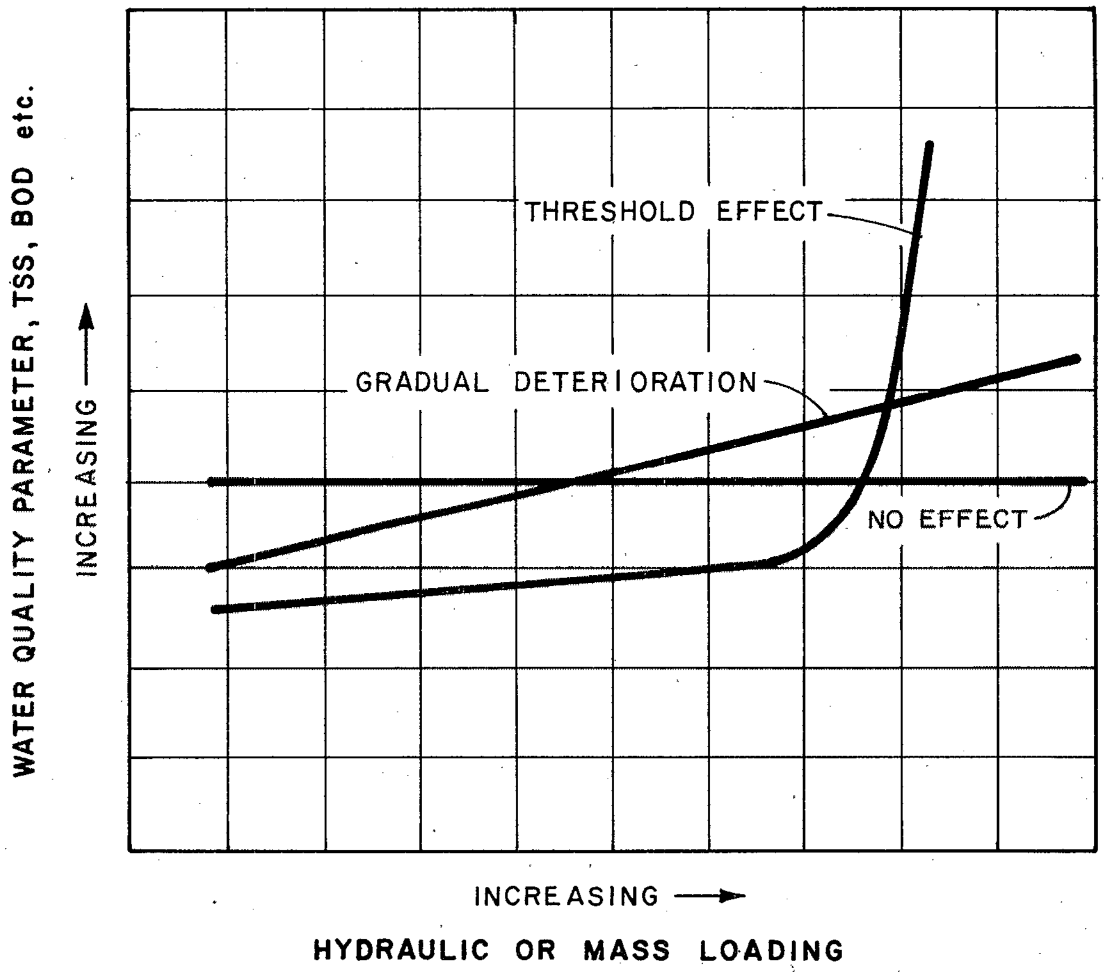

Figure 77. Classification of responses to loading variations by wastewater treatment processes

performs two basic functions: removal of solids with potential for causing problems in downstream processes; and to provide partial treatment of BOD and suspended solids to minimize the size of more costly downstream processes.

From the standpoint of settleable solids removal, rather high peak flows (peak overflow rates) are acceptable. Primary sedimentation tanks typically provide effective settleable solids removal at peak overflow rates of up to 2,000 gallons/ square foot/day (gpd/ft^2) and higher. This assumes that the overflow rate and not other factors (e.g., poor inlet baffling, insufficient depth, etc.) is controlling process performance. As overflow rates increase above the 2,000 gpd/ft level, loss of settleable solids to the primary effluent will begin to occur. However, this portion is that which is most easily resuspended; and it is the least significant concerning gross solids in equalization basins receiving primary effluent.

As settleable solids removal drops, a corresponding drop will occur in BOD removal efficiency in primary sedimentation. However, if equalization is located between primary sedimentation and downstream processes, the drop in BOD removal efficiency may be kept from affecting the downstream processes.

<u>Activated Sludge Systems</u>--The effects of peaking on biological and sedimentation processes making up an activated sludge system are discussed separately since peak loading effects can be different for each process. The basic function of the biological unit is to convert dissolved and nonsettleable organics to settleable biological solids. The sedimentation process serves to separate these solids from the liquid stream, providing a clarified effluent. Adverse effects of toxic or poorly degradable substances are discussed in a subsequent section.

Peak hydraulic loadings in the biological portion of an activated sludge system result in reduced residence times in the aeration zone as compared to residence times at average flow conditions. Such conditions reduce the period of time available for microorganisms under aeration to remove organic elements. Design practice for activated sludge systems provides considerably more residence time in the aeration zone than the minimum needed for removal of organic constituents (particularly soluble organics) from the wastewater feed. As a result, no deterioration in effluent quality ordinarily results from reducing the residence time available in the aeration cell at peak hydraulic loads. Figure 78 graphically depicts the relationship between the influent concentration of soluble organics (C_O, expressed as COD, in mg/l), the suspended solids concentration in the mixed liquor (X_{AO}, a measure of the microorganism population, in mg/l), and the time required for the microorganisms to remove the organics from solutions. These

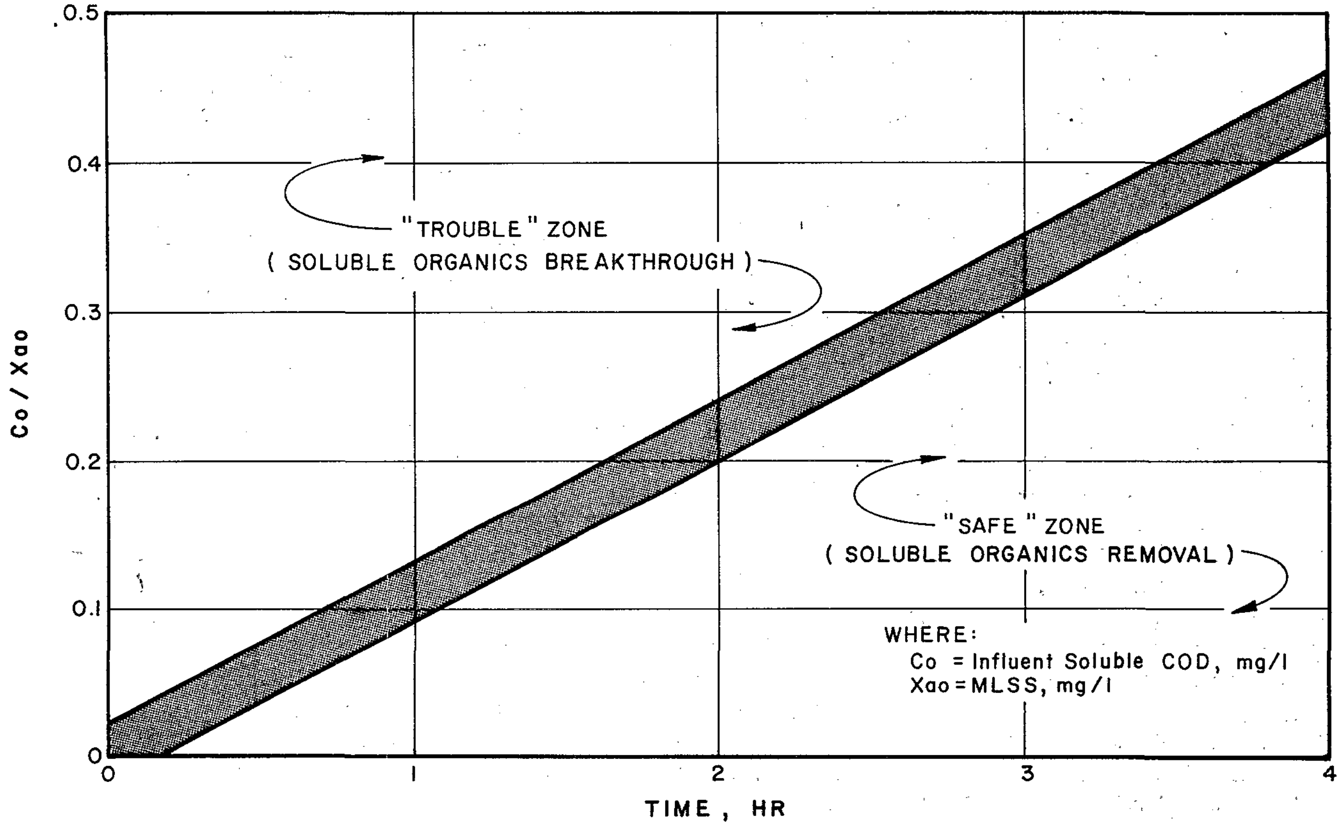

Figure 78. Estimate on limits on soluble organics removal in activated sludge (36, 37)

data, derived from batch tests, have been confirmed recently on large scale pilot activated sludge units in Seattle.(35)

A large number of studies of existing, full-scale activated sludge plants, as well as a number of well-controlled pilot scale investigations, have shown that average effluent quality of activated sludge units is <u>seldom</u> adversely effected by peak loads. Variations in effluent quality due to load changes tend to be "averaged out". These investigations were conducted over a wide range of organic loadings, mixed liquor concentrations, hydraulic loading rates, and activated sludge modifications. An example of such data is given in Figure 79. All of these systems were operating well within the range considered to be "safe" according to Figure 78, even though many were very heavily loaded. Typical design criteria for activated sludge units will result in operations well within the "safe" zone.

In summary, flow equalization for the purpose of reducing hydraulic and organic peaks to the biological process in activated sludge systems cannot ordinarily be expected to result in improved performance of the biological process. Except in unusual cases, when C_O/X_{AO} is high and the available residence time is very low, the soluble organics content of the process effluent will be unaffected by peak loading conditions.

<u>Activated Sludge Sedimentation</u>--Design and operation of sedimentation processes in activated sludge systems have been implemented in a wide variety of ways. Shapes of the tanks (circular, rectangular), inlet and outlet configurations (center feed - peripheral feed - peripheral withdrawal, etc.), sludge removal mechanisms (suction, scraping, etc.), and other factors all contribute to variations in the performance of existing or proposed sedimentation units. The principal feature distinguishing activated sludge sedimentation units from that for trickling filter units is the large quantity and different settling rates of solids that must be removed. Typical activated sludge units process about ten times more solids through the sedimentation tank as compared to a trickling filter system receiving comparable influent loadings.

Available data on activated sludge sedimentation tank performance indicate that the effect of peak hydraulic loadings is small until a certain "threshold" is reached (see Figure 77). This threshold can be defined in hydraulic terms (gallons per minute per square foot, gpm/sf) or solids loading units (pounds per day per square foot, ppd/sf), depending upon whether the ultimate limitation in the sedimentation tank is related to discreet settling of individual particles (hydraulic effects) or the ability to remove settled particles from the tank at an adequate rate (solids flux effects) under peak loads. Sometimes changing (increasing <u>or</u> decreasing) the rate of return sludge

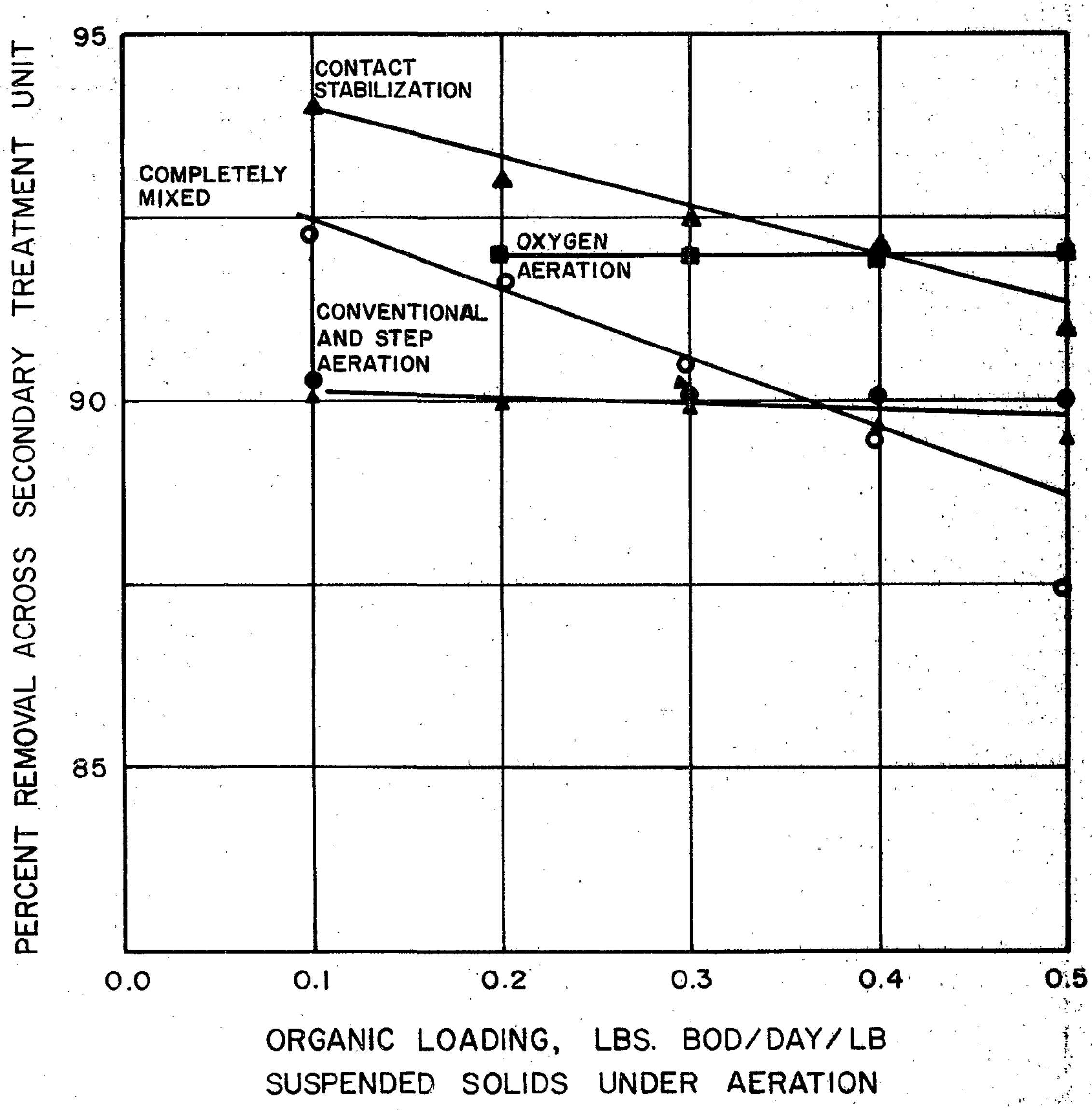

Figure 79. BOD removal as a function of organic loading for activated sludge modifications (35)

from the secondary sedimentation tank, or pacing the return rate with influent flow, can have the effect of reducing or eliminating loss of solids at peak loadings. Simple observation of the sedimentation tank at peak loads can often confirm the cause for excessive loss of solids. A number of possible alternatives for reducing excessive solids loss from the secondary clarifier of an activated sludge plant are included in Table 23.

Data was collected at the Ypsilanti Township (28) activated sludge plant on rectangular secondary sedimentation tanks designed for an average hydraulic loading in the range of 1,340 gpd/sf to about 1,500 gpd/sf. Performance deterioration appeared to occur at the lower overflow rate (1,340 gpd/sf) if, at the beginning of the test, a large quantity of sludge was already present in the tank. Process performance appeared to be adequate for periods of several hours at rates to about 1,500 gpd/sf when the sludge quantities in the sedimentation tank were controlled with an adequate return rate. In two separate test runs at 1,440 gpd/sf, performance was excellent and stable, with a sharp decline in performance after about 12 hours. The researchers in Ypsilanti concluded that the deterioration in effluent quality of an activated sludge sedimentation tank can considerably lag the increased flow rate.

At the activated sludge plant in Amarillo, Texas (29), secondary sedimentation tank overflow rates of 1,100 gpd/sf were sufficient to cause a significant loss of solids over the effluent weir. Design recommendations for activated sludge secondary clarifiers contained in the USEPA Technology Transfer Design Manual entitled "Suspended Solids Removal" (38) include average overflow rates of 400-800 gpd/sf with peak rates in the range of 1,000-1,200 gpd/sf. For new plants, tank sizing should take into account <u>both</u> average and peak rate limitations, with the tank actually constructed according to the criteria resulting in the largest recommended size.

It appears, from the foregoing full scale test data and design recommendations, that overflow rates beyond about 1,000 gpd/sf can cause effluent deterioration in an activated sludge sedimentation tank. In some cases, peak sustained rates to 1,500 gpd/sf can be tolerated for a period of several hours depending upon the design configuration of the tank, the quantity of sludge in the tank at the start of peak loading conditions, sludge settleability, and the ability of the sludge withdrawal equipment to remove sludge at peak sludge loading rates.

Flow equalization is one of the mitigating measures which may be effective in improving activated sludge effluent quality in existing systems when effluent quality deterioration is due to loss of solids from the secondary sedimentation tanks at peak flow rates. Flow equalization should be considered for existing activated sludge systems when the hydraulic rate to the clarifier exceeds about 1,000 gpd/sf for periods exceeding 4 hours per day where effluent quality deterioration results.

Trickling Filter Systems--The removal of organics in the biological process occurs through flocculation and agglomeration of the suspended and colloidal organic particles, and diffusion of the soluble organics into the trickling filter biological film. The overall removal (conversion to settleable form for ultimate removal in sedimentation) of organics in a trickling filter is related to a number of factors; including the hydraulic loading rate, the type and surface density of the trickling filter media, media depth, temperature, and the composition of the waste stream.

Data from a number of investigations are given in Figures 80 and 81. These data show that in several rock and plastic trickling filter applications BOD removal was found to be related linearly to the hydraulic loading rate. The significance of these data is that benefits from flow equalization on biofilter performance would be negligible since process efficiency reductions at peak loads would be "averaged out" by efficiency increases at loadings below the average rate.

The potential for increasing trickling filter performance through use of flow equalization, therefore, does not appear to be great. Subsequent discussions will focus on benefits which may by achieved through flow equalization if a trickling filter system receives shock loads of toxic or slowly degradable materials from time to time.

Trickling Filter Sedimentation--The underflow from a trickling filter typically contains biological solids and some undegraded influent solids. The particle size of this material ranges from colloidal and near-colloidal to large, readily settleable. The large particles are easily removed in even the most rudimentary sedimentation basins, but a large portion of the particulates are small and settle very slowly. Removal of the particles on the smaller end of the particle size spectrum frequently is a crucial factor in the ability of a trickling filter plant to meet secondary effluent TSS and BOD standards of 30 mg/l.

Very little data is available to allow prediction of the effects of peak hydraulic and solid loads on trickling filter sedimentation tanks. For the average trickling filter sedimentation basin, performance deteriorates gradually, and increases at lower than average loadings tend to average out peak effects.

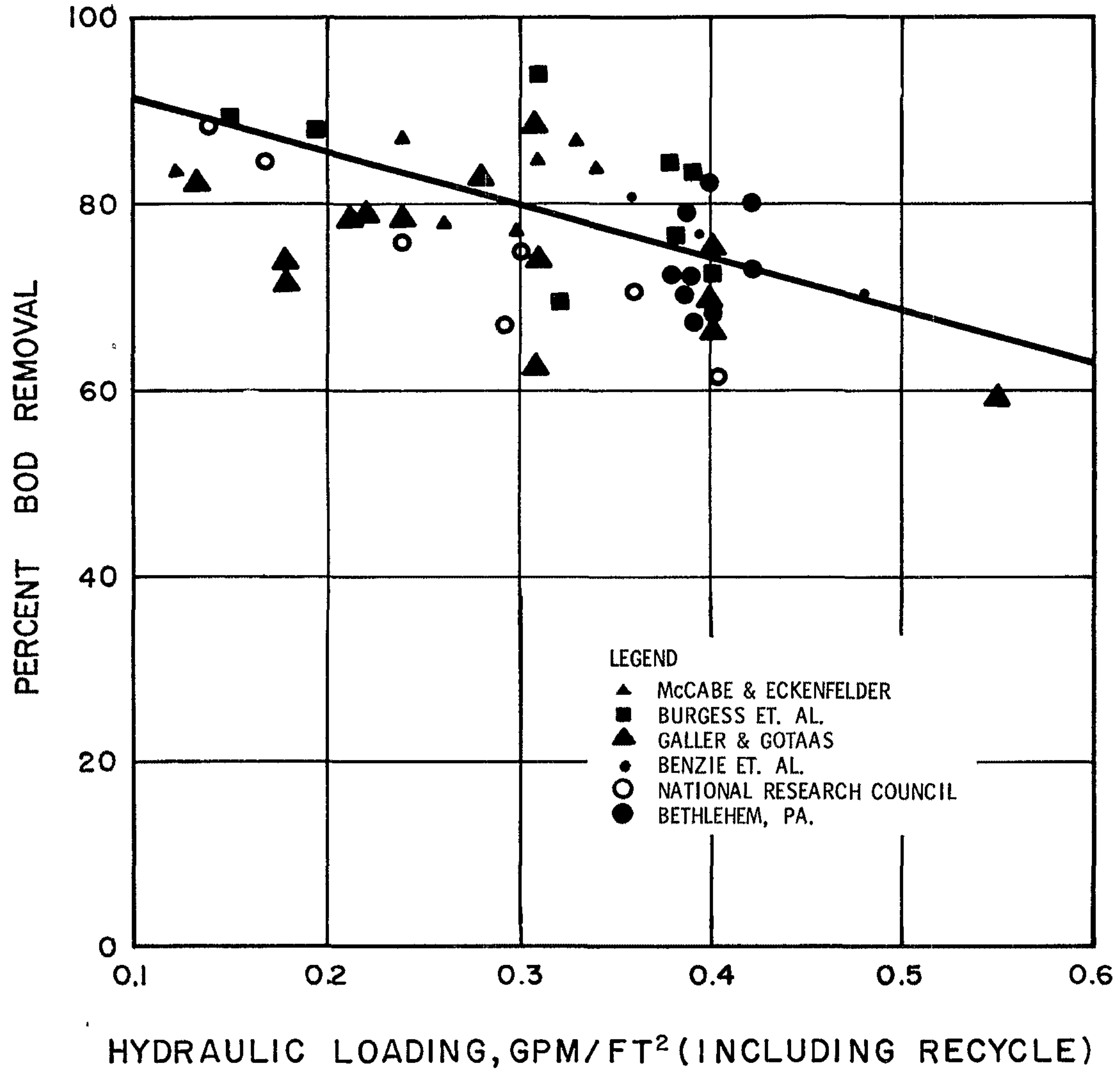

Figure 80. Effect of hydraulic loading on stone media trickling filter BOD removal (35)

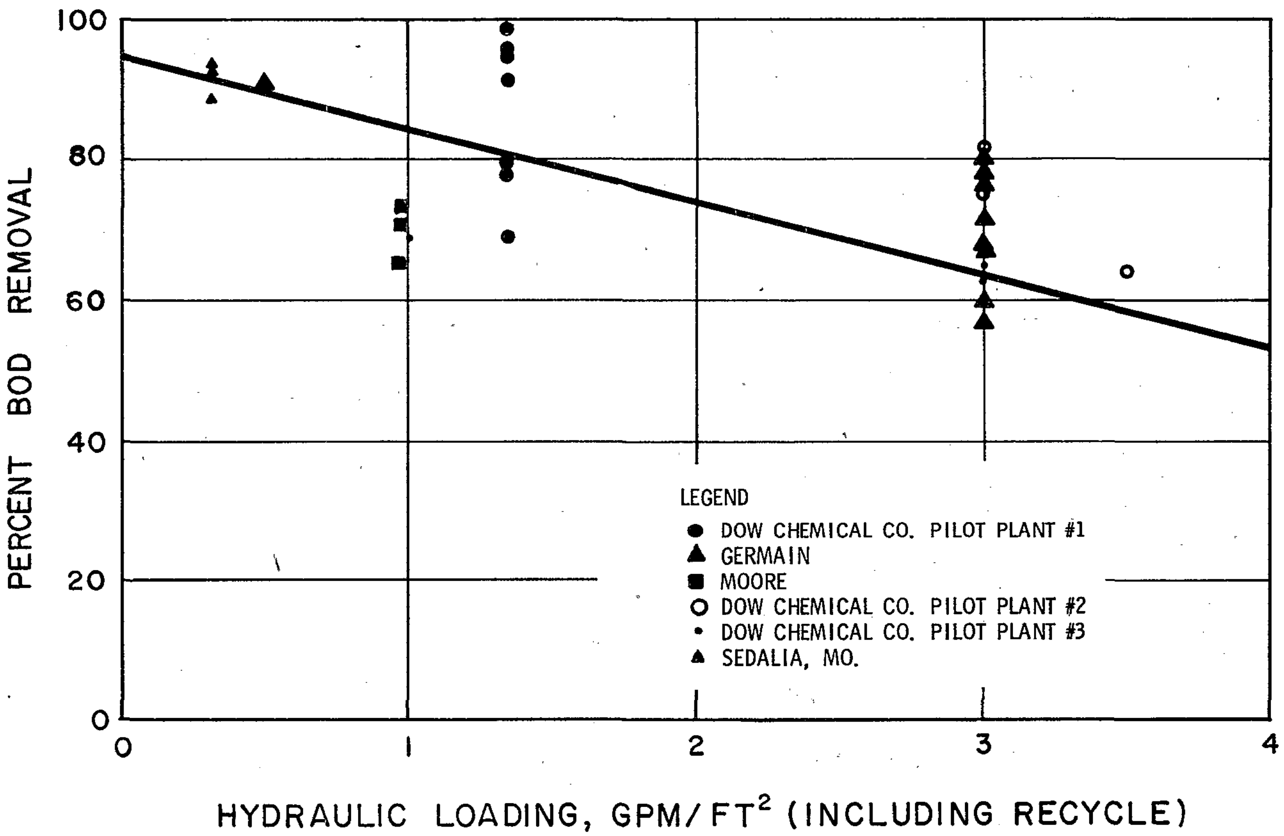

Figure 81. Effect of hydraulic loading on plastic media trickling filter BOD removal (35)

In such conditions flow equalization benefits would be minimal. An example of such an observation is given in Figure 82. Data are taken from a trickling filter system recently tested in Seattle. Figure 82 shows a gradual change in effluent TSS from about 12 mg/l to 40 mg/l as the feed TSS to the sedimentation unit is increased from 100 pounds TSS/day/1,000 ft^2 to 600 pounds TSS/day/1,000 ft^2 of sedimentation tank surface area.

Also included in Figure 82 are data on the effectiveness of tube settlers in reducing effluent TSS levels across the same range of TSS loadings. Tube settlers were found to provide some improvement in effluent TSS after about 16 percent of the sedimentation tank surface area was covered with the settlers. This condition corresponded approximately to a tube settler hydraulic loading rate equal to that of the uncovered sedimentation tank. Beyond 16 percent coverage, the improvement in effluent TSS with increasing coverage was dramatic. As noted in Table 23, tube settlers are one important alternative to be considered if a trickling filter plant sedimentation unit must be upgraded.

Plant and Process Overloading--

In many cases, the actual average loads received at a particular existing treatment plant or wastewater collection system are at or near design loadings; but the peak rates of flow, organic or solids loadings are beyond the capability of existing facilities. Obvious examples of such conditions would be hydraulic overloads, which cause wastewater to overflow manholes and short duration dissolved oxygen deficiencies during peak organic loads to activated sludge aeration basins. Flow equalization should be one of the corrective alternatives in such circumstances. Design of flow equalization to achieve a given reduction in hydraulic or organic peak rates may be carried out according to the criteria set forth in Section 2.

Toxic Upset Conditions--

The effects of toxic substances on wastewater treatment performances may be reduced or eliminated through proper use of flow equalization. Intermittent shock loadings of toxic substances can adversely affect the viability of biological processes for periods much longer than the actual exposure period. Frequently the toxic substances may be diluted in equalization facilities to harmless concentrations. Design of such basins, however, requires that the magnitude and duration of the shock load be known or estimated, as well as the time of occurrence. Design of equalization basins for dampening the concentration of toxic substances should be carried out according to the procedures in Section 2; except that the time and concentration factors associated with the toxic substance are used in the analysis rather than the flow and concentration factors indicated in the Section 2 discussion.

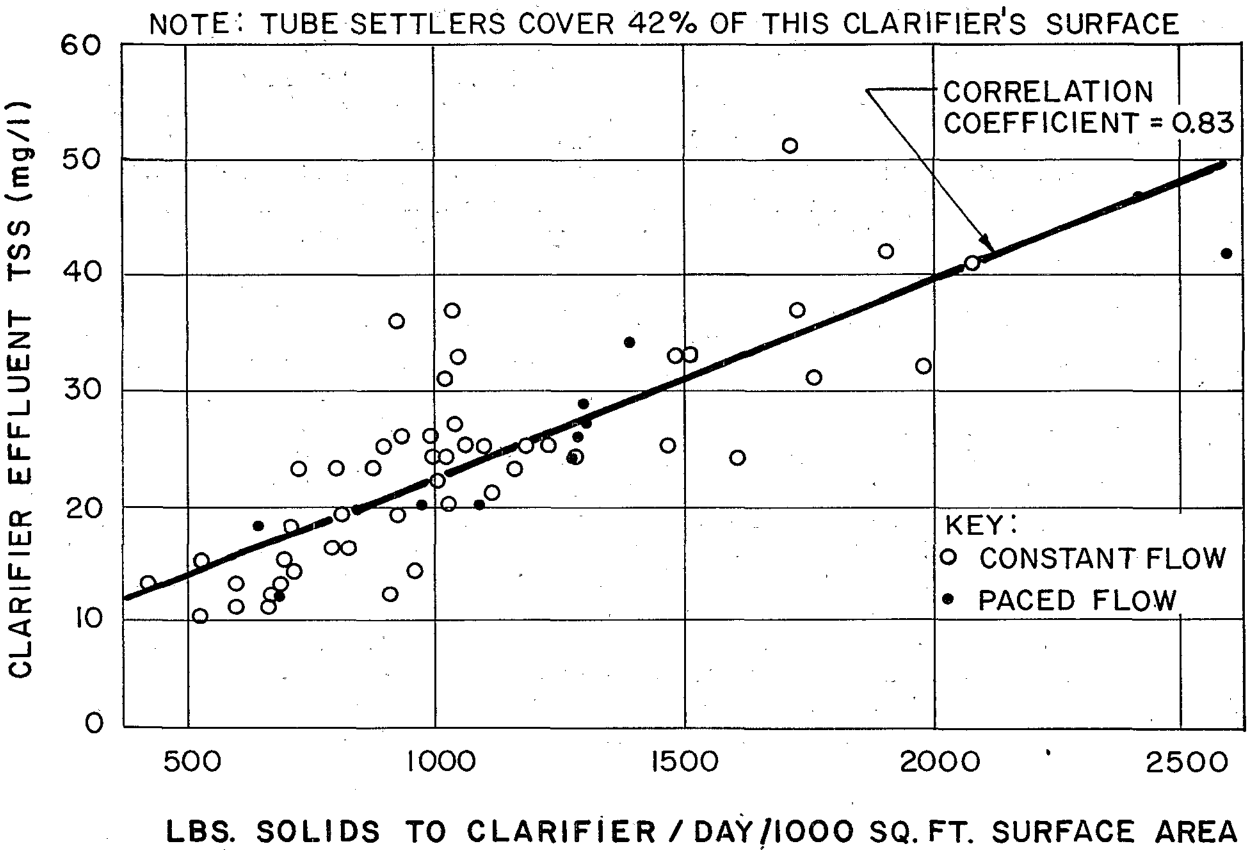

Figure 82. Sedimentation tank effluent TSS versus solids loading - normal hydraulic loadings (36)

Performance benefits can be significant when flow equalization is applied in situations where toxic upsets are otherwise known to occur. An example of the magnitude of such benefits is available in data from Tecumseh, Michigan. Before equalization, toxic upsets from the intermittent discharge of oils and plating wastes from local industries were partially responsible for a yearly average (in 1970) effluent BOD concentration of 28.4 mg/l. After equalization (in 1973), the annual average effluent BOD had been reduced 73 percent to 7.7 mg/l. A comparable, and sometimes greater, improvement in performance following addition of flow equalization is attainable at other plants, depending upon the severity of upsets resulting from toxic substances and the type and volume of flow equalization provided.

Operational Problems--

The benefit most frequently claimed for flow equalization by operators of equalized plants was that the plant was easier to operate (operational problems were reduced) following addition of flow equalization facilities. Though this benefit is difficult to quantify, it is a factor which should be carefully considered in the initial design or renovation of wastewater treatment works.

Process Complexity--

The operational and cost benefits associated with flow equalization increase as wastewater treatment plant complexity increases. This is by virtue of the fact that the size, cost, and operation of flow equalization facilities are essentially independent of the number and type of treatment process(es) downstream. Flow equalization benefits (such as reduced peaking capacity requirements) increase as the number of affected processes increases. A more complete analysis of the cost/benefit factors associated with treatment plant complexity is given in the cost analysis examples in Section 5.

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SECTION 6

THE DEVELOPMENT OF COSTS FOR EQUALIZATION AND TREATMENT PROCESSES

BASIS OF UNIT COST DEVELOPMENT

Information required for preliminary estimation of construction and operating costs for equalization facilities and appurtenances is developed in this section. Relationships between capacities, costs of construction and other initial investment costs of major components of flow equalization facilities are defined along with major operation and maintenance costs. The data presented are appropriate for average conditions and, when used with discretion for specific local situations, will permit development of preliminary estimates for flow equalization facilities.

Construction costs can be expected to change with time, in keeping with corresponding changes in the economy and regional variations in materials and labor. A good barometer of these changes is the Engineering News Record (ENR) Construction Cost Index. It is computed from prices on construction materials and labor and is based on a value of 100 for the year 1913. Experience has shown that the costs of municipal treatment works follow quite closely the cost escalation reported in the ENR cost index.

The estimates of construction costs presented in this report for both existing cost data received and cost curves developed are representative of the national average price levels (ENR Index = 2600) as of August 1977. If the estimating data are to represent costs in a particular area at a different date, they must be adjusted accordingly. The cost adjustment can be based on the ENR index; which permits comparison of price levels at differing locations and dates.

Cost curves have been developed for four major equalization facility components: holding tanks or basins, wastewater pumping stations, aeration/mixing systems, and facility washdown systems (manual). Cost estimates for each component are related to a single parameter for sizes of individual facilities ranging in capacity from 0.1 to 100 million gallons. Use of these estimating data is dependent on the design engineer establishing, on the basis of appropriate design criteria, the overall complexity of facilities to be provided and the sizes of components required.

These estimating data cannot in any way be used as a substitute for cost estimating based on detailed knowledge of a particular local situation. Construction cost estimates must take into account not only the constraints and design characteristics of a particular equalization facility, but also such items as labor and material costs and availability, time allowed for construction, climate and seasonal factors, local site conditions, and other variables such as whether the equalization facility is to be added to an existing sewage treatment plant or is to be part of a new facility.

The information on estimating both construction costs, and operation and maintenance costs requirements is intended to be applicable for average situations throughout the continental United States. Of course, any estimating method having such a broad applicability must of necessity be regarded as suitable only for determining preliminary estimates. Based upon variations in construction costs, it is estimated that the costs of individual components for any specific situation may vary in excess of 100 percent from the average cost values shown in this study. In particular cases this variation may be substantially greater. Variations are due to differences in factors which are not readily quantified or regulated, as discussed previously. The user should recognize the inherent limitations of such estimates and should develop applicable cost estimates as warranted by local circumstances.

CONSTRUCTION COSTS

Data for estimating initial investment costs for wastewater flow equalization facilities are presented here for four principal components:

1. Holding tank or basin
2. Wastewater pumping station
3. Aeration/mixing system
4. Manual-type washdown or solids removal system

The estimated construction costs shown for each of the components listed include all costs a contract bidder would normally encounter in completing the specific facility component named. Such costs include materials, labor, equipment, electrical work, and normal excavation, etc.

Estimates of construction costs do not include extraordinary costs associated with rock excavation, wet conditions or site dewatering, and piling. Such costs cannot readily be incorporated into average cost estimates. If such conditions are encountered, the designer must include an additional cost allowance for all extraordinary expenses involved, based on local experiences.

Estimated quantities have been extended by a material price to develop an estimated construction price. The material price used is a reasonably close average for the United States, excluding Alaska and Hawaii. Typically, freight costs of mechanical equipment are assumed at 1,500 miles transit to job site. Sales tax is not included in the construction cost estimate. Construction equipment and labor prices have been averaged out by multiplying manhours or equipment hours by crew, equipment or composite rates (for the United States, excluding Alaska and Hawaii) of the trades or equipment involved for a particular component. The manhours projected or estimated for each component of the flow equalization facility are those which, under average conditions, would be anticipated for any such project built in any semi-urban area. The estimates also take into account the workmanship experienced in building typical municipal wastewater work projects.

Cost estimates of flow equalization facilities may be developed by combining the basic structure, basin or tank, with desired appurtenances. The system cost is determined as the sum of all component unit costs.

Holding Tank and Basin Costs

Estimates of cost for the basic structures are based on a square concrete tank with 15-foot sidewater depth, 17-foot wall height, 15-inch average wall thickness, and 12-inch average bottom slab thickness for facilities in the range of 0.1 to 10 million gallon capacity. A cost versus capacity graph (see Figure 83) is presented for three conditions: 16-foot, 10-foot and 4-foot installed depth. Excavation was assumed for a 1 on 1 slope starting four feet from the outside wall. Prices used for earth excavation assumed dry conditions. A disposal site for excess excavation material was assumed to be within a distance of one mile.

For equalization facilities in the range of 0.5 to 100 million gallons, estimates were developed for a 3-1/2-inch thick, gunite lined, earthen basin with a 6-inch thick concrete bottom slab. Mechanical work items such as connecting piping, overflow structures, and miscellaneous electrical systems were estimated at 10 percent of the concrete and earthwork costs. A typical cross section of the tank or basin is shown in Figure 84.

It may be noted that the cost of a simple basin or tank, according to Figure 83 or 84, is significantly lower than

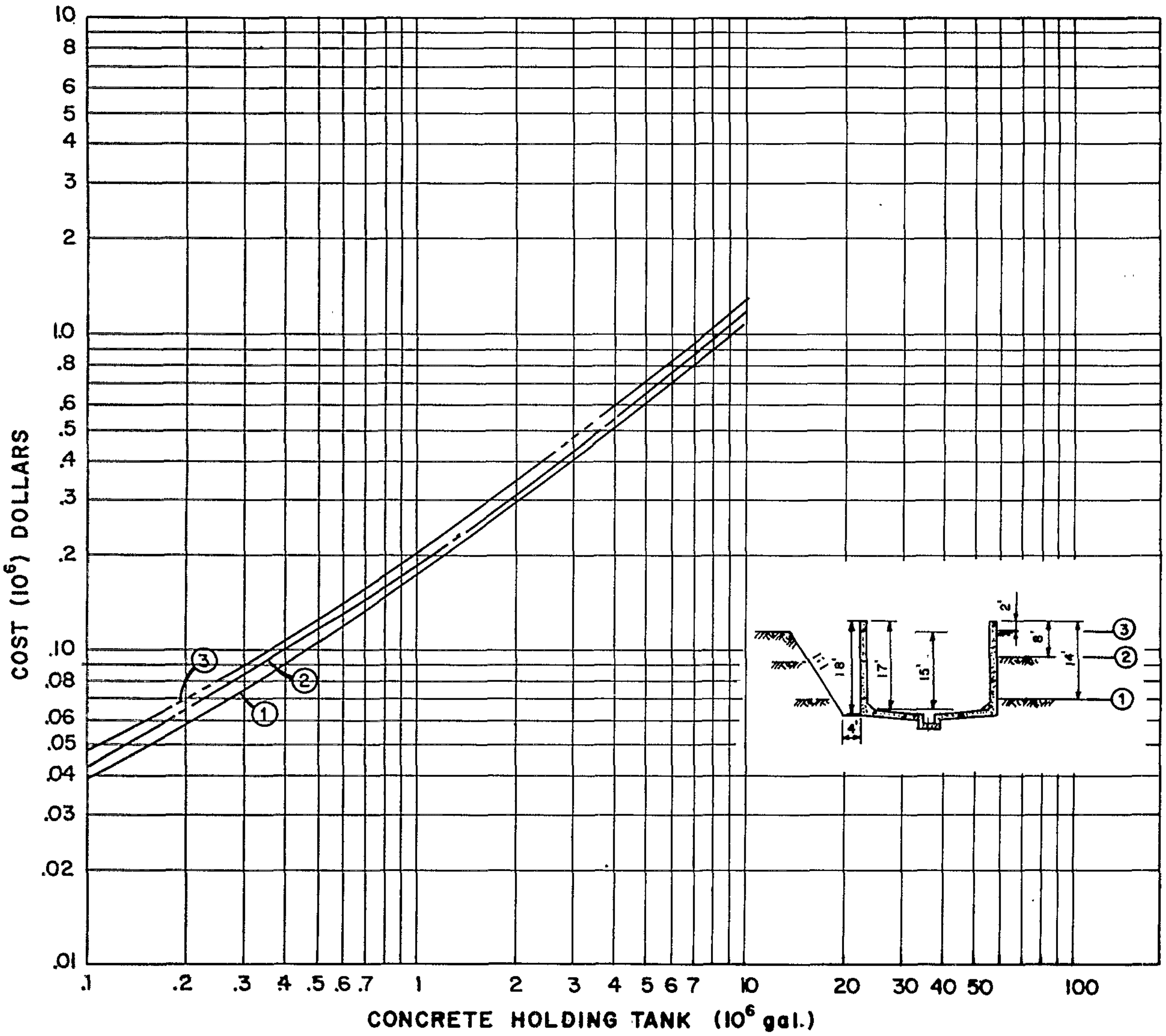


Figure 83. Construction cost as a function of concrete holding tank size

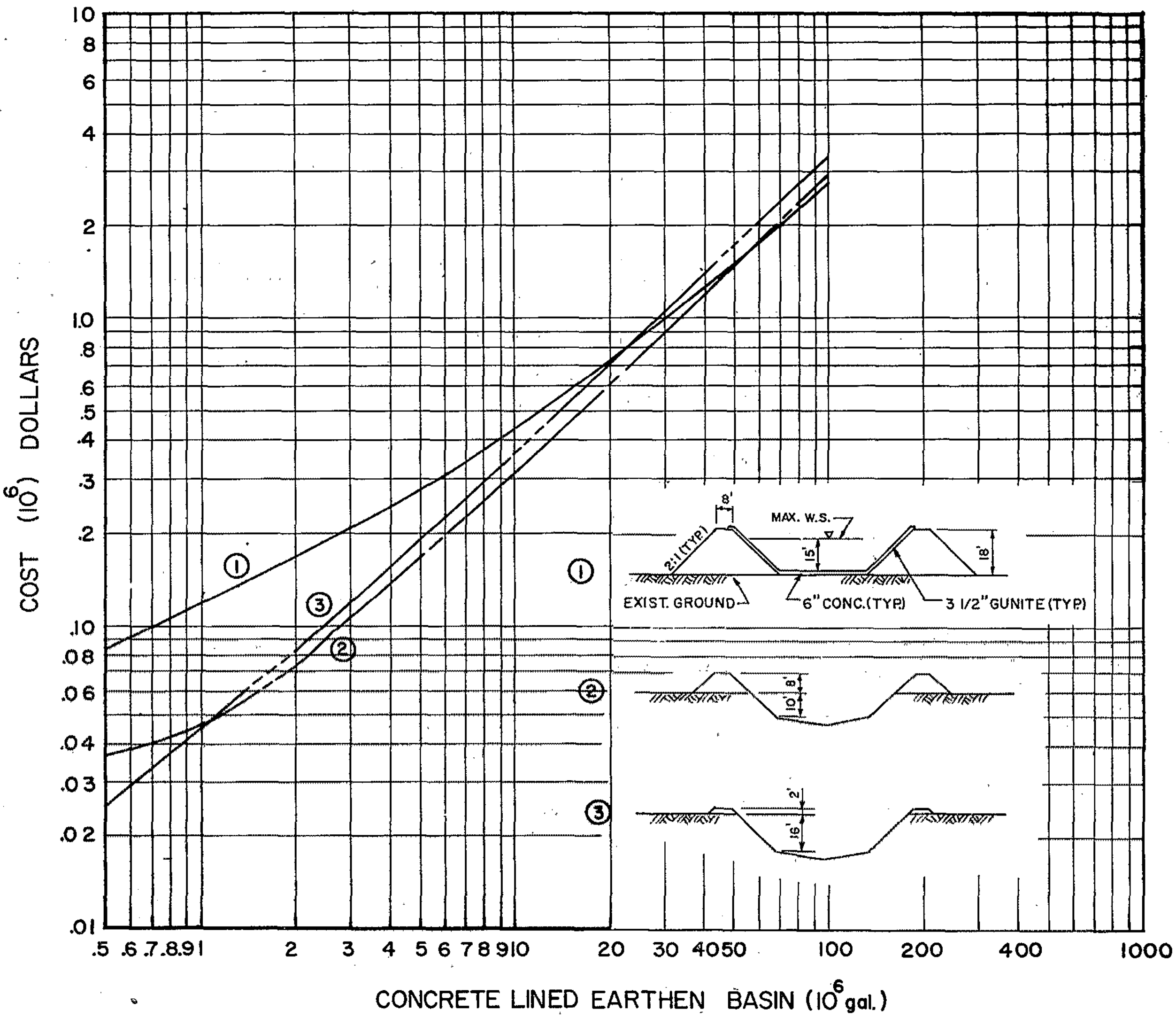


Figure 84. Construction cost as a function of concrete lined earthen basin volume

comparable complete equalization systems identified in the equalization facilities survey, Figure 18. The additional cost of existing systems is accounted for in the cost of appurtenances. For example, 50 percent of all systems surveyed include mixing or aeration, 70 percent include a pump station, and 65 percent include washdown facilities. Addition of such costs from the cost curves for appurtenances, Figures 85 through 88, results in estimated equalization system costs comparable to those summarized in Figure 18.

Wastewater Pumping Costs

Capital costs for pumping stations located downstream of the sewage treatment plant headworks are shown in Figure 85. The pumping volumes listed in the figure are based on a 24-hour pumping rate. Typically, the pumping capacity of a flow equalization facility should be sufficient to transfer the equalization structure volume in or out within a 12-hour period. Cost curves are for combinations of fixed speed, low lift, centrifugal pump installations without auxiliary or standby power facilities. Costs include structure, pumping units, all electrical and mechanical components, and control equipment.

Aeration and Mixing Costs

The construction costs of aeration equipment vary widely depending on the type and size of the equipment used. The most common types of equipment are surface aerators (fixed and floating) and diffusion systems. Inasmuch as the type, size, and cost of equipment will depend to a great extent on local conditions, aeration cost curves developed in this report are based on using floating, low speed, mechanical surface aerators. The cost curve (see Figure 86) includes all mechanical and electrical equipment and assembly necessary for a working installation.

Floating aerators with low water shutoff were used for the typical flow equalization facility, because their operation is complementary to a wide range of flow conditions. For existing or planned equalization facilities accompanying a treatment process system having diffused aeration equipment, it may be more economical to share use of common equipment and install a diffused air system.

Floating or fixed aerators are independent of all other sewage treatment plant unit processes and equipment, and, therefore, may provide more reliable data for a preliminary estimation of the additional cost of supplying an aeration component.

Aeration or mixing necessary to maintain oxygen levels ranges typically from 0.015 to 0.04 hp/1,000 gallons. Aeration

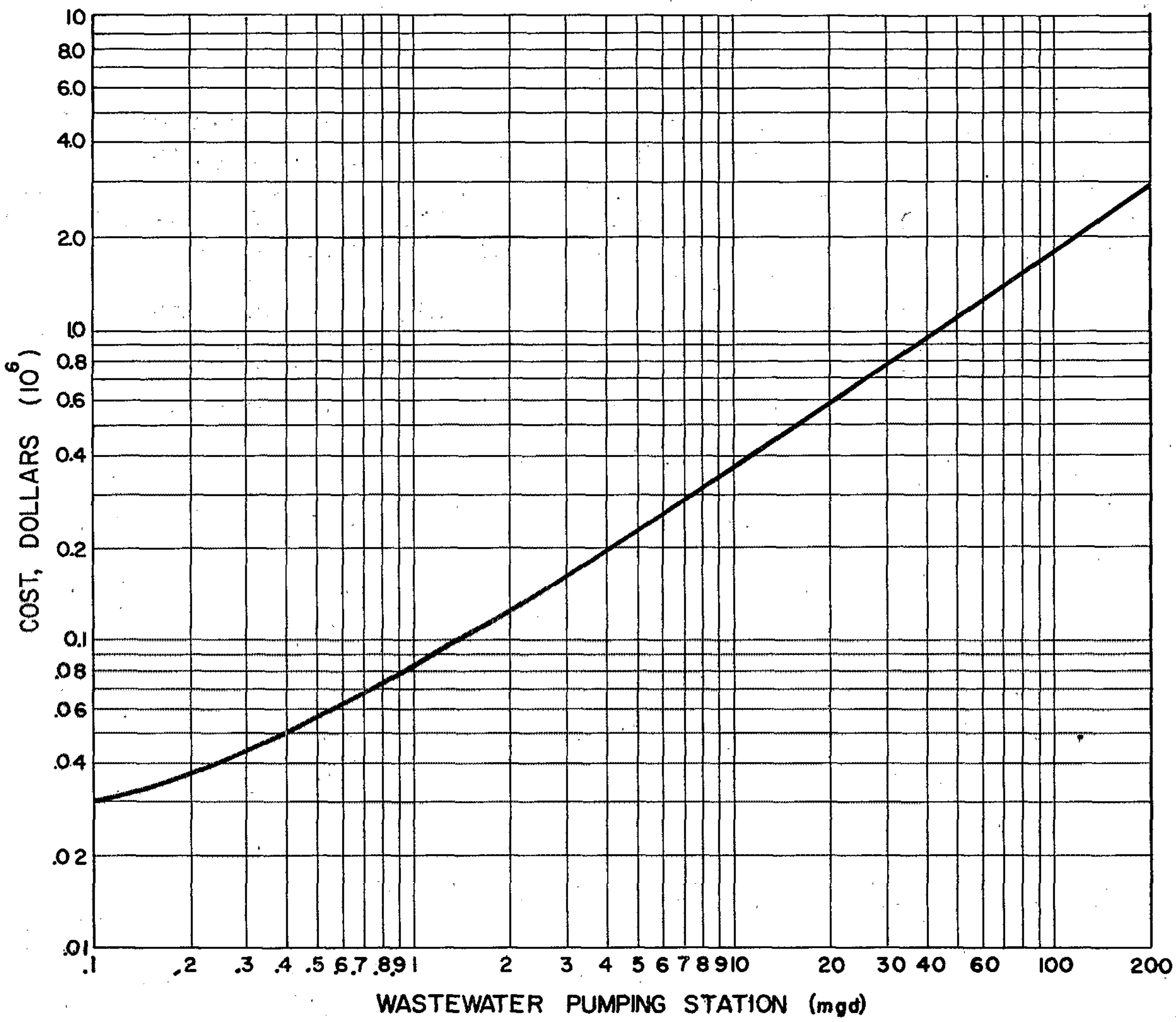


Figure 85. Capital cost as a function of pump station capacity

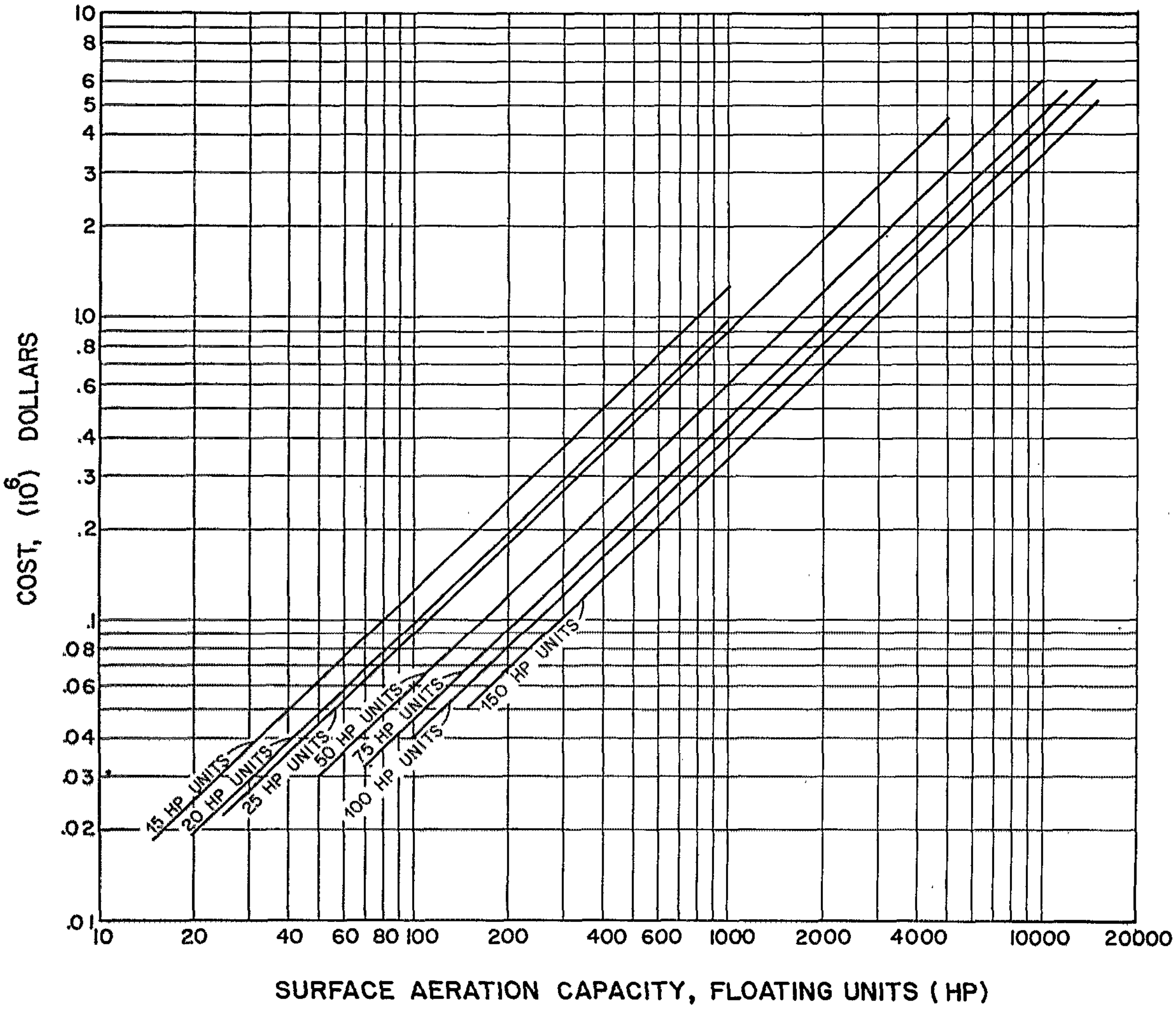


Figure 86. Surface aerator capital cost as a function of rated capacity

required for complete mixing (to keep all solids in suspension) ranges up to 2 hp/1,000 gallons. By virtue of this broad range, it is felt that each designer must determine the amount of horsepower required for each individual facility, and must select the appropriate value; again depending on where the facility is located, and the type and duration of flow stored. These items will significantly affect the level of aeration or mixing required.

Washdown and Solids Removal Costs

Capital costs associated with two manual washdown systems have been prepared. The first of these, suggested for use with concrete tank structures, includes a piping system around the inside of the top of the tank structure, with spray nozzles located on approximately four-foot centers. This system would also include hydrant assemblies at strategic locations for use in the washdown operation.

The second system, for use with concrete-lined earthen basins, is provided with a piping network installed around the periphery of the basin, and also across the basin at set intervals (as required). This is fitted for use either with fire boat-type hydrant assemblies, or fire hose connections. A schematic illustration of each system is shown in Figure 87. Within the suggested volume range of each holding tank or basin, the additional capital cost required for a manual washdown or solids removal system can be estimated from Figure 88.

Additional Capital Costs

No allowance for additional capital costs, such as construction contingency costs, and legal and administrative costs, has been included in the construction cost curves. For budgetary purposes, these costs are assumed to be 30 percent of the total construction cost. Land costs also have not been included because local land prices around the nation vary significantly. These costs should be added to the preliminary construction cost to arrive at a more meaningful estimate.

OPERATION AND MAINTENANCE COSTS

From an analysis of survey data and a review of the many variables associated with the use and operation of flow equalization facilities, it was concluded that no singular plot or grouping of operation and maintenance values for each of the individual equalization facilities components could reasonably be developed with any acceptable level of responsibility. Equalization facilities used as an integral component of the daily operation of a sewage treatment plant would have a wide range of possible O&M costs depending on where in the plant it

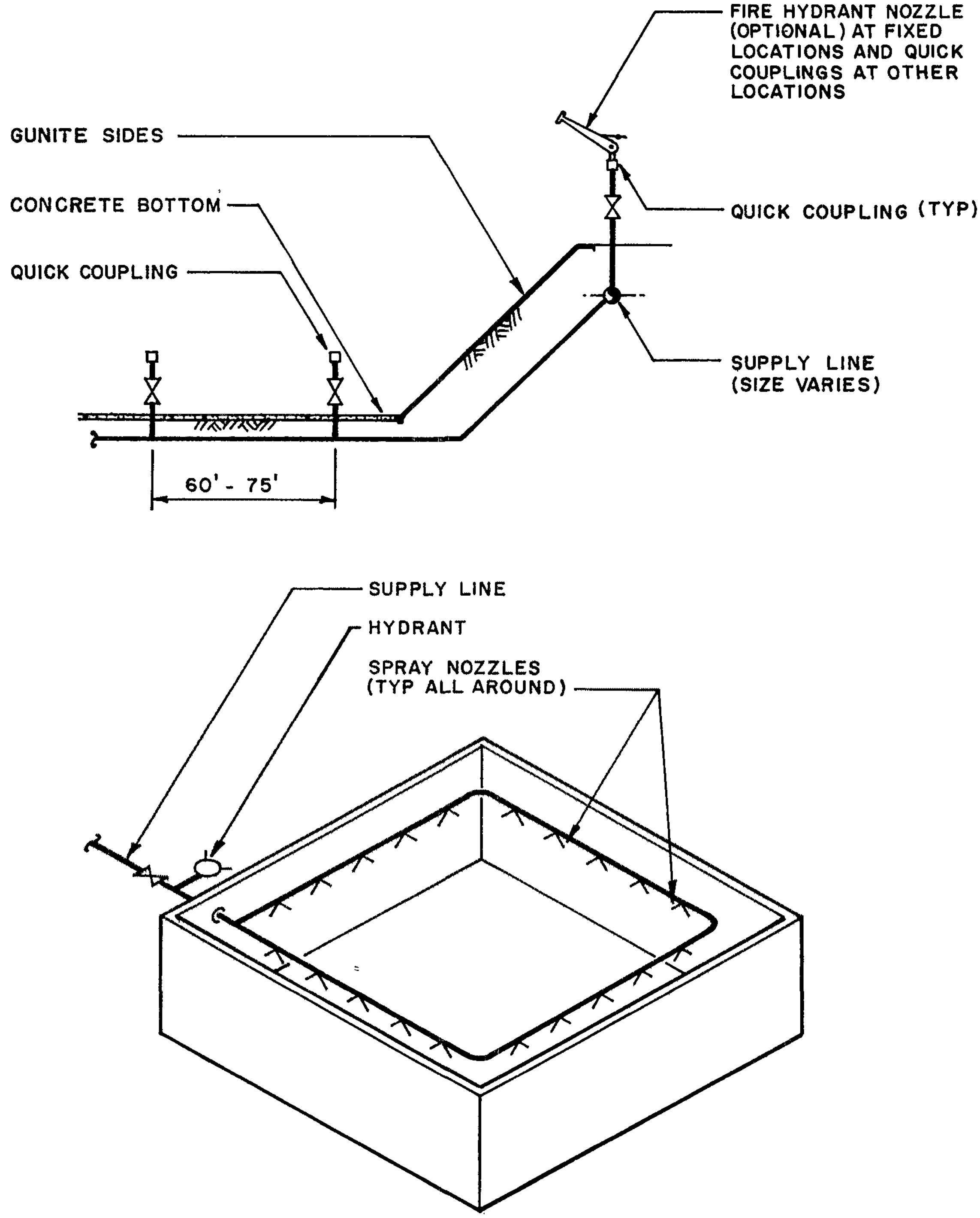


Figure 87. Equalization basin washdown system details

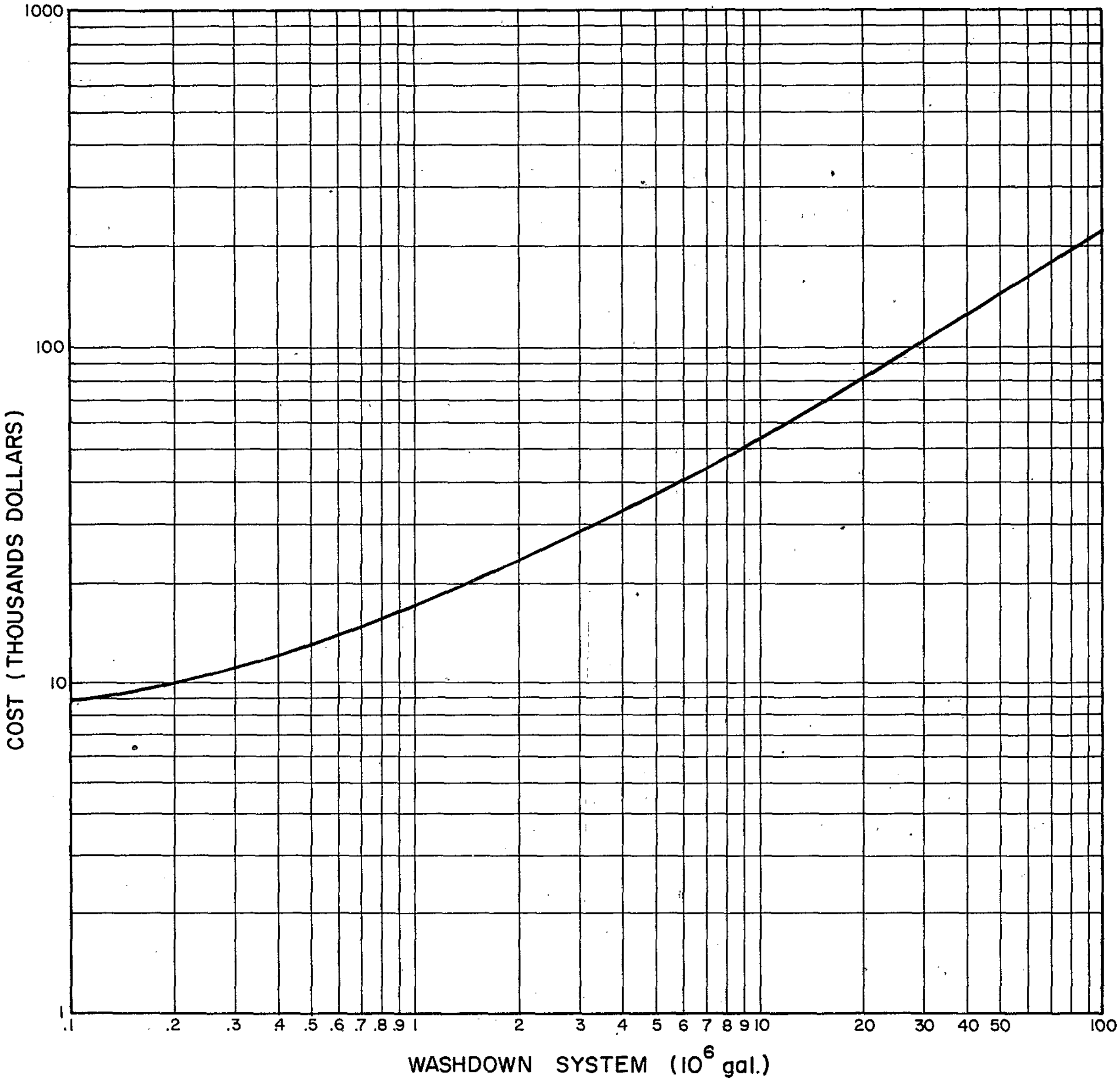


Figure 88. Equalization basin washdown system capital cost as a function of system capacity

is located, the complexity of the installation, and the duration and frequency of flow bypassed to the equalization structure. Structures used as side-line flow equalization facilities would have a still greater variance in yearly O&M costs, and would, to a great degree, be dependent on the number of times the facilities were used. Therefore, any attempt to outline typical or representative O&M cost figures is quite restricted. Additionally, O&M costs associated with equalization facilities sharing components of the treatment plant are difficult to quantify.

Operation and maintenance costs associated with aeration components are based on using the equipment 12 hours per day at a level of 0.02 hp/1,000 gallons of installed storage tank capacity. Power costs are estimated at $0.02/kwh. If requirements and operating conditions vary significantly from the assumed values, then the cost curve should be adjusted accordingly.

Manpower costs included in the various O&M cost curves were developed by assuming 6.5 hours of productive labor per man-day, with the labor cost assumed at $13.30 per productive man-hour, including fringe benefits.

Washdown or solids removal operation and maintenance costs are based on equalization facilities having some form of mixing/ aeration, and an assumed cleaning interval of twice per month.

The O&M cost relationships presented in this study were derived from local experience, survey data from existing flow equalization facilities, and from several EPA technical reports. The O&M cost figures should be used only to establish the relative magnitude or range of O&M costs for each component. Based on O&M data received as part of this survey, the cost curves shown in Figure 89 would appear to be the upper limit of O&M costs to be anticipated with flow equalization facilities.

COMPARISON OF EQUALIZATION COSTS AND TREATMENT PLANT PEAKING CAPACITY COSTS

Sample calculations are summarized in this section to compare the estimated cost of wastewater treatment plants designed for peak flow conditions without equalization, and for average flow conditions with either in-line or side-line flow equalization. Costs are estimated for plants of 1 mgd, 3 mgd and 10 mgd capacity, each for three moderate advanced treatment: secondary treatment (activated sludge); moderate advanced treatment (primary with alum, nitrifying activated sludge, filtration); and high level advanced treatment (primary with alum, high-rate activated sludge, nitrification-denitrification by the "three sludge" process, and filtration). These three treatment systems correspond to systems 5, 9, and 11 in the Areawide Assessment Procedures Manual, (39)9

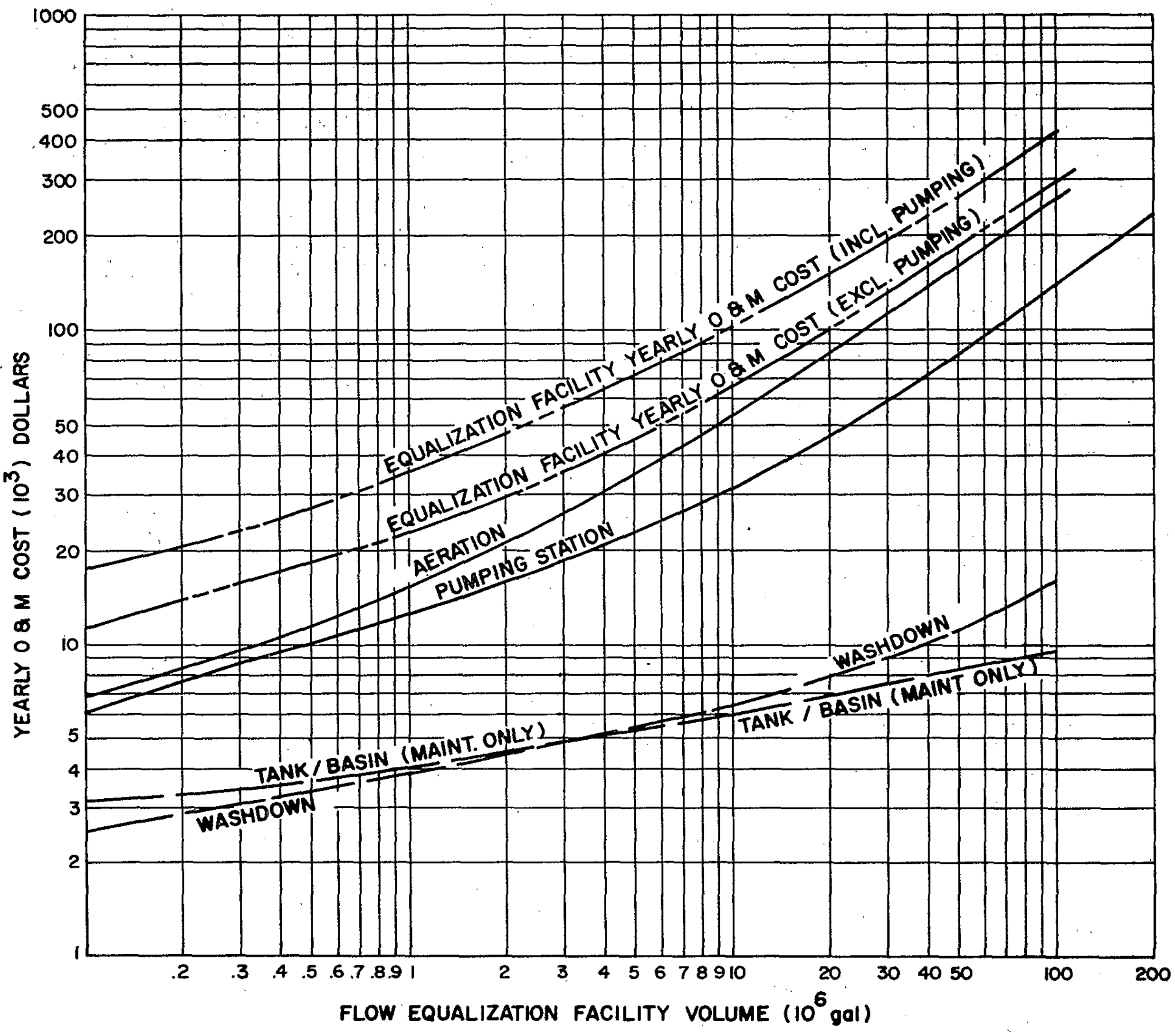


Figure 89. Equalization facility operation and maintenance costs as a function of flow equalization facility volume

as illustrated in Figure 90. Comparisons are made for peak-to-average flow ratios of 1.3:1, 1.6:1 and 2.0:1. Selection of these peaking ratios was based on preliminary calculations conducted to identify the region of transition from cost effective to not cost effective defined subsequently in Table 24 and Figure 91.

Cost information for treatment systems is taken from the Areawide Assessment Procedures Manual (39) using systems 5, 9 and 11. Construction costs are increased by 30 percent to allow for site work, piping, electrical work, and instrumentation. Also, in order to present data for mid-1977 cost levels, 5.05 percent is added; cost levels are based on an Engineering News-Record Construction Cost Index of 2600. Cost differences due to change in peak treatment flow are computed based on estimates of the impact of equalization on each process in the plant; care is taken to assure that equalization's benefits are credited where appropriate, and not elsewhere. As noted in Section 4, a simple and rigorous estimate of benefits is not possible in many cases, but the best practical approximations are made.

Cost information for equalization facilities is taken from unit cost curves in this section. Costs are estimated separately for side-line concrete tanks not requiring separate pumping, and for in-line concrete tanks. To drain the side-line equalization tank, the main plant influent pumps are assumed to suffice. An additional pumping station with capacity for total plant flow is required for in-line equalization.

Additional assumptions used in the cost comparisons include:

1. Equalization volumes used are 9 percent at P/A = 1.3, 17 percent at P/A = 1.6, and 29 percent at P/A = 2.0.

2. Cost savings in equalized flow plants result from:

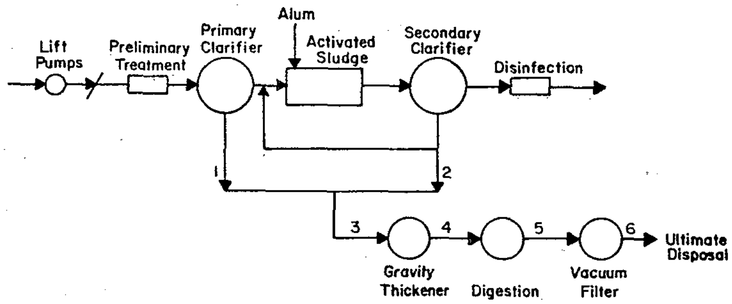
    a. Treatment system 5: secondary clarifier, chlorine contact, and aeration O&M.

    b. Treatment system 9: aeration tank, secondary clarifier, chlorine contact, filtration, chlorine contact, and aeration O&M.

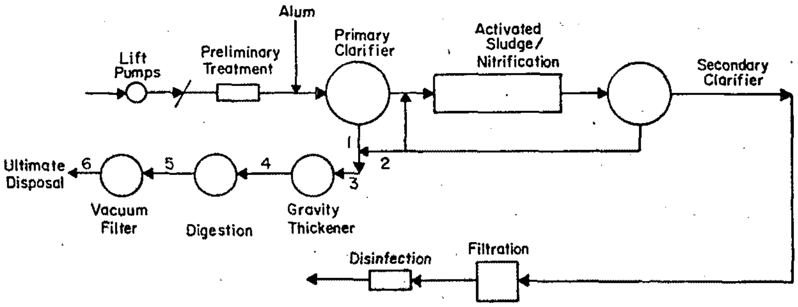
    c. Treatment system 11: second-stage aeration, secondary clarifiers, filtration, chlorine contact, and first and second stage aeration O&M.

The selection of equalization volumes used, 9, 17 and 29 percent of average daily flow, was based on mass diagram estimation using hypothetical daily hydrographs corresponding to the respective P/A ratios, 1.3, 1.6, and 2.0. The unit processes

SYSTEM NO. 5



SYSTEM NO. 9



SYSTEM NO. 11

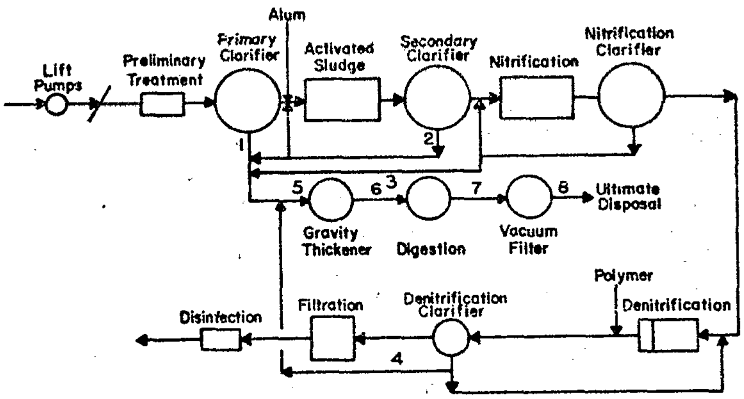


Figure 90. Treatment systems used in equalized vs. unequalized facilities cost comparison

TABLE 24a. EQUALIZATION COST EFFECTIVENESS EXAMPLE

Treatment System: System No. 5

Plant Size, mgd	Peaking Factor for Flow	Construction Costs - $ Millions: Plant Without Equalization	Equalized Plant	Equalization	Pumping Station	Net Benefit of Equalization	Net Benefit of Equalization w/P.S.
1	1.3	2.1856	2.1201	0.0702	0.1079	-0.0047	-0.1126
1	1.6	2.3222	2.2106	0.0930	0.1079	+0.0186	-0.0893
1	2.0	2.5954	2.4315	0.1313	0.1079	+0.0326	-0.0753
3	1.3	3.9614	3.8520	0.1196	0.2210	-0.0102	-0.2312
3	1.6	4.5078	4.2899	0.1872	0.2210	+0.0307	-0.1903
3	2.0	5.0542	4.7263	0.2561	0.2210	+0.0718	-0.1492
10	1.3	9.2888	8.9746	0.2561	0.4940	+0.0581	-0.4359
10	1.6	10.2450	9.7396	0.4056	0.4940	+0.0998	-0.3942
10	2.0	12.2940	11.4607	0.6604	0.4940	+0.1729	+0.3211

Treatment System: System No. 9

Plant Size, mgd	Peaking Factor for Flow	Plant Without Equalization	Equalized Plant	Equalization	Pumping Station	Net Benefit of Equalization	Net Benefit of Equalization w/P.S.
1	1.3	3.1418	2.9286	0.0702	0.1079	+0.1430	+0.0351
1	1.6	3.4150	2.9721	0.0930	0.1079	+0.3499	+0.2420
1	2.0	3.8248	3.2661	0.1313	0.1079	+0.4274	+0.3195
3	1.3	5.8738	5.4599	0.1196	0.2210	+0.2943	+0.0733
3	1.6	6.8300	6.0344	0.1872	0.2210	+0.6084	+0.3874
3	2.0	8.0594	6.7713	0.2561	0.2210	+1.0320	+0.8110
10	1.3	15.0260	13.4800	0.2561	0.4940	+1.2899	+0.7959
10	1.6	16.3920	14.1108	0.4056	0.4940	+1.8756	+1.3816
10	2.0	17.7580	14.0561	0.6604	0.4940	+3.0415	+2.5475

Treatment System: System No. 11

Plant Size, mgd	Peaking Factor for Flow	Plant Without Equalization	Equalized Plant	Equalization	Pumping Station	Net Benefit of Equalization	Net Benefit of Equalization w/P.S.
1	1.3	4.0980	3.8998	0.0702	0.1079	+0.1280	+0.0201
1	1.6	4.4395	4.1157	0.0930	0.1079	+0.2308	+0.1229
1	2.0	5.1908	4.6335	0.1313	0.1079	+0.4260	+0.3181
3	1.3	7.9228	7.5390	0.1196	0.2210	+0.264	+0.0432
3	1.6	9.2888	8.5415	0.1872	0.2210	+0.5601	+0.3391
3	2.0	10.6548	9.3873	0.2561	0.2210	+1.0114	+0.7904
10	1.3	19.1240	18.1405	0.2561	0.4940	+0.7274	+0.2334
10	1.6	21.8560	19.9983	0.4056	0.4940	+1.4521	+0.9581
10	2.0	24.5880	21.2550	0.6604	0.4940	+2.6726	+2.1786

TABLE 24b. EQUALIZATION COST EFFECTIVENESS EXAMPLE

Treatment System: System No. 5

		O&M Costs - $ Millions/Year					
Plant Size, mgd	Peaking Factor for Flow	Plant Without Equalization	Equalized Plant	Equalization	Pumping Station	Net Benefit of Equalization	Net Benefit of Equalization w/P.S.
1	1.3	0.1575	0.1507	0.0057	0.0140	+0.0011	-0.0129
1	1.6	0.1680	0.1560	0.0061	0.0140	+0.0059	-0.0081
1	2.0	0.1890	0.1736	0.0063	0.0140	+0.0091	-0.0049
3	1.3	0.2941	0.2309	0.0064	0.0185	+0.0487	+0.0383
3	1.6	0.3362	0.3107	0.0071	0.0185	+0.0184	+0.0001
3	2.0	0.3887	0.3496	0.0078	0.0185	+0.0313	+0.0128
10	1.3	0.6825	0.6427	0.0078	0.0320	+0.0320	-0.0000
10	1.6	0.7875	0.7164	0.0089	0.0320	+0.0622	+0.0302
10	2.0	0.9450	0.8256	0.0098	0.0320	+0.1097	+0.0777

Treatment System: System No. 9

1	1.3	0.2521	0.2356	0.0057	0.0140	+0.0108	-0.0032
1	1.6	0.2626	0.2263	0.0061	0.0140	+0.0302	+0.0162
1	2.0	0.2941	0.2396	0.0063	0.0140	+0.0482	+0.0342
3	1.3	0.4412	0.3466	0.0064	0.0185	+0.0882	+0.0697
3	1.6	0.5147	0.4262	0.0071	0.0185	+0.0814	+0.0629
3	2.0	0.5985	0.4577	0.0078	0.0185	+0.1330	+0.1145
10	1.3	1.0500	0.9386	0.0078	0.0320	+0.0334	+0.0015
10	1.6	1.2600	1.0209	0.0089	0.0320	+0.2302	+0.1982
10	2.0	1.4700	1.0776	0.0098	0.0320	+0.3826	+0.3506

Treatment System: System No. 11

1	1.3	0.3152	0.2876	0.0057	0.0140	+0.0219	+0.0079
1	1.6	0.3467	0.2802	0.0061	0.0140	+0.0604	+0.0464
1	2.0	0.3887	0.3100	0.0063	0.0140	+0.0724	+0.0584
3	1.3	0.6093	0.5476	0.0064	0.0185	+0.0553	+0.0368
3	1.6	0.7038	0.5763	0.0071	0.0185	+0.1204	+0.1019
3	2.0	0.8194	0.6128	0.0078	0.0185	+0.1988	+0.1803
10	1.3	1.4700	1.2462	0.0078	0.0320	+0.2160	+0.1840
10	1.6	1.6800	1.2914	0.0089	0.0320	+0.3797	+0.3477
10	2.0	2.1000	1.4286	0.0098	0.0320	+0.6616	+0.6296

TABLE 24c. EQUALIZATION COST EFFECTIVENESS EXAMPLE

Plant Size mgd	Peaking Factor for Flow	Present Worth: P.W. of O&M Net Benefit $ Millions	Present Worth: P.W. of O&M Net Benefit w/P.S. $ Millions	Overall Net Benefit Construction + O&M: Total P.W. of System w/Equal. $ Millions	Overall Net Benefit Construction + O&M: Total P.W. of System w/Equal. w/P.S. $ Millions
Treatment System: System No. 5					
1	1.3	+0.0124	-0.1482	+0.0086	-0.2599
1	1.6	+0.0676	-0.0930	+0.0862	-0.1823
1	2.0	+0.1048	-0.0558	+0.1374	-0.1311
3	1.3	+0.6515	+0.4393	+0.6413	+0.2081
3	1.6	+0.2110	+0.0011	+0.2417	-0.1914
3	2.0	+0.3590	+0.1468	+0.4308	-0.0024
10	1.3	+0.3668	-0.0002	+0.4249	-0.4361
10	1.6	+0.7140	+0.3469	+0.8138	-0.0473
10	2.0	+1.2577	+0.8966	+1.4306	+1.2117
Treatment System: System No. 9					
1	1.3	+0.1239	-0.0367	+0.2669	-0.0016
1	1.6	+0.3464	+0.1858	+0.6963	+0.4278
1	2.0	+0.5528	+0.3923	+0.9802	+0.7118
3	1.3	+1.0117	+0.7995	+1.3060	+0.8728
3	1.6	+0.9337	+0.7215	+1.5421	+1.1089
3	2.0	+1.5255	+1.3133	+2.5575	+2.1243
10	1.3	+0.3834	+0.0172	+1.6733	+0.8131
10	1.6	+2.6404	+2.2734	+4.5160	+3.6550
10	2.0	+4.3884	+4.0214	+7.4299	+6.5689
Treatment System: System No. 11					
1	1.3	+0.2512	+0.0906	+0.3792	+0.1107
1	1.6	+0.6928	+0.5322	+0.9236	+0.6551
1	2.0	+0.8304	+0.6698	+1.2564	+0.9879
3	1.3	+0.6343	+0.4221	+0.8985	+0.4653
3	1.6	+1.3810	+1.1688	+1.9411	+1.5079
3	2.0	+2.2802	+2.0680	+3.2916	+2.8584
10	1.3	+2.4775	+2.1105	+3.2049	+2.3439
10	1.6	+4.3552	+3.9881	+5.8073	+4.9462
10	2.0	+7.5886	+7.2215	+10.2612	+9.4001

realizing cost savings by design to meet average flow capacity instead of peak flow have been identified as a design decision. Flexibility of such decisions is necessarily limited in such an example due to dependence on widely available cost curves for treatment components (39). Nevertheless, such decisions must be made in the design process in accord with details of individual treatment conditions based on the knowledge and experience of the design engineers.

The cost comparison of equalized and unequalized treatment facilities is summarized in Table 24 (a, b, and c). In reviewing the tabulated figures it is important to note the relative contributions of construction and O&M costs, using O&M costs expressed in terms of present worth. The final two columns of Table 24c indicate the overall net benefit of the respective treatment systems, with equalization facilities compared to unequalized treatment plants. This shows that, adopting the assumptions used in developing the costs, for systems using the side-line equalization without an additional pumping station, the unequalized systems designed to accommodate full diurnal peak flows. Systems using in-line equalization requiring an additional pumping station, however, are more expensive for the simplest treatment system, and for the smallest size intermediate treatment system. Regions of plant size and peak-to-average ratio, for which equalization is not cost effective for this example, are summarized in Figures 91 and 92.

Examination of this example enables some generalizations to be made. Extending generalizations to the vast range of potential applications differing from this example in important details is, however, not possible. Costs and cost comparisons are heavily dependent upon the underlying assumptions used in their development. The range of required assumptions necessary to allow a valid cost comparison for a given treatment plant is very broad. It is therefore necessary that each potential application of equalization be analyzed according to its own situation in a manner similar to that above.

The example shows that, based on capital plus operation and maintenance cost considerations, use of equalization facilities should be considered in all but a few situations when new treatment plants are being planned. Situations that do not favor equalization are: small plant sizes (less than 5 mgd); simple secondary treatment plants required to meet only 30-30 standards; low peak-to-average flow ratios with little wet weather influence; and physical settings requiring in-line equalization, including a pump station. Situations tending to favor equalization include: large plants with high peaking factors, plants with treatment in addition to simple secondary required; and physical settings not requiring a duplicate pump station.

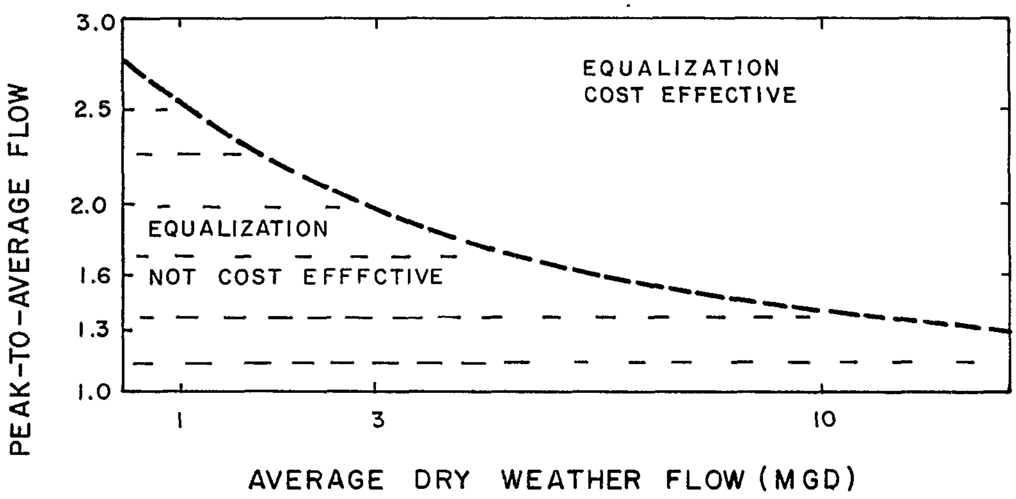


Figure 91. Regions of Cost Effective Equalization for Cost Example, Treatment System 5

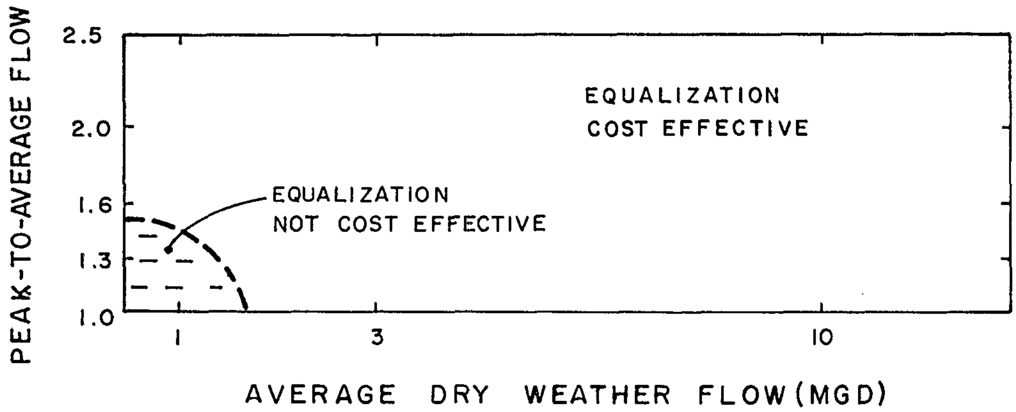


Figure 92. Regions of Cost Effective Equalization for Cost Example, Treatment System 9

REFERENCES

39. Areawide Assessment Procedures Manual, Volume III, Appendix H, U. S. EPA, MERL, ORND, EPA-600/9-76-014, July, 1976.

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16. ABSTRACT

This study was conducted to analyze the impact of flow equalization on the operation and performance of municipal wastewater treatment plants. Objectives of the study were: (1) establish the effects of flow equalization on plant performance; (2) summarize current experience with design and operation of equalization facilities; and (3) summarize unit costs of equalization facilities and appurtenances. A national survey identified facilities and provided detailed information on design; operating practices; and construction, operation, and maintenance costs. Quantitative effects of equalization on plant performance were analyzed. Quantitative design methodology is presented for the sizing and estimation of costs for equalization facilities.

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17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
SEWAGE TREATMENT: activated sludge process, trickling filters, packaged sewage plants; SEWERS: combined sewers, sanitary sewers; BASINS (containers); FLOW CONTROL; FLOW MEASUREMENT; COSTS: cost effectiveness, cost estimates, cost comparison, construction costs.	FLOW EQUALIZATION: in-line, side-line; COSTS: capital costs, operation and maintenance costs; SEWAGE FLOW: peak flows, flow prediction; TREATMENT PLANT DESIGN; EQUALIZATION FACILITIES DESIGN	13B

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