METROPOLITAN SEATTLE SEWERAGE AND DRAINAGE SURVEY

1956–1958

METROPOLITAN SEATTLE
SEWERAGE AND DRAINAGE SURVEY

CHARLES V. GIBBS

A REPORT FOR

THE CITY OF SEATTLE, KING COUNTY AND THE STATE OF WASHINGTON

ON

THE COLLECTION, TREATMENT AND DISPOSAL OF SEWAGE AND THE COLLECTION AND DISPOSAL OF STORM WATER IN THE METROPOLITAN SEATTLE AREA

MARCH 1958

BROWN AND CALDWELL
CIVIL AND CHEMICAL ENGINEERS
SEATTLE, WASHINGTON
SAN FRANCISCO, CALIFORNIA

METROPOLITAN SEATTLE SEWERAGE SURVEY
1331 3RD AVENUE, ROOM 415
SEATTLE 1, WASHINGTON
SENECA 2873

BROWN AND CALDWELL
CIVIL AND CHEMICAL ENGINEERS
66 MINT STREET
SAN FRANCISCO 3, CALIFORNIA

SURVEY STAFF
H. E. MILLER, PROJECT ENGINEER
H. E. BABBITT
F. J. KERSNAR
E. R. MCMASTER
W. M. ROMAN

May 19, 1958

Honorable Mayor and City Council
City of Seattle, Washington

Honorable Board of Commissioners
County of King, Washington

Pollution Control Commission
State of Washington

Gentlemen:

As provided under the terms of an agreement authorized by Ordinance No. 85375 of the city of Seattle on July 30, 1956 and executed August 29, 1956, we have completed the necessary engineering studies and are submitting herewith a report on a long-range program of sewerage and drainage improvements for the metropolitan Seattle area.

For convenience in presentation, the report is divided into six parts. Of these, Parts I and II are devoted to general, historical and background information; Part III to existing facilities and problems; Part IV to technical aspects of sewerage and drainage planning; Part V to alternative projects, stage construction programs, and financing procedures; and Part VI to a summary of findings and a list of recommendations.

As set forth in the report, construction of the proposed sewerage improvements is scheduled to take place in three stages, with the first stage to begin as soon as necessary financing can be arranged. Improvements called for under Stage I are estimated to cost a total of $83,215,000. This total includes (1) $46,810,000 for facilities which, primarily, will serve the city of Seattle; (2) $30,391,000 for facilities which will serve areas bordering the east and south shores of Lake Washington; and (3) $6,014,000 for small systems along Puget Sound and for temporary treatment works in isolated inland sections of the metropolitan area.

Because much of the metropolitan area is as yet undeveloped, it is not possible at present to prepare a comprehensive drainage plan for the entire area. To provide a basis for the planning of drainage improvements and for determining their approximate costs, unit costs were developed for each of five categories or types of areas. In that manner, it was estimated that the ultimate cost of constructing storm drainage facilities to serve the metropolitan area outside Seattle will approximate $145 million.

Seattle itself is faced with a continuing program of storm water separation to prevent overloading of local and trunk collection systems. An analysis of a representative number of areas indicated that necessary relief of combined sewers could be

achieved generally by partial separation. At an average estimated cost of $1,860 per acre, the total cost of partial separation will approximate $69 million. This cost, as well as all others given in the report, is based on present prices and makes no allowance for possible future increases.

A preliminary review of the financing problem indicates that the proposed sewerage improvements can be financed by means of revenue bonds, supported solely by sewer service charges. The extent to which present service charges will have to be increased for such a purpose will depend upon financial and related conditions prevailing within the various participating agencies and on market conditions at the time the bonds are issued. For household connections, the resulting increase in present service charges may vary from a modest amount to as much as $2.50 per month.

Finally, we are firmly convinced that the provision of service on a metropolitan and watershed basis represents the only practicable and certainly the most economical solution to the sewerage and drainage problems of metropolitan Seattle. We are equally convinced that such a program will require the formation of a central agency vested with authority to construct and to operate and maintain all of the proposed facilities.

Respectfully submitted,

BROWN AND CALDWELL

K. W. Brown

D. H. Caldwell

H. E. Miller

KWB/wb

PROJECT STAFF — BROWN AND CALDWELL

ENGINEERING

O. E. Albertson
H. E. Babbitt
R. H. Bogan
L. N. Hoag, Jr.
G. M. Jones
F. J. Kersnar
E. R. McMaster
H. E. Miller
J. T. Norgaard
D. A. Reinsch
W. M. Roman

DRAFTING

F. O. Bolton
A. H. Lee
A. C. O'Byrne
F. L. Parrick
J. J. St. Amand

REPORT PREPARATION

W. S. Bolton
M. M. FitzGerald
J. M. LeDieu
P. J. Ludke
A. J. Wilson

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PART I

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Chapter 1
INTRODUCTION

Through the medium of newspaper and radio publicity and frequently by direct experience, residents of Seattle and its surrounding communities have become acutely aware of the fact that the metropolitan area is currently faced with serious problems of sewerage and drainage. In addition to the significance of these problems from the standpoint of public health, it is now recognized that they involve such matters as individual safety and comfort, recreational activity, industrial productivity, and the value of land and property. If not resolved, sewerage and drainage deficiencies will become increasingly serious and will emerge sooner or later as a major obstacle to continued growth and development of the entire metropolitan community.

Sewerage and drainage problems are a matter of concern not only to the metropolitan area itself but to the state of Washington and the counties of King, Snohomish and Kitsap. The state is concerned because of its responsibility for the control of water pollution, and because metropolitan Seattle is its most populous, most productive and, economically, its most valuable area. King and Snohomish counties are concerned because they are direct contributors of sewage and drainage and because they are affected directly by any adverse conditions stemming from sewage and industrial wastes disposal operations in the metropolitan area. Kitsap county may be said to be a sideline observer, the interests of which will be affected detrimentally by any failure to resolve present problems. But the position of Seattle is unique. Aside from being the major producer of sewage and drainage, it lies across the major drainage outlets of the metropolitan watershed into Puget Sound and thus is subject to the terminal or cumulative effects of upstream conditions.

IMPORTANCE OF ENVIRONMENTAL WATER

The metropolitan Seattle area enjoys an abundance of diversified water resources unsurpassed anywhere. Large quantities of high quality fresh water in nearby mountain streams insure a plentiful and excellent supply for public use. Numerous fresh water lakes provide outstanding recreational opportunities and fisheries resources. Puget Sound, in addition to being an important recreational and fisheries resource, provides a protected navigational outlet to the sea and contains in its bays and estuaries excellent facilities for docking and shipping operations.

Shore lines adjacent to streams and lakes of the area and to Puget Sound provide excellent sites for homes and parks, for recreational pursuits, and for industrial and commercial developments. From the viewpoints, therefore, of the private citizen, of the business and industrial developer, and of the tourist, these waters represent a community asset of inestimable value. As a consequence, it is imperative that they be protected against degradation resulting from pollution and contamination brought about by discharges of industrial wastes and untreated sewage.

NEED FOR SEWERAGE AND DRAINAGE SURVEY

Among the many factors contributing to the sewerage and drainage problems of the metropolitan Seattle area, both present and future, the most significant are:

Growth of Population

Growth of population in the city of Seattle, and particularly in the surrounding communities is taking place at a rate such that the number of people residing within the metropolitan area can be expected to reach one and one-quarter million by 1980 and to be over two million in approximately 70 years. At the latter time the metropolitan sewerage service area is expected to exceed 500 square miles.

The present trend toward suburban living has extended the problem across political boundaries. As a result, the responsibility for providing sewerage service now rests with 41 separate jurisdictions, including 19 cities and 22 sewerage districts.

Raw Sewage Discharges

Although there are 25 sewage treatment plants in the metropolitan area, raw sewage is discharged through some 60 outfalls scattered along the shore lines of Duwamish River, Elliott Bay, and Puget Sound. Sewage from approximately 425,000 persons, or about 53 per cent of the total population presently residing in the metropolitan area, is discharged without treatment into these waters. As might well be expected, bacteriological analyses of the shore waters indicate that all beaches within the area are subject to dangerous contamination. It can be expected also that continued evidences of raw sewage discharges will have an adverse effect on the use of Puget Sound for boating, fishing and other recreational activities.

Raw Sewage Overflows

Due to summer rain storms, overflows of raw sewage into Lake Washington and Green Lake frequently occur during the recreational season. Studies indicate that, under average rainfall conditions, overflows occur more than 40 times per summer at each of about 30 points of discharge. Following these storms, virtually the entire west side of Lake Washington and all of Green Lake are rendered unfit for swimming.

In addition to the lake discharges, 30 other points of emergency overflow are scattered along the shores of Duwamish River, Elliott Bay, Lake Union, the Ship Canal, and Puget Sound. These overflows also discharge raw sewage following summer storms.

Lake Washington Pollution

Treated sewage from an estimated 80,000 persons now enters Lake Washington through direct discharges from ten community sewage treatment plants, and through indirect discharges from at least 4,000 private septic tanks. Biological and chemical conditions indicate that the lake is in the first stage of degradation due to nutrient enrichment, resulting in part from the discharge of sewage effluents and overflows. If this situation is not remedied, the inestimable value of Lake Washington as a recreational and scenic asset is likely to be greatly reduced, or perhaps even lost completely.

Duwamish River Pollution

Because of the discharge of raw and partially treated sewage and industrial waste, the Duwamish River is approaching the limit of its capacity to receive putrescible material. Tests indicate that the dissolved oxygen content of the river water is reduced at times to the minimum level considered satisfactory for aquatic life.

Suburban Sewerage Problems

About one-third of the total population residing in the metropolitan area is without public sewer service. To keep pace, therefore, with residential development, 6,000 private septic tanks are being constructed each year at a cost of approximately $2 million. Many of these installations are doomed to failure. Furthermore, gradual seepage from septic tanks is aggravating, and will continue to aggravate the already serious condition of nutrient enrichment of Lake Washington. It can be expected also to provoke a similar situation in other lakes within the metropolitan area.

Soil conditions in much of the suburban area are completely unsuitable for septic tank disposal. In situations of this kind, home building is prohibited until public sewers become available.

Combined Sewer Problems

Most of the older areas of Seattle are served by combined sewers which carry both sanitary sewage and storm runoff. Because of inadequate provision for storm flow capacity, these sewers become overloaded during periods of intense rainfall. Even at times of relatively light rainfall, many of them cause basement floodings and discharge sewage into streets through storm water inlets. It is not surprising, therefore, that every heavy rain results in numerous claims for storm water damage.

Metropolitan Aspects of Sewerage and Drainage

Sewerage and drainage problems of the metropolitan area are not restricted to individual cities and communities. Watershed boundaries, which define storm drainage areas and also limit sewerage service areas, are not confined to city, district and county lines. In other words, sewerage and drainage problems of the metropolitan communities are area-wide in scope and solutions must be formulated accordingly. This in turn implies that the construction and operation of trunk and interceptor sewers, major pumping stations, and treatment plants should be delegated to a single area-wide authority.

OBJECTIVES AND SCOPE OF SURVEY

Based on the problems outlined above, it is evident that there is an urgent need for the development of a comprehensive, long-range plan under which provision would be made for the systematic, orderly, economic, and properly integrated construction of necessary sewerage and drainage improvements. Such a plan, of course, calls first for a detailed engineering survey, taking into account all facts pertinent to the sewerage and drainage needs of the entire metropolitan area.

A survey to determine metropolitan needs should be concerned only with trunk and intercepting sewers, major storm sewers, main pumping stations, and treatment and disposal works. It should not, on the other hand, be concerned with local sewers, storm drains, and pumping stations which are not related directly to the development of a long-range metropolitan program. In general, the provision of local sewerage and drainage should be an independent function delegated to and retained in each political entity contained within the metropolitan area.

Objectives of Survey

An engineering survey should culminate in a detailed report setting forth in general, nontechnical language complete information concerning every phase of the required investigation. Such a report would serve throughout the period required for design of the recommended facilities and would be invaluable thereafter as a continuing reference.

Stated briefly, the principal objectives of the survey here reported are:

1. The development of a long-range sewerage plan for the metropolitan Seattle area, including such trunk and intercepting sewers, pumping stations, sewage treatment works, and outfall sewers or other methods of final disposal, as will be required to assure orderly and economic provision of needed services.

2. The development, insofar as permitted by existing conditions, of a basis of planning major storm drainage facilities throughout the metropolitan Seattle area, including such trunk drain lines, culverts, and combined sewer separation as will be required to provide for future development in each drainage basin.

3. The integration into the long-range sewerage and drainage plans, to the fullest possible extent, of all existing facilities found to be serviceable.

4. The protection of the shores and shore waters of Puget Sound and of Lake Washington and other inland waters, both surface and underground, from pollution, contamination and nuisance caused by discharges either of raw sewage, sewage plant effluents, or industrial wastes.

Scope of Survey

To attain the objectives outlined above, the work of the survey, as stipulated in the agreement cited later, included but was not limited to the following phases:

1. A review of existing reports and data, including previous reports on sewerage and drainage problems, planning reports, topographic surveys and maps, land use studies, climatological reports and data, and studies of lakes and estuaries having special reference to sewage disposal.

2. A study of drainage areas tributary to Lake Washington to determine the nature and degree of their development, both present and future. This phase also included areas tributary to Lake Sammamish, areas draining directly to Puget Sound, and other areas having natural drainage in the direction of metropolitan Seattle.

3. A study and analysis of population data and an estimate of population distribution and density, both present and future.

4. A study of the geographical characteristics of the metropolitan area as they relate to sewerage and drainage planning, including topography, geology, climate, natural resources, and economic and social development.

5. A study of existing sewage collection, treatment and disposal systems and the preparation of maps and diagrams showing the locations and functional details of all such systems. This phase was undertaken primarily for the purpose of determining the extent to which existing facilities could be incorporated, either at present or in the future, in a long-range program of sewerage improvements.

6. An analysis of the quantity and composition of sewages and industrial wastes now being collected in the area and a determination of the probable characteristics of those likely to be collected in the future.

7. A determination of present and future loadings on sewers, storm drains and treatment works, based on analyses of sewages and industrial works in the area and on experience and data secured elsewhere.

8. A study of the rates of storm water inflow and ground water infiltration into separate sanitary sewers, especially in the Lake City area, with the purpose of establishing realistic criteria applicable to the design of trunk sewers, interceptors, and treatment works.

9. A study of the effect of existing sewage disposal practices on the waters of Puget Sound, Duwamish River, and Lake Washington and on other waters in the metropolitan area.

10. A study of future disposal requirements to protect public health, to maintain receiving water quality consistent with its beneficial uses, and to prevent nuisance in the vicinity of disposal areas.

11. The development of design criteria and preliminary costs data for the design of sewerage and drainage facilities.

12. An investigation of problems concerned with the use of sewers carrying combined flows of storm water and sanitary sewage and, insofar as permitted by existing conditions, the development of a realistic, workable plan for storm water separation.

13. The development of all reasonable alternative sewerage and drainage projects and an analysis thereof as to physical and economic feasibility. This phase dealt with such factors as location, area served, population served, and sewage flow, and the location of trunk sewers, storm drains, interceptors, pumping stations and treatment plants.

14. An estimation of costs, both construction and annual, for sewerage and drainage facilities and a comparison of alternative projects.

15. The presentation of recommendations as to the most suitable long-range programs for both sewerage and drainage of the metropolitan area, together with preliminary plans and descriptions giving essential features of the proposed projects.

16. The development of a program of stage or incremental construction of both sewerage and drainage improvements.

17. A discussion of the various methods of financing construction and operation of the recommended sewerage and drainage improvements.

AUTHORIZATION OF SURVEY AND REPORT

On August 29, 1956, the engineering firm of Brown and Caldwell was engaged by the city of Seattle to undertake, in accordance with the foregoing objec-

tives, a comprehensive survey of sewage and drainage in the metropolitan area. The agreement then negotiated provides for financial participation by both the state of Washington, acting through the Pollution Control Commission, and the county of King. It calls also for completion of the project and submission of a report by March 1, 1958. Separate agreements relating to joint participation were subsequently negotiated between Brown and Caldwell and the state and county. Under all three agreements, payment for the work is on a time and expense basis, with the total fee limited to $130,000. Of this total, $90,000 is being contributed by the city, $30,000 by the county, and $10,000 by the state.

In the agreement with the city, it is stipulated that Brown and Caldwell shall prepare and submit 1,000 copies of a written report. This report is to present all information and data developed during the survey, together with descriptions of and estimates of costs for the recommended improvements.

FIELD AND LABORATORY WORK

Field and laboratory work was concerned primarily with the following activities:

1. A determination of the quantities of sewage and industrial waste from all parts of the metropolitan area. This phase of the work was marked by the use, for the first time, of a radioactive isotope technique which greatly simplified the difficult problem of flow measurement in large sewers. Continuous flow records at selected locations on trunk sewers in the city of Seattle were observed with the aid of pneumatic recording equipment developed specifically for the survey. Further details of these methods are given in Chapter 7.

2. A determination of the composition of sewages and industrial wastes from selected locations in Seattle and from outlying communities in the metropolitan area. Samples were collected at each location with the aid of one of two types of automatic sampling equipment. For small sewers and treatment plants, a rotating scoop-type sampler was employed which picked up a representative portion of the flow at ten minute intervals. For deep sewers, a new type of sampler was developed specifically for use during the survey. Detailed descriptions of both types are given in Chapter 7.

Samples collected over 24 hour periods were analyzed by survey personnel in the sanitary engineering laboratories of the University of Washington. Seattle engineering department personnel assisted in setting up the equipment for flow measurements and sampling.

3. A study of the waters of Puget Sound to determine current velocity and direction at selected points along the shore, and to determine the effects of existing sewage disposal practices, particularly in the vicinity of the point of discharge of the North Trunk sewer in Shilshole Bay. For this phase of work, use was made of a boat and crew furnished through the courtesy of the U.S. Coast Guard. Chapter 11 describes the results of the Puget Sound studies.

4. A study of the water of Lake Washington and of other lakes and streams in the area to determine the biological and chemical conditions of nutrient enrichment, and to determine the degree of pollution due to discharge of sewage effluents and raw sewage overflows. This study was carried out in conjunction with similar work being done by the University of Washington, Department of Zoology, under a grant from the U.S. Public Health Service. Samples were collected by survey personnel and university students and were analyzed in the sanitary engineering laboratories of the university. Although a great deal more still needs to be known about Lake Washington, sufficient information is now available to permit conclusions with respect to the effects of sewage effluent discharges and overflows.

5. A survey of the existing sewerage system within the metropolitan area, including trunk sewers, major pumping plants, treatment works and outfalls, or other means of final disposal. Results of this work are summarized in Chapter 6.

6. A field inspection of the routes of trunk sewers and of the locations of pumping stations, treatment plants and special structures. Reconnaissance surveys were made from the ground, by water, and from the air of all such routes and locations. In addition, foundation borings were made at the site proposed for the West Point treatment plant. Aerial reconnaissance and photography were performed through the cooperation of the U.S. Navy.

7. The collection, through numerous field trips, of information on development of the metropolitan area. This information served to establish the pattern and nature of urban growth, and to determine the logical extent of the future area to be served by the metropolitan facilities.

OFFICE WORK

Office work was concerned with the following principal activities:

1. A careful review of all reports, basic data and general information furnished by the staff of the Seattle engineering department, by other cities and districts, by consulting engineers, and by other agencies and individuals.

2. An investigation of the entire subject of present and future population development as to rates, distribution, and total numbers.

3. An estimation of sewage flows and treatment

plant loadings based on field quantitative measurements and on laboratory analyses of representative samples.

4. A determination of all necessary present and future trunk sewers with respect to the areas to be served, and their routes, capacities and sizes.

5. An investigation of sewage pumping in terms of pumping necessity, location, capacity and economy.

6. An estimation and review of all design factors.

7. The preparation of preliminary layouts for additional main sewers, treatment works and principal appurtenances.

8. The development and assembly of unit costs, and the preparation of necessary construction and total annual costs.

9. The preparation of the final report.

INFORMATION AND DATA AVAILABLE TO SURVEY

Existing reports, maps, plans, specifications, and statistical information relating to the various cities and districts in the area were furnished by city and district officials. Personnel of the Seattle Department of Engineering and engineering personnel of other cities and districts assisted in every way possible. Nevertheless, a great deal of time had to be spent by the survey staff in obtaining from the city engineering files necessary information concerning the existing Seattle sewerage system.

Other public agencies within the metropolitan area have generously made available reports, maps, files and other data. Particularly helpful material was furnished by the Seattle Planning Department, King County Planning Department, Snohomish County Planning Department, Seattle-King County Health Department, State Pollution Control Commission, and State Department of Health. All consulting engineering firms in the area concerned with sewerage graciously cooperated by releasing information and plans from their files and by furnishing facts based on their knowledge of local conditions. Soils engineering firms and many industrial firms cooperated by releasing the results of soil and foundation investigations which had been performed by and for them in the past.

Information was obtained from a number of federal agencies, particularly the U.S. Geological Survey, U.S. Coast and Geodetic Survey, U.S. Weather Bureau, U.S. Census Bureau, U.S. Engineer Corps, U.S. Navy, and the U.S. Public Health Service. Valuable data were obtained also from the Department of Oceanography, the Bureau of Governmental Research and Services, the Department of Civil Engineering, and the Department of Zoology, all of the University of Washington. Other state agencies furnishing information included the Department of Fisheries, Department of Game, Department of Employment Security, Highway Department, Census Board, and the Office of the Secretary of State.

PROGRESS REPORTS

Verbal reports on the progress of the survey were made from time to time to the sponsoring agencies as well as to quasi-official, technical and lay groups. Two reports were presented to the Mayor and City Council of Seattle during 1957, the first in April and the second in October. Additionally, two reports were made to the Mayor and members of the Streets and Sewers Committee, the first in November 1956 and the second in February 1957. A progress report was given also to the State Pollution Control Commission in May 1957. In the same month, a detailed description of the progress being made was presented to the members of the Metropolitan Problems Committee meeting jointly with officials of communities and agencies throughout the metropolitan area.

ACKNOWLEDGEMENTS

The task of assembling and compiling the facts and figures required for a report of the magnitude here presented involved the work of many persons. In addition to the survey staff, numerous individuals contributed, either directly or indirectly, to the preparation of the necessary material.

It is difficult, in retrospect, to recall everyone who, in some measure, contributed to the survey and report. We trust, however, that the list of acknowledgements contained in the appendix is reasonably complete. The help given us by the individuals there listed is deeply appreciated.

Chapter 2
HISTORY OF SEWERAGE PROBLEM

White settlers began arriving in the area that is now Seattle only a little over 100 years ago. Requiring little in the way of public services, they took water from the nearest convenient sources and disposed of sewage by primitive means.

As the settlement grew, water sources became contaminated and nuisances developed. By 1865, the population had reached 300 and the need for public control had become so imperative that the territorial legislature incorporated the little community.

A board of public works, created by Ordinance Number 4 of the new municipal council, undertook construction of the first public sewers in the area. Lacking any semblance of a "system", these early sewers consisted of wood troughs or boxes, which discharged individually at the most convenient point, usually into Elliott Bay and Lake Union. It was not until 1875, by which time the population had increased to about 1,500, that planning and construction of sewers were commenced on an organized basis. Bonds for financing construction were passed the following year but, because the validity of bonds voted under a territorial franchise was questioned, nothing was accomplished until enabling legislation was enacted by the United States Congress in 1882.

The first sewer of more or less permanent character was constructed in 1883. Made of "iron stone", a mixture of clay and iron slag, this sewer was laid in Madison Street from Fifth Avenue to Elliott Bay. Vitrified clay pipes, 12 inches in diameter, were first laid in 1885 when the population of Seattle had grown to over 5,000. By that time, local nuisances had become widespread and pollution of near-shore waters, especially in Lake Union, had become a serious community problem. Finally, the situation became so serious that the city council obtained the services of Colonel George E. Waring, Jr., a sanitary engineer of national repute, to design a comprehensive system of sewers.

The Waring Report

Submitted to the city council in March 1889, Colonel Waring's report presented a plan for a comprehensive system of separate sewers designed to carry domestic sewage only, with no provision for flow resulting from rainfall. On this subject the report stated:

"The system of sewerage adopted is arranged for the removal of foul wastes only. That is to say, it is to receive no rain water from any source whatever. It was found on estimating the sizes needed for carrying roof water that this would add materially to the cost of the work.

"You are so well situated for the removal of surface water that it is not worth while to spend public money for increasing the facilities unless possibly hereafter with reference to certain localities where storm water may accumulate to an inconvenient or dangerous degree."

An important feature of the sewerage problem, as established by the Waring report, was the need for the construction of a tunnel, later known as the Lake Union tunnel, to prevent the discharge of sewage into Lake Union.

Shortly after receipt of the Waring report, interest in public affairs was distracted from normal channels by the great fire of 1889, which destroyed a large part of the city. Preoccupied then with reconstruction of the city, public attention was diverted temporarily from further consideration of the sewerage problem. In the end, the Waring report was rejected, apparently because the proposed sewers were believed to be of insufficient capacity.

Renewing its effort to find a solution to the growing problem, the city council, on November 18, 1889, retained by Benezette Williams, a consulting engineer from Chicago, to prepare plans for a comprehensive sewer system. At the time, Williams was working also on plans for a water supply system for Seattle.

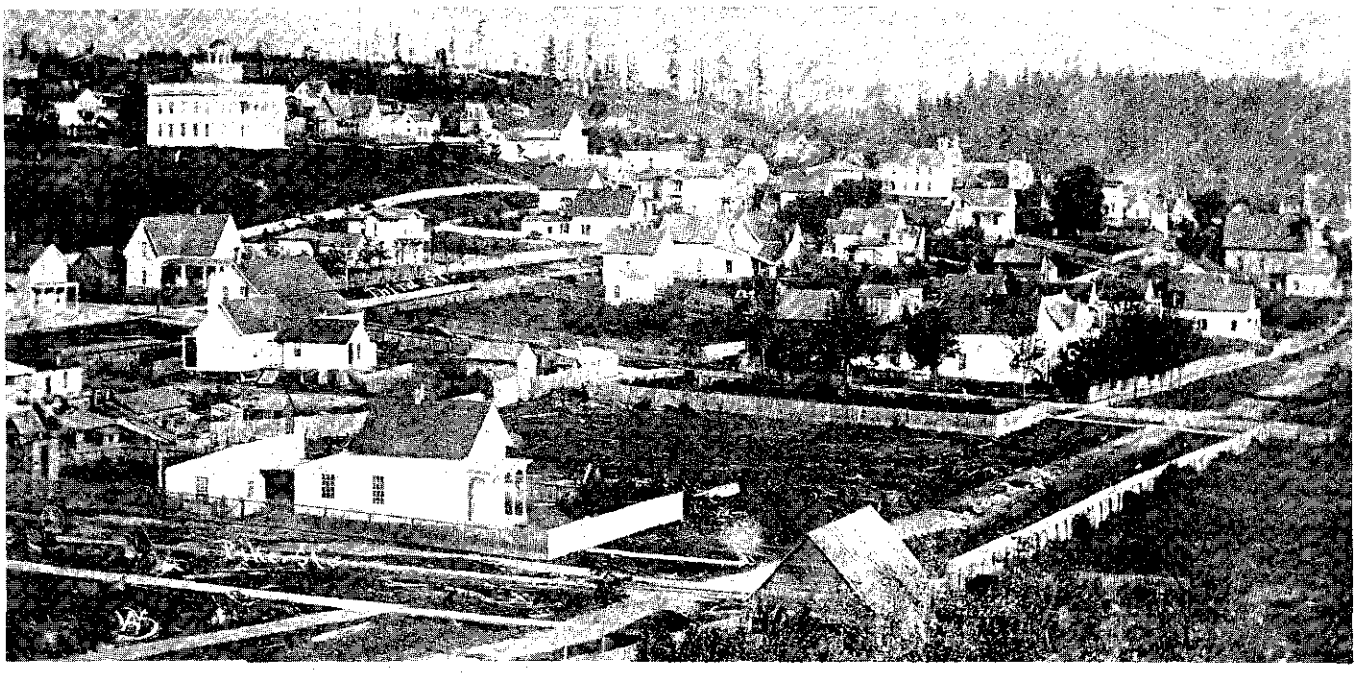
In 1890, while Williams was preparing his report, Washington was admitted to the Union and the new State Legislature granted a municipal charter to the city of Seattle. This charter was adopted at an election held on October 1, 1890.

That the sewerage problem must have been one of the first concerns of the newly chartered city is evidenced by the following statement contained in the 1890 annual report of the Honorable Harry White, Mayor of Seattle, to the City Council:

"Your attention is called to the question of providing additional sewerage facilities especially in the vicinity of Lake Union. I would recommend that a committee be appointed by the Council...and that said committee...report...whether the sewerage from that portion of the city can be disposed of without using Lake Union for the purpose."

The Williams Report

The Williams report was received in August 1891. In contrast to the Waring proposals, Williams recom-



SEATTLE in 1878, about the time the city's first sewerage system was being planned. Street intersection in foreground is 2nd and Pike Streets. The large building at upper left was the University of Washington located on the present site of the Olympic Hotel.

mended a system of sewers to carry both sanitary sewage and storm runoff, thus necessitating pipes and conduits of much larger capacity. Domestic sewage and low storm runoff were to be diverted from the Lake Union, Green Lake, and part of the Lake Washington drainage basins by means of three major systems, each discharging into salt water. During storms, surface runoff mixed with sewage would be discharged to adjacent waterways through a number of overflow and bypass structures located along the principal trunk and intercepting sewer lines.

While the Waring and Williams philosophies were in direct conflict from the standpoint of separate versus combined sewers, it is interesting to note that their proposals were in agreement in one significant respect. Both urged construction of the Union Lake tunnel.

Combined Versus Separate Sewers

The decision to use combined instead of separate sewers in Seattle has had, and will continue to have, an important bearing on Seattle's sewerage problems and the pollution of its surrounding waters. It is desirable, therefore, to review the reasons for the Williams recommendations. These reasons are of particular interest because more recent advisers have recommended separation of the sewer system, at least insofar as existing conditions will permit.

Williams pointed out that surface soils are constantly soaked to saturation during a considerable part of the six rainy months. This condition, together with steep surface slopes, results in rapid runoff and high concentration of storm water in gutters and at street intersections. In his report, Williams stated:

"In short, the accumulated experience of all populous cities and towns leaves no chance for being mistaken in the assertion that the underground removal of storm water is a necessity in a modern city. Any attempt to dispense with it is a retrograde movement, and one not to be tolerated at the present day.

"If sewage and storm water are not removed together by means of a combined system of sewers, underground conduits will have to be provided for rain water, as well as sewers to be used exclusively for organic waste, thus substituting a double system for a single one, and materially increasing the cost.

"Much has been said in advocacy of complete separation of sewage and storm water...but nevertheless the stubborn fact remains that in all cities of large size, it is considered necessary to remove storm water by underground conduits, and in nearly all such cases it has been found most convenient, and the cheapest to do it by means of a combined system of sewers.

"The sanitary plea that is often urged in behalf of the separate system...cannot properly be held sufficient to outweigh the many advantages afforded by the combined system.

"...If a system of sewers is built so small as to be adapted merely to the removal of house sewage, it will be overcharged in a few years, through storm water connections, made perhaps without the knowledge of the city departments."

Treatment of sewage for the prevention of water pollution was not practiced in 1890, and little or nothing was known about it at that time. Probably neither Waring nor Williams fully realized that treatment ultimately would be required. Certainly Williams did not mention the additional cost which would be entailed should it become necessary to intercept and pump sewage combined with storm runoff, and convey the combined flow to treatment sites at some distance from the various outfalls.

Williams was aware of the nuisances which were bound to result from the discharge of sewage into near-shore waters. His report, in the remarks quoted below, recognized that construction of a comprehensive sewer system would probably result in widespread pollution.

"Of the 28 square miles within the city limits, about 10 square miles drain naturally to the salt water Sound, and 18 square miles to the fresh water lakes, Washington, Union and Green. Of the fresh water drainage area, approximately 8 square miles drain to Lake Union and 10 square miles to Lake Washington directly and indirectly.

"It is thus seen that following the natural topography the sewage from about 64 per cent of the city's population would go into fresh bodies of water where it would stagnate, and as it grew in magnitude would become offensive in every way. To prevent this as far as possible, should be one of the main purposes in outlining a sewerage plan.

"As the sewage of the city increases it will become more and more of a problem to dispose of it along the waterfront without creating a nuisance, and the fewer the points of outfall the more readily can some system be adopted to throw the sewage far out in the Sound should it become necessary.

"...When the quality of sewage shall have become large, this stranding (on the beach) of the sewage may become objectionable. It is also a question whether the Bay itself for a considerable way from shore, will not become offensive in the course of years. This, however, cannot be told with any degree of assurance without a full knowledge of the currents occasioned by the ebb or flow of the tides."

Time has proven that William's fears over the possible consequence of discharging sewage along the shorelines were well founded. How best to resolve this problem, in view of the large shows which are experienced during every storm, has perplexed city officials and engineers for nearly fifty years.

Nevertheless, under conditions of sanitary engineering practice in 1890, Williams arguments were convincing and his recommendations were approved. Seattle, therefore, started to construct combined sewers.

Early Trunk and Intercepting Sewers

Construction of the existing Seattle sewerage system may be said to have commenced coincident with the preparation of the reports by Waring and Williams. By the end of 1891, about 14.9 miles of sewers had been constructed, ranging from 8 inches to 20 inches in diameter. The rapidity of the subsequent growth of the sewer system is indicated by the following tabulation of the miles of sewers which had been constructed up to the years indicated:

1891	14.9 miles
1900	60.45 miles
1908	212.32 miles
1924	628.63 miles
1930	802.10 miles
1940	863.15 miles
1950	988.09 miles
1956	1059.59 miles

Full responsibility for the construction of sewers was placed in the hands of R. H. Thompson who, except for a brief period in 1894, served as city engineer from 1892 to 1911. Following in general the plans laid out in the Williams report, Thompson directed the design and construction of major sewers and sewer tunnels which to this day comprise the "backbone" of most of the system. These projects included the Lake Union tunnel system, most of the North Trunk sewer, and the Beacon Hill tunnel system and marked the first important steps in the protection of the fresh water bodies of the area. But beneficial as they were in diverting dry weather flow from Lake Union, Green Lake and parts of the Lake Washington drainage basin to tidal waters, they aggravated conditions along the Elliott Bay waterfront and created a new problem in Puget Sound proper.

Concern regarding the discharge of sewage from numerous small systems along the Elliott Bay waterfront was expressed in the city engineer's report of 1901 which stated:

"More than six years ago sewage from a large portion of Renton Hill was carried down Twelfth Avenue to Lane Street, where it is turned to the west and is discharged at the foot of Lane Street into the waters of Elliott Bay...The sewage cast upon the flats at the foot of Lane Street becomes extremely offensive, and will soon be the occasion of public uprising unless some steps are taken to abate the nuisance."

Although corrective steps have been taken from time to time, discharges of raw sewage into near-shore salt waters have increased steadily and today constitute principal problems. Among the most serious are the discharges from the major systems constructed in the early 1900's, namely, the North Trunk sewer to the sound in the vicinity of West Point, the Lake Union tunnel system to Elliott Bay near Denny

Way, and the Rainier Valley system to the Duwamish River at Harbor Island.

While these early and important developments were taking place in Seattle, events of historical significance were occurring elsewhere in the area. In Ballard, which was not a part of Seattle until 1907, four large trunk sewers with outfalls into Salmon Bay were constructed during 1903 and 1904. Although these sewers still serve the Ballard district, an interceptor was constructed in 1937 and the dry weather flow is now conveyed to the North Trunk sewer through a siphon laid along the bottom of Salmon Bay.

In 1910, construction of sewers commenced in the cities of Auburn and Renton. Discharging, respectively, into the Green and Cedar rivers, these were the first of many systems to follow which would discharge to the fresh waters of the area at numerous and scattered points. Meanwhile, the principal developments continued to center in Seattle where the sewerage system was being expanded rapidly throughout the city. Much of this expansion consisted of small systems with independent discharges.

In 1920, a major intercepting sewer was constructed along Henderson Street from Forty-Second Avenue South to its point of discharge into Lake Washington. This line served the lower Rainier Avenue and Empire Way areas and, upon its completion, provided the developed sections of Seattle a moderately adequate network of sewers.

Overloaded Sewers

Because of the rapid growth of the city, and the attendant extension of tributary areas served by the various combined sewer systems, many trunk sewers soon became overloaded. As a consequence, numerous problems were subsequently encountered and many opinions, expert and otherwise, were frequently expressed. Here, briefly, are some typical examples:

1. A flow measurement at an overflow manhole on Roanoke Street on January 5, 1914 showed a depth of flow of over two feet, indicating that the entire length of the main sewer was overloaded from the Lake Union tunnel to Roanoke Street.

2. In a report to the city engineer made in 1921, it was stated that, on September 8, 1916, the Lake Union tunnel backed up and that water flowed from the manholes in Roy Street.

3. A report in 1927, prepared in the office of W. D. Barkhuff, city engineer, listed a large number of inadequate sewers in the city and called for "immediate relief either by sewers paralleling the existing sewers or by the construction of a single sewer of the proper dimensions."

4. In a letter dated September 27, 1927, A. H. Dimock, who had previously served as city engineer for 12 years, commented upon a recommendation regarding relief of overloaded sewers: "A more realistic approach now will be to start providing separate storm sewers."

5. In 1928, W. D. Barkhuff, city engineer, recommended "the construction of a separate system of storm water sewers to give the necessary relief for present and future development."

6. Dr. Abel Wolman, in his report to the City Council in 1948, recommended "The policy should be promptly adopted. . . . prohibiting any further extension . . . . of combined sewers. The continuing installation of combined sewers will aggravate the disposal problems. . . . when excessive amounts of storm water, sullied by domestic sewage in combined sewers, will create major fiscal problems."

Numerous other engineers, employed by the city and in private practice, have expressed themselves as favoring either complete or partial separation of the Seattle sewer system.

The Lake Washington Interceptor System

Although the North Trunk and Bayview tunnel systems were designed to divert the dry weather flow from parts of the Lake Washington drainage basin, they did not, initially, serve areas lying along the easterly slope of the ridge in Seattle which extends along practically the entire Lake Washington shoreline. Consequently, a number of small systems was constructed in those areas, each discharging raw sewage into the lake. Several large trunks, all with lake discharges, were also constructed. Others were added from time to time until there were 30 raw sewage outfalls to the lake by 1922.

To correct these undesirable conditions, a plan was devised during the term of J. D. Blackwell as city engineer which called for construction of five pumping plants and a system of intercepting sewers which would divert some of the flow from the Lake Washington basin to salt water. This plan also included a new tunnel under Hanford Street to relieve flooding and backups, which were occurring frequently in Rainier Valley because the capacity of the Bayview tunnel was no longer sufficient to serve the tributary area. Since, however, the estimated cost of $4,200,000 was considered to be too high, an alternative plan was developed by the city engineer. The latter contemplated the construction of 16 sewage treatment plants along the lake front which, with their connecting sewers, were estimated to cost about $2,500,000.

In reviewing both plans, the State Department of Health wrote to the city of Seattle in August 1922, stating among other things:

"Accordingly, if it is contemplated to construct a combined storm and septic sewer system, the treatment works should be adequate to take care of all the effluent of the sewer, including both septic and

storm sewage; but if it is preferred to build two separate systems, one for the storm water and one for the septic sewage, the storm water will not have to be treated.

"This Department cannot permit any increase in the pollution of Lake Washington which affords the only possible water supply for certain settlements on its shore..."

Despite what appears to have been a valid stand on the part of the Department of Health, permission was granted subsequently for the construction of plants to treat combined sewage. Three such plants, comprising Imhoff tanks only, were constructed in 1924, one each at Perry Street, Alaska Street, and Massachusetts Avenue.

A resolution of the King County Board of Health, dated October 20, 1925, stated that from and after November 1, 1925 any sewage discharged into Lake Washington must be so purified that it will not contain the germs of human diseases, and will conform with the United States Public Health Service bacteriological standard for drinking water. Meanwhile, however, the State Department of Health had, in effect, withdrawn from its earlier stand by approving plans for sewage treatment plants which were incapable of meeting the demanded standard.

By Resolution Number 2005 of October 1925, the King County Board of Commissioners condemned the use of Lake Washington for the disposal of sewage. This resolution also required that any effluent discharged into the lake should meet the standards of the United States Public Health Service for drinking water.

In 1925, strong complaints about nuisances arising from the plants already constructed were expressed by residents of nearby areas. Finally, conditions became so offensive that suit was brought to enjoin their operation. With public sentiment turned against these plants, a bond issue of $2,125,000 was voted on March 6, 1926 for the construction of intercepting sewers and pumping stations to divert sewage from the lake, apparently along the lines first recommended by Blackwell.

While the above controversy was in progress, the city council employed A. H. Dimock to develop a plan for abating pollution of Lake Washington. A statement contained in his report of May 17, 1926 bears repeating in the light of subsequent developments:

"The population of Seattle will soon overflow its present boundaries both north and south. It will jump across the lake and its eastern shores will become urban property. And, coincident with the growth of Seattle, there will be a corresponding growth in population and industries throughout the entire habitable watershed. The area to the north and northeast, together with other tracts will become suburban property. This area will comfortably house, without crowding, some 2 or 3 millions of people. This will be a long time ahead, but it is manifest that an increasing population will deliver a constantly increasing amount of contamination to the lake.

"It is my opinion, therefore, that sewage even after treatment should not be disposed of in the waters of Lake Washington within the limits of the city of Seattle, but should be removed completely..."

Estimated to cost $3,000,000, the plan recommended by Dimock called for an intercepting sewer system along the shoreline of Lake Washington from twelve pumping stations, and for a tunnel through Beacon Hill under Hanford Street to replace the Bayview tunnel. Interceptors were to be constructed in three separate sections: (1) a northerly section with pumping stations to lift sewage into the North Trunk system; (2) a central section with pumps to lift the flow into the Hanford Tunnel system; and (3) a southerly section with pumps to lift flow to a new Henderson Street system which would convey it to the south end of Beacon Hill where construction of a treatment plant was contemplated. Storm water overflows were to be provided at intervals along the interceptor system and the existing Henderson Street intercepting sewer was to become an overflow outfall. After considerable controversy over the relative merits of the various plans that had been proposed, the Dimock plan was adopted and has since been followed to a substantial degree.

The principle of eliminating and preventing the discharge of sewage into Lake Washington had now become so well established that by 1936 all outfalls to the lake had been intercepted. Storm overflows, however, could not be avoided and continue to occur at some 30 scattered points along the lake, even during relatively light rainfalls.

Final Stages in Development of Existing Sewerage System

At the time the south Lake Washington intercepting sewer system was being planned, it had become apparent that an interceptor was needed also to protect the waters of Duwamish River. Under the Dimock plan, it was contemplated that dry weather flow from the new Henderson Street system would ultimately be treated at a plant south of Beacon Hill. In view, however, of the concurrent need for an interceptor along the Duwamish, a plan was devised whereby the Henderson Street sewer would be routed around the south end of Beacon Hill to a junction with the proposed Duwamish interceptor. The latter would then convey sewage along East Marginal Way to a treatment plant at Diagonal Avenue. This change was approved and construction of the system was completed about 1940.

Design of the East Marginal Way interceptor was based on serving the Henderson Street system only

until its full capacity would be required by developments along the Duwamish. It that event, the plan was to construct the proposed treatment plant south of Beacon Hill.

Sewerage construction slackened during World War II, being confined principally to the provision of facilities needed to serve war housing areas. In 1945, a major program of sewer system extensions was recommended by the office of the city engineer. Involving construction over the following ten years and a bond issue of $3,000,000, this program was approved by the voters at a general election.

As the volume of raw sewage being discharged and the number of points of discharge increased, state and local health authorities, the Pollution Control Commission, and other state and local agencies interested in water resources expressed increasing concern. Numerous surveys of conditions in the waste receiving waters were conducted, out of which came warnings regarding the growing menace to the public health and the urgent need for corrective action. For instance, a letter from the State Department of Public Health, dated July 12, 1945 stated in part:

"In the spring of 1941 and again in 1944 extensive bacteriological investigations were made of the waters at Alki and Golden Gardens bathing beaches. These studies were made by the State and City of Seattle Health Departments. Conclusions of both investigations were that the beaches were seriously polluted. The outfall sewers mainly responsible for these conditions were the Ballard and North Trunk at Golden Gardens; and Jersey Street, Arkansas Street, and Fifty-Third Avenue, southwest at Alki.

"These bacteriological studies revealed coliform bacteria concentrations as high as 10,000 per 100 c.c. and average values of over 1,000 per 100 c.c. Although no national standard has been adopted establishing the maximum limit of quality of water in which it is safe to swim, it is generally indicated by the several standards in use that 50 coliform organisms per 100 c.c. is desirable, but in a few instances, upper limits of 1,000 per 100 c.c. have been set to permit swimming when other quality of water was unavailable. Therefore, it is indicated that the public bathing at Alki and Golden Gardens bathing beaches have been made unsatisfactory by the discharge of city of Seattle sewage.

"While only two established city bathing beaches have been discussed, all the salt water bathing beaches within the city limits of Seattle must also be considered, on the basis of their location with reference to city sewer outfalls, as unsatisfactory for bathing purposes.

"Other factors of concern are: There are continual complaints arising from floating solids, scum and slick throughout all of Elliott Bay and extensive stretches of the Sound, depending on tide conditions. Also, preliminary studies of the Duwamish Waterway reveal that at times the dissolved oxygen is depleted to such an extent that fish life cannot exist. This situation is attributable in part to the city of Seattle sewer outfalls into these waters."

The Wolman Report

Manifestly unsatisfactory sewerage conditions and a lack of unanimity in proposals for their solution led the city council, in 1947, to authorize the city engineer to engage expert advice. The services of Dr. Abel Wolman, Professor of Sanitary Engineering at Johns Hopkins University and a widely recognized authority on sanitary engineering matters, were then obtained.

Dr. Wolman submitted his report on September 25, 1948. A condensation of his recommendations follows:

"The capacity of the North Trunk sewer should be increased to carry three times the dry weather flow, without overflows, and its outlet should be extended into a depth not less than 75 feet. All sewage, before discharge into the Sound, should be subjected to fine comminution. Whether grease or solids should be removed should wait on results with comminution. Land should be purchased for the installation of comminution facilities and plain settling.

"All sewers discharging into Elliott Bay should be extended into deeper water.

"All sewers into Puget Sound at the south of Alki Beach should be extended into at least 35 feet of water. For the time being comminution is not recommended for these areas.

"The policy should be promptly adopted of prohibiting any further extension of combined sewers.

"An industrial waste survey must be instituted and maintained.

"Vigilant administrative checking should be conducted to detect on-shore pollution as a result of breaks in outfalls, over-balancing of assimilative capacities, and other indices of good or bad performance.

"A master plan for progressive sewer extension and for controlled sewage disposal should be developed for Seattle and those environs, outside the city limits, which would normally empty into the same surface waters."

The Wolman report was approved in a letter from the city engineer to the city council on November 17, 1948. This letter stated: "This department is in hearty accord with Dr. Wolman's findings. This department recommends: That Dr. Wolman's report be adopted as the future policy governing our sewage disposal."

Approval was expressed also by the Director of the State Department of Public Health who, in a letter addressed to the city council on December 20, 1948,

stated: "We see no valid technical reasons, public health, engineering or otherwise, why these recommendations should not be acceptable to all concerned."

In November 1948, a brochure was published by the State Pollution Control Commission in which the Wolman report was severely criticized. The following quotations are indicative of the tenor of the publication:

"It was the understanding of the Pollution Control Commission that the purpose of the Wolman Report was to provide the city of Seattle with a basis for future planning for sewage and industrial waste collection, treatment and disposal, and to furnish detailed information as to methods of accomplishment. The report does not give any of the expected details. The basis for future planning seems to hinge on the interpretation of the recommendations. Some first interpretations have appeared in the newspapers to the effect that Seattle will never need to provide sewage treatment facilities. Since this is a far reaching conclusion and affects not only Seattle but all of the salt water areas in the state, it appears advisable to consider all of Wolman's recommendations and conclusions and not only those which appear to recommend no treatment.

"It will be the requirement of the Pollution Control Commission that the minimum acceptable degree of treatment for the sewage and industrial wastes of the city of Seattle will be primary treatment."

Public officials, civic groups and the citizenry were disturbed and confused by the conflict of opinions and recommendations. Some urged the appointment of a board of engineers to advise the city council on a solution.

In an effort to resolve differences and to provide a basis for action, the city engineer commenced an investigation of overload, overflow and pollution problems and the development of a long term program of improvements. At the same time, the Pollution Control Commission undertook investigations to evaluate the extent and effect of pollution and nuisances in Puget Sound.

The Sylvester Report

In 1949, the Pollution Control Commission employed Professor R. O. Sylvester and associates of the Department of Civil Engineering, University of Washington, to conduct a comprehensive survey of pollution conditions in Puget Sound and its tributaries. Among other things, the report on this survey discusses (1) the various beneficial uses of the waters of the Sound, Elliott Bay, Shilshole Bay, the Ship Canal and the Green-Duwamish River; (2) the various points where wastes were being discharged into these waters; (3) the conditions found as they relate to the beneficial uses; and (4) the suitability of possible sites for waste disposal.

Details as to the conditions in the receiving waters, as described by Silvester, will be set forth elsewhere in this report. As a matter, however, of historical interest, it is pertinent here to include the following summary of some of the more important findings:

"1. The biochemical conditions of the Lake Washington Ship Canal system is not satisfactory.

"2. All of the bathing beaches surveyed at times show sewage pollution (presence of coliform organisms) to be in excess of the standard established by the State Pollution Control Commission and the State Department of Public Health.

"3. The following beaches would not be recommended for bathing...due to either high coliform counts or the observed presence of sewage materials in the beach area: Carkeek Park..., Ballard Beach, West Point Beach, Magnolia Bluff in the vicinity of Thirty-Second Avenue West, the major portion of Alki Beach, the major portion of the entire beach from Alki Point south to the city limits, and the beach at Salmon Creek.

"4. The Green-Duwamish River is receiving heavy discharges of polluting materials from Auburn north to its confluence with Elliott Bay. The bacteriological condition is very poor in the river stretch past the city of Kent. In the Duwamish Waterway the dissolved oxygen depletion...is particularly severe during...August and September.

"5. In spite of the extension of eight Seattle outfalls, the conditions of beach pollution are not satisfactory."

Recent Reports of the City Engineer

One of the most urgent problems pointed up by the Wolman and Sylvester reports was the pollution of West Seattle recreational beaches. In 1948, following refusal of the Pollution Control Commission to grant permission for the construction of a storm water overflow device in West Seattle, the city agreed to proceed with planning for an interceptor system and a sewage treatment plant which would eliminate the discharge of raw sewage from numerous independent systems. Three years later, in 1951, the city engineer presented a predesign report for this project.

In preparing the predesign report, careful consideration was given to the Wolman and Sylvester reports and to tentative agreements reached with the State Pollution Control Commission. Based on these and other pertinent studies, it was recommended that an area of 3,843 acres be served by two interceptors with a total length of about five miles, by seven pumping stations, and by a primary type treatment plant to be constructed at Alki Point. Raw sludge removed in sedimentation tanks at the treatment plant was to be mixed with salt water and pumped at a slow rate through an outfall separate from the effluent outfall.

Under this plan, sludge was to be pumped into a depth of 100 feet of water at low tide and at times of favorable current conditions in the sound. Disposal of sludge in this manner was considered to be experimental and was to be tried for the purpose of determining the validity of the Wolman recommendations for deep-water assimilation of raw sewage. As matters turned out, the proposal ended in controversy and construction of the West Seattle system was delayed.

In 1952, the city engineer issued a report entitled "Planning and Progress, Seattle City-Wide Sewage Disposal Problem". This report points out that "Subsequent to 1949, and with the aid of 1946 sewer bond funds, an intensive sewer rehabilitation program has been undertaken by the Engineering Department to overcome the decline in progressive sewer planning for the city that set in with the depression of the 1930's and reached its lowest point during the war period (1941-1945). To overcome the recession period, and to modernize the system, the spending of approximately $25,000,000 will be necessary to create a network of sewers to serve the sanitary requirements of Seattle's growing population."

Information is presented regarding results of sewage characteristic and pollution surveys, studies of overflows into Lake Washington, and studies of sewer capacities. On the subject of sewer capacity, it is said:

"These studies have revealed some startling facts that tend to account for failures in the past, and are now the basis for not allowing sewer permits to be issued for connection to systems already overloaded. This policy in the past has piled trouble onto existing trouble, when new relief sewers should have been constructed. The results of this survey will help to reduce claims against the city for damages from sewer backups into basements and save maintenance funds from being wasted on temporary repairs to undersized sewers."

A further effort to initiate necessary action came in 1954 with the holding of an election to authorize issuance of $5,000,000 in general obligation bonds for financing construction of sewerage additions and improvements. This program, however, failed to receive voter approval.

With reference to new construction, the 1952 report summarizes a proposed program as follows:

"The program, which is planned for progressive construction stages over a 10-year period, will involve the construction of 29 pumping plants with their associated force mains, to provide new low level, shoreline sewer interception to existing higher sewers, approximately 8 miles of new interceptors and replacement sewers to concentrate existing sewers in conformance with this plan, and a minimum of four sewage treatment plants."

Three new treatment plants were to be constructed, one each at Alki Point, Fort Lawton and West Denny Way. In addition, the existing Diagonal Avenue plant was to be enlarged.

The 1956 Bond and Service Charge Election

In 1955, the city council passed an ordinance which provided for extensive improvements and additions to the sewerage system and the issuance of revenue bonds in the amount of $6,250,000. Another ordinance provided for a sewer service charge, setting a fixed fee of $1.00 per month for a single family residence and a fee for larger contributors based on water use. These actions were approved by the voters at an election held on March 13, 1956. As a result, the service charge is now in effect and the proposed improvements and additions are in various stages of design and construction. Among these are:

1. Construction of the West Seattle interceptor system and sewage treatment plant. This program involves an intercepting sewer with sufficient capacity to limit the number of overflows to twelve per summer. The sewage treatment plant, employing primary sedimentation and separate sludge digestion, is being constructed at Alki Point as originally proposed.

2. Construction of an interceptor to eliminate raw sewage discharges in the vicinity of Golden Gardens.

3. Enlargement of the Lake City Sewage Treatment plant, and improvements to other systems located in the area between Eighty-Fifth Street and One Hundred and Forty-Fifth Street, which was annexed to the city in 1954.

Growth and Problems of the Suburbs

Engineering reports dealing with Seattle's sewerage problem have given increasing attention to the relationship between the problems of the city and those of the outlying areas. Between 1890 and the beginning of World War II, eight communities were incorporated within what is now the metropolitan Seattle area and much of the unincorporated fringe area surrounding the city became heavily populated. During the war, a sharp growth in population occurred and has since continued.

As is the case in most of the major population centers in the United States, recent heavy growth has been coupled with a desire on the part of people to live in the suburbs. In consequence of that trend, large areas to the north and south of Seattle, and east across Lake Washington, have become heavily developed. Since the war, ten cities have been incorporated within a 15 mile radius of downtown Seattle and the unincorporated area within this radius has become urbanized to such an extent that the demand for public sewerage service has brought about the formation of

22 sewerage districts. As a result, 26 independent sewerage systems and as many sewage treatment plants now operate within the metropolitan Seattle area. Yet, nearly a third of the population is without public sewerage service. Furthermore, because of unfavorable soil conditions, there are frequent instances in unsewered areas when household wastes flow over the ground surface and along street gutters or stagnate in pools.

Problems of the unsewered areas are discussed in a report published in 1952 by the Pollution Control Commission entitled, "The Sewage Disposal Problem in the Seattle Metropolitan Area". This report states in part:

"The movement of people from cities to suburban areas has created a multitude of problems. Included among these is the difficult one of providing adequate sewer systems and sewage treatment plants. In the initial stages of the development of suburban areas, septic tanks and drain fields may solve the initial household problems. As the areas develop further, health and sanitation problems are created and, eventually, community sewerage systems become a vital necessity. Such is the condition in much of the Seattle metropolitan area.

"...The hodge-podge development of sewerage facilities in the suburban Seattle area during the past decade has been a matter of concern to all interested in or affected by sewage collection and disposal. Sewer districts have been and are being formed to include only that immediate area, no matter how irregular in patterns, needing and desiring sewers at the moment...Little regard has been given to the nearby sewer districts or intervening areas when a new district is established. Construction of sewerage facilities in this fashion is expensive, as many small treatment plants and trunk sewers will have to be abandoned eventually in favor of other units designed for the entire drainage basin or basins."

On the same subject, a report issued in 1956 by the Bureau of Governmental Research and Services at the University of Washington, entitled "Government in the Metropolitan Seattle Area" states in part:

"A geographic concentration of population carries with it a need for basic services...The current significant movement to the surrounding suburban unincorporated areas and small incorporated municipalities produces a large number of communities which do not have the governmental machinery to provide the basic services necessary in urban areas. Consequently, the residents of the unincorporated areas form special districts to obtain the necessary urban services. Each district is formed separately under its own state laws and has its own budget. The result is layers of government...each performing certain specific services to the residents of its particular area. This jig saw, patch-work pattern has been appropriately called 'fractionated' government...From an overall view, the difficulties experienced by the present small, scattered sewer districts in attempting to solve the problems of adequate sewage facilities are:

"(1) Small districts are unable to cope with the sewage problem. Eighty per cent of the present districts have an area less than two square miles. Generally, a sewer district is formed only when there is an acute sewage problem, and then only the immediate area is considered in solving the problem. When formed inland, the district is faced with the next-to-impossible task of financing long outfall lines for the disposal of sewage.

"(2) Future planning in sewer system designed by small districts for the Lake Washington drainage basin is very difficult, if not impossible. The boundaries of sewer districts are not drawn according to topography to create a unit capable of economical and efficient operation. In King County, there are 60 sewer district commissioners and 139 city councilmen concerned with sewerage problems. Each governmental unit has its engineers and legal advisers. Coordination of plans with adjacent units is extremely difficult; planning for the entire area is virtually impossible.

"(3) Contractors building large housing developments would often like to install a central sewer system, but do not, as they have no information on how their system might be coordinated with the surrounding areas' future sewerage development. The abandonment of many of the small sewage treatment systems that have been built by contractors now appears to be imminent if they do not fit into the comprehensive plan of trunk sewers and treatment plants being developed for the area as a whole.

"(4) Treatment plant and outfall sewer sites become increasingly difficult and more expensive to obtain as the areas increase in development.

"(5) Furthermore, there is at present no legal method for the 73 square miles in Snohomish County to cooperate with the development of sewage facilities to prevent the deterioration of the lake, even if the area were to form a sewer district."

The difficulties discussed above were underscored in 1956 when the Bellevue Sewer District proposed a program for removing sewage and sewage effluent from Lake Washington by contracting with other sewer districts and jointly financing a trunk sewer to serve the communities on the east side of the lake. Estimated to cost between 2.5 and 3 million dollars, this program called for the construction of an intercepting sewer from Houghton to Renton, with the treatment plant discharging its effluent to the Duwamish River. In the initial stage, service was to be provided for the

Bellevue Sewer District, Three Points Annexation, Enatai and Beaux Arts Annexation, Eastgate, Houghton, Mercer Island Sewer District, East Mercer Sewer District as well as the new Safeway Industrial Center and the proposed Overlake Commercial Development. In later stages, facilities were to be constructed whereby it would be possible eventually to serve the entire Lake Washington - Lake Sammamish area.

At about the same time as the east side project was proposed, the State Department of Health and the State Pollution Control Commission adopted similar statements of policy with respect to the discharge of sewage into Lake Washington. Plans for the intercepting sewer apparently met all requirements set down by these authorities.

On August 8, 1956, before the Bellevue plan could be consummated, the Lake Hills Sewer District proposed an alternative plan involving the construction of a 24-inch effluent line under Lake Washington to discharge ultimately into tide water in Shilshole Bay. Under this plan, the Lake Hills Sewer District was to build a treatment plant at Yarrow Bay and the Bellevue District was to deliver its sewage thereto under a contractual arrangement. Construction was to be undertaken by such a time as 6,000 properties were under contract to utilize the proposed facility. This project, however, has fallen by the wayside because the several agencies involved therein have been unable to reach a mutually satisfactory agreement.

Limitations on Discharge of Sewage into Lake Washington

Coincident with urbanization of the area surrounding Lake Washington, several sewage treatment plants were constructed with the outfalls into the lake. To safeguard domestic water supplies being taken from the lake, the State Department of Public Health and the Pollution Control Commission adopted regulations requiring that all treatment plants discharging thereto must provide facilities for secondary treatment and thorough disinfection. While not entirely satisfactory from a domestic water supply standpoint, it was believed that the degree of sewage treatment thus attained would safeguard the recreational and aesthetic values of the lake.

Recent experiences in similar situations in other parts of the country have focused attention on the fact that the nutrient substances contained in sewage and sewage effluents can cause algal and other biological activities of nuisance proportions in waste receiving waters. Reports of investigations by the Pollution Control Commission and by Dr. W. T. Edmundson and associates of the University of Washington have indicated that increased biological activity is presently taking place in Lake Washington. As a result, heavy emphasis has been placed on the need for removal of sewage and sewage effluent discharges from the Lake Washington drainage basin.

As part of the abatement effort, the Pollution Control Commission, in 1956, established a policy aimed at progressive correction of the discharge condition. Attendant publicity regarding this and the threatened deterioration of Lake Washington, coupled with the inability of independent agencies to unite on an effective program, has impressed upon public officials and residents of the metropolitan Seattle area the need both for comprehensive sewerage planning and for a central sewerage authority.

Central Sewerage Authority

Creation of a central sewerage authority has been advocated from time to time for many years. In 1934, E. French Chase, former sewerage maintenance engineer of the city of Seattle, stated in a public address, "Since 1915... I have advocated the formation of a metropolitan sewer district to include the entire area around Lake Washington." In a report of May 17, 1926, previously cited, A. H. Dimock stated that a central sewerage authority was needed in the Seattle area. To that end, he commented in part as follows:

"It is clear that the problem is far larger than Seattle alone can solve. It outruns our authority but not our interest... Whatever may be reasonable and necessary to preserve the purity of the lake should be done through the watershed as well.

"There is at present no provision for a central authority to deal with the problems arising from the sanitation of a single watershed... It is quite evident that a divided authority... is incapable of formulating and carrying out a unified program which requires scientific study, engineering skill, and sound financing."

In the 1948 publication of the State Pollution Control Commission commenting on the Wolman report, the following statement appears:

"There is urgent need for the establishment of a competent utility organization within the city of Seattle whose sole purpose is the administration of the sewerage problem... It is expected that this utility organization might later be expanded to include surrounding territory forming a metropolitan sewer area."

State, city, county and sewer district officials, chambers of commerce, the Municipal League, and other civic organizations have expressed themselves as favoring some form of centralized sewerage authority. Early in 1956, former Governor Langlie appointed a committee to consider and to assist in the development of a solution to the metropolitan sewerage problem. Concurrently, the mayor of Seattle, and the commissioners of King County appointed a

metropolitan problems committee to consider area-wide problems and to make recommendations for their solution. In so doing, the sewerage situation was cited as a principal reason for the appointment of the committee. With the support of the Board of Commissioners of King County, the committee was later expanded to include representatives of the entire metropolitan area.

Community pressure, together with efforts of interested agencies and the appointed committees, culminated in the passage, by the 1957 State Legislature, of an enabling act providing for the formation of Metropolitan Municipal Corporations. Under this act, such corporations are empowered to plan, finance and administer certain services, including sewerage, on a metropolitan basis.

As a final note in recounting the history of the sewerage problem, it should be recognized that the assistance and support of the appointed committees played an important part also in the decision of the state, county and city of Seattle to finance jointly the conduct of the survey here reported.

PART II

CHARACTERISTICS OF METROPOLITAN SEATTLE AREA

Chapter 3. Physical Geography
Chapter 4. Economic Development
Chapter 5. Population

Chapter 3
PHYSICAL GEOGRAPHY

Physical geography is concerned with the physical features of the earth and their form, movement, and changes. Local geography, as defined by topography, geology, climate and water areas, influences in many ways the design, construction, cost, and operation of sewerage and drainage facilities. At Seattle, water areas are of particular significance in that they require protection from pollution and nuisance and provide at the same time a means for the safe disposal of storm runoff and suitably prepared sewage effluents.

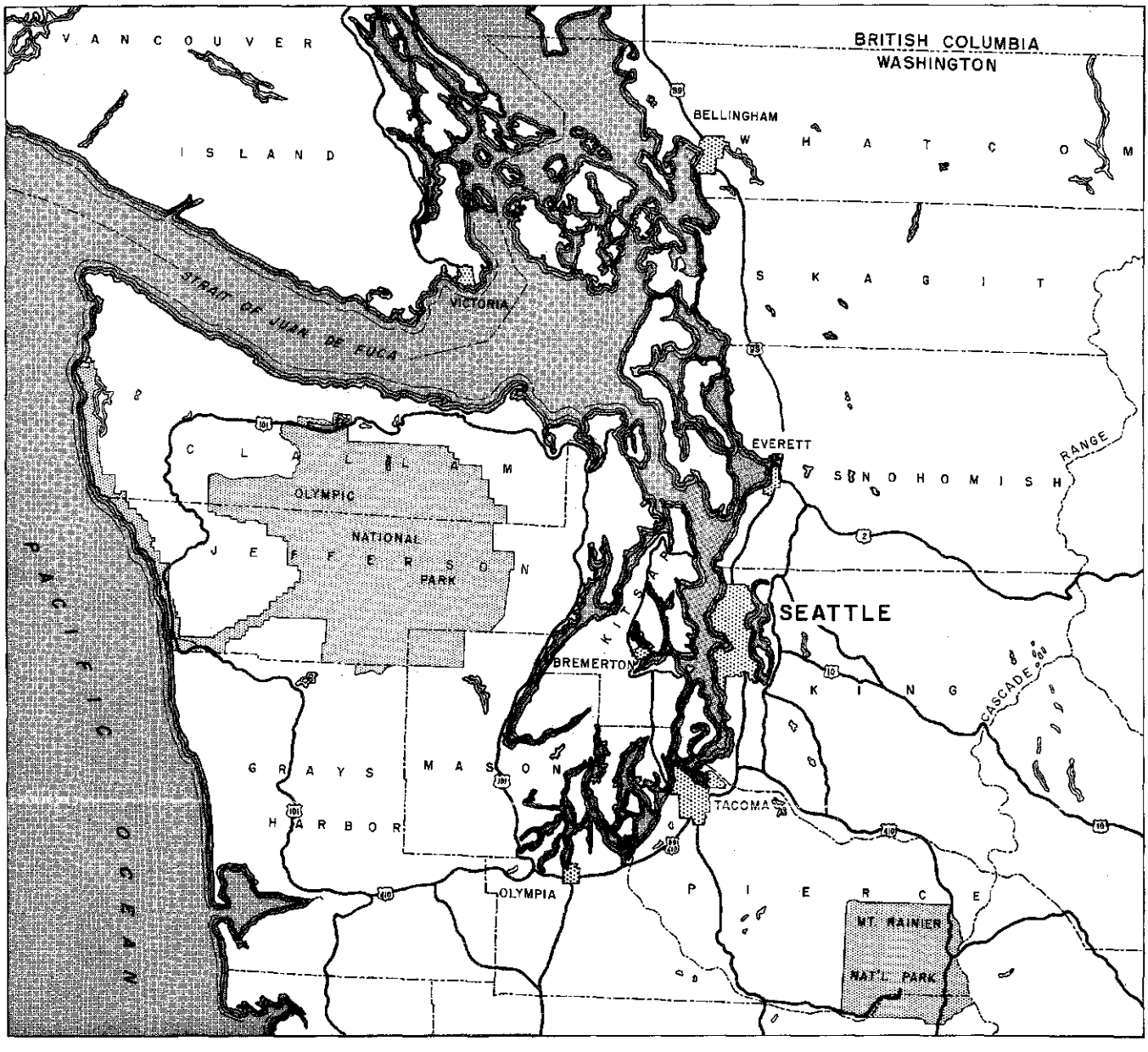


Fig. 3-1. Puget Sound Basin

The Cascade Mountains to the east and the Olympic Mountains to the west define the limits of the section of the Puget Sound Basin lying within the United States. Seattle and its metropolitan area are situated along the southeasterly shoreline of Puget Sound about 90 miles inland from the Pacific Ocean and 100 miles south of the Canadian border.

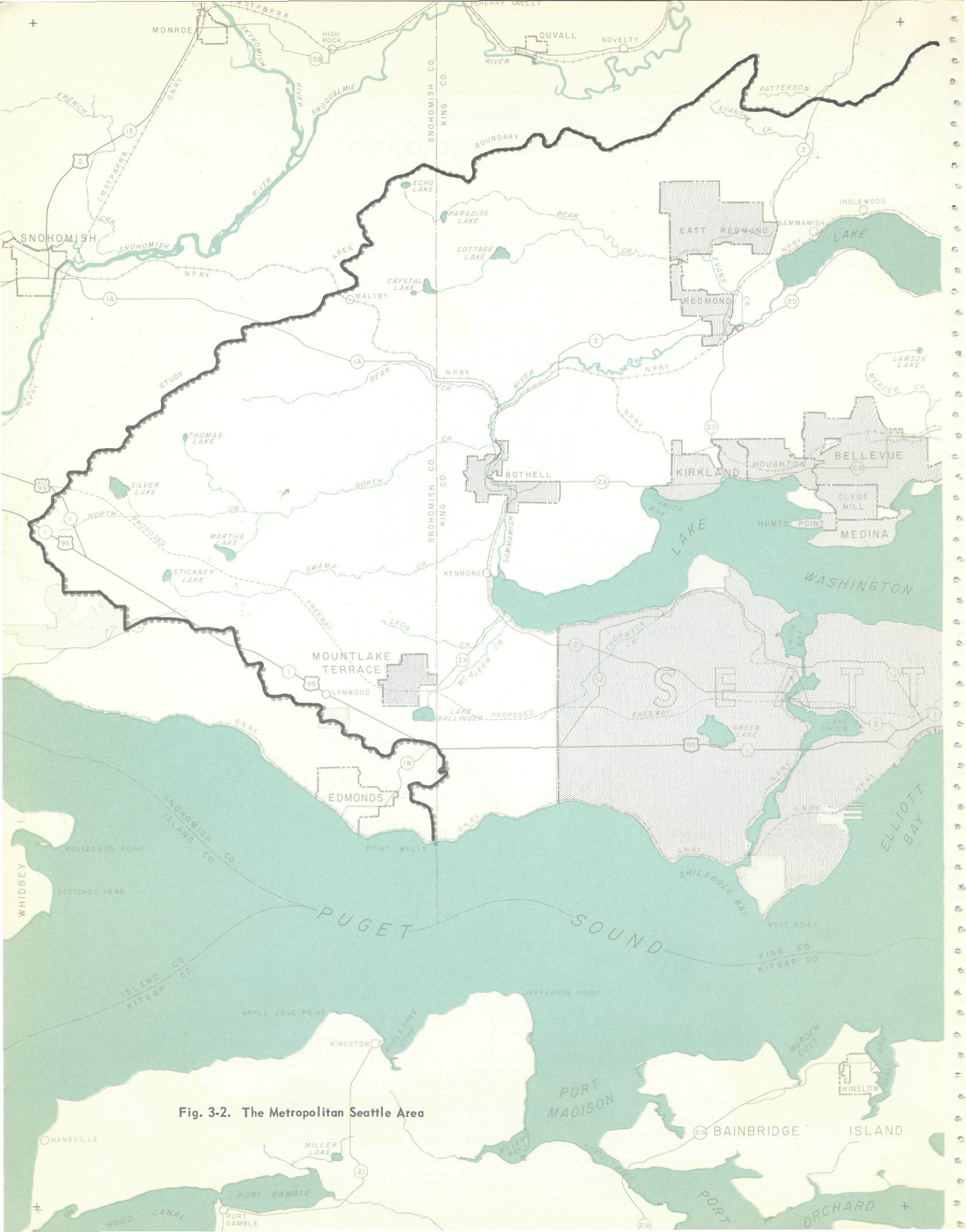
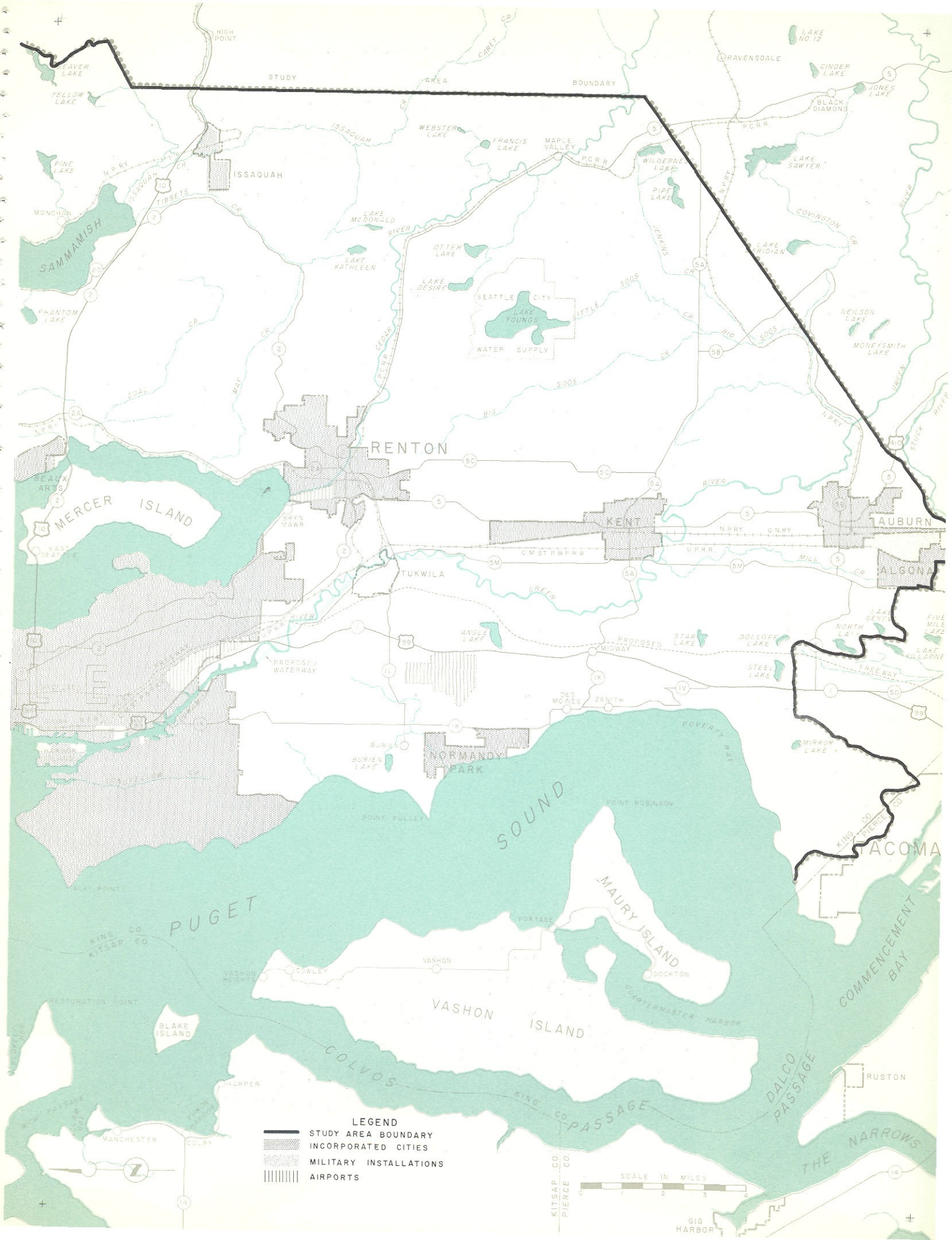


Fig. 3-2. The Metropolitan Seattle Area

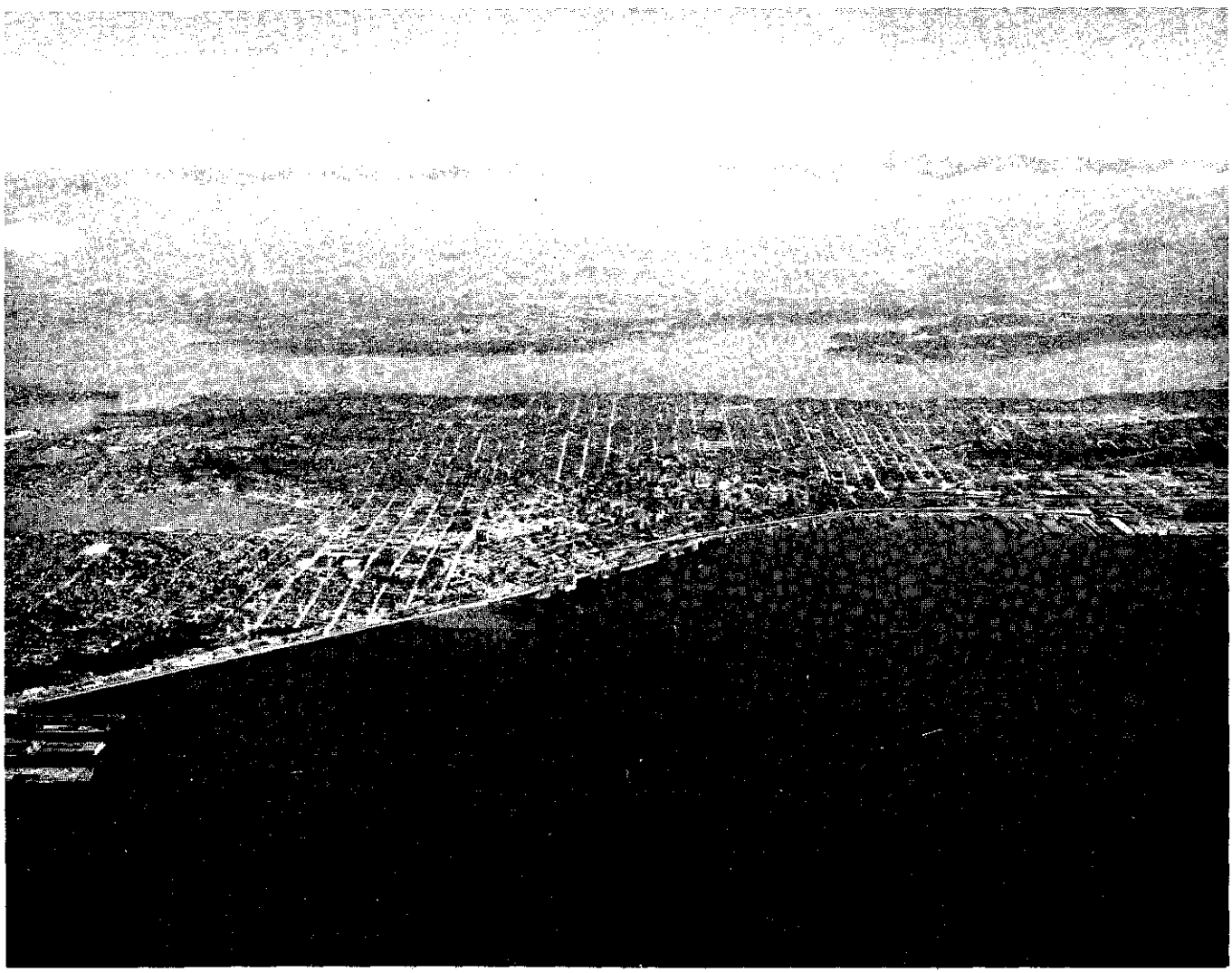

STUDY AREA BOUNDARY
HIGH POINT
BEAVER LAKE
YELLOW LAKE
ISSAQUAH
SAMMAMISH
MONOHON
PINE LAKE
PHANTOM LAKE
WEBSTER LAKE
FRANCIS LAKE
MAPLE VALLEY
LAKE MCDONALD
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
RENTON
MERCER ISLAND
BEACH ARTS
EAST SEATTLE
BRYN MAWR
TUKWILA
KENT
AUBURN
ALGONA
RAVENSDALE
LAKE NO. 12
CINDER LAKE
JONES LAKE
BLACK DIAMOND
LAKE SAWYER
WILDERNE LAKE
PIPE LAKE
LAKE MERIDIAN
NEILSON LAKE
MONEYSMITH LAKE
FIVE MILE LAKE
LAKE GENEVA
NORTH LAKE
LAKE KILLARNE
ANGLE LAKE
STAR LAKE
DOLLOFF LAKE
STEEL LAKE
MIRROR LAKE
PROPOSED WATERWAY
PROPOSED FREEWAY
MIDWAY
DES MOINES
ZENITH
BURIEN
BURIEN LAKE
NORMANDY PARK
POVERTY BAY
POINT ROBINSON
POINT PULLEY
PUGET SOUND
ALKI POINT
KING CO.
KITSAP CO.
PIERCE CO.
TACOMA
MAURY ISLAND
PORTAGE
DOCKTON
VASHON
EWLEY
VASHON ISLAND
QUARTERMASTER HARBOR
COMMENCEMENT BAY
RESTORATION POINT
BLAKE ISLAND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
RUSTON
HARPER
MANCHESTER
COLBY
GIG HARBOR
LEGEND
STUDY AREA BOUNDARY
INCORPORATED CITIES
MILITARY INSTALLATIONS
AIRPORTS
SCALE IN MILES
0 1 2 3 4

LOCATION AND LIMITS OF THE METROPOLITAN AREA

At the outset of a survey concerned with sewerage and drainage problems of a large metropolitan area, the most difficult task from a technical standpoint is that of establishing geographically logical limits for study and planning purposes. This task involves a consideration of many factors. It can be said, however, that economy in construction and operation of sewerage and drainage systems requires that boundaries be established on a watershed rather than political basis.

In Seattle at present there is no legal or physical boundary by which the metropolitan area may be effectively defined. For sewerage planning purposes, it may be regarded broadly as the area stemming from the central city, Seattle, which has common or topographically related problems and is likely to become fully urbanized during the life of the planned facilities. This basic concept, together with an analysis of topographic features and expected future development, was taken into account in determining the boundaries of the planning area. Shown in Fig. 3-2, this area extends from the vicinity of Silver Lake on the north to the city of Auburn on the south and from Puget Sound inland for a distance of 18 miles at the point of its greatest width. In all, it encompasses an area of about 575 square miles, or 370,000 acres, of which 320,000 acres lie in King County and 50,000 acres lie in Snohomish County. Actually, the boundary crosses the Pierce County line at several points, but the acreage within that county is quite small.

In addition to Puget Sound, the principal physical features of the area are its rather rugged terrain, much of it heavily wooded, and its numerous lakes and rivers. Lake Washington, with a length of about 25 miles and an area of about 22,000 acres, practically bisects the area from north to south. Lake Sammamish, which is the next largest, has an area of about 4,700 acres and lies in the east-central section. These, together with numerous smaller



SEATTLE, central city of the metropolitan area, lies between Puget Sound (foreground) and Lake Washington (center). These bodies of water and the many other smaller lakes are one of the area's outstanding physical features.

lakes, provide about 140 miles of fresh water shoreline. There are three principal rivers, the Green-Duwamish, the Cedar, and the Sammamish, and many minor creeks and streams.

Taken as a whole, perhaps the most impressive physical feature of the metropolitan area is its outstanding natural beauty. With the rugged background formed by the Cascade Mountains on the east and the Olympic Mountains on the west across Puget Sound, the combination of large water areas, many miles of shoreline, and irregular wooded terrain is truly one of nature's masterpieces.

Compared with other metropolitan areas of the United States, the Seattle area is fairly new. In addition to Seattle, which was incorporated less than 100 years ago, the area now contains 18 incorporated cities. Two of these, Auburn and Kent, were established before the turn of the century, and six of them, Renton, Kirkland, Tukwila, Bothell, Redmond and Issaquah, were incorporated between 1900 and 1914. The remaining ten, Algona, Houghton, Bellevue, Beaux Arts, Clyde Hill, Normandy Park, Hunts Point, Medina, East Redmond and Mountlake Terrace became cities during the past 10 years and are indicative of the rapidity with which fringe areas have been developing.

TOPOGRAPHY

Topography, as related to ground slope and natural drainage features, determines to a large degree the route, size and slope of collection sewers and, in general, the extent to which an area is tributary to a common point. It determines also the necessity for pumping of sewage and the location of pumping stations. Moreover, topography has a direct bearing on population distribution and growth.

Fig. 3-3 shows the over-all configuration of the ground surface and defines the primary watershed areas. It should be noted that the watersheds of Cedar River and Green-Duwamish River extend beyond the boundaries of the metropolitan area and that the total areas given below refer only to those portions of these watersheds which lie within the study area.

Principal Topographic Features

Bounded on the west by Puget Sound and on the east by the foothills of the Cascade Range, the metropolitan area is topographically an area of striated hills, rolling glaciated uplands, and deeply incised adjoining troughs. Its principal physical features, such as large lakes, major stream valleys and intervening ridges, have a general north-south trend, paralleling the axis of Puget Sound.

An exception to the glaciated north-south topography is the prominent east-west ridge between the south end of Lake Washington and the foothills of the Cascade Range to the east. Composed of sedimentary rocks flanking a core of igneous rocks, this ridge rises to about elevation 2,000 at Squak and Tiger mountains near Issaquah, dividing the eastern part of the study area into north and south sections. North of the east-west ridge, the upland areas lie generally above elevation 300 and rise to a maximum of about elevation 500. South of the ridge, the upland areas are approximately 100 feet higher, lying above elevation 400 and containing ridges which rise to elevation 600 near the boundaries of the area.

In the city of Seattle, south of the Lake Washington Ship Canal, the dominating topography is one of more gently rounded hills lying 200 to 300 feet above sea level, with the summits rising to approximately elevation 400. West Seattle has a similar topography with corresponding features lying about 100 feet higher.

Upland sections of the metropolitan area present a glacial relief in which stream courses are often poorly defined, and local basins and depressions of retarded drainage are occupied by areas of marsh, swamp, or lake. These sections terminate generally in steep bluffs which descend about 200 feet to adjoining troughs occupied by interhill stream valleys, lake basins, and Puget Sound.

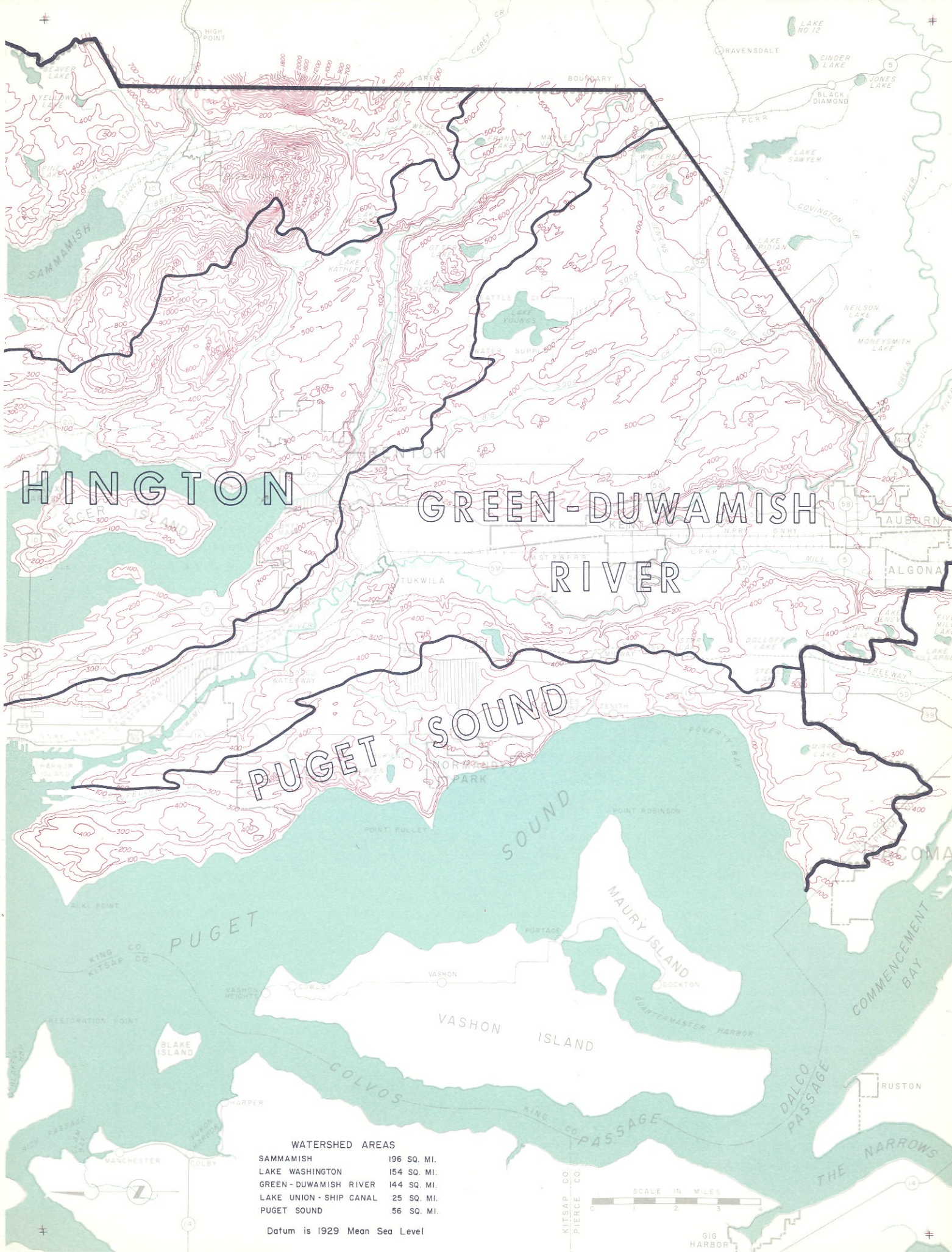
Major Watersheds

Although it drains ultimately to Puget Sound, the metropolitan area is divided naturally into four major watersheds. The largest of these, the Lake Washington watershed, encompasses almost 350 square miles and thus constitutes about 60 per cent of the study area. Bounded by uplands and ridges on the west, Lake Washington cannot drain directly to Puget Sound but is connected to the Sound through the Lake Washington Ship Canal. Formerly both the lake and Cedar River drained into the Duwamish via the Black River, but lowering of the lake level at the time the Government Locks were constructed changed the natural drainage direction and the Cedar became instead a principal tributary to Lake Washington.

Topographically, the Sammamish River valley is a part of the Lake Washington drainage basin. In the planning of sewerage facilities, however, this valley is considered separately because of its large area (196 square miles) and the elevation of the intervening ridge which separates it from Lake Washington.

The Green-Duwamish watershed, which drains directly to Elliott Bay, comprises an area of 144 square miles, making it the second largest in the study area. Direct drainage of the southeastern portion of the study area to the Green-Duwamish River valley is prevented by a slight ridge paralleling the top of the easterly valley wall. Instead, this area

Fig. 3-3. Topography and Major Watersheds



WATERSHED AREAS

SAMMAMISH	196 SQ. MI.
LAKE WASHINGTON	154 SQ. MI.
GREEN-DUWAMISH RIVER	144 SQ. MI.
LAKE UNION-SHIP CANAL	25 SQ. MI.
PUGET SOUND	56 SQ. MI.

Datum is 1929 Mean Sea Level

drains southerly to Big Soos Creek and then into the Green River east of Auburn.

Of the remaining 81 square miles of watershed in the metropolitan area, 25 square miles drain to the Lake Washington Ship Canal system. On that basis, only 56 square miles, or 10 per cent of the total, are tributary directly to Puget Sound.

Principal Hydrographic Features

Lake Washington has an east-west bottom configuration shaped essentially in the form of a "W". Deeper in the central portion, the lake shoals toward the north and south ends. It has an average depth of over 150 feet and reaches its maximum depth of 220 feet north of the Lake Washington floating bridge off Madison Park. While its steeply inclined sides below surface level are a continuation of the adjacent bluffs, the lowering of the lake level to its present elevation exposed a wave-cut terrace around much of the perimeter. Fathometer soundings, as performed by the Department of Oceanography of the University of Washington, indicate that fragments of this terrace extend into the lake from the present shoreline.

Puget Sound, which bounds the metropolitan area on the west, is one of the deepest salt water basins in the United States. A partially drowned, glacially modified drainage system, the sound is generally "U" shaped below water and has an average bottom elevation of about 600 feet below sea level. While the bottom normally drops off sharply a short distance from shore, it slopes gradually in the vicinity of West Point to a depth of 100 feet approximately 3,000 feet from shore, then breaks sharply to a depth of about 800 feet. Similar shoaling conditions do not exist elsewhere within the study area except in the vicinity of Point Wells near the north King County line.

GEOLOGY

The geology of an area is significant in the planning of sewerage works in that formations encountered in the construction of pipelines, tunnels and other structures affect design and construction requirements and thus have a direct bearing on cost. Geological conditions may be determining factors in choosing routes of principal sewers and may even influence the over-all scheme of sewerage to a marked degree. Moreover, soil formations, both surface and subsurface, determine the feasibility of local sewage disposal by means of septic tanks and leaching fields.

Geological History

Early geologists recognized two periods or stages of glaciation, giving the name Admiralty to the earliest stage and Vashon to the last. In recent years, geologists have found evidence of two and possibly three glacial stages in the formations previously assigned to the Admiralty period. As in other glaciated regions, glaciation of the metropolitan area occurred over a period of hundreds or thousands of years.

Glaciers carry with them debris ranging from clay size particles to stones of all dimensions. This material, referred to collectively as drift, is deposited by (1) glacial ice, (2) ice and streams working together, and (3) streams issuing from the melting glacier. As the forward movement occurs, unsorted drift or till is plastered over the existing topography by the bottom of the glacier. The weight of the overlying ice compresses the till to a dense, compact, often cemented material. When the forward movement stops and the ice stagnates and melts, drift is deposited in irregular heaps due to the varying debris load within the ice. Kettles or undrained depressions varying from a few feet to hundreds of yards in diameter are formed by the melting of isolated masses of ice surrounded or buried by drift.

As the ice retreats and melts, the meltwater forms streams which serve to carry debris away from the glacier front. Depending on the size and velocity of these streams, material may be deposited immediately outward from the glacier, or previously deposited materials may be picked up and differentially transported. Melting rates fluctuate and are manifested, in the case of a fast rate, by large outpourings of material which aggrade outwash plains of great extent. Lakes are formed by the accumulation of meltwater in depressions or by dammed-up valleys. In these lakes, fine clay and silt-size sediments are deposited, often in thick beds of soft material. Similar clay deposits from previous glacial periods, subsequently consolidated under later glacial ice, are the source of the hard tough blue clays exposed in the bluffs throughout the metropolitan area.

Deposition of sediments during retreat of the glacier preceding the Vashon glaciation resulted in an aggraded flood plain extending over the entire Puget Lowland. Following uplift of the plain, an interglacial period of extensive stream erosion occurred which cut deep valleys in the surface of the flood plain, forming intervalley hills and establishing the general topography as it exists today.

Glaciers of the Vashon period, in their advance southward from Canada, modified the interglacial topography by plastering till over the upland surfaces and deepening the valleys which were most in sympathy with the over-all trend of ice movement. Within the metropolitan area, the Vashon till sheet generally is a relatively thin veneer which covers the slopes and summits of the interglacially formed hills and, in some places, the floors of the interhill valleys. Increasing in thickness in the northern part of the area, the till sheet reaches its maximum thickness of about

150 feet north of the Snohomish-King County line. Extensive morainic and outwash deposits laid down during the early Vashon retreat appear to have obliterated completely the interglacially eroded valleys in the Puget Sound basin south of the metropolitan area. While this obscuring of interglacial topography occurred locally in some instances, the pre-Vashon valleys within the metropolitan area which were exposed by the ice retreat were generally little modified by outwash and recessional morainic deposits.

As the ice retreated northward, a series of glacial lakes formed between the ice front and the drainage divide south of Olympia. As this artificial basin filled with glacial meltwater, these lakes coalesced to form a master lake. At its maximum, the master lake, given the name of Lake Russell, extended from Olympia to the vicinity of Everett and its surface lay at 160 feet above the present sea level. After the ice retreated to a point where the Strait of Juan de Fuca was reopened, the lake was destroyed and marine waters invaded the basin.

Since the disappearance of the glacial ice, the area has been uplifted slightly, and moderate warping or tilting done to the south has occurred, with the axis of tilting lying just south of the city of Seattle. Moderate postglacial erosion of the upland areas has taken place, as have deposition and delta-building in the valleys. Wave-cutting at the bases, and subsequent sloughing of the upper parts, has caused retreat of the cliffs, particularly those facing Puget Sound.

Available Geological Information

Although mapping of the surface geology in Seattle has not been completed, unpublished data are available from various sources. Definitive subsurface geological data, however, are not available from any source and little is known about the interior composition of the hills, such as the distribution or continuity of formations. Surface soils have been classified and mapped on a pedological basis by the United States Department of Agriculture and results thereof have been published for King[1] and Snohomish[2] counties. Well logs, together with data from previous soils explorations, are available but the latter are generally limited to highway routes and to lowland areas and filled tidelands where industrial expansion has taken place. In the upland areas, particularly east of Lake Washington, drilling data are generally lacking. Considerable unpublished information concerning bottom conditions in Lake Washington was obtained from the Department of Oceanography, University of Washington. On the other hand, little is known about bottom conditions in Puget Sound.

Although available information and data concerning the geology of the metropolitan area are generally meager, they are sufficient for preliminary planning purposes. It should be pointed out, however, that the sewer routes selected in this report should not be considered as specific in every instance and that, as a part of final design, minor departures from selected routes may, and probably will, be necessary because of soil conditions. On the whole, such departures will not affect the relative merits of the various alternatives presented, nor should they have a significant effect on total costs. It should be emphasized, however, that adequate exploration programs and geological examinations of samples must be undertaken as a part of final design and prior to selection of precise routes.

Soil and Foundation Conditions

On the basis of available information, certain generalizations may be made as to the foundation conditions which exist in the metropolitan Seattle area. It is important to note at the outset, however, that because there have been at least three and possibly four periods of glaciation, considerable horizontal and vertical variation in subsurface geology can be expected. Nevertheless, generally similar conditions will occur in similar land forms, such as the upland regions and hills, the lowland valleys, and the troughs occupied by Lake Washington and Puget Sound. Notable exceptions to the generalized conditions are the prominent slide areas found in the bluffs and the bedrock exposures which exist in certain locations.

Upland Areas and Hills. In general, a shallow weathered soil from two to six feet in depth has formed over the upland areas and is underlain extensively by hard cemented till. Where post-glacial erosion has removed the till cap, firm compact clay (hard pan) often underlies the surface soil. East of Lake Washington, along the area south of Highway 10, sandstone and shale rock are found at shallow depths. Morainic deposits of sand and gravel and recessional outwash deposits are also found over much of the uplands, extending in some instances to great depths and over a considerable area. Peat deposits have accumulated along the courses of many small creeks draining the upland areas as well as in local kettle-like depressions. Ground water tables vary seasonally but may be generally high where underlain by shallow impermeable strata. Except where peat deposits are encountered, the upland soils have excellent bearing capacities for foundations of structures and good bedding conditions for pipelines.

Soil formations composing the interior of the hills within the metropolitan area may be grouped into three

[1] *Soil Survey, King County Washington, U.S. Department of Agriculture, Series 1938, No. 31, 1952.*

[2] *Soil Survey, Snohomish County Washington, U.S. Department of Agriculture, Series 1937, No. 19, 1947.*

general classifications: (1) firm, generally dry, compact clays and silts, (2) hard cemented till, and (3) water-bearing sands and gravels. Of these, the firm clays and silts predominate at the lower elevations and require excavation with mechanical equipment. In tunnels, the firm clays and silts stand with minimum support. The cemented till also requires minimum support and, when encountered in confined places such as trenches or tunnels, may necessitate blasting for economical excavation.

Sand and gravel generally constitute a subterranean pocket or reservoir of water, and are of primary concern when encountered in tunneling operations. In such cases, the quantity of water which must be handled is dependent upon the gradation or size of the particles as well as upon the extent of the deposit. Horizontal well-points installed in advance of the tunnel face may be necessary if large quantities of water are encountered. Chemical stabilization or solidification processes, developed in recent years, also may be resorted to in order to reduce the water inflow and the amount of tight lagging required for support. Of the ten major tunnels driven in Seattle during the last sixty years, four encountered difficult water conditions. Of these, three were constructed in the period from 1892 to 1912 prior to the development of modern construction equipment and techniques for handling such problems.

Lowland Valleys. Soil conditions in the lowland areas consist of alluvial deposits of soft clay, silts, and fine to coarse sands intermixed with gravel. These soils may be grouped into two general classifications: (1) silty alluvium composed of the fine grained, low strength, more compressible soils occupying the low areas and depressions, and (2) sandy alluvium composed of the more granular, permeable soils having better supporting characteristics and lying at slightly higher elevation. Being alluvial in origin, the lowland soils are generally sorted as to size but occur in lenses or pockets rather than as stratified deposits of uniform elevation. The higher sandy soils, being more granular, are easily drained. Peat, up to 12 feet in depth, is widely distributed throughout the lowland areas but is found more frequently in areas adjacent to the bluffs forming the valley walls.

High groundwater tables are encountered in the lowland areas. Water is found at depths from one to six feet below the surface during summer months and is generally at or above the ground surface during the winter.

Valley soils frequently require piles or other special foundations for the support of structures and usually require a special bedding of granular material for pipelines. While similar to the alluvial valley soils, soils underlying the reclaimed tidelands are often more compressible and may require piling for support of larger size pipelines.

Lake Washington Trough. Considerable unpublished data have been obtained by the Department of Oceanography during a core sampling study of the bottom sediments in the deeper portions of Lake Washington. These data disclose that much of the lake bottom consists of a layer, up to 40 feet deep, of soft, almost fluid, organic peat-like sediment overlying an unknown depth of soft, compressible glacial blue clay. A core drilled in 190 feet of water between Union Bay and Evergreen Point extended 140 feet below the lake bottom without penetrating the soft clay deposits. Closer to shore, data from borings indicate that the steeply inclined sides of the lake bottom are composed of firm, compact glacial deposits similar to those exposed in the adjoining bluffs. Evidence indicates also that only relatively shallow soft sediments have been deposited on the wave-cut terrace where it extends outwards from shore. In the shallow area at the south end of the lake, loose silts and sands have been deposited during delta-building by the Cedar River. At the north end of the lake, in the Kenmore area, the lake bottom is composed of sand and gravels washed into the lake from the upland morainic deposits to the north. Near the mouth of the Sammamish River, these sands and gravels are overlain by alluvial soils.

Puget Sound Trough. Little is known about the bottom conditions of Puget Sound. Sampling has been confined to grab samples of sediments lying immediately on the bottom. Work performed offshore from Point Wells indicates that the steeply inclined sides of Puget Sound may be composed of firm materials similar to those found in the steep bottom slopes in Lake Washington. Bottom areas denoted as rocky on the navigation charts may be either these firm glacial deposits or glacial erratics ranging up to many feet in diameter.

Slide Areas. Slide areas are prevalent in the bluffs bounding the upland areas where a considerable depth of sand overlies impermeable clay formations. Where these conditions are encountered, the cause of sliding may be ascribed to water percolating downward through the sand deposit to the upper face of the clay layer and draining outward to the cliff face, causing sloughing and slumping in the cohesionless sand. Large slide areas are particularly noticeable along the cliffs facing Puget Sound where wave-cutting of the underlying clay formations occurs. A pronounced example of areal sliding is located along the south and west side of Magnolia Bluff extending from Smith Cove to south of West Point. North of West Point toward Shilshole Bay, four large slide areas are located.

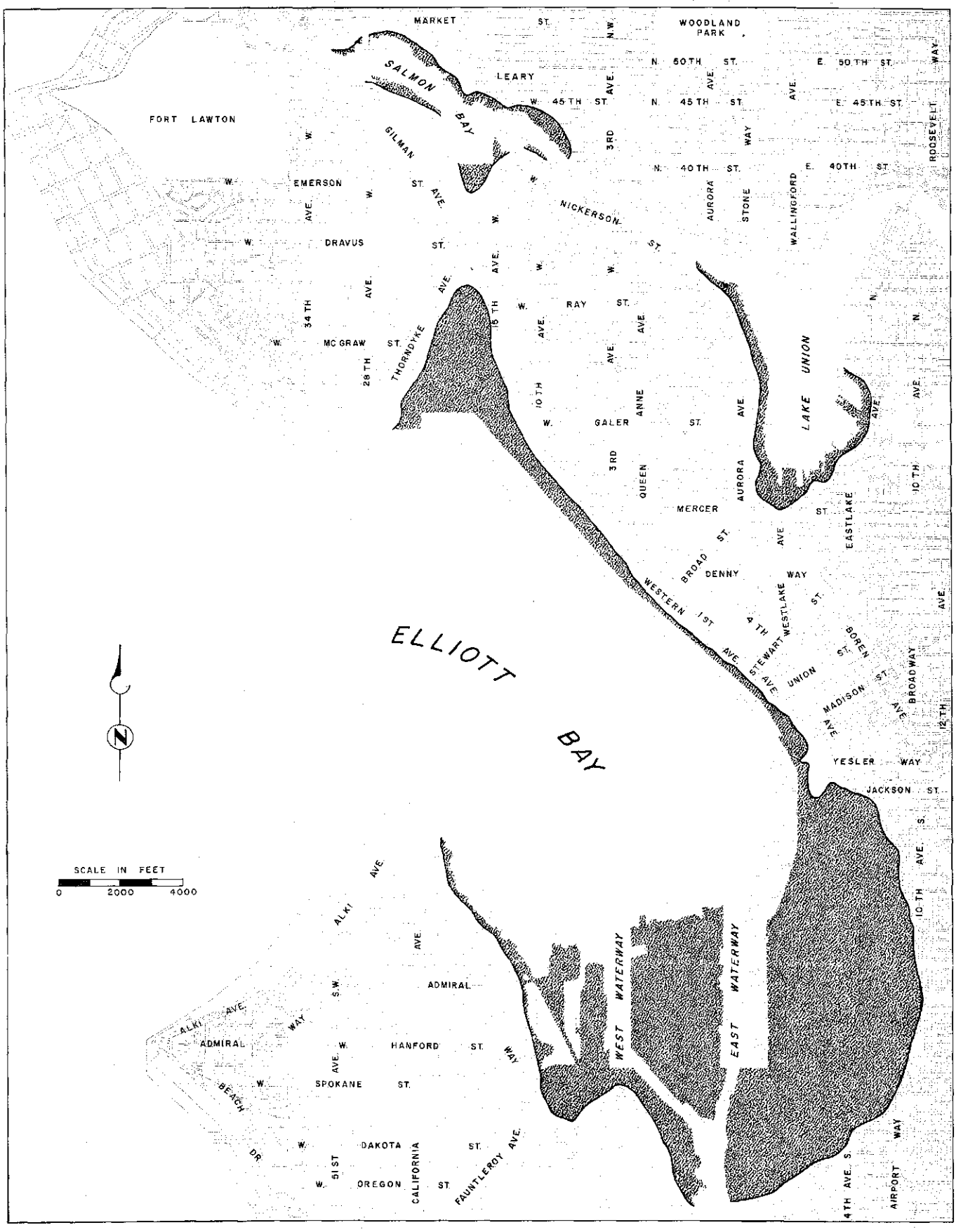


Fig. 3-4. Reclaimed Tidelands

Formerly submerged, the shaded areas have been filled and reclaimed for industrial use. As the need for industrial land developed, shorelands were reclaimed first around Lake Union and later along Elliott Bay. Following construction of the Government Locks, additional land was reclaimed along the Ship Canal. Special problems are involved in the planning of sewerage and drainage facilities across these areas because of lack of surface slope and difficult foundation conditions.

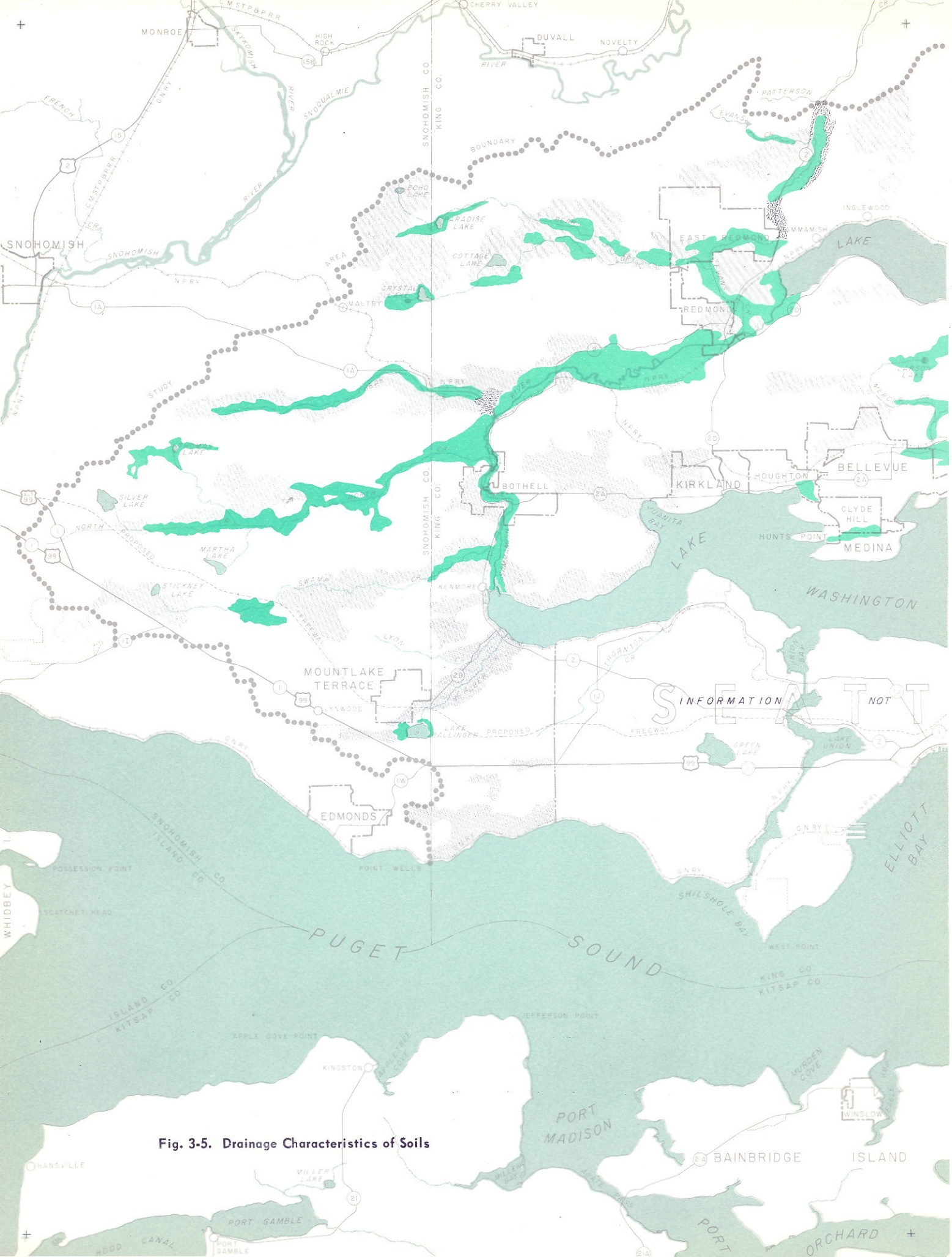
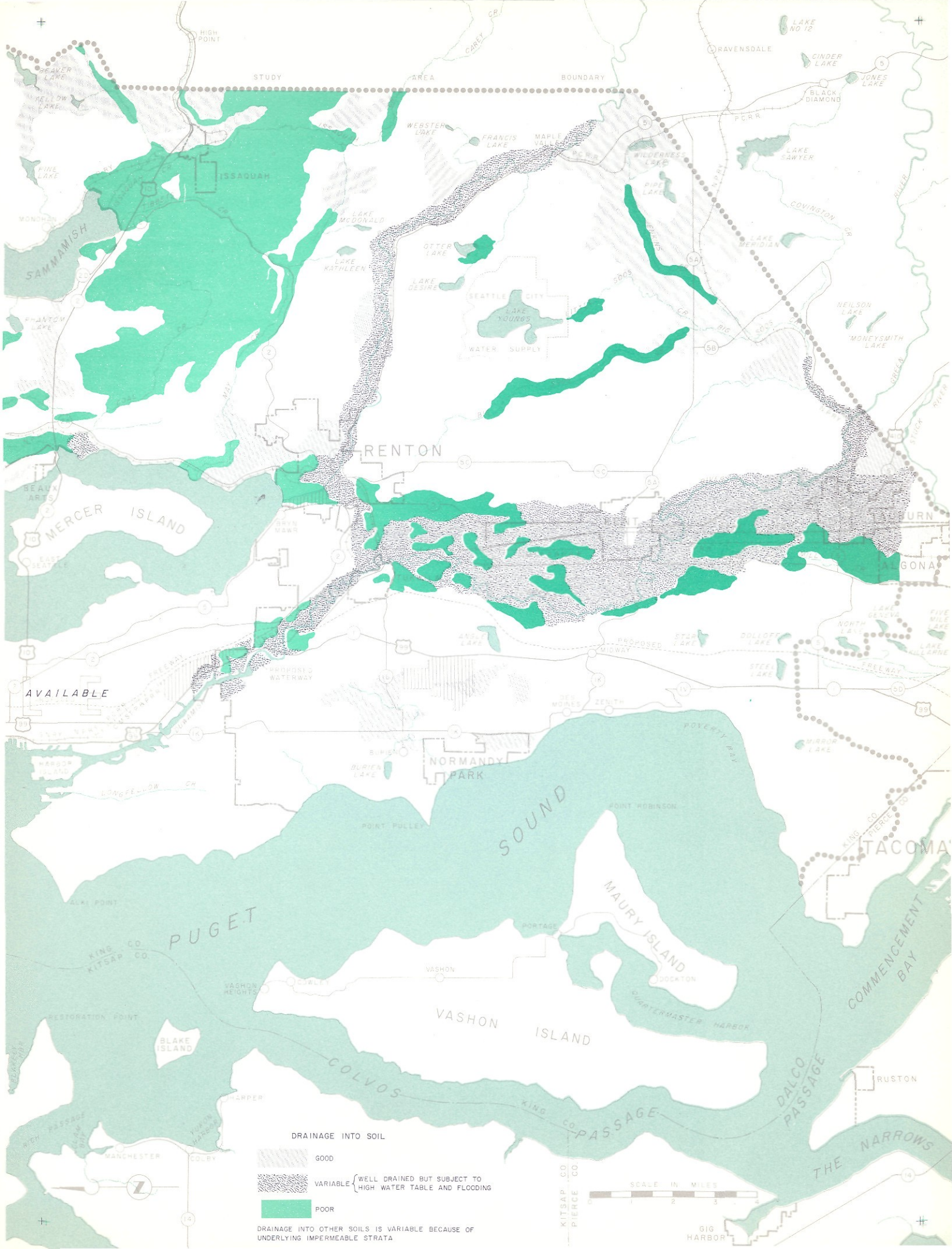


Fig. 3-5. Drainage Characteristics of Soils


STUDY AREA BOUNDARY
HIGH POINT
CAREY CR
LAKE NO 12
RAVENSDALE
CINDER LAKE
JONES LAKE
BLACK DIAMOND
BEAVER LAKE
YELLOW LAKE
WEBSTER LAKE
FRANCIS LAKE
MAPLE VALLEY
WILDERNESS LAKE
LAKE SAWYER
PIPE LAKE
ISSAQUAH
PINE LAKE
MONOHON
SAMMAMISH
LAKE MCDONALD
OTTER LAKE
LAKE KATHLEEN
LAKE DESIRE
LAKE MERIDIAN
COVINGTON CR
SEATTLE CITY
LAKE YOUNGS
WATER SUPPLY
NEILSON LAKE
MONEYSMITH LAKE
PHANTOM LAKE
RENTON
BEAUX ARTS
MERCER ISLAND
EAST SEATTLE
BRYN MAWR
ALGONA
LAKE GENEVA
NORTH LAKE
FIVE MILE LAKE
LAKE KILLARNE
ANGLE LAKE
PEARL LAKE
DOLLOFF LAKE
PROPOSED MIDWAY FREEWAY
STEEL LAKE
PROPOSED WATERWAY
AVAILABLE
DES MOINES
ZENITH
POVERTY BAY
MIRROR LAKE
HARBOR ISLAND
BURIEN
BURIEN LAKE
NORMANDY PARK
LONGFELLOW CR
POINT ROBINSON
POINT PULLEY
PUGET SOUND
KING CO
PIERCE CO
TACOMA
ALKI POINT
MAURY ISLAND
PORTAGE
KING CO
KITSAP CO
VASHON
VASHON HEIGHTS
DOCKTON
QUARTERMASTER HARBOR
COMMENCEMENT BAY
RESTORATION POINT
BLAKE ISLAND
VASHON ISLAND
COLVOS PASSAGE
KING CO
DALCO PASSAGE
RUSTON
HARPER
YOUNG HARBOR
MANCHESTER
COLBY
THE NARROWS
KITSAP CO
PIERCE CO
SCALE IN MILES
0 1 2 3 4
GIG HARBOR
DRAINAGE INTO SOIL
GOOD
VARIABLE
WELL DRAINED BUT SUBJECT TO HIGH WATER TABLE AND FLOODING
POOR
DRAINAGE INTO OTHER SOILS IS VARIABLE BECAUSE OF UNDERLYING IMPERMEABLE STRATA

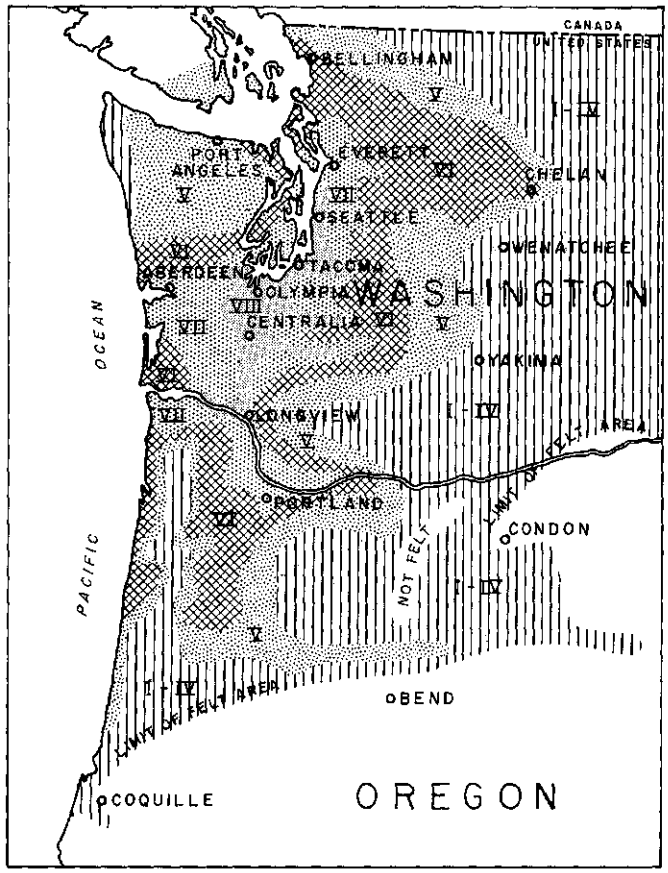


Fig. 3-6. Isoseismal Map for Earthquake of April 13, 1949

Very strong to ruinous motion was observed in the metropolitan area during this earthquake, which had its epicenter near Olympia.

One of these, extending for approximately 1,300 feet along the bluff and from 300 to 500 feet back from the toe, is centered above the portal of the North Trunk sewer tunnel. An estimated 10,000 to 200,000 cubic yards of material are sliding at this location.

Bedrock. Bedrock in the metropolitan area consists of marine shales, sandstones and conglomerates, with intercalated basalts. Bedrock is not well exposed except in a relatively narrow belt extending westerly from Issaquah across Seward Park to the Duwamish valley. It appears also at Alki Point where it passes beneath Puget Sound.

Major exposures of bedrock occur on the high ridge west of Issaquah and at various locations along the west side of Beacon Hill south to the Black River junction. Bedrock is also exposed along the west side of the Duwamish valley in the vicinity of Tukwila.

Reclaimed Areas. In recent years, some of the shoreland areas have been filled in and reclaimed for industrial purposes, thus modifying their natural condition. Since a wide variety of materials, including sawdust, have been used as a fill, piling or other special foundations may be required therein for structural support. Fig. 3-4, prepared from old topographic maps, shows the extent of reclaimed areas in the vicinity of Seattle.

Limitation on Local Sewage Disposal

Downward drainage characteristics of the subsoils, as well as of the surface soils, have an important bearing on the efficiency of the septic tank and leaching field method of local sewage disposal. As shown by Fig. 3-5, which was prepared from Department of Agriculture Soil Survey maps, drainage characteristics of soils in the metropolitan area range from good to poor. Rapid downward drainage occurs where the surface soils are underlain by porous morainic deposits. Much of the area, however, is underlain by till, firm clay or rock at depths varying from two to six feet. Under such a condition, leaching field drainage may serve adequately for a number of years until population density increases and the area builds

Table 3-1. Comparison of Local Climatological Data

Station	Index number[a]	Elevation, feet	Record, years		Temperature, °F		Precipitation, inches	
			Temp.	Precip.	Normal	1956	Normal	1956
Bothell	0826	105	23	24	50.0	b	38.10	38.29
Everett	2675	120	42	42	50.1	50.0	32.46	39.83
Kent	4169	40	31	39	51.2	50.7	36.59	37.61
Seattle								
Boeing Field	7483	14	28	28	52.2	51.5	34.28	32.17
Federal Office Building	7488	14	66	79	53.2	52.3	32.05	32.11
Maple Leaf Reservoir	7463	422	–	10	–	–	34.56	30.68
Naval Air Station	7468	21	–	26	–	–	30.87	30.48
University of Washington	7478	60	36	46	51.9	52.5	32.04	32.75
Seattle-Tacoma Airport	7473	366	12	12	50.7	49.7	33.93	36.85

Source: "Local Climatological Data - Washington, Annual Summary, 1956" and earlier years, U.S. Weather Bureau.

[a]*Official U.S. Weather Bureau designation for station.*

[b]*Data missing.*

up to such an extent that effluent contributions exceed the absorptive capacity of the soil. Similarly, seasonal variation in the ground water table below the more permeable sandy soils limits the use of septic tanks in the valley and lowland areas. In both of these cases, drainage conditions are shown as variable (Fig. 3-5) because adequate drainage would not be available at all times.

Areas shown as having poor drainage characteristics, and therefore as being unsuitable for septic tank use, include the peat deposits in the uplands and the silty alluvial deposits in the valleys and lowlands. Also included in this category is the east-west bedrock ridge extending from Issaquah to Lake Washington.

Seismic Conditions

The Pacific Northwest area is considered to be a seismically active area, or major earthquake zone. Because of the thickness and extent of Pliestocene sediments in the Puget Lowland section, however, earthquake faults are not generally apparent and are therefore unmapped.

In the period from 1841 to 1949, 108 strong motion earthquakes were noted in Washington, Oregon, western Idaho and western Montana. Major earthquakes of intensity VII on the modified Mercalli scale occurred in the immediate area on November 12, 1939 and February 14, 1946.

The most recent major shock, which occurred on April 13, 1949, produced strong motion in an area of more than 10,000 square miles in western Washington and northwestern Oregon. This shock caused a loss of 8 lives and injury to 62 persons, and resulted in property damage variously estimated as costing anywhere from 15 to 50 million dollars. As shown by the isoseismal map in Fig. 3-6, intensities of VI, VII, and VIII on the modified Mercalli scale were felt in the metropolitan area at that time[3].

CLIMATE

Sewerage and drainage functions are affected in various ways by climatic factors. Air temperature, for example, determines the time and extent of the season of outdoor sports involving intimate contact

[3]*Source: "The Trend in Engineering", University of Washington, January 1953.*

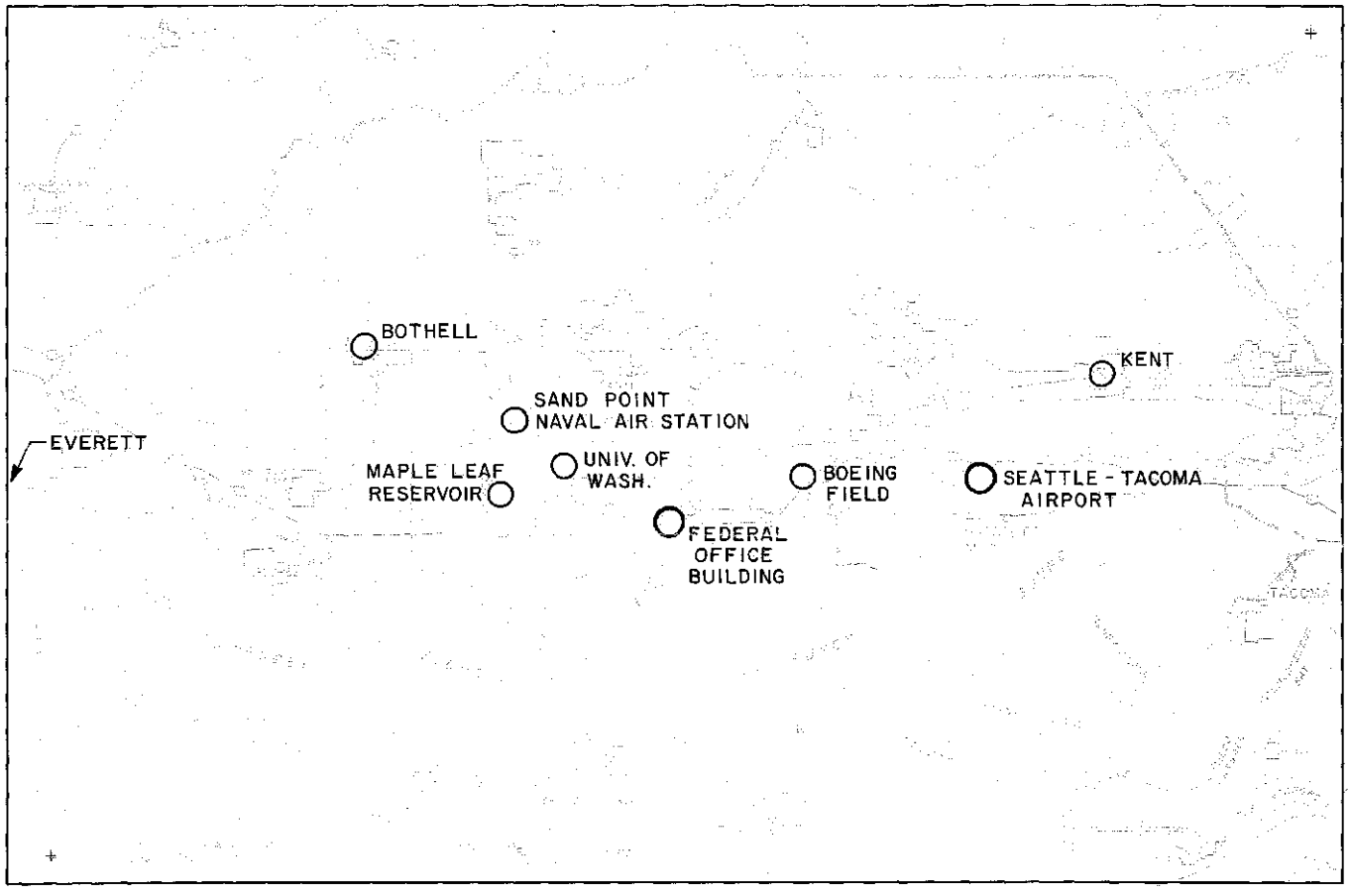


Fig. 3-7. Location of Climatological Stations

Climatological data are reported by the U.S. Weather Bureau for the nine locations shown.

with surface waters, both salt and fresh. Because surface water temperatures are related to air temperatures, a high air temperature condition accelerates biological activity, which in turn brings about the destruction and stabilization of organic matter contained in sewage and treated sewage effluents. During cold weather a greater amount of heat is required for heating sludge digestion tanks than in warm weather.

Wind has both beneficial and detrimental effects. When a wind is blowing, the rate of oxygen absorption in bodies of water is increased as is the rate of evaporation. Oxygen absorption is of importance in supplying oxygen for stabilization of wastes discharged to receiving waters, while evaporation aids in the drying of digested sewage sludge on open air beds or in lagoons. Wind also determines the distance and direction which disagreeable odors may travel and, in some cases, may require that provision be made for odor control or elimination. In recreational areas situated in the vicinity of waste discharges, wind induced onshore surface currents may impose added requirements for a higher degree of treatment. On the other hand, winds may cause choppy surface conditions which aid in the mixing of sewage effluent with the main body of water.

Rainfall is by far the most significant of the climatic factors. It governs the required capacity of storm drainage facilities and, through infiltration, may have a profound effect on collection systems designed solely for sanitary sewage. An analysis of the relationship between rainfall frequency, duration, and intensity is of prime importance in the design of storm drainage facilities. When utilized with appropriate runoff coefficients, these factors determine the quantity of storm water which must be conveyed to a suitable point of disposal. This relationship must be considered also in the design of intercepting sewers which serve combined systems carrying both storm runoff and sanitary sewage.

General Climatic Conditions

Marine air from the Pacific Ocean, 90 miles to the west, readily penetrates inland and is, to a large degree, the source of the equable year-round temperatures prevailing in the metropolitan Seattle area. To a lesser degree, the extensive water surfaces of Puget Sound serve to equalize temperature extremes. On the other hand, the Cascade mountains usually serve during winter months to block the westward movement of cold continental air masses. Average daytime temperatures during the winter are in the 40° to 50° F range, while those at night are in the 30° to 40° F range. Summer temperatures range generally from 70° to 80° F in the afternoon and from 50° to 60° F during the night.

Rainfall, modified considerably by the Olympic Mountains to the west, occurs during a pronounced, though not sharply defined rainy season and totals between 30 and 40 inches per year. The Olympics serve further to deflect or retard inland movement of the more severe storms occurring along the coast.

Local variations in temperature and precipitation, as recorded by the nine stations which report to the United States Weather Bureau, are indicated by the data in Table 3-1. Locations of these stations are shown in Fig. 3-7. For the area as a whole, temperatures in 1956 were slightly below normal, while precipitation was above normal. Variations between stations are attributable to the effects of local topography and the locations of the stations with reference to the sound and adjacent waters. It should be noted also that the actual instrument location at a given station can be such as to cause variations or erroneous readings of the same order of magnitude as the area-wide variations indicated in Table 3-1.

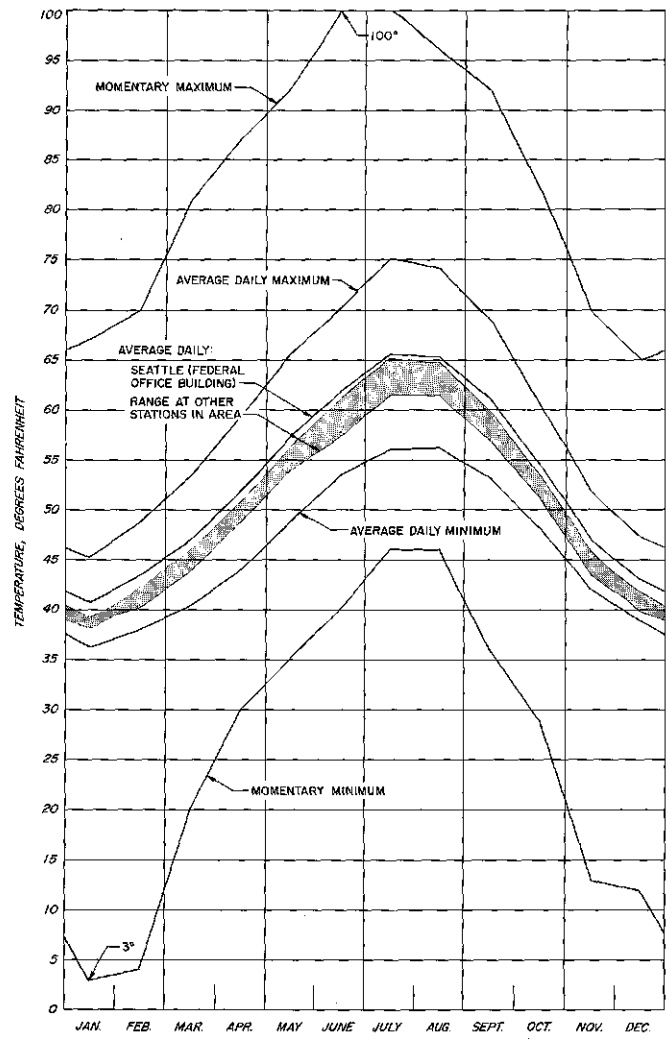


Fig. 3-8. Air Temperatures in Seattle

Air temperatures in Seattle are indicative of those in other portions of the metropolitan area. Average monthly temperatures at the other climatological stations vary by a maximum of 4° F.

Table 3-2. Temperature Data for Seattle

Month	Normals[a]			Extremes of record[b]		Normal degree days[a]
	Monthly	Daily Maximum	Daily Minimum	Highest	Lowest	
January	40.7	45.2	36.2	67	3	753
February	43.5	48.8	38.1	70	4	602
March	47.0	53.4	40.5	81	20	558
April	51.8	59.4	44.1	87	30	396
May	57.3	65.6	48.9	92	35	246
June	61.8	70.2	53.4	100	40	107
July	65.6	75.1	56.1	100	46	49
August	65.2	74.2	56.2	96	46	45
September	61.0	68.8	53.2	92	36	134
October	54.4	60.5	48.2	82	29	329
November	47.0	51.8	42.1	70	13	540
December	43.1	47.3	38.9	65	12	679
Year	53.2	60.0	46.3	100	3	4,438

Temperatures expressed in degrees Fahrenheit.

Source: "Local Climatological Data with Comparative Data – Seattle, 1956", U.S. Weather Bureau.

[a]*For 30-year period 1921-1950.*

[b]*For 64-year period ending in 1956.*

The season during which significant use of beach areas takes place normally extends from May through September. In any one year, however, actual use may begin earlier or end later, depending on climatic conditions then prevailing.

Temperature

Average annual temperatures do not vary greatly from year to year. Since 1905, averages recorded at the downtown Seattle station, now located at the Federal Office Building, have fallen within a range of 6.3° F, marked by a high of 55.5° F for the year 1940 and a low of 49.2° F for the year 1916.

Monthly and daily temperatures, of course, are subject to more variation. Fig. 3-8 shows temperature data for the downtown Seattle station and indicates by a shaded zone the range in average temperatures at the other climatological stations. Basic data from which the Seattle curves are plotted are given in Table3-2.

Table 3-3. Relative Humidity Data

Month	Seattle, Boeing Field[a]				Seattle-Tacoma Airport[b]			
	4:30 a.m.	10:30 a.m.	4:30 p.m.	10:30 p.m.	4:30 a.m.	10:30 a.m.	4:30 p.m.	10:30 p.m.
January	86	80	74	83	88	85	83	87
February	87	78	69	83	89	82	75	85
March	86	70	59	78	88	77	68	83
April	86	63	52	75	88	67	54	76
May	86	61	50	74	88	65	53	76
June	86	63	52	74	90	67	57	79
July	86	63	47	72	91	69	50	77
August	88	68	51	76	92	71	53	80
September	91	72	57	81	94	75	59	84
October	92	80	69	87	94	82	74	88
November	90	82	77	87	89	84	81	87
December	87	83	78	85	89	87	84	87
Year	88	72	61	80	90	76	66	82

Relative humidity is expressed in per cent.

Source: "Local Climatological Data with Comparative Data – 1956" for Seattle, Federal Office Building station and for Seattle-Tacoma Airport station.

[a]*Period of record 17 years; equipment formerly at Federal Office Building station moved to this location in 1939.*

[b]*Period of record 7 years.*

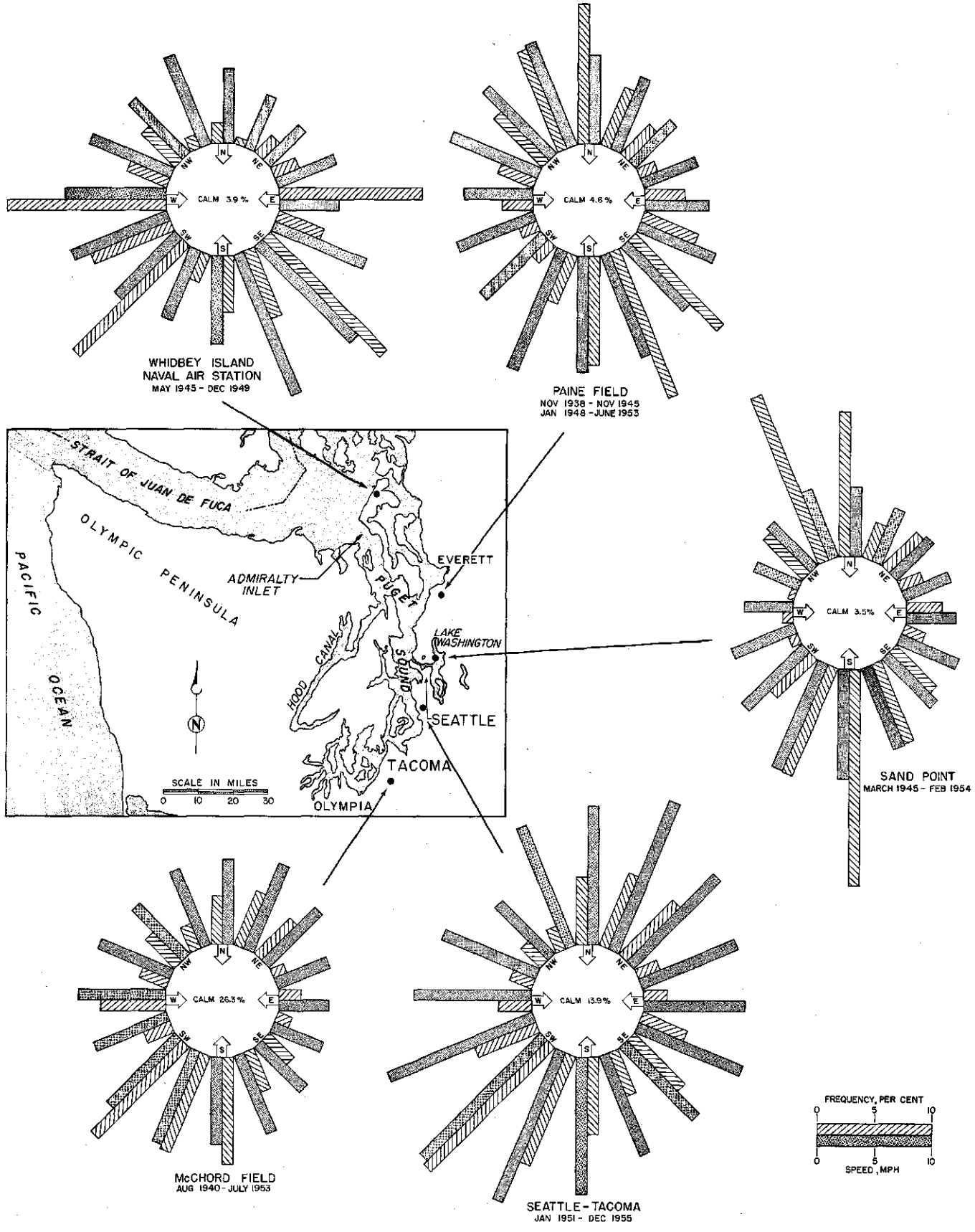


Fig. 3-9. Annual Wind Summary for Central Puget Sound Region

Frequency and velocity data were developed by the Air Weather Service and are given in Table 3-4. The prevailing wind direction varies from southwest at McChord Field to east at Whidbey Island and reflects the topographic effects of the Olympic and Cascade mountain ranges.

Table 3-4. Wind Summary for Central Puget Sound Region

Wind origin	Frequency of occurrence, per cent					Average velocity, mph				
	Whidbey Island	Paine Field	Sand Point	Seattle-Tacoma	McChord Field	Whidbey Island	Paine Field	Sand Point	Seattle-Tacoma	McChord Field
N	1.8	12.1	12.7	5.9	4.2	6.5	7.7	6.1	12.0	7.4
NNE	0.6	5.4	3.3	6.9	4.6	5.1	6.8	5.1	14.1	8.1
NE	2.1	3.5	4.2	6.7	4.4	4.4	5.1	4.6	10.8	6.5
ENE	2.3	1.5	2.1	2.1	1.2	5.5	5.1	4.4	9.1	4.8
E	12.5	3.5	3.2	2.2	1.9	5.2	5.6	4.4	9.0	4.3
ESE	4.2	5.1	1.9	4.3	1.3	8.6	7.2	5.5	10.0	4.6
SE	14.6	11.5	5.0	6.3	3.0	11.8	7.8	6.0	9.0	5.5
SSE	6.3	13.8	7.5	4.1	3.8	13.2	9.6	7.6	9.4	7.4
S	4.9	9.5	19.0	6.8	9.4	7.8	10.1	9.7	12.0	7.7
SSW	2.5	4.5	10.2	10.3	9.1	6.5	11.2	10.1	14.2	8.8
SW	13.6	2.1	4.8	15.4	11.3	7.5	7.5	7.3	15.0	8.5
WSW	7.5	1.7	0.8	5.8	5.6	9.0	7.2	5.8	13.7	8.9
W	14.0	2.6	0.9	2.8	5.7	8.9	6.0	4.3	10.3	7.7
WNW	3.8	3.4	0.5	2.0	2.5	7.9	8.3	4.2	8.9	6.9
NW	4.2	6.0	4.9	2.4	3.5	6.8	8.3	5.4	9.5	6.2
NNW	1.2	9.4	15.7	2.1	2.3	8.4	9.7	6.4	11.3	7.1
Calm	3.9	4.6	3.5	13.9	26.3	–	–	–	–	–
Total	100	100	100	100	100	8.1	7.9	6.9	10.3	5.6

Source: "Surface Wind Summaries" for stations listed obtained from Air Weather Service, Data Control Unit, U.S. Weather Bureau.

Period of Record: Whidbey Island Naval Air Station, May 1945 - December 1949.
Paine Field, November 1938 - November 1945 and January 1948 - June 1953.
Sand Point Naval Air Station, March 1945 - February 1954.
Seattle-Tacoma Airport, January 1951 - December 1955.
McChord Air Force Base, August 1940 - July 1953.

Normal monthly temperatures at the downtown station, computed for the 30-year period 1921-1950, vary about 25° F from January, the coldest month, to July, the warmest month. At other stations within the metropolitan area, average monthly temperatures may vary by 5° F from those at the Seattle station (Fig. 3-8). Consistently higher averages at the latter are attributable to the fact that minimum temperatures in the central or downtown business district rarely fall as low as they do in outlying areas.

Monthly averages of maximum and minimum temperatures reached each day (Table 3-2) indicates that maximum temperatures in July average 75.1° F as compared to minimums of 36.2° F in January. In 64 years of record, including 1956, the highest instantaneous or momentary temperature recorded at the Seattle station was 100° F in June 1955, while the lowest was 3° F in January 1893. Since 1893, the lowest reading was 11° F in January 1950.

Relative Humidity

Relative humidity data for the downtown Seattle station and for Seattle-Tacoma airport (Table 3-3) indicate essentially equal values at both stations. Lowest humidities normally occur in July and the highest in December. As applied to sewage treatment, relative humidity is significant in that a high humidity tends to retard the drying of digested sewage sludge on open air drying beds.

Wind

Surface winds at various locations within the metropolitan area are subject to considerable variation in direction, velocity and frequency. In general, prevailing winds in the Puget Sound basin are from a southerly direction during the winter months, while northerly winds occur more frequently during the summer. The strongest winds are almost always from the southwest.

Variations in wind direction, speed and frequency are illustrated in Fig. 3-9. This figure was prepared from annual summaries (Table 3-4) of wind observations made at Whidbey Island Naval Air Station, Paine Field (Snohomish County Airport), Sand Point Naval Air Station, Seattle-Tacoma Airport, and McChord Air Force Base just south of Tacoma. It will be seen that prevailing directions vary from southwest at McChord Field to southeast at Whidbey Island. Monthly variations at Seattle-Tacoma Airport are given in Table 3-5 and are shown graphically in Fig. 3-10.

Evaporation

Since 1948, evaporation data have been collected at Maple Leaf Reservoir, a unit of the Seattle water system situated in the north central section of the city. Table 3-6 gives the total evaporation and total precipitation averaged by months for the period 1948 to 1956 inclusive. It also gives the net evaporation, or the difference between total evaporation and precipitation, for the year 1956. Monthly total and net evaporation and total precipitation are shown graphically in Fig. 3-11 as averages for the period of record.

For 1956, as a whole, precipitation exceeded evaporation by 1.03 inches (Table 3-6). During the summer months, with higher temperatures and lower precipitation, evaporation was in excess of precipitation, while the reverse was true during the winter months. The data for June 1956 reflect an abnormal condition wherein precipitation (3.56 inches) was much greater than the average (1.81 inches) for June.

As applied to sewerage, a negative value for net evaporation indicates that sludge drying beds either must be equipped for removal of excess water or must be provided with covers. In addition, their capacity must be sufficient to hold sludge produced during winter months for subsequent drying during summer months.

Occurrence of Sunshine, Clouds and Fog

Table 3-7 lists data pertaining to sunshine, clouds and fog, as recorded at the downtown Seattle station. Similar data for Seattle-Tacoma Airport show generally comparable conditions.

On the average, these records indicate that 72 days per year are completely clear from sunrise to sunset, and that 46 of these, or 64 per cent, occur during the

Table 3-5. Monthly Wind Frequency and Speed Data for Seattle-Tacoma Airport

Direction	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
						Frequency, per cent						
N	3	4	5	7	7	7	8	7	10	7	3	3
NNE	5	6	5	8	7	6	8	7	11	10	5	5
NE	4	5	5	8	9	7	8	8	10	8	4	4
ENE	1	1	2	3	2	2	3	3	2	2	2	2
E	4	3	4	2	2	1	1	1	1	2	3	3
ESE	9	6	6	2	2	1	1	1	2	4	9	9
SE	9	7	7	5	4	2	2	3	4	8	13	12
SSE	6	5	4	4	3	2	2	3	3	5	6	6
S	10	9	7	5	6	5	4	4	6	9	9	10
SSW	16	12	12	8	8	9	7	8	10	11	9	13
SW	15	14	18	16	18	24	16	15	12	11	11	14
WSW	3	5	7	7	8	10	8	7	5	4	3	2
W	1	2	2	4	4	4	6	5	3	1	1	1
WNW		1	1	2	3	3	4	4	2	2	1	
NW		1	1	3	4	3	5	6	3	1	1	
NNW	1	2	1	3	4	3	3	3	3	1	1	
Calm	12	17	11	13	11	12	12	15	15	14	19	15
						Speed, mph						
N	12.2	11.3	13.5	12.9	12.2	12.6	11.5	11.3	12.3	10.5	10.3	11.6
NNE	12.0	11.5	12.1	13.0	11.4	12.1	11.4	11.3	12.1	12.5	11.6	13.5
NE	9.7	9.0	10.3	11.4	11.2	11.5	11.3	10.6	10.9	10.2	11.5	10.8
ENE	7.4	7.3	9.2	9.7	9.4	9.9	10.1	9.1	7.9	7.9	9.3	9.3
E	10.1	7.3	10.5	8.9	7.4	8.9	7.9	6.3	6.4	6.9	9.7	11.2
ESE	10.1	9.3	10.1	8.3	8.6	7.7	7.6	6.1	7.7	8.8	11.6	12.4
SE	9.1	8.4	8.4	9.0	8.0	8.0	7.3	7.3	7.5	9.6	9.8	10.3
SSE	9.9	8.9	9.3	10.3	8.9	8.2	8.3	7.8	8.2	9.9	10.3	10.1
S	13.7	12.2	12.4	11.2	9.5	8.8	9.1	8.3	9.5	11.7	12.6	13.4
SSW	16.9	16.3	15.6	14.6	11.4	11.2	10.2	10.1	11.0	14.6	16.5	16.5
SW	18.4	17.8	17.3	17.0	14.6	13.5	11.4	11.6	11.9	15.2	16.2	17.2
WSW	16.4	15.4	17.2	16.3	13.3	12.3	10.7	10.9	11.9	13.6	15.2	16.4
W	8.3	10.3	12.4	11.4	11.6	9.8	10.2	9.6	9.0	9.3	8.1	11.8
WNW	8.7	6.0	8.8	9.2	9.3	9.0	9.7	9.2	8.3	8.0	8.1	8.0
NW	6.4	7.7	8.7	10.1	9.9	10.3	9.3	9.4	8.7	8.1	11.8	10.4
NNW	11.7	10.5	12.7	12.4	11.8	11.7	10.8	11.0	10.6	9.3	10.6	9.5

Source: "Summary of Hourly Observations — Seattle-Tacoma Airport, January 1951 - December 1955", U.S. Department of Commerce, Weather Bureau.

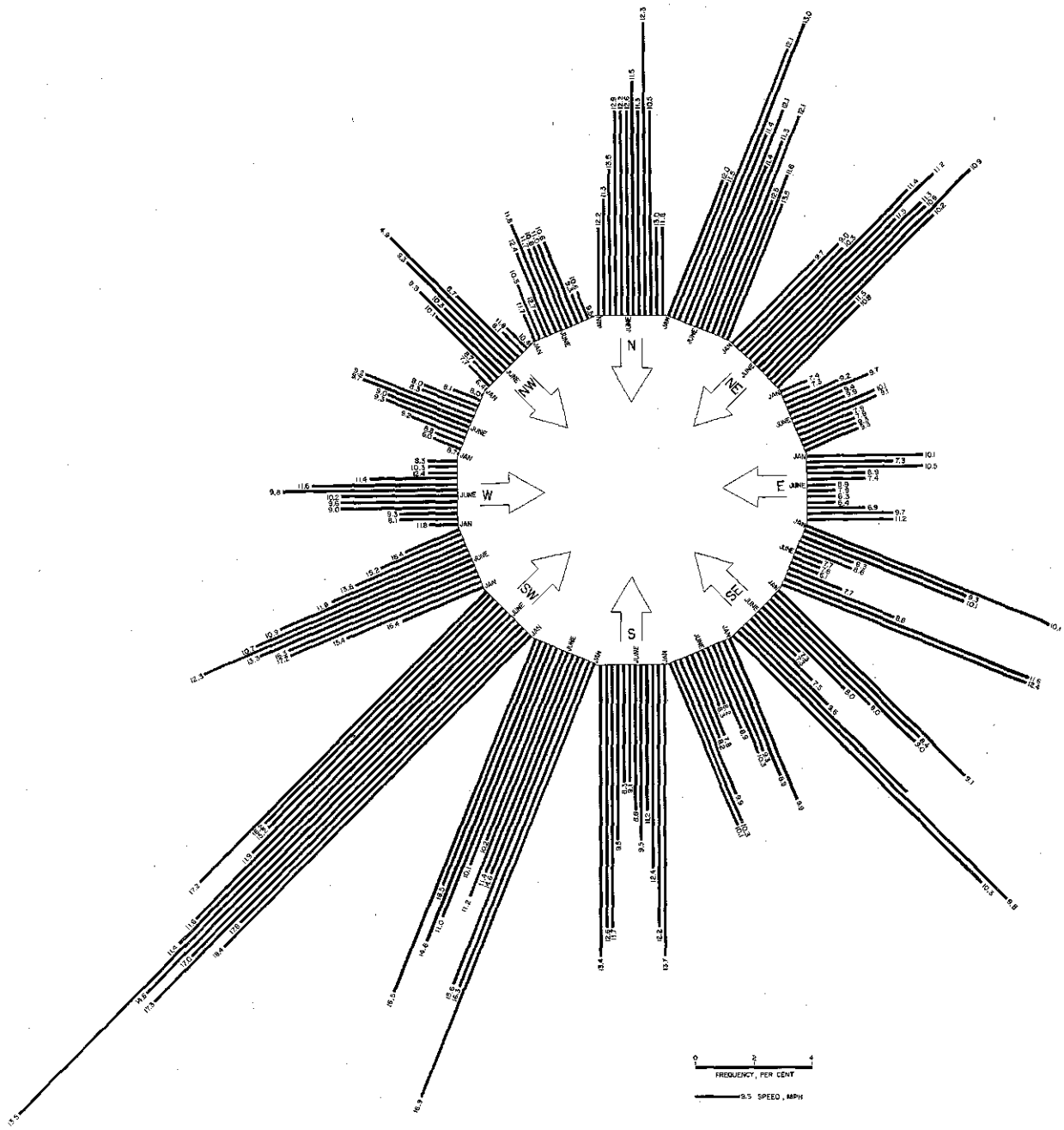


Fig. 3-10. Monthly Frequency, Speed and Direction at Seattle-Tacoma Airport

In contrast to Fig. 3-9, which shows average conditions for the entire year, this figure illustrates monthly variations. Data for the 5 years, 1951 - 1955, were utilized.

five-month period, May through September. Considering the year as a whole, 20 per cent of the days are clear, 25 per cent partly cloudy, and 55 per cent completely overcast.

About 45 per cent of the total possible sunshine occurs during an average year, while a higher average, 54 per cent, occurs during the five summer months. The degree or extent of cloudiness, expressed in terms of relative sky cover, is shown in Table 3-7. A value of 10 represents complete sky cover from sunrise to sunset, whereas lower values indicate increasing sunshine and clear periods. For the 23-year period of record, sky cover averages 6.8 per year and 5.7 per summer.

Heavy fog occurs infrequently, averaging only 24 days per year. This condition develops on an average of about 5 times during summer months and is confined mostly to winter months.

Table 3-6. Evaporation Data for Maple Leaf Reservoir, Seattle

Month	Long term average			1956
	Total evaporation inches	Precipitation[a] inches	Net evaporation inches	Net evaporation inches
January	0.62	6.17	−5.55	−7.47
February	0.86	4.48	−3.62	−2.01
March	1.68	2.85	−1.17	−4.32
April	3.17	1.82	1.35	2.99
May	4.61	1.69	2.92	5.36
June	5.07	1.81	3.26	0.74
July	6.29	1.13	5.16	7.20
August	5.09	0.97	4.12	4.84
September	3.61	1.79	1.82	1.10
October	1.69	3.15	−1.46	−2.49
November	0.78	4.92	−4.14	−1.53
December	0.57	4.29	−3.72	−3.09
Year	34.04	35.07	−1.03	1.32

Source: "Climatological Data - Washington, Annual Summary", 1948 to 1956 inclusive, U.S. Weather Bureau.

[a]*Averaged from individual monthly data excluding those with no corresponding evaporation data.*

Snowfall

Although the higher peaks of the Cascade and Olympic mountains are covered with snow for a good portion of the year and snow falls frequently at elevations as low as 2,000 feet, snowfall within the metropolitan area is not frequent and varies considerably from year to year. For the 23-year period ending in 1956, the mean yearly fall is 8.6 inches. Because the snow very often melts before it reaches measurable depth, falls of one inch or more are rare, occurring on an average of two times per year during the period of record.

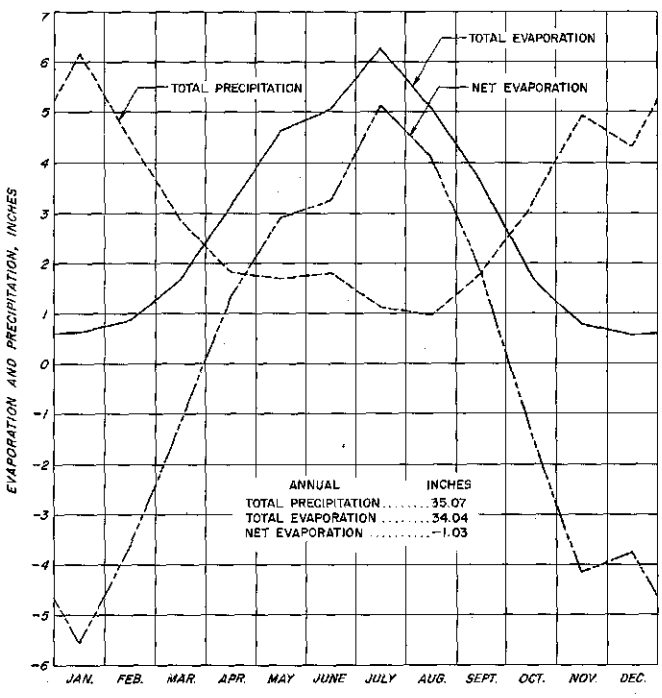


Fig. 3-11. Evaporation at Maple Leaf Reservoir, Seattle

Data for the period 1948-1956 indicate that, on an annual basis, precipitation exceeds evaporation by 1.03 inches.

Table 3-7. Long-Term Sunshine and Cloud Data

Month	Per cent of possible sunshine	Average sky cover[a]	Mean number of days[b]			
			Clear	Partly cloudy	Cloudy	Heavy fog[c]
January	28	8.0	3	5	23	2
February	34	7.7	3	6	19	2
March	42	7.4	4	8	19	2
April	49	6.8	5	10	15	1
May	53	6.4	7	10	14	
June	48	6.4	7	8	15	
July	61	4.8	12	10	9	
August	55	5.2	11	9	11	1
September	53	5.7	9	8	13	4
October	36	7.1	5	8	18	5
November	27	8.0	3	6	21	4
December	24	8.1	3	5	23	3
Year	45	6.8	72	93	200	24

Source: "Local Climatological Data, Seattle (Federal Office Building), 1956". U.S. Weather Bureau.

Period of record 23 years, 1933-1956, except heavy fog.

[a]*Expressed as a relative proportion of 10 which would indicate complete sky cover from sunrise to sunset.*

[b]*From sunrise to sunset, except fog which is for 24-hour period.*

[c]*61-year period of record as reported in above source for 1955.*

Table 3-8. Normal and Long-Term Precipitation Data, Seattle

Month	Monthly				Maximum in 24 hrs. inches
	Normal inches	Average inches	Maximum inches	Minimum inches	
January	4.49	4.80	10.93	1.43	2.47
February	3.87	3.68	8.10	0.34	2.69
March	3.06	2.96	7.23	0.42	2.92
April	1.94	2.13	5.53	0.16	1.74
May	1.61	1.73	4.67	0.09	1.35
June	1.25	1.38	3.70	0.03	1.42
July	0.52	0.60	2.36	0.00	1.22
August	0.87	0.68	2.50	Trace	1.43
September	1.56	1.67	3.46	0.08	1.91
October	3.08	2.92	7.43	0.02	2.22
November	4.46	5.07	9.50	1.04	3.20
December	5.34	5.26	15.33	1.00	3.52
Year	32.05	32.88	15.33	0.00	3.52

Source: "Local Climatological Data, Seattle, 1956", U.S. Weather Bureau, also 1955.

Period of record 65 years, 1892-1956, except normal which is based on 1921-1950.

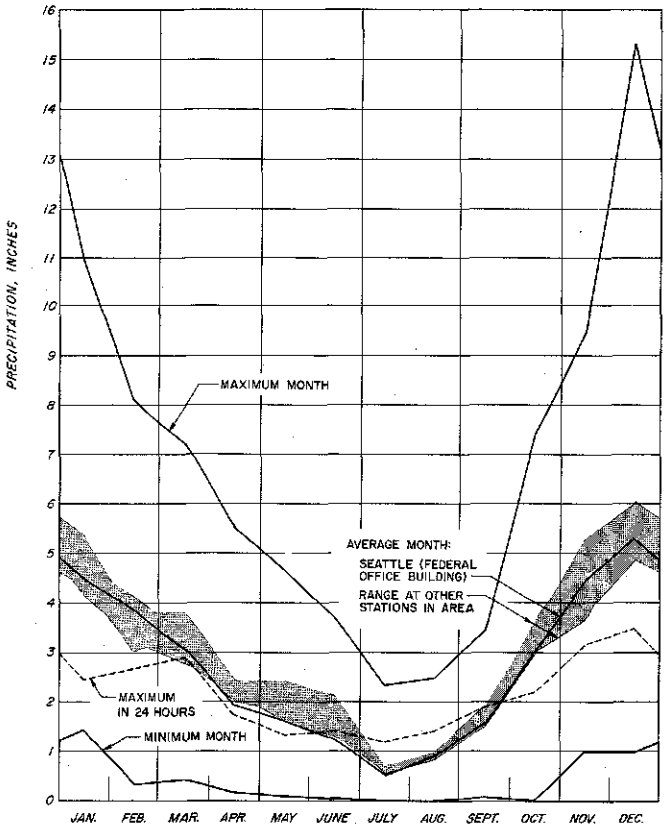


Fig. 3-12. Precipitation in Seattle

Only 18 per cent of the normal yearly precipitation occurs during the five months, May - September. Monthly normal precipitation at stations in the metropolitan area varies by about 1 inch.

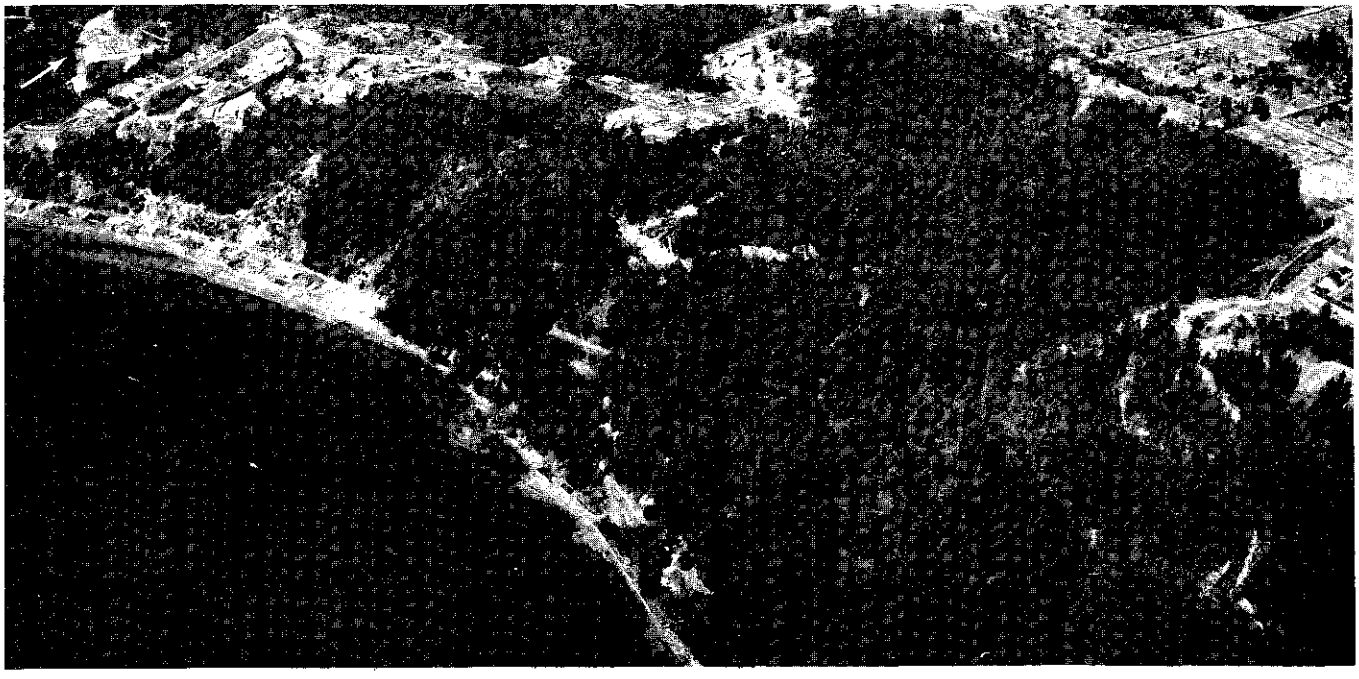
Deep snowfalls in the Seattle area and elsewhere along the shores of Puget Sound occur only when a storm is so oriented that it brings cold air directly out of Canada, or when it has traveled only a short distance over water. Since 1904, the records show traces of snow for seven winters and falls totaling over 20 inches for four winters. The maximum for one winter occurred in 1915-1916 and totaled 60.9 inches, while the greatest fall in 24 hours, 11.5 inches, occurred during January 1943.

Precipitation

As shown previously (Table 3-1), total annual precipitation over the metropolitan area varies somewhat with location. Annual and monthly averages for the downtown Seattle station are given in Table 3-8, together with the minimum and maximum months of record.

Variation in normal monthly precipitation is shown in Fig. 3-12 for downtown Seattle, while the range for other weather stations in the metropolitan area is indicated by the shaded zone in the same figure. A total of 5.81 inches normally falls during the five-month period, May through September. This represents 18 per cent of the annual normal precipitation.

Table 3-9 lists both the number of hours and the number of days during which various amounts of precipitation occurred. These data are based on an analysis by the U.S. Weather Bureau and represent annual averages for a five-year period, 1951 through 1955.



RAINFALL, through infiltration, has a significant effect on sewerage systems designed solely for sanitary sewage. Infiltration studies for various systems, including that tributary to the sewage treatment plant of the Southwest Suburban Sewer District (arrow) are described in Chapter 7.

Table 3-9. Occurrence of Precipitation at Boeing Field, Seattle

Range of precipitation, inches	Hours per year in stated range			Days per year in stated range		
	Summer (May - September)	Winter (October - April)	All Year	Summer (May - September)	Winter (October - April)	All Year
Trace	230	609	839	18	26	44
0.01	66	256	322	6	12	18
0.02 - 0.09	96	422	518	14	35	49
0.10 - 0.24	11	53	64	10	30	40
0.25 - 0.49	0.2	0.8	1	7	21	28
0.50 - 0.99	0.2	0	0.2	2	12	14
1.00 - 1.99	0	0	0	0.4	3	3
2.00 and over	0	0	0	0	0.2	0.2
Total	403	1,341	1,744	57	139	196

Source: Unpublished tabulation "Frequency of Precipitation Amounts, Boeing Field, Seattle, Washington". U.S. Weather Bureau.

Based on period January 1951 through December 1955. Values greater than 1 have been rounded to nearest whole number.

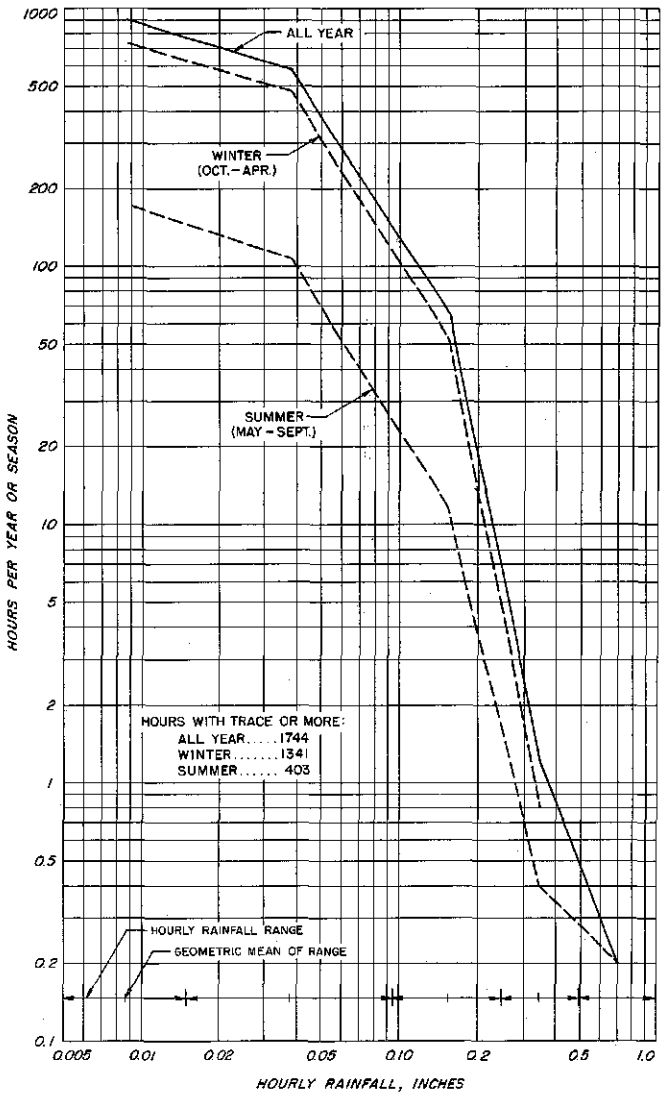


Fig. 3-13. Frequency of Occurrence of Hourly Precipitation

Based on data given in Table 3-9, the curves show the total hours during which precipitation equaled or exceeded each range.

Relative frequencies of occurrence are shown by grouping the data into summer and winter periods. For the summer season, rainfall of 0.01 inch or more occurred during 173 hours, or 4.6 per cent of the total number of hours in the season. On a daily basis, total falls of 0.01 inch or more occurred on 39 days, or 25.5 per cent of the days in the five-month period.

Fig. 3-13 shows the total number of hours per year and per season in which the hourly precipitation equaled or exceeded each indicated range of values. This figure was plotted by cumulating the hourly values in Table 3-9 from the highest range of rainfall to the lowest. Similarly, Fig. 3-14 shows the number of days per year and per season in which daily precipitation amounts equaled or exceeded each range.

Types of Storms

Most of the rainfall in the Seattle region comes from moving storms or areas of low barometric pressure which are common to the middle latitudes. Precipitation from these storms covers relatively large areas.

Thunderstorms, or thunder showers, which cover a relatively small area with short duration rainfall of high intensity, are rather infrequent (Table 3-10. During the 20 years analyzed, a total of 127 thunderstorms occurred in Seattle and of these, 66 took place during the summer for an average of about 3 per summer. It is evident, therefore, that this type of storm is not significant with respect to either recreational activities or, to any great extent, the frequency, intensity and amount of rainfall during the summer months.

Frequency, Duration and Amount of Rainfall

Design of storm drainage facilities requires a knowledge of the relationship between the frequency, intensity and duration of rainfall. Similar information is

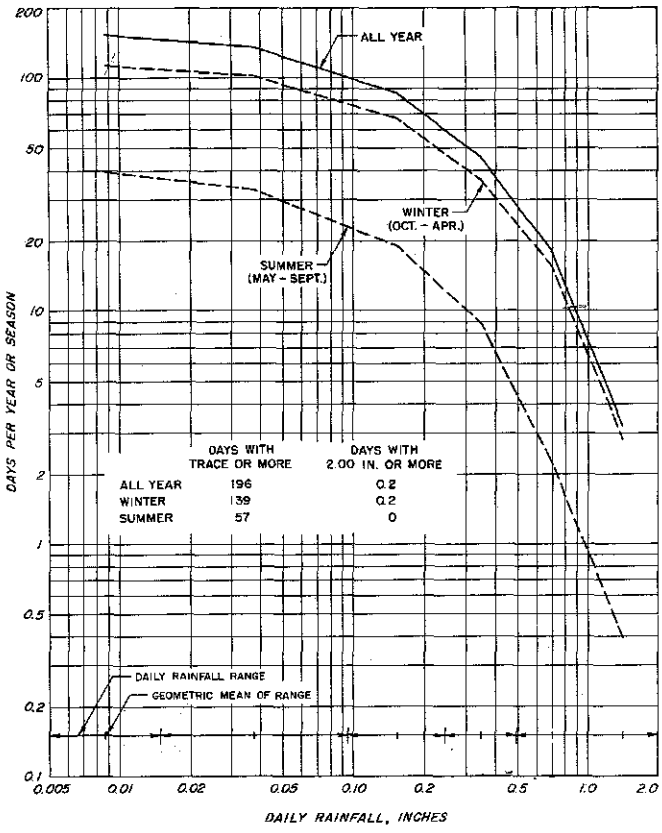


Fig. 3-14. Frequency of Occurrence of Daily Precipitation

Prepared in the same manner as Fig. 3-13, this figure shows the number of days in which precipitation amounts equaled or exceeded specified ranges.

required also for the design of intercepting sewers in areas served by combined sewers. In developing rainfall frequency-intensity-duration data for the Seattle study area, full use was made of records and data obtained from the U.S. Weather Bureau and from a recently completed sewerage and drainage study for the city of Tacoma, Washington[4].

Because the frequencies of storm recurrence considered for storm drain design and for intercepting sewer design are not of the same magnitude, it was necessary in each case to develop separate frequency-intensity-duration relationships. For storm drainage, recurrence intervals of one year and more are of most importance and the relationships must be based on an analysis of data covering the twelve months of the year. For intercepting sewers and related structures, however, recurrence intervals of less than one year are of most importance. In this case, conditions during the summer or recreational season must be anticipated and the relationships must be based on an analysis of data for the five summer months, May through September.

[4] *"Metropolitan Tacoma Sewerage and Drainage Survey", Brown and Caldwell, June 1957.*

Table 3-10. Total Number of Thunderstorms at Seattle in 20 years

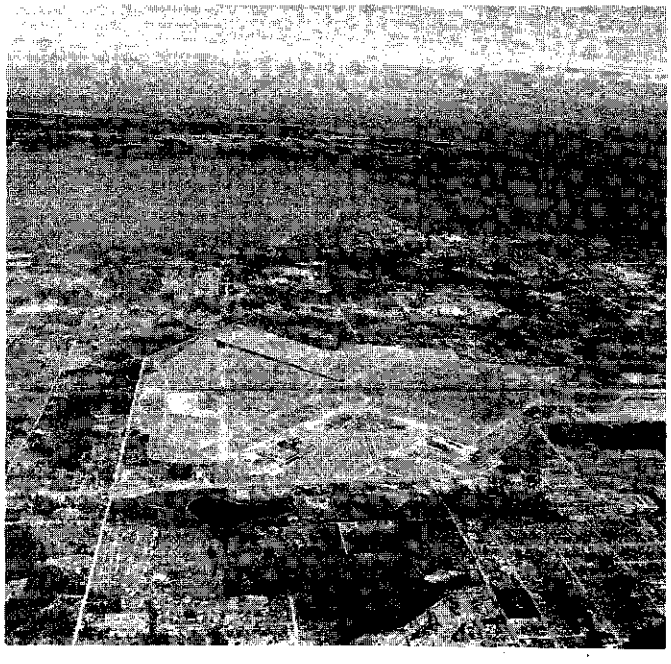
Season	Time of day				Total
	0000-0600	0600-1200	1200-1800	1800-2400	
Winter	0	0	0	2	2
Spring	1	1	20	6	28
Summer	6	9	23	28	66
Autumn	2	4	19	6	31
20 year total	9	14	62	42	127

Source: "Hydrometeorological Report No. 5, Thunderstorm Rainfall". U.S. Weather Bureau 1945.

Period of record 1906 to 1925, inclusive.

Analysis for Storm Drainage Design. Analyses of frequency-intensity-duration relationships have been made by the U.S. Weather Bureau for the period from 1903 to 1951 and are based on the automatic rain gage records of the downtown Seattle station. Results of these analyses were published recently in graphic form[5]. Curves plotted in Fig. 3-15 are based on information there presented and show the amount or depth of rainfall which occurs during a specific time interval or duration and which will be equaled or exceeded at a particular frequency over a long period of time. It must be pointed out that these curves do not represent the variations in rainfall which occur

[5] *"Rainfall Intensity-Duration-Frequency Curves", U.S. Department of Commerce, Weather Bureau Technical Paper No. 25, December 1955.*



U.S. WEATHER BUREAU STATION at Seattle-Tacoma Airport is one of the two principal sources of climatological information in the metropolitan area.

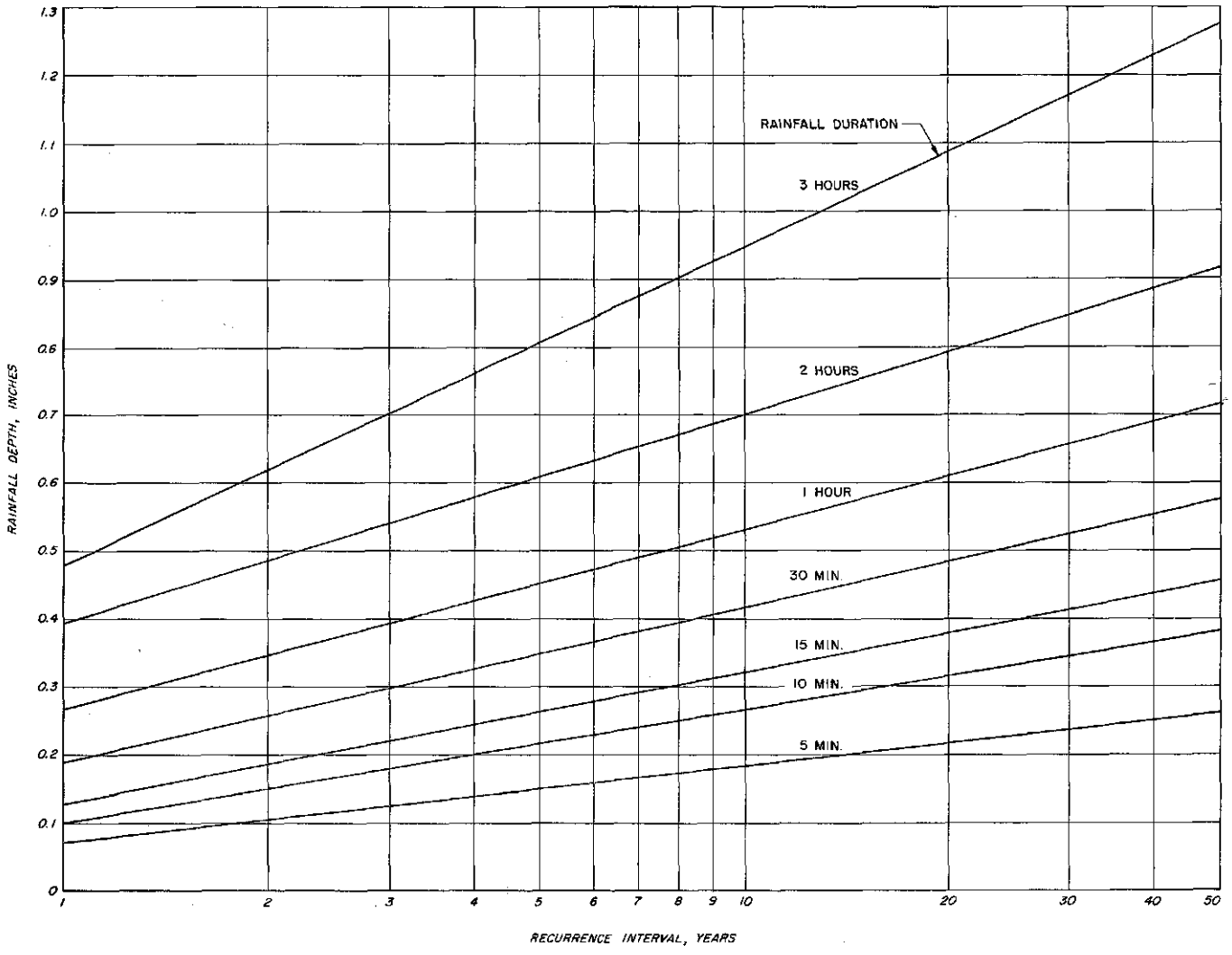


Fig. 3-15. Rainfall Frequency at Seattle, January through December

Based on curves developed by U. S. Weather Bureau and published in "Rainfall Intensity-Duration-Frequency Curves", Technical Paper No. 25, the curves indicate frequency of occurrence of specified rainfall amounts during various time intervals. Curves were derived from analysis of data covering the period 1905 - 1950 and do not represent the pattern of a typical storm.

during the course of a particular storm. It is extremely unlikely that any one storm would follow more than a small portion of a particular frequency pattern.

A design curve for a specific recurrence interval can be made by converting rainfall amounts for each duration to intensity. Curves thus prepared are presented later in Chapter 13 in connection with a discussion of design criteria for storm drainage improvements.

Analysis for Intercepting Sewer Design. Because of the availability of similar data developed for the Tacoma area, it was not necessary to make a detailed analysis of summer rainfall in Seattle. The Tacoma curves, based on an analysis of automatic rain gage charts for the summer months of 1940 through 1949, were modified to account for known differences in the frequency-intensity-duration relationships between the two cities.

Two independent analyses were made to determine the necessary modification. Of these, the first involved a determination, for summertime frequencies, of the relationship between 60-minute duration rainfalls at Tacoma and Seattle. In so doing, use was made of clock-hour data for the 10-year period as published by the U. S. Weather Bureau in "Hourly Precipitation Data". To develop the required information, the first step was that of tabulating the measured hourly amounts of rainfall in descending order of magnitude for each station. The frequency of recurrence for each amount was then determined by dividing the period, 10 years, by the order of magnitude of that amount. For example, the amount which was 10th in order of magnitude was equaled or exceeded a total

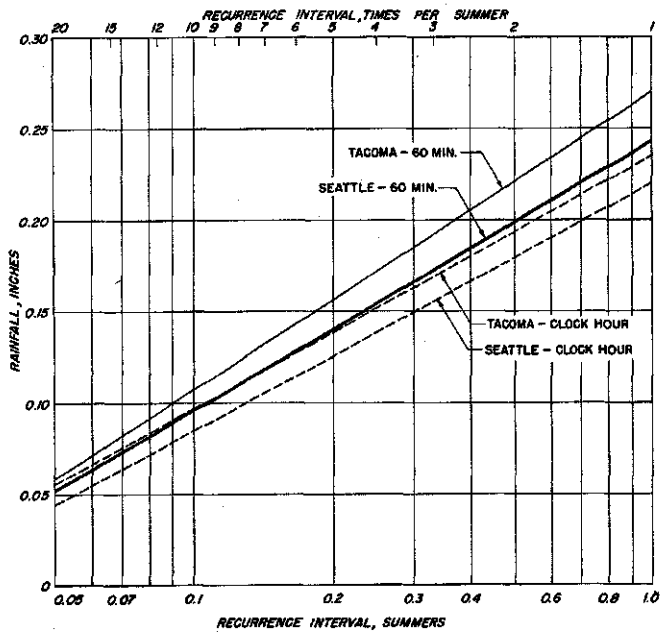


Fig. 3-16. Frequency of Clock-Hour and Maximum 60-Minute Rainfalls at Tacoma and Seattle

Clock - hour rainfalls at Seattle were compared with those recorded at Tacoma to determine the relationship between rainfalls at these stations.

of 10 times in 10 summers, and therefore has a recurrence interval of one year. This amount may be expected, on the average, to be equaled or exceeded once per summer. Similarly, the amount 50th in order of magnitude represents the amount which will be equaled or exceeded five times per summer.

Fig. 3-16 shows recurrence intervals for clock-hour amounts of precipitation at Seattle and Tacoma. The amounts of rainfall which are recorded in a clock-hour are not necessarily the maximum amounts which may fall in a 60-minute period. To illustrate this fact, the figure also shows the maximum 60-minute rainfalls for various frequencies as determined in the Tacoma study. For a given frequency, the maximum 60-minute fall is about 115 per cent of the clock-hour fall.

A comparison of the clock-hour amounts for Seattle and Tacoma indicates that, for a given frequency, rainfalls at Seattle are 90 per cent of those at Tacoma. The 60-minute curve for Seattle was drawn accordingly.

The second analysis was made for the purpose of determining whether the same factor is applicable to other rainfall durations. This was accomplished by a comparison of the intensities recorded in Weather

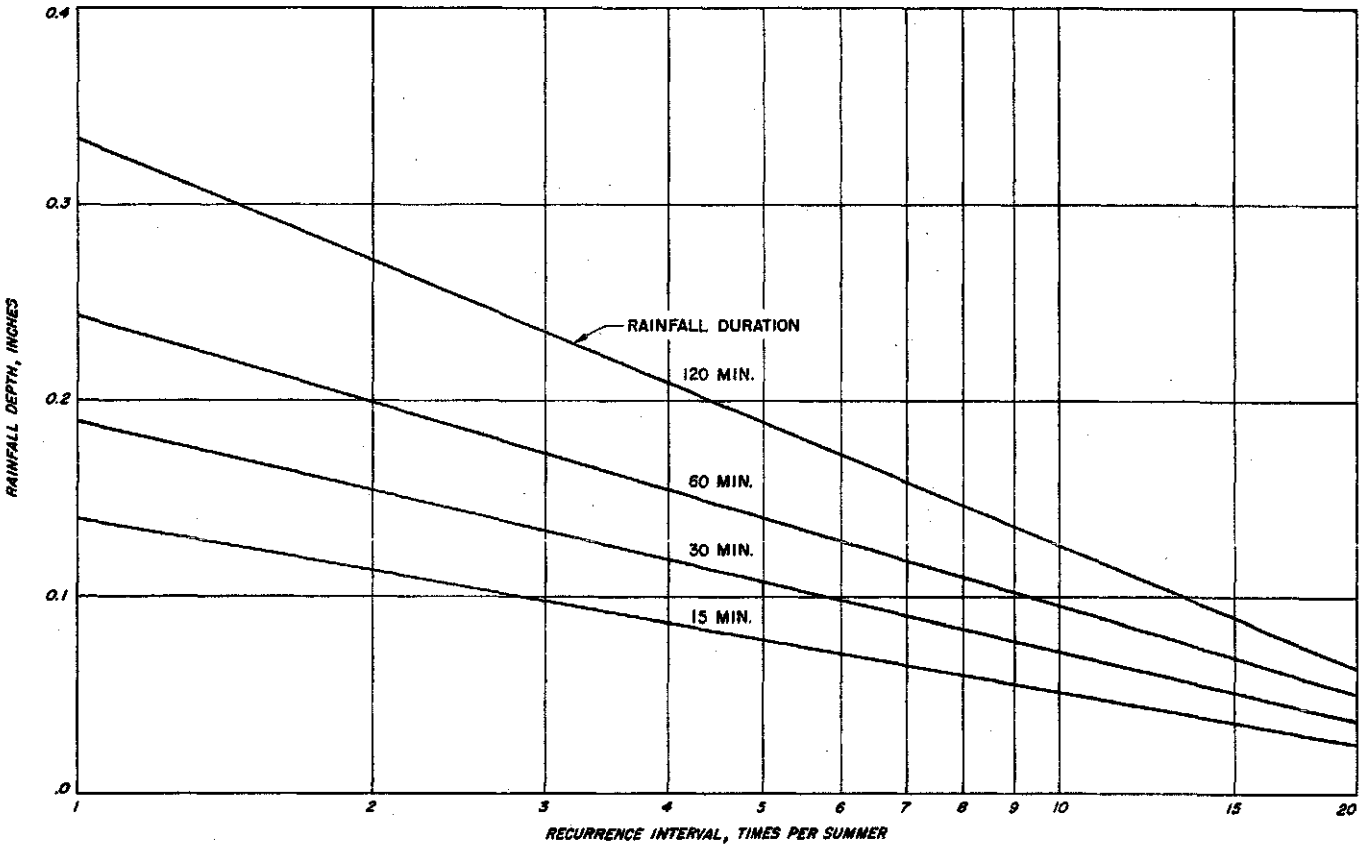


Fig. 3-17. Rainfall Frequency at Seattle, May through September

It was found that rainfalls at Seattle were 90 per cent of those recorded at Tacoma for frequencies and durations of interest in design of intercepting sewers. Curves for Seattle were adapted from similar Tacoma data based on analysis of individual storms during the summer months from 1940 - 1949.

Bureau Publication No. 25. For the more frequent occurrences and for durations between 15 and 120 minutes, Seattle intensities are about 90 per cent of those at Tacoma. This value was applied to the Tacoma curves in developing the summer rainfall curves for Seattle (Fig. 3-17).

Chapter 4
ECONOMIC DEVELOPMENT

As a background or basis for the study of population presented in Chapter 5, this chapter deals with the economic environment of the metropolitan area, including land and water uses, trade and commerce, and public utilities. It deals also with future land use patterns as they relate to sewerage and drainage planning.

Statistical data for the many factors affecting economic environment are not available for the metropolitan area as a whole. It is necessary, therefore, to present and evaluate such data as are available for city, county and regional units within and contiguous to the study area.

The metropolitan Seattle area has been long recognized as the major center of economic activity in the state of Washington, if not in the entire Pacific Northwest. The pioneer economy, initially based on lumber, soon expanded to include mining, agriculture, fishing and, of necessity, shipping. While still important, these activities have been surpassed by manufacturing and service industries.

By far the biggest factor in the development of the present economy of the metropolitan area is the growth of the Boeing Airplane Company. Employment in aircraft production in King County has increased from 6,000 in 1940 to 64,000 in June 1957. During the same period, the total number of all employed persons increased from 195,000 to 381,000.

Fig. 4-1 shows the relative magnitude of some of the economic factors discussed herein and illustrates the proportions applicable to King County, the Puget Sound area, the state of Washington and the Pacific Northwest.

LAND AND WATER USE

Land uses in the metropolitan Seattle area range from intense residential, commercial and industrial in the city of Seattle, to undeveloped cut-over lands and second growth timber in the northeastern, eastern and southeastern portions. Developed lands outside of Seattle include numerous small cities and suburban residential areas (Fig. 3-2). Agricultural uses are centered in the Green and Sammamish River valleys, with marginal uses in the upland areas.

Surface waters of the area are utilized for shipping, commercial fishing, harbor commerce, recreation, various types of industrial operations and, to a limited extent, domestic water supply. Use of the waters of Puget Sound, and of the Lake Washington and Green-Duwamish drainage basins is disclosed in detail in Part IV.

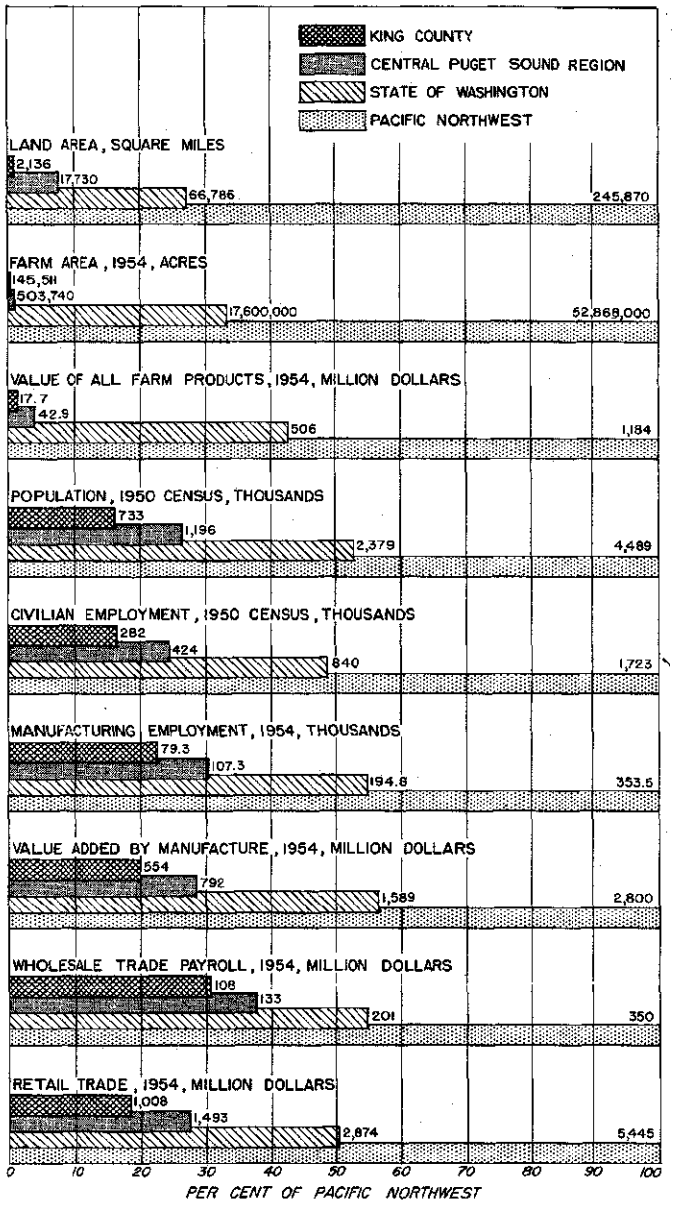


Fig. 4-1. Indices of Economic Activity

King County and the central Puget Sound region (King, Kitsap, Pierce and Snohomish Counties) are more highly urbanized and less dependent on agriculture than are the State of Washington and the Pacific Northwest (Washington, Oregon and Idaho). King County in particular is the center of manufacturing and trade. Data obtained from publications of U.S. Department of Commerce.

Table 4-1. Land Uses in City of Seattle, 1952 - 1954

Land use	Area, acres
Single family homes	17,435
Duplexes	578
Apartments	927
Business	999
Commercial	1,063
Manufacturing	1,590
Railroads	653
Public, semi-public	8,113
Streets	13,930
	45,288
Vacant	8,583
Total, land area	53,871
Submerged lands	1,186
Total city area	55,057

Source: City of Seattle Planning Commission.

Land uses in the city of Seattle, as determined during the period 1952-1954 by the Seattle Planning Commission, are listed in Table 4-1. Since about 10,000 single family dwellings have been constructed in the city subsequent to the Planning Commission study, the amount of vacant land (Table 4-1) has been reduced by about 25 per cent.

Residential Development

The major portion of the metropolitan area is now, or in the future can be made suitable for residential use. From Seattle, the central city, development has taken place to the north, east, and south, and has been spotty rather than uniform. Fig. 4-2 shows the number of residential units constructed and the type of construction within King County as a whole from 1950 through 1956. Construction outside the city of Seattle amounted to 38 per cent of the total for the county in 1950, to 65 per cent in 1955, and to 54 per cent in 1956. For the seven-year period, 55 per cent of all residential construction was in areas outside Seattle. Furthermore, 98 per cent of the outside construction consisted of single family homes as compared to 69 per cent within the city.

Residential expansion by tract type construction, such as that currently taking place, results in fully developed areas surrounded by relatively unimproved areas. This in turn tends to magnify and complicate the problems involved in providing necessary municipal utilities and services, including sewerage and drainage.

Industrial Development

Extractive industries, such as forestry, fishing, and mining, were developed early in the history of Seattle. In fact, the first lumber shipment was to San Francisco in 1852, the year the first permanent settlement was established. Early manufacturing operations were concerned primarily with the production of machinery and equipment for logging, fishing and shipbuilding.

Almost all of the standard manufacturing classifications are now represented in the metropolitan area. Growth of manufacturing industries has, of course, brought with it the usual proportion of nonmanufacturing and service type industries.

Many of the manufacturing categories represented in the early development of Seattle have been expanded by the addition of new products. For example, forest products now include plywood, pulp, furniture, and prefabricated structures in addition to cut lumber; food products include condiments and synthetic vanilla in addition to seafood and fruit and vegetable packs; and transportation equipment, after being limited initially to shipbuilding, is now expanded to include military and civilian aircraft, trucks, and refrigerated railroad cars. Additionally, the over-all industrial base has been broadened by the introduction of new industries and the manufacture of such products as plastics and fibreglass, certain types of building materials, textiles, and sporting goods.

As to the future, the outlook is toward a continuing expansion of the industrial base. For example, construction of several oil refineries is now or soon will be under way. These in turn will provide a local

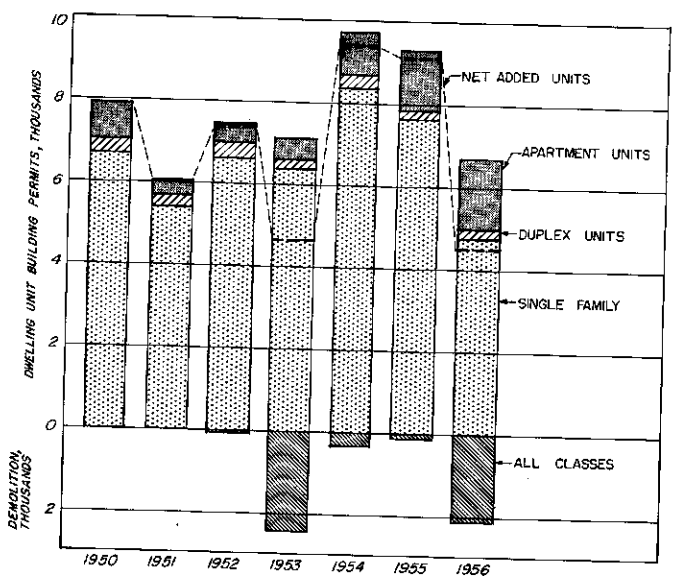


Fig. 4-2. Residential Construction in King County

In the seven-year period, 1950-1956, permits were issued for construction of almost 54,000 residential units. Of these, 85 per cent were single family residences. Demolished units, totaling over 5,000, were largely temporary war housing structures. Total residential units increased from 248,638 in 1950 to 297,716 by the end of 1956. Data obtained from Seattle Planning Commission.

source of petroleum products, as well as raw materials for the manufacture of petrochemical materials. A possible source of raw material for synthetic chemical industries became available in 1956 with the advent of natural gas.

Recently, a great deal of emphasis has been placed on long-range programs for industrial expansion. Several organizations are very active locally in this field, as is the Department of Commerce and Economic Development, a newly established agency of the state of Washington. One of the principal aims of the latter is the expansion of the industrial base of the entire state.

Industrial Areas. According to a study by the Seattle Planning Commission in 1952, 4,648 acres within King County were then utilized by industries, of which 1,590 acres were within the city of Seattle. Most of the heavy industrial development in the metropolitan area is situated in the lower reaches of the Green-Duwamish River valley and along the shores of Elliott Bay, Lake Washington Ship Canal, and Lake Union.

Future developments of any magnitude in the heavy industrial field are expected to be confined, at least initially, to the Green-Duwamish Valley. This is because of availability in that area both of flat land and of rail and water transportation. At present, the first stages of long-range plans for industrial development in the Duwamish valley are being implemented by the Port of Seattle Industrial Development District. Improvements now under way, together with those planned for the future, will provide a new industrial area of slightly over 3,000 acres.

Light industry is considerably more dispersed than heavy industry and is centered generally in developed areas adjacent to rail and arterial highway facilities. Future growth of light industry is expected to follow the same pattern and to utilize presently undeveloped land in areas where adequate transport services are available. It is possible, however, that operations of this type may develop also in close proximity to residential areas. Current experience in the San Francisco Bay area indicates that, with proper planning, neighboring development of light industrial and residential areas results in mutual advantages.

Industrial Classifications. Although a wide diversification of industries in the metropolitan area is in-

Table 4-2. King County Employment Statistics

Classification	Employees, thousands						
	1940 March	1944 July	1946 July	1950 Year	1954 Year	1956 Year	1957 January to June
Manufacturing industries							
Food, kindred products	7.1	8.2	8.4	8.3	7.4	7.4	6.8
Apparel, textiles	1.9	2.4	2.4	2.3	2.4	2.5	2.4
Lumber, wood products	7.1	7.3	5.6	7.9	6.1	6.8	6.5
Primary metals	2.0	4.7	3.0	2.3	2.0	2.5	2.7
Metal products, machinery	4.2	7.8	5.9	7.2	7.8	8.1	7.8
Transportation equipment							
Aircraft	6.0	38.8	10.0	19.5	36.8	43.2	58.1
Shipbuilding, repair	1.2	30.0	7.9	1.6	3.1	3.2	3.2
Other	0.7	1.1	1.8	1.9	2.7	3.2	2.8
Other manufacturing	6.9	6.6	7.2	8.4	9.5	10.3	10.1
Total manufacturing	37.1	106.9	52.2	59.4	77.8	87.2	100.4
Non-manufacturing							
Mining, forestry, fishing	2.4	1.6	1.6	1.7	1.6	1.5	1.2
Contract construction	8.6	9.0	12.9	13.4	12.5	15.0	15.4
Transportation, utilities, communication	16.9	22.1	24.1	25.9	25.9	27.5	27.6
Trade, services	64.4	84.8	97.3	110.1	119.9	130.0	129.0
Government	22.7	58.8	45.1	37.9	42.9	44.9	45.2
Total, non-manufacturing	115.0	176.3	181.0	189.0	202.8	218.9	218.4
Agriculture	6.7	10.8	10.6	8.1	7.9	7.8	6.5
Miscellaneous[a]	36.7	24.4	32.3	39.2	39.8	40.0	40.2
Total employment	195.5	318.4	276.1	295.7	328.3	353.9	365.6

Source: "Labor Force and Employment in the Seattle Area (King County)", Washington State Employment Security Department.

[a]*Includes self-employed, domestic servants, and others.*

Table 4-3. King County Manufacturing Statistics

Major industry group	1954				1947	
	Establishments, number	Employees, number	Payroll, $1,000	Value added,[a] $1,000	Employees, number	Value added,[a] $1,000
Transportation equipment	60	40,479	193,284	177,942[b]	20,156	67,097
Food and kindred products	226	9,393	38,723	67,389	8,113	45,566
Lumber and wood products	187	4,931	20,846	32,364	5,562	37,901
Fabricated metal products	146	4,134	19,058	32,290	3,574	21,351
Printing and publishing	190	3,631	16,601	27,004	3,225	22,830
Machinery, excluding electrical	128	3,137	15,262	23,811	3,248	16,860
Primary metal industries	41	2,304	10,588	17,103	2,499	12,943
Apparel and related products	88	2,372	7,080	11,407	2,293	8,520
	1,066	70,381	321,442	389,310	48,670	233,068
All other industries[c]	401	8,892	36,427	164,567	6,100	32,354
Total	1,467	79,273	357,869	553,886	54,770	265,422

Source: "1954 Census of Manufacturers, Washington, Preliminary Report", U.S. Department of Commerce, Bureau of the Census.

[a]*Defined as the value of shipments less costs of materials, supplies, fuel and electric energy.*

[b]*Incomplete value; see next footnote.*

[c]*Also includes values withheld from specific groups to avoid disclosing figures for individual companies.*

dicated by King County employment statistics (Table 4-2), it is nevertheless evident that employment is dominated by aircraft production. This is attested to by the fact that three out of five workers in the manufacturing industries are employed in the production of aircraft.

A similar domination is reflected by the manufacturing statistics given in Table 4-3 for 1954. This year was chosen because it is the latest one for which U.S. Bureau of the Census figures are available.

In considering the figures for the values added by manufacturing (Table 4-3), it should be realized that the specific value given for the transportation industry does not represent the true total for that group. In compiling statistics of this type, the practice is to exclude any values which would disclose information pertaining to the specific operations of a single company. On the other hand, the true values are actually included in the total both for all other industries and for the county as a whole.

On a state-wide basis, the leading manufacturing industry is again that of transportation equipment, principally aircraft[1]. For a twelve-month period ending September 30, 1956, aircraft industry payrolls amounted to a total of $222.9 million, representing 21.8 per cent of the total manufacturing payroll of $1,021 million for the entire state. During the same period, employment in aircraft production averaged 41,800, representing 20.3 per cent of the state-wide manufacturing industry employment total of 206,000. Table 4-4 lists state-wide payrolls in 1954 and 1956 for major manufacturing industry groups.

[1]"Summary of Pacific Northwest Industries, Annual Review," *Seattle First National Bank, April 1957.*

Transportation

In the field of passenger and freight transportation, the Seattle area not only serves western Washington and the Pacific Northwest but is the gateway to the shortest air and sea routes from a mainland port in the United States to Alaska and the Orient. Because of the favorable location of the area with respect to both overseas markets and mainland commerce, the transportation industry will continue to expand and will become an increasingly important segment of the total economy.

Water Transport. Marine transportation always has been and will continue to be a major feature of business activity in the metropolitan area. Elliott Bay is deep enough to accommodate all types of vessels and has a total shoreline of 10.6 miles. Of this

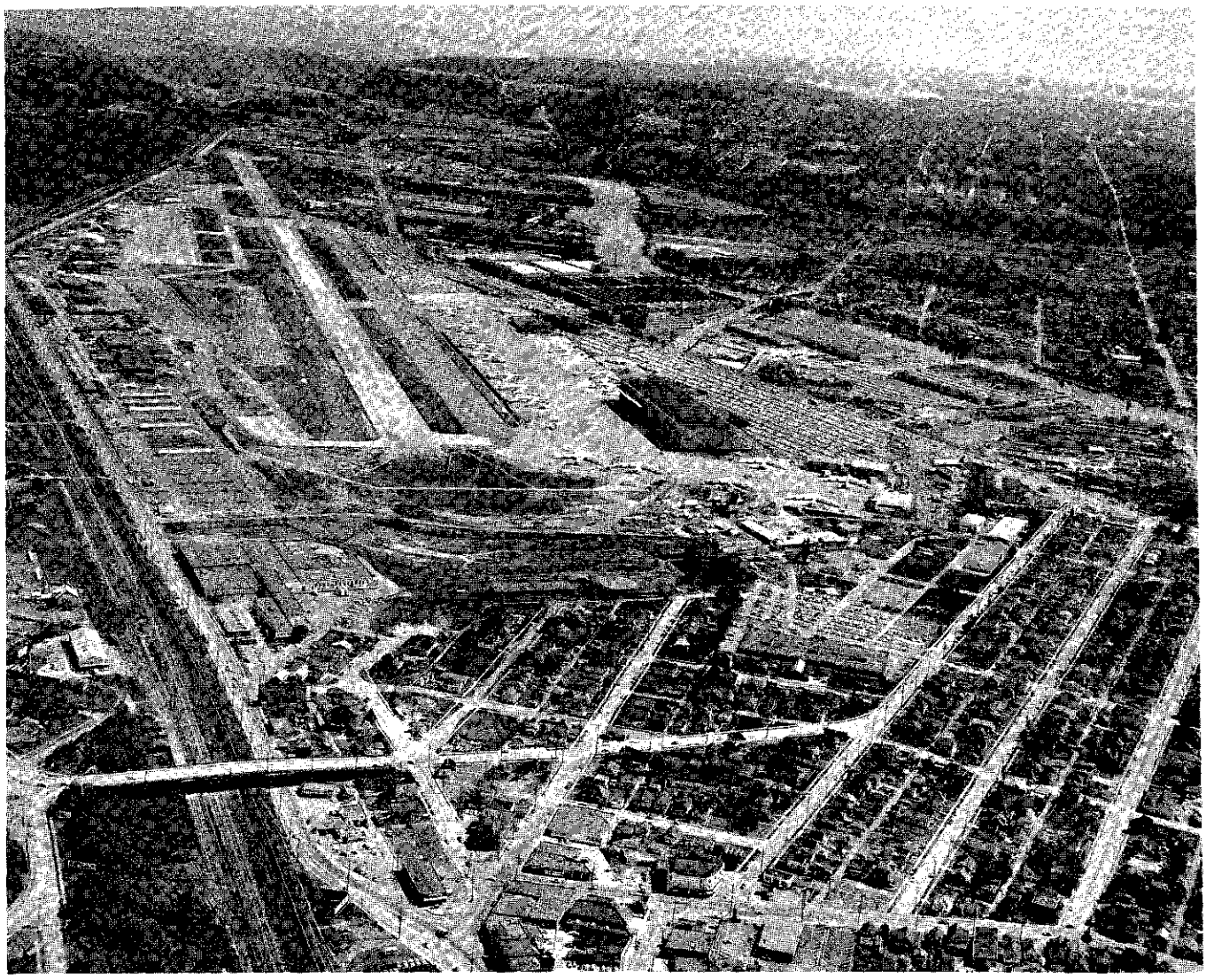
Table 4-4. Payrolls in Washington Manufacturing Industries

Industry group	Payroll,[a] million dollars		
	State of Washington		King County
	1956	1954	1954
Food products	102.8	97.4	38.7
Forest products	348.8	284.8	27.4
Metals and chemicals[b]	483.2	402.1	246.9
Other	86.2	70.7	43.8
Total	1,021.0	855.0	356.8

Source: "Summary of Pacific Northwest Industries, Annual Review", Seattle First National Bank, June 1955 and April 1957.

[a]*For 12-month period ending September 30 of year stated.*

[b]*Includes aircraft production.*



BOEING FIELD and the main Seattle Boeing plant (right) are located adjacent to Duwamish River. Employment at Boeing, largest individual industry in the metropolitan area, increased from 6,000 in 1940 to 64,000 in June 1957. Headquarters for air cargo and non-scheduled air lines are located on left side of field.

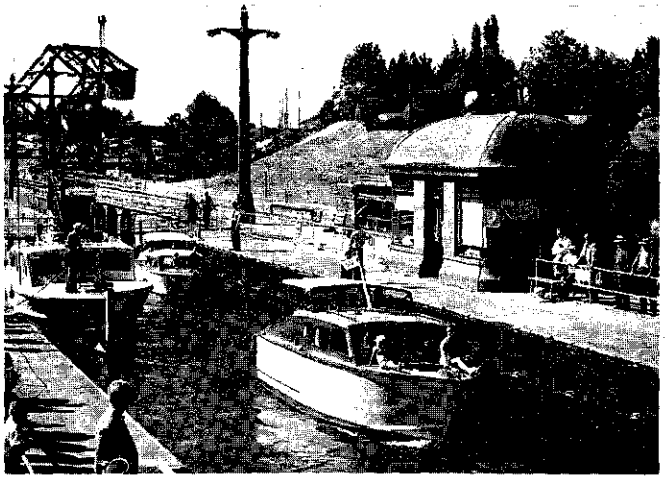
total, 8.7 miles, or 82 per cent, are occupied by marine enterprises. Duwamish waterway provides an additional 10.5 miles of shoreline, of which 1.7 miles are developed for marine uses.

Waterfront loading facilities consist of more than 90 piers, docks and wharves. Public docks and port services are provided by the Port of Seattle, a municipal corporation which has the power to purchase, lease, construct and operate various types of marine facilities. In 1956 the Port of Seattle owned properties valued at $37.7 million.

Lake Washington ship canal and Chittenden (Government) Locks, constructed in 1916, provide access to Lake Union and Lake Washington for vessels with a maximum draft of 30 feet. Of the 8.8 miles of shoreline in the ship canal area, which includes Lake Union and Salmon Bay, 6.3 miles are utilized by marine users. Moorage space is available for 1,000 fishing boats at the Fishermen's Terminal in Salmon Bay. This area is used extensively by vessels fishing Pacific Northwest and Alaskan waters. Use of the Lake Washington shoreline for water oriented industries, including shipyards, has declined greatly in recent years.

In the period from 1946 to 1956, the total traffic tonnages for Seattle Harbor increased from 9.6 to 13.7 million tons, or over 40 per cent. Seattle Harbor, as defined by the U.S. Corps of Engineers, extends from Point Wells on the north to Fauntleroy Ferry slip on the south and includes the ship canal, Lake Union, and Lake Washington,.

A breakdown of the number and types of vessels inbound and outbound during 1955 is given in Table 4-5. Although the number reported for the ship canal,



CHITTENDEN LOCKS, constructed in 1916 and operated by the U.S. Corps of Engineers, are located on the Lake Washington ship canal. Ship traffic on the canal, connecting Lake Washington and Puget Sound, totaled 60,000 vessels in 1955. Over half the number of vessels were recreational craft.

Lake Union and Lake Washington represents nearly half of the total for the harbor as a whole, this particular group includes a preponderance of recreational craft.

Highway Facilities. A system of state and county roads and highways serves developed sections of the metropolitan area. In addition, the area is traversed by two major transcontinental routes, U.S. 99, the Pacific Coast highway, and U.S. 10, the northern east-west highway. Auto ferries provide connections with the Olympic Peninsula to the west across Puget Sound, with various islands in the sound, and with Victoria, B.C. on Vancouver Island. Consideration is presently being given to the construction of a bridge across the sound.

Construction in 1940 of the floating bridge across Lake Washington has been a major factor in hastening development of the communities east of the lake. Traffic, however, has reached such proportions, particularly during morning and evening rush hours, that planning of a second floating bridge is now in progress.

Design of a limited access freeway connecting Seattle with Everett to the north and Tacoma and Olympia to the south is a current project of the State Highway Department. When constructed, this freeway will alleviate congested traffic conditions on U.S. 99 both north and south of Seattle and will tend to stimulate further residential and light industrial development in contiguous areas.

U.S. Highway 10 which is planned for an ultimate six to eight lane width, is now being widened to four

Table 4-5. Seattle Harbor Traffic in 1955

Class	Seattle Harbor[a]		Ship Canal[b]	
	Inbound	Outbound	Inbound	Outbound
Self-propelled vessels, number				
Dry cargo or passenger	36,128	36,163	821	823
Tanker	835	831	101	99
Tug and towboat	22,374	22,384	4,987	4,987
Fishing boats	c	c	6,179	6,540
Recreational craft	c	c	17,798	17,974
	59,337	59,378	29,886	30,243
Non-self-propelled vessels, number				
Dry cargo	4,709	4,688	1,488	1,465
Tanker	2,374	2,368	298	289
	7,083	7,056	1,786	1,754
All vessels, total number	66,420	66,434	31,672	31,997

Source: *"Waterborne Commerce of the United States, 1955. Part 4: Waterways and Harbors, Pacific Coast, Alaska, and Pacific Islands", Department of the Army, Corps of Engineers.*

[a]*Extending from Pt. Wells to Fontleroy ferry slip and including ship canal, Lake Union and Lake Washington.*

[b]*Includes Lake Union and Lake Washington.*

[c]*Included in other classes.*

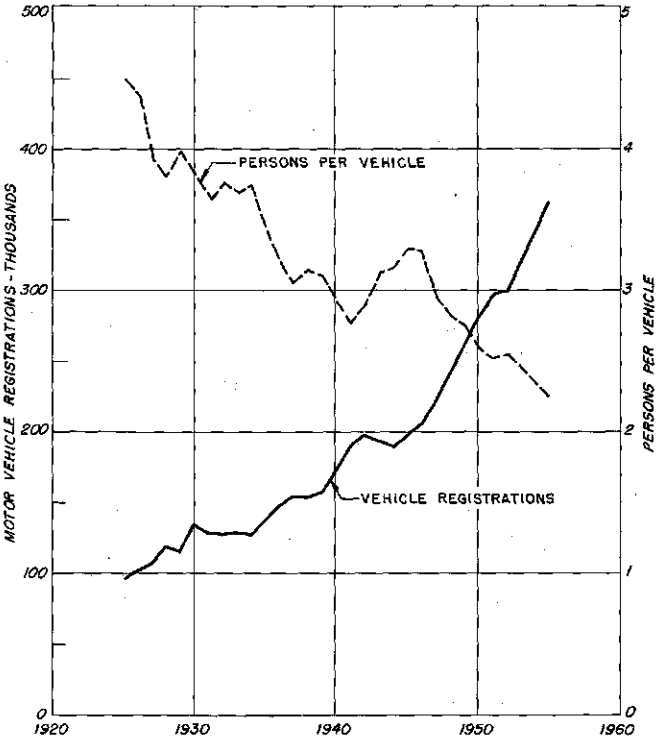


Fig. 4-3. Motor Vehicle Registrations in King County

From 1925 to 1955, motor vehicle registrations increased from 95,000 to 362,700. Passenger cars account for 80 percent of the vehicles in the county. Increasing urbanization and dependence on highway transportation contribute to the decreasing number of persons per vehicle. Data from State Department of Licenses.

Table 4-6. Motor Vehicle Registrations in 1955

Type	Washington State, thousands	Central Puget Sound Region[a]		King County		
		thousands	per cent of state	thousands	per cent of state	per cent of region
Passenger	927.0	459.6	49.6	292.0	31.5	63.5
Trucks	205.0	80.0	39.0	47.8	23.3	59.7
Trailers	76.7	29.8	38.9	16.4	21.4	55.0
Miscellaneous[b]	30.0	10.9	36.4	6.5	21.6	59.6
Total	1,238.7	580.3	46.9	362.7	29.4	62.6

Source: Department of Licenses, State of Washington.

[a]*Includes King, Pierce, Snohomish and Kitsap Counties.*

[b]*Includes vehicles for hire, private busses, motorcycles, and vehicles of all governmental agencies.*

lanes in the section over Snoqualmie Pass. A new limited access freeway paralleling the east shore of Lake Washington ultimately is to connect with both the proposed Tacoma-Everett freeway and Highway 2 across Stevens Pass.

Fig. 4-3 illustrates graphically the great increase in the number of motor vehicles registered in King County since 1925 together with the concomitant decrease in number of persons per vehicle. King County has over 30 per cent of the passenger vehicles in the state and over 60 per cent of those in the four counties, King, Pierce, Snohomish and Kitsap, making up the Central Puget Sound region (Table 4-6).

More than 100 carriers are engaged in furnishing motor freight service to Pacific Northwest points and the rest of the country. Transcontinental bus service is offered by two interstate lines.

Railroads. The metropolitan area is served by four transcontinental railroads, Great Northern, Milwaukee, Northern Pacific, and Union Pacific. These provide access to marketing points east and south as well as a connection to Vancouver, B.C. In addition, there is a local network of railway routes, dating back to early logging and mining activities.

Air Transportation. Seattle-Tacoma International Airport (Sea-Tac) and Boeing Field are the major airports serving the area. The former is the center of commercial air transport activity, while the latter is used mostly by Boeing Airplane Company for aircraft produced at its adjacent plant. Boeing Field is used also by air cargo and non-scheduled passenger carriers. Other fields in the area include those at Renton Municipal Airport, Bellview Airport, and Paine Field. Sand Point Naval Air Station is situated on the west shore of Lake Washington.

Traffic at Sea-Tac, which is owned and operated by the Port of Seattle, is increasing yearly (Table 4-7). In the short period from 1954 to 1956, the number of revenue passengers and the weight of air freight and air express increased over 20 per cent, and the amount of air mail increased more than 60 per cent. In terms of overseas passengers, Sea-Tac ranks third nationally, following New York and Miami. Runways are adequate for the largest transport planes now in

Table 4-7. Air Traffic at Seattle-Tacoma Airport

Classification	1954	1955	1956
Revenue passengers, number	1,048,383	1,181,564	1,286,126
Air mail, pounds	11,489,440	17,276,289	18,872,471
Air freight, pounds	26,259,194	29,962,543	32,259,329
Air express, pounds	2,297,904	2,542,549	2,837,967
First class mail[a], pounds			2,211,150

Source: "Annual Report", Port of Seattle, 1955 and 1956.

[a]*Service started in January 1956.*

SEATTLE - TACOMA AIRPORT is center for commercial air traffic, both domestic and overseas. In 1956, the total number of revenue passengers was 1,286,126, a 22 per cent increase since 1954. Construction now underway will extend runway to 10,400 feet, long enough for use by jet airliners and transport planes.

Table 4-8. Value of Farm Production in King County

Product	Value in million dollars	
	1954	1949
Field crops		
Vegetables	2.1	2.7
Fruits and nuts	1.0	0.8
Horticultural specialties[a]	2.4	2.6
Other field crops	0.04	0.06
	5.54	6.16
Livestock and products		
Dairying	6.6	6.0
Poultry	3.9	3.7
Other livestock	1.5	2.5
	12.0	12.2
Forest products[b]	0.18	0.07
Total farm products	17.7	18.4

Source: "1954 Census of Agriculture", U.S. Department of Commerce, Bureau of the Census, Vol. I, Part 32, Washington and Oregon.

[a]*Includes nursery and greenhouse products, flower and vegetable seeds and plants.*

[b]*Includes saw timber, posts and Christmas trees.*

service. At present, however, the main north-south runway is being lengthened from 7,500 feet to 10,400 feet to accommodate jet transports.

Boeing Field is owned and operated by King County and is conveniently located for transportation of air cargo in the industrialized Duwamish Valley. A single runway, 10,000 feet in length, is adequate for present and anticipated future needs.

Renton Municipal Airport is used by the Renton plant of the Boeing Airplane Company and by private planes. Its 5,400 foot runway would be difficult to lengthen because of adjacent highly developed land on the south and Lake Washington on the north.

Paine Field, a former Air Force base in the northern part of the area, is now partially owned by the U.S. Army but the runway section is owned by Snohomish County. The latter is operated as the Snohomish County Airport and is used also by the Army under a lease arrangement.

Agriculture

Soil classification studies by the State College of Washington indicate that there are about 540,000 acres of farm land in King County. These include 70,000 acres suitable for commercial farming, 45,000 acres of below-average land yielding a low economic return, and 425,000 acres of marginal or poorer land with little or no agricultural value.

As reported by the U.S. Census Bureau in the 1954 Census of Agriculture, the number of farms in King County decreased from 5,496 in 1950 to 5,181 in 1954 and the occupied farm acreage decreased from 153,301 to 145,111 acres. For the central Puget Sound region, farm acreage declined nearly 8 per cent between 1950 and 1954, whereas the total for the state increased about 1 per cent.

A comparison of the occupied farm acreage in King County with the figures given above for various classes of land indicates that a minimum of 30,000 acres classified as poor and marginal is actually utilized for farming. Undoubtedly, this contributes to the fact that operators of over 60 per cent of the farms in King County also engage in some type of off-farm employment.

Agricultural operations in the metropolitan Seattle area are confined largely to the fertile valleys of the Green and Sammamish rivers. It is expected, however, that the continuing pressure for residential and

Table 4-9. Washington and Puget Sound Fisheries Statistics

Species	Fish landings, million pounds				Washington value, million dollars			
	Washington		Puget Sound[a]		Catch[b]		Processed[c]	
	1955	1956	1955	1956	1955	1956	1955	1956
Salmon	61.8	31.1	48.7	20.0	11.8	10.9	21.0	20.8
Bottom fish	42.3	42.5	41.7	41.8	2.3	3.0	3.4	4.5
Halibut	16.1	16.2	16.1	16.2	2.6	3.6	3.4	5.0
Other food fish and livers	7.9	10.9	5.3	8.5	0.8	1.2	3.6	4.2
Oysters	10.1	10.6	3.4	3.4	1.3	1.5	6.0	7.4
Other shellfish	9.8	11.2	2.8	2.0	1.2	1.6	2.3	2.6
Total[d]	148.6	122.5	118.1	91.9	20.1	21.6	39.8	44.5

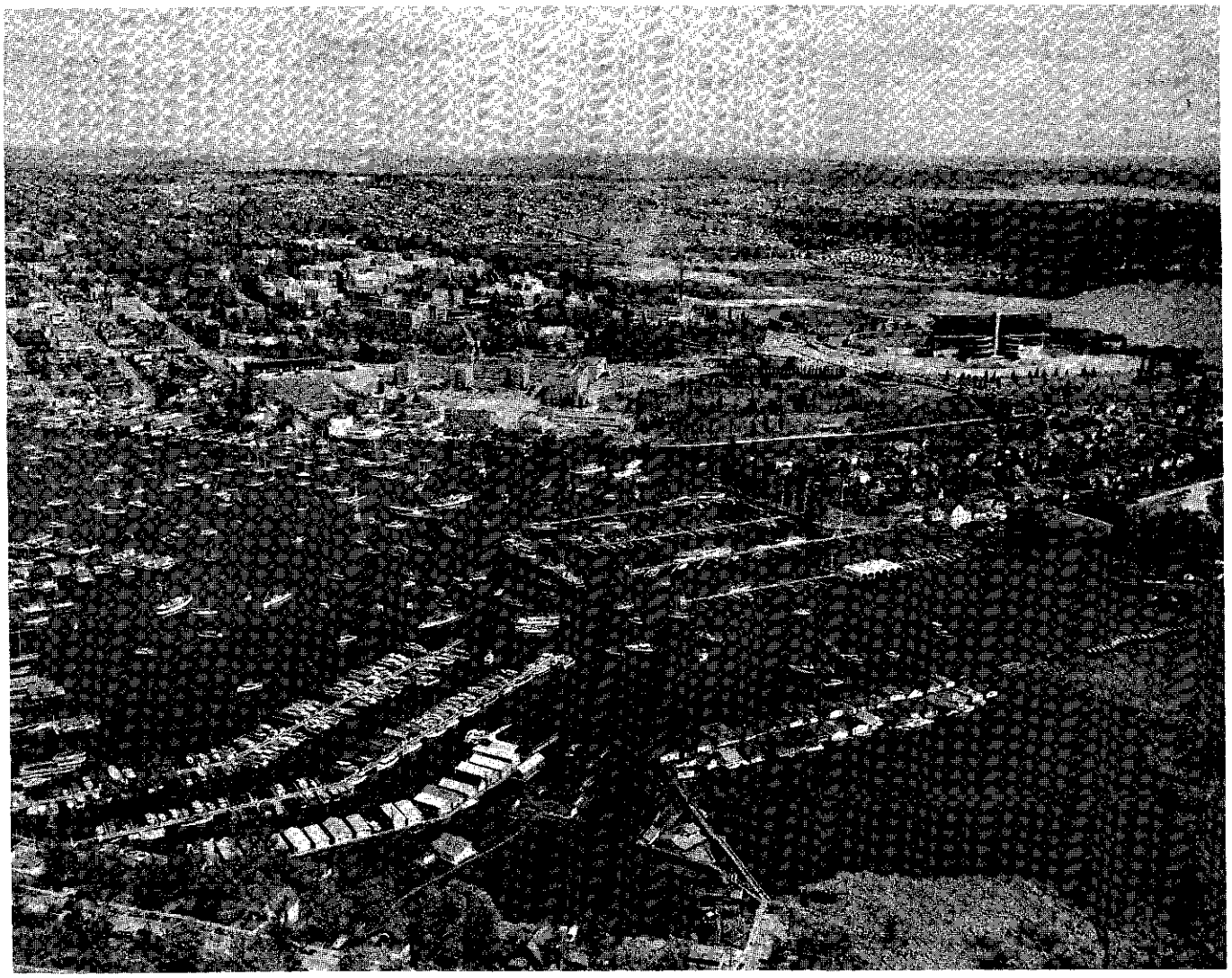
Source: "1956 Fisheries Statistical Report", State of Washington, Department of Fisheries.

[a]*Includes all fish landed at Puget Sound ports from catches in the Sound and Pacific Ocean.*

[b]*Price paid to fisherman for his catch.*

[c]*Wholesale, before shipment, value after canning, filleting, salting, or freezing; includes all by-products.*

[d]*Determined from sum of individual figures before rounding off.*



PORTAGE BAY, west of Lake Washington, is extensively used for mooring recreational craft. University of Washington, top, has an enrollment of over 14,000.

industrial development of the valley lands will lead to a further decline in farm acreage.

Farm Products. For the state as a whole, the value of farm products increased from $365 million in 1949 to $506 million in 1954. By comparison, values in the central Puget Sound region decreased from $43.7 million to $42.9 million during the same period.

Similarly, farm products in King County decreased from $18.4 million in 1949 to $17.7 million in 1954 (Table 4-8). In 1954, the value of dairying and dairy products amounted to $6.6 million or 37 per cent of the total for the county. Employment in agriculture is highest during the three months, July through September, and lowest during January. In the last 10 years, the number of employees in this category ranged from an average minimum of about 5,500 in January to a maximum of 11,000 to 14,000 in September.

Fisheries

Long recognized as a major center of commercial fishing, Seattle receives catches from Puget Sound, Alaska and the offshore waters of the Pacific. About two-thirds of the commercial salt water licenses sold in the state during 1956 were issued in the Puget Sound district.

Salmon is by far the most important commercial catch, accounting for roughly one-half of both the annual catch and the processed value of the Puget Sound fisheries industry (Table 4-9). Although the 1956 catch was quite low compared to 1955, total values for the two years were about the same. Good demand, combined with a short supply, served to nearly double unit values in 1956.

Sport or pleasure fishing in the waters of the sound is enjoyed by many thousands of residents and visitors every year. No accurate count of their numbers can be made, however, because licenses are not required.

Table 4-10. Parks and Other Recreational Areas in Seattle

Type	Number	Area, acres	
		Total	Water
Major parks	7	1,188	349
Minor parks	23	358	
Squares, view points	70	14	
Boulevards	8	361	
Waterfront parks and beaches[a]	23	103	85[b]
Golf courses	4	425	
Special areas[a]	19	370	
Total	–	2,819	434

Source: "Planning for Recreation", City of Seattle Planning Commission, 1954.

[a]*Includes some duplication with other types listed; area stated only for those not elsewhere covered.*

[b]*Total shoreline length, 18.2 miles.*

According to the State Department of Fisheries, sport catches of silver and chinook salmon totalled 476,000 in Puget Sound in 1956. For all the waters of the state, including the Columbia River, the total was 680,000.

Recreation

Endowed as it is with an abundance of natural attractions, the metropolitan Seattle area affords outstanding opportunities for recreational activity. This asset, coupled with that of scenic beauty, brings many thousands of visitors and a resulting tourist trade of considerable economic magnitude.

Recreational Facilities. Two national parks, Rainier and Olympic, and several state parks are within a few hours drive of the metropolitan area. Additionally, many public and private resorts are conveniently situated along the shores of lakes and streams and in the Olympic and Cascade ranges. Unlimited opportunities are available for hiking, camping, picnicking, and fishing in summer months and for hunting, skiing, and tobogganing in the colder months.

Boating. Puget Sound and Lake Washington, as well as the many small lakes in the metropolitan area, are used extensively for boating, sailing, and water skiing. In 1955, for example, the number of trips through Chittenden Locks by pleasure craft of all kinds totalled nearly 35,000 (Table 4-5). It is claimed, furthermore, that one out of every seven to eight Seattle families owns a power-driven boat.

From the standpoint of spectator and general community interest, the outstanding sports event in the boating field is the annual hydroplane competition on Lake Washington. The Gold Cup races, together with the attendant activities during Seafair week, attract nationwide attention.

Local Parks and Beaches. Within the metropolitan area itself there are three state parks: Saltwater, on the sound near Des Moines; Bridal Trail, east of Lake Washington near Houghton; and Lake Sammamish, at the south end of that lake.

The Seattle Park Department operates and maintains 7 major and 23 minor parks in addition to a large number of community recreation centers, playfields, squares and parkways (Table 4-10). This department also operates 23 waterfront parks and beaches on Puget Sound and Lake Washington. Recreational facilities occupy over 18 miles of shoreline. Attendance at supervised bathhouses and pools in 1955, as reported by the Park Department, totalled nearly 229,000 persons.

Military Installations

The Army, Navy, Air Force, and Coast Guard maintain establishments in the metropolitan area. Locations of the major installations are shown in Fig. 3-2.

Army. Fort Lawton, which borders Puget Sound south of Shilshole Bay, is the principal Army installation in the area. Others are the Seattle District Engineers Office on East Marginal Way, and the Quartermaster Depot at Auburn. Operations at the Army Terminal were terminated in mid 1957 with the transfer of its functions to a Navy installation at Pier 91. Chittenden Locks, though not a strictly military establishment, are owned by the U.S. Army and operated by the Corps of Engineers.

Table 4-11. Wholesale Trade in 1954 and 1948

Operation	Number of establishments		Sales, $1,000,000	
	1954	1948	1954	1948
Washington				
Merchant wholesalers	2,796	2,091	1,725	1,225
Manufacturers' branches	500	463	799	526
Petroleum and other fuels	698	608	340	236
Merchandise agents	355	297	389	346
Assemblers of farm products	309	323	205	227
Total	4,658	3,782	a	a
King County				
Merchant wholesalers	1,296	1,034	867	645
Manufacturers' branches	314	323	673	438
Petroleum and other fuels	51	49	108	77
Merchandise agents	272	227	298	258
Assemblers of farm products	8	6	8	13
Total	1,941	1,639	a	a

Source: *"1954 Census of Business, Wholesale Trade, Washington", U.S. Department of Commerce, Bureau of the Census, Bulletin W-1-47.*

[a]*State and county values are classified by the Census Bureau as "not applicable".*

ELLIOTT BAY waterfront is center of Seattle harbor activity. In 1955 the harbor ranked fourth in terms of domestic trade tonnage and sixth in foreign trade tonnage of Pacific Coast ports. Alaska Way viaduct, along waterfront, carries U.S. 99 highway traffic through congested downtown area of city.

Navy. Navy installations in the metropolitan area consist of the Sand Point Air Station on the shore of Lake Washington, the Naval Station and Supply Depot in Smith Cove at the northern extremity of the Elliott Bay waterfront, and the Reserve Ship Yards on Harbor Island at the mouth of Duwamish River. Sand Point Air Station is used also as the headquarters of the thirteenth Naval District and as a training base for a reserve unit of the U.S. Air Force.

Coast Guard. Coast Guard operations are centered at a base just upstream from the Government Locks in Salmon Bay. This base is used by patrol boats operating in Puget Sound and Lake Washington and serves in addition as a supply point for light ship stations. Pier 70 on the Elliott Bay waterfront also is utilized by the Coast Guard.

Air Force. There are no Air Force installations in the metropolitan area but reserve training units are based at Paine Field and at Sand Point Naval Air Station.

TRADE AND COMMERCE

Over one-half of the wholesale and retail establishments in the state are located in the Central Puget Sound counties. These establishments account for more than 50 per cent of the dollar value of the total trade.

Wholesale Trade

Wholesale trade, as measured by the number of establishments and the dollar value of total sales, increased substantially in the seven-year period from 1948 through 1954 (Table 4-11). During this period, the total number of establishments increased 123 per cent in the state as a whole and 119 per cent in King County. The largest gain was in the category of merchant wholesalers, which includes wholesalers, jobbers, distributors, foreign trade merchants and other related operations. In 1954, this category accounted for 60 per cent of the state's total number of wholesaling operations and for 67 per cent of those in King

Table 4-12. Regional and Local Wholesale Trade in 1954

Area	Total		Merchant wholesalers		Other Others types	
	Establishments, number	Payroll, $1,000,000	Establishments, number	Payroll, $1,000,000	Establishments, number	Payroll, $1,000,000
Washington	4,658	201	2,796	1,725	1,862	1,733
Central Puget Sound Region[a]	2,503	133	1,708	1,105	795	1,187
King County[b]	1,941	108	1,296	867	645	1,087
Auburn	8	0.08	5	0.7	3	1.0
Renton	17	0.7	14	4.2	3	1.4
Seattle	1,818	105	1,220	846	598	1,035
Rest of county	98	2.6	57	14	41	49

Source: "1954 Census of Business, Wholesale Trade, Washington", U.S. Department of Commerce, Bureau of the Census, Bulletin W-1-47.

[a]*Includes Kitsap, King, Pierce and Snohomish Counties.*

[b]*County payroll and sales totals computed before rounding off figures for political subdivisions.*

County. It also accounted for roughly one-half of the total sales value.

As indicated by the data in Table 4-12, the central Puget Sound Region accounted in 1954 for 54 per cent of the state's wholesale establishments and for 66 per cent of the payroll. These data reveal also that more than 75 per cent of the establishments and payroll were in King County and that this type of business activity is centered in the metropolitan Seattle area.

Retail Trade

Retail trade statistics for 1948 and 1954 (Table 4-13) indicate approximately equal rates of growth for the state, the Puget Sound Region, and King County. During the seven-year period, the number of retail establishments increased just under 10 per cent, while the dollar value of retail sales increased about 30 per cent. The latter is due to the decreased value of the dollar, to increased purchases per capita, and to the increase in population between 1948 and 1954.

Within King County, the greatest percentage increase in sales occurred in the cities of Auburn and Kent where the 1954 values were 158 and 162 per cent of the 1948 values. At Seattle, retail trade increased to 135 per cent of the 1948 value, and accounted for over 80 per cent of the county total in both 1954 and 1948.

Retail sales are divided by the Bureau of the Census into eleven major groups. In 1954, the food group accounted for 22 per cent of the total value in King County as a whole (Table 4-14). This was followed by the automotive and the general merchandise groups each with 14 per cent. In other words, 50 per cent of the total value falls in these three groups.

Foreign Commerce

In foreign trade tonnage, Seattle ranked sixth among the major Pacific Coast ports in 1955 (Table 4-15). In 1956, trade with Canada and Mexico accounted for 60

Table 4-13. Regional and Local Retail Trade in 1954 and 1948

Area	Establishments, number		Sales, $1,000,000	
	1954	1948	1954	1948
Washington	26,806	24,875	2,874	2,204
Central Puget Sound Region[a]	13,034	12,173	1,493	1,117
King County[b]	8,178	7,607	1,008	749
Auburn	145	118	19	12
Kent	131	81	11	6.8
Kirkland	112	89	10	7.1
Renton	231	187	28	19
Seattle	6,381	5,583	827	610
Rest of county	1,178	1,549	113	95

Source: "1954 Census of Business, Retail Trade, Washington", U.S. Department of Commerce, Bureau of the Census, Bulletin R-1-47.

[a]*Includes King, Kitsap, Pierce and Snohomish Counties.*

[b]*Total sales value computed before rounding off figures for political subdivisions.*

Table 4-14. Distribution of Retail Sales in King County, 1954

Business group	Sales in thousand dollars						
	Total	Auburn	Kent	Kirkland	Renton	Seattle	Remainder
Food stores	220,457	4,468	4,018	2,400	6,667	166,449	36,455
Eating, drinking places	83,267	824	595	508	1,713	70,605	9,022
General merchandise	143,214	813	606	884	1,859	134,327	4,725
Apparel	54,492	568	191	126	1,585	50,035	1,987
Furniture, appliances	43,853	604	673	494	1,911	35,650	4,521
Automotive	143,325	6,318	2,247	3,466	5,974	111,168	14,152
Gasoline stations	62,671	1,066	729	707	2,440	46,144	11,585
Lumber, building equipment	51,473	831	633	580	2,657	36,437	10,335
Drug stores	31,320	a	305	360	1,155	24,635	4,152
Other retail	113,902	2,593	978	757	2,097	92,631	a
Non-store retail[b]	60,082	a	184	72	36	59,120	a
Total	1,008,056	18,668	11,159	10,354	28,094	827,201	112,580

Source: "1954 Census of Business, Retail Trade, Washington", U.S. Department of Commerce, Bureau of the Census, Bulletin R-1-47.

[a]*Omitted to avoid disclosure of information which could be associated with operation of a single establishment.*

[b]*Includes mail order houses, house-to-house sales, and vending machine operations.*

per cent of the imports, while 45 per cent of the exports went to Japan and Korea. Limestone, limerock and gypsum rock accounted for 40 per cent of the import tonnage, and wheat and barley for 60 per cent of the export tonnage.

Fig. 4-4 portrays the development of foreign trade at Seattle since 1900. Variations in the totals of import and export tonnage follow those of national and world economy and, for the latter, reflect the status of trade with the orient.

Domestic Commerce

Domestic waterborne commerce includes four major classifications: coastwise, internal, intraport, and local. For the entire group in 1955, Seattle ranked fourth on the Pacific Coast, with a total of 11.1 million tons. Los Angeles Harbor ranked first with 14.4 million tons and was followed in order by Richmond Harbor and Portland.

Seattle Harbor receipts and shipments in 1955 are listed in Table 4-16 for coastwise, internal, and intraport categories. Local commodity movements totalled 629 thousand tons. Of these, the principal items, in thousand tons were: residual fuel oil, 238; rafted logs, 125; gas oil distillate fuel oil, 83; motor fuel and gasoline, 63; and lumber and shingles, 23. Total tonnage of domestic shipping in 1956 was 11.8 million

Table 4-15. Comparative Foreign Trade Figures for 1955

Harbor	Total, 1,000 tons	Exports, 1,000 tons	Imports, 1,000 tons
Los Angeles, Calif.	4,885	3,155	1,730
Long Beach, Calif.	3,066	2,277	789
Richmond, Calif.	1,748	767	981
San Francisco, Calif.	1,661	801	860
Portland, Ore.	1,408	1,258	150
Seattle, Wash.	1,357	698	659
Carquinez Strait, Calif.	1,283	145	1,138
Tacoma, Wash.	1,257	817	440
Oakland, Calif.	868	773	95
Redwood City, Calif.	174	93	81
San Diego, Calif.	149	93	56

Source: "Waterborne Commerce of the United States, 1955. Part 4, Waterways and Harbors: Pacific Coast, Alaska and Pacific Islands". Department of the Army, Corps of Engineers.

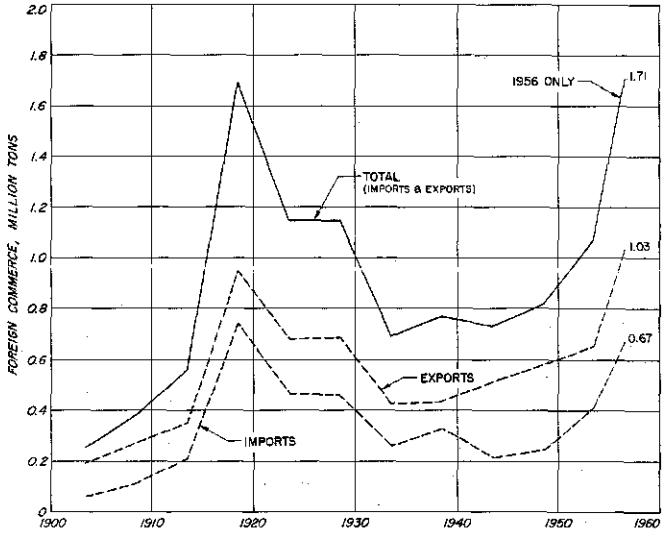


Fig. 4-4. Foreign Trade and Commerce, Seattle Harbor

Average values for five-year periods from 1905 through 1955. In 1956, total tonnage of foreign trade items exceeded the average for any five-year period. Exports have consistently exceeded imports.

Table 4-16. Leading Domestic Receipts and Shipments, Seattle Harbor, 1955

Receipts		Shipments	
Commodity[a]	1,000 tons	Commodity[a]	1,000 tons
Coastwise		Coastwise	
Gas, oil, distillate, fuel oil	1,759	Lumber and shingles	50
Motor fuel and gasoline	1,424	Building cement	31
Residual fuel oil	1,266	Motor fuel and gasoline	21
Crude petroleum	118	Rolled finished steel mill products	19
Petroleum asphalt	89	Posts, poles and piling	18
	4,656		139
All other commodities	469	All other commodities	510[b]
Total	5,125	Total	649
Internal		Internal	
Sand, gravel, crushed rock	1,515	Gas, oil, distillate, fuel oil	395
Rafted logs	597	Residual fuel oil	377
Motor fuel and gasoline	170	Motor fuel and gasoline	322
Gas, oil, distillate, fuel oil	155	Rafted logs	139
Residual fuel oil	114	Wheat	85
	2,551		1,318
All other commodities	284	All other commodities	289
Total	2,835	Total	1,607
Intraport		Intraport	
Rafted logs	62	Gas, oil, distillate, fuel oil	132
Lumber and shingles	7	Motor fuel and gasoline	26
Posts, poles and piling	2	Residual fuel oil	26
	71		184
All other commodities	3	All other commodities	21
Total	74	Total	205

Source: "Waterborne Commerce of the United States, 1955. Part 4: Waterways and Harbors, Pacific Coast, Alaska and Pacific Islands". Department of the Army, Corps of Engineers.

[a]*Department of Commerce classifications.*

[b]*Indicative of wide variety of commodities; about half of this total accounted for by commodities not specifically covered by the classifications.*

tons, a 6 per cent increase over the 11.1 million tons in 1955.

Tourist Trade

On a statewide basis, the value of the tourist industry has more than doubled since 1948. According to estimates by the State Department of Conservation and Development[2], tourists and vacationists spent a total of $271 million in the state in 1956, as compared to $116 million in 1948. These totals include such items as food and lodging, gasoline and other transportation costs, entertainment, gifts, and photographic and other equipment.

In 1956, over one million out-of-state automobiles entered the state and the total number of tourists, including those using other forms of transportation was estimated at 3.7 million persons. The average tourist party numbered 3.3 persons, stayed 7.38 days and spent $9.87 per day per person.

Seattle is the leading convention city in Washington. Data compiled by the Bureau of Economic and Business Research of Washington State University[3] indicate that, in 1952, Seattle had a total of 72,000 convention visitors who spent a total of $4.2 million.

A long-term, continuing program aimed at attracting increased numbers of tourists to the state is presently being developed by the new State Department of Commerce and Economic Development.

PUBLIC UTILITIES

Abundant and relatively cheap sources of water and power are available to the metropolitan Seattle area and have played a major role in its economic development.

[2]"A Report on Washington 1956 Vacation Travel," *Department of Conservation and Development, Bureau of Progress and Industrial Development.*

[3]"An Analysis of Convention Business in Washington State."

Table 4-17. Operating Water Supply Agencies

Name	Area, square miles	Connections 1957[a]	Source of supply[b]	Capacity,[c] mgd
Cities				
Auburn		2,750	S	6.0
Bothell	1.00	700	W	0.3
Houghton			d	
Issaquah		931		0.230
Kent		2,225		2.7
Kirkland	3.70	2,521	W	
Pacific				
Redmond		600	W	0.237
Renton			e	
Seattle	91.57[f]	172,060	R	205
Tukwilla	1.00	280	e	
Water Districts				
1 Yarrow	0.40	162	S	0.300
4 Three Tree Point	1.50	475	S	0.500
14 Bryn Mawr				
17 Hunts Point	0.30	90	g	0.112
20 Boulevard Park	9.50	6,950	e	1.000
22 Beaux Arts	0.13	94	W	0.108
23 Medina	1.40	400	g	2.000
25 Duwamish		420[h]	e	0.105
35 Foster		460	e	0.325
38 Riverton Heights			e	5.970
42 North City			e	42.214
43 Riverton Heights	7.00	1,725	W	1.080
45 White Center		1,000	e	0.062
49 Burien	1.00	3,000	W	
53 Angle Lake	1.00	181	W	
54 Des Moines	1.00	525	W	0.650
56 Redondo	3.40	581	S	1.000
57 Lake Ridge		57	e	
58 Spring Glen	4.00	725	e	0.800
61 White Center			e	3.118
62 Earlington			e	
63 Lake Ridge		735	e	
64 Steel Lake	6.00	601[h]	W	0.200
68 Bellview	8.10	6,100	L	8.000
69 Skyway	0.16	187	e	0.052
72 Juanita	9.90	500	W	0.360
75 Midway	11.40	3,500	W	2.450
77 Skyway			e	4.668
78 Cedar River	0.75	220	e	0.898
79 Kenmore	8.50	1,510	e	0.176
80 Christopher	16.00	500	S	0.160
81 Rose Hill	4.00	600	d	2.000
82 Pine Lake	1.40	98	W	0.050
83 Lake Forest	4.00	650	W	
85 Seahurst	1.00	392[h]	S	
87 Thomas	2.00	89	S	0.043
88 Skyway	0.19	135	e	0.729
90 Coalfield	6.50		W	0.350
91 Mercer Island	0.22	207	W	0.140
93 Mercer Island	1.67	1,092	W	0.750
97 Eastgate Hills	1.87	800	W	2.000
Private Companies				
East Hill Comm. Well Co.	2.00	180	W	0.060
Independent Water Co.		120	S	0.090
Lake Center Water Coop.	21.00	800	W	1.00
Lakota Coop. Water Assn.		35[h]	S	
Meridian Water Co.			i	
Normandy Park Water Co.		350	W	0.395
Richmond Beach Water Co.		630	W	0.720
South Auburn Water Assn.			W	0.010
Star Lake Water Co.	4.00	200	W	0.400
Washington Water Service	15.00	2,000	L	1.500
Weowna Beach Corp.	0.06	30		0.010

Source: "Government in Metropolitan Seattle, Bureau of Governmental Research and Services, University of Washington, 1956.

[a]*Number of services, not persons. Figures include any services outside district or city boundary.*

[b]*W - well, S - spring, L - lake, R - river; primary source of supply.*

[c]*Capacity of existing facilities.*

[d]*Purchases water from Kirkland.*

[e]*Purchases water from Seattle.*

[f]*Includes outside areas; actual city land area is 84.17 sq. mi.*

[g]*Purchases water from WD 68.*

[h]*1955 data.*

[i]*Purchases water from Kent.*

Water

There now are a total of 76 governmental and private agencies engaged in supplying water in the metropolitan area (Fig. 4-5 and Table 4-17). These include 10 cities, 43 water districts, and 11 private companies or cooperatives in King County, and 4 water districts in Snohomish County. Additionally, there were 8 more districts in 1956 which, although legally formed, were not yet actually delivering water to consumers.

Sources of Supply. At present, water is obtained from rivers, wells, lakes, springs and creeks. Water for the city of Seattle and for contiguous areas served by the city system (Fig. 4-5) is derived from Cedar River. The city sells water on a wholesale basis to

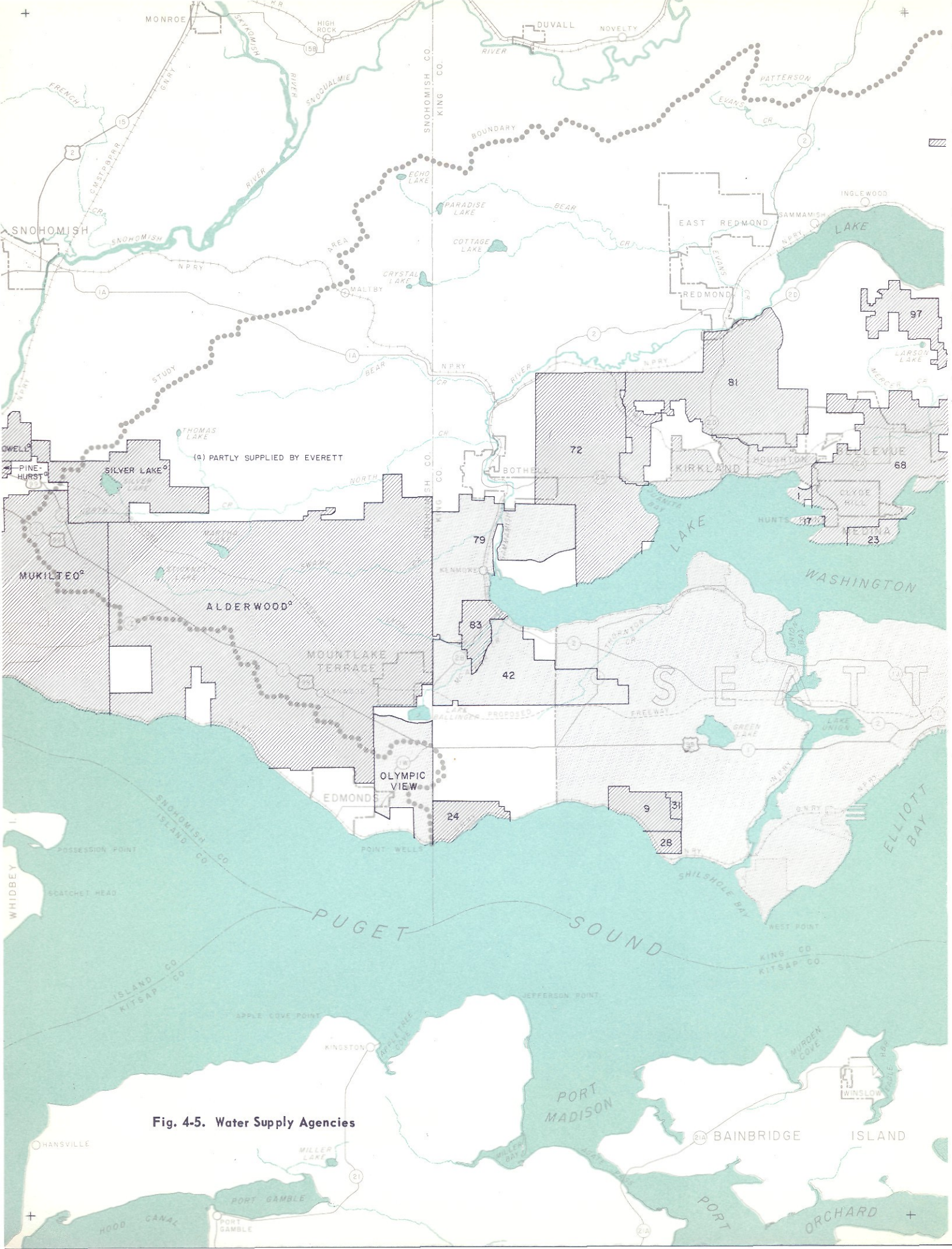
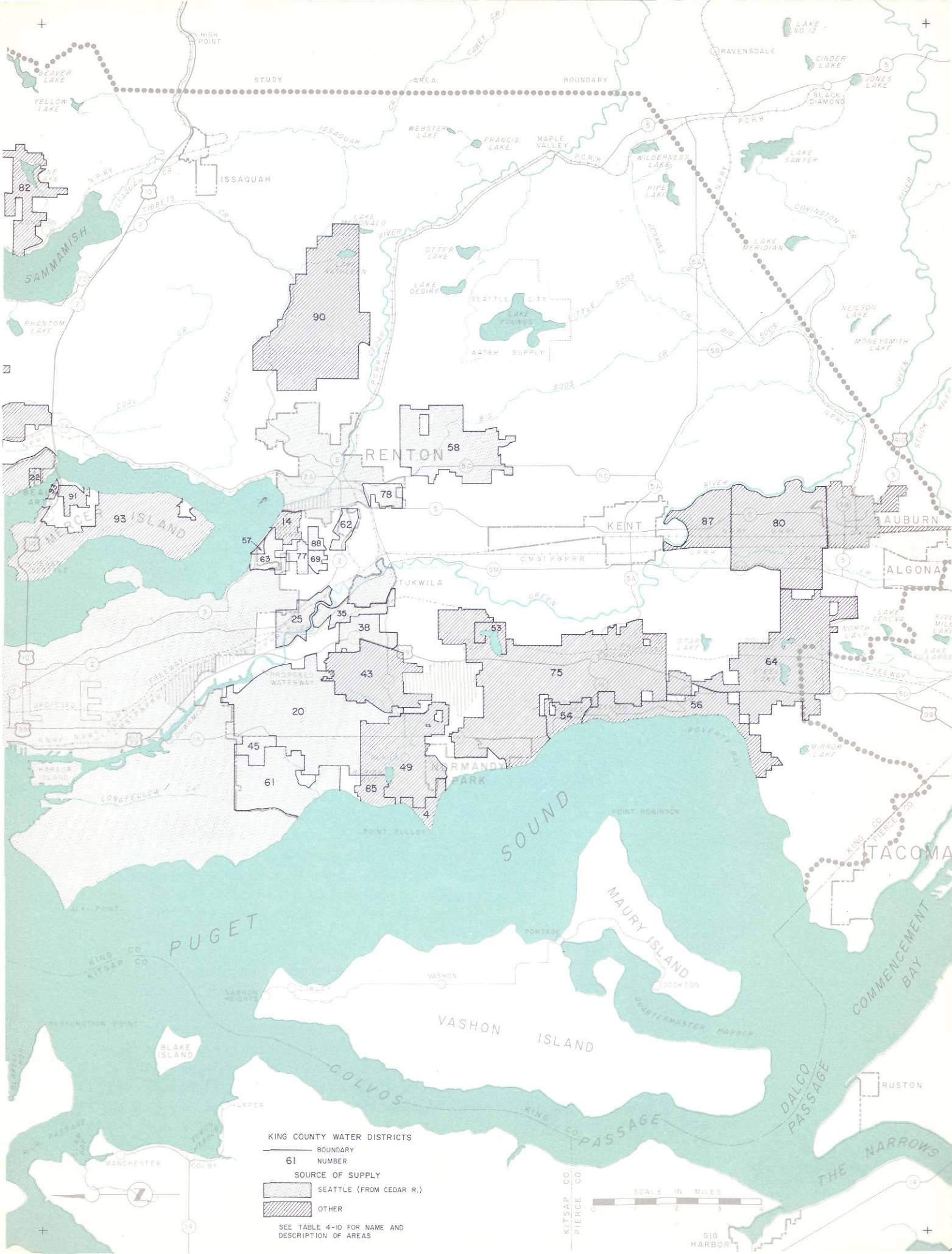


Fig. 4-5. Water Supply Agencies


STUDY AREA BOUNDARY
HIGH POINT
BEAVER LAKE
YELLOW LAKE
ISSAQUAH
SAMMAMISH
PHANTOM LAKE
WEBSTER LAKE
FRANCIS LAKE
MAPLE VALLEY
RAVENSDALE
LAKE NO. 12
CINDER LAKE
JONES LAKE
BLACK DIAMOND
WILDERNESS LAKE
PIPE LAKE
LAKE SAWYER
COVINGTON CR
LAKE MERIDIAN
NEILSON LAKE
MONEYSMITH LAKE
LAKE MCDONALD
OTTER LAKE
LAKE DESIRE
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
RENTON
KENT
AUBURN
ALGONA
TUKWILA
MERCER ISLAND
NORMANDY PARK
TACOMA
LAKE GENEVA
NORTH LAKE
MIRROR LAKE
POINT ROBINSON
POINT PULLEY
PUGET SOUND
MAURY ISLAND
PORTAGE
VASHON
DOCKTON
VASHON ISLAND
QUARTERMASTER HARBOR
COMMENCEMENT BAY
BLAKE ISLAND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
RUSTON
GIG HARBOR
MANCHESTER
COLBY
HARPER
YELLOW HARBOR
KING CO
KITSAP CO
PIERCE CO
SCALE IN MILES
0 1 2 3 4
KING COUNTY WATER DISTRICTS
BOUNDARY
61 NUMBER
SOURCE OF SUPPLY
SEATTLE (FROM CEDAR R.)
OTHER
SEE TABLE 4-10 FOR NAME AND DESCRIPTION OF AREAS

17 outlying agencies in King County and to Water District No. 1 in Snohomish County. In 1956, the water department reported that 586,600 persons were served directly by the city system and than an additional 74,750 persons were served by districts which purchase water from Seattle. These together represent approximately 80 per cent of the population of the metropolitan area.

Wells are the primary source of supply for most water districts. District 68, however, supplies Bellview and adjacent areas with filtered water taken from Lake Washington. Lake Washington is the source of supply also for the Shorewood Apartments on Mercer Island and many individual private users. The Washington Water Service Company, which serves part of the eastern portion of the metropolitan area takes its supply from Lake Sammamish.

Although it is possible in some cases to augment local sources now in use, it is evident that future water supply developments will involve the Cedar, Tolt, and Skykomish rivers. It might become possible also to serve a portion of the southern part of the metropolitan area, particularly Auburn and Kent, with water from the Green River. This river is currently the major source of supply for the city of Tacoma.

Because of the magnitude of the expenditures involved in developing new sources, it is anticipated that many of the existing districts will, in the future, become dependant upon Seattle for water. At maximum development, the Cedar and Tolt rivers will yield an estimated 455 mgd (million gallons per day), comprising 305 mgd from the Cedar and 150 mgd from the Tolt. At prevailing rates of consumption, these two sources could supply the metropolitan area for the next 40 to 60 years. As a part of its long-term program, however, the city of Seattle has filed for water rights on the north fork of the Skykomish River.

Table 4-18. Chemical Quality of Cedar River Water

Constituent	Concentration, ppm[a]
Silica (SiO_2)	7.0
Iron (Fe)	0.09
Aluminum (Al)	0.005
Calcium (Ca)	3.76
Magnesium (Mg)	1.2
Potassium (K)	0.28
Sodium (Na)	1.99
Bicarbonate (HCO_3)	19.4
Sulfate (SO_4)	2.8
Chloride (Cl)	0.5
Total hardness ($CaCO_3$)	14.2
Total solids	44
pH	7.8

Source: "Annual Report, 1956," City of Seattle Water Department. Sample collected from Beacon Avenue sample tap November 11, 1956.

[a]*Except pH.*

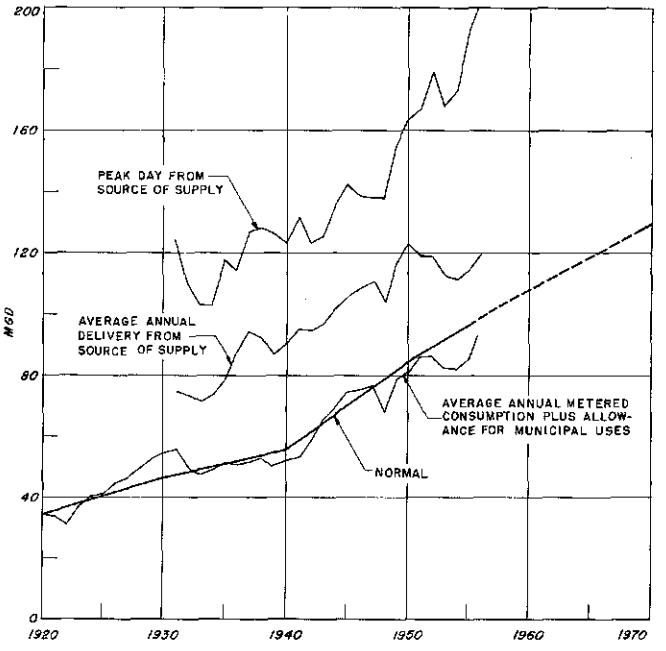


Fig. 4-6. Water Consumption in Seattle System

System is 100 per cent metered. Estimated normal consumption in 1970 is 190 mgd. Data and estimates from Seattle Water Department.

The city of Everett also derives water from rivers of the Cascade Range and has long-term plans to serve substantial portions of the area north of the Snohomish County line. In view of this and the Seattle potential, it can be concluded that Lake Washington will be utilized for domestic water to a lesser extent in the future than it is at present.

Water Quality. Cedar River water, as supplied by the city of Seattle, is of excellent quality (Table 4-18) and is suitable for almost all domestic and industrial purposes without any treatment other than chlorination. A similar situation obtains with respect to other existing and potential sources in the Cascades. Local sources, on the other hand, are of generally less acceptable quality.

Annual Water Consumption. Because of wide variations in the type of development in areas served by the various systems, water consumption figures vary greatly when expressed on a population or per capita basis. For the Seattle system, average annual consumption, exclusive of transmission and distribution losses, increased from 35 mgd in 1920 to 80 mgd in 1950 and 93 mgd in 1956 (Fig. 4-6). The 1956 figure includes 7.2 mgd used for various unmetered municipal services. In terms of population served, the water department estimates that average uses during

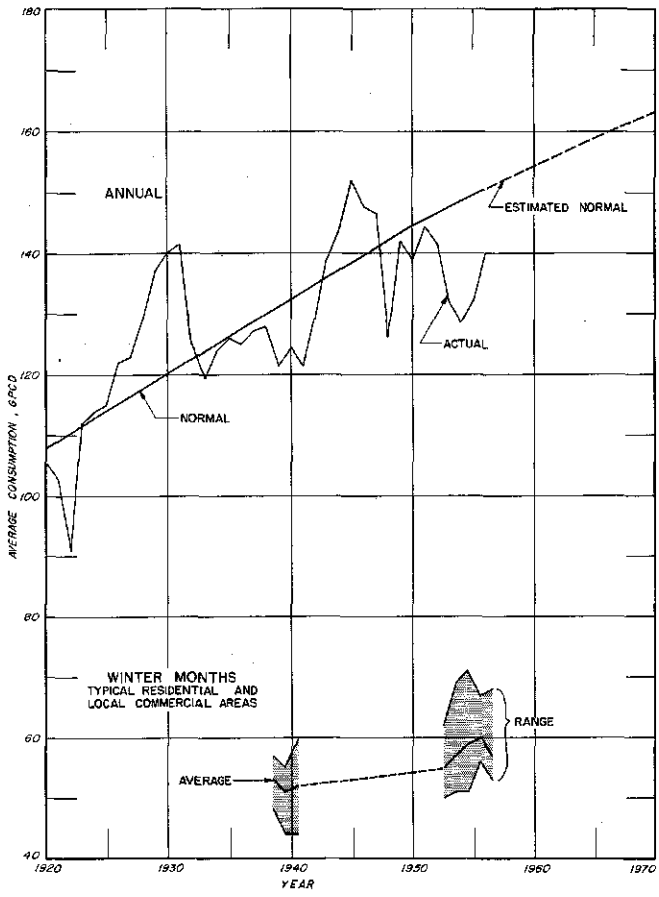


Fig. 4-7. Per Capita Water Consumption in Seattle System

Anticipated increases in industrial uses and increases in residential uses of automatic washing machines, dishwashers and garbage grinders indicate that per capita consumption, exclusive of transmission and distribution losses, will increase to 160 gpcd in 1970. Data and estimates from Seattle Water Department.

the three years were 105, 138 and 140 gallons per capita daily. These and the other per capita figures represent combined domestic, industrial and municipal uses and are typical for large systems having intensively developed industrial and commercial areas, as well as a large residential population. Variations from year to year in the average daily consumption stem principally from differing requirements for irrigation water during relatively dry and wet summers.

Increases in per capita water use (Fig. 4-7) have been taking place for many years not only in Seattle but in many other systems throughout the country. This may be attributed in part to (1) increased use of water for industrial purposes, and (2) increased household use of automatic washing machines, dishwashers and garbage grinders. The Seattle Water Department estimates that per capita consumption will increase from the present level of about 150 gpd to over 160 gpd by 1970. Should that trend continue, the rate will be in excess of 180 gpd by the year 2000.

Seasonal variations in consumption in the Seattle system are shown in Fig. 4-8. Summer demands, which are affected primarily by temperature and rainfall and the corresponding necessity for irrigation, are about 137 per cent of the yearly average and may vary considerably from year to year. Winter demand, on the other hand, is more nearly the same from year to year and approximates 85 per cent of the yearly average.

Winter Water Consumption. From the standpoint of sewerage, water consumption figures for the winter or nonirrigating season are particularly significant. At that time, practically all water delivered to consumers is discharged to the sewers and thus is a direct measure of sewage volume other than that added by infiltration of ground water and inflow of surface drainage.

Winter water consumption data, as indicated by quarterly readings of consumer meters, were obtained from the Seattle Water Department for three meter routes in specific residential-local commercial areas. In each case, these areas were nearly identical to those tributary to a sewage metering point (Chapter 7). Additional data were obtained, however, for other meter routes in areas of similar nature.

Information concerning metered sales by the city to suburban water districts and records of water pro-

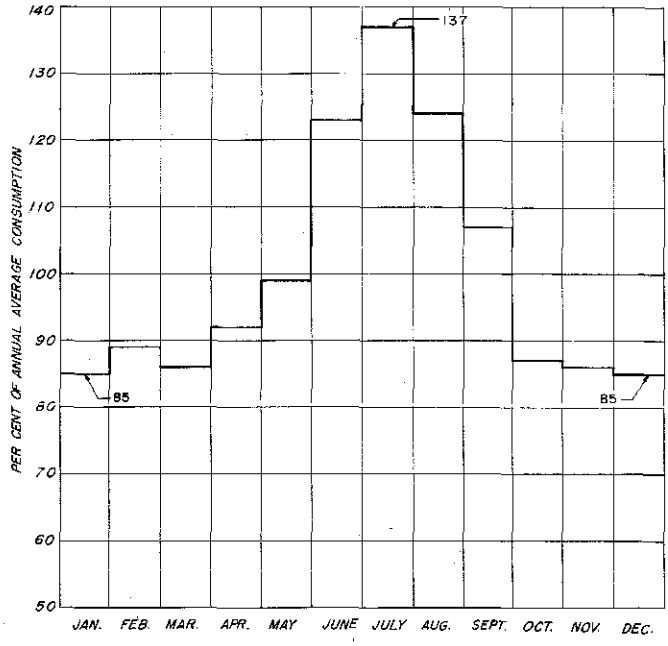


Fig. 4-8. Monthly Variation in Water Consumption

Average monthly consumption in Seattle's system normally ranges from 135 to 85 per cent of the annual average consumption. Consumption, particularly in summer months, varies considerably from year to year because of fluctuations in rainfall and lawn sprinkling. Data from Seattle Water Department.

Table 4-19. Winter Water Consumption, 1955 - 1956

Description of area[a]				Consumption	
Designation	Number of services	Persons per household[b]	Total connected population	mgd	gpcd
Auburn	2,500	3.5[c]	8,750	0.50	57
Kent	2,040	3.5[c]	7,150	0.47	66
Seattle - Ballard	8,108	3.1	25,000	1.45	58
Seattle - Greenwood	4,957	3.2	16,000	1.00	62
Seattle - Lake City	7,011	3.3	23,200	1.24	53
Seattle[d]					
13	277	3.3	915	0.06	65
95	370	3.3	1,220	0.06	49
58	278	-	-	0.05[e]	
59	265	-	-	0.06[e]	
89	379	3.1	1,180	0.07	59
Shorewood Apartments			2,300[f]	0.16	70
King County Water District No.					
20	5,810	3.3	19,200	1.13	59
25	415	3.0	1,250	0.07	56
42	5,187	3.3	17,200	0.73[g]	43
45	587	3.5	2,050	0.09	44
57	48	3.3	160	0.01[h]	48
61	2,783	3.2	8,900	0.52[g]	58
68	4,500	3.2	14,400	0.82	57
77	582	3.3	1,920	0.11	57
79	1,238	3.1	3,840	0.20	52
Total			154,535	8.68	
Weighted average					56

[a]*See Fig. 4-5.*

[b]*Unless otherwise shown, determined from 1950 census tract data.*

[c]*King County Planning Commission estimate, 1957.*

[d]*Water meter routes located as follows: 13 - Ennis Arden Tract; 95 - upper Rainier Valley; 58 and 59 - commercial along Bothell Way; 89 - Magnolia.*

[e]*Omitted from total; included in Lake City total.*

[f]*Estimated from number of units.*

[g]*Water department delivery less 10% for system losses.*

[h]*Rounded from 7,700 gallons per day.*

duction in districts and cities having independent supplies were obtained from the individual operating agencies. In utilizing these data, net consumption was determined by deducting an allowance of 10 per cent for system losses.

Per capita consumption was obtained by dividing the net consumption by the product of the number of meters and the average 1950 household population. Figures for the latter were taken for the particular census tract or tracts in which the water district or water meter route was situated.

Winter water consumption in the residential-local commercial areas averages 56 gpcd and ranges from 65 to 44 gpcd (Table 4-19). Deviations from the average bear no relation to the size of the area but do, to some extent, reflect its economic status. For example, meter route 13 in the Seattle Highlands area shows an average winter consumption of 65 gpcd, whereas meter route 95 in the lower income neighborhood bordering Yesler Way shows 49 gpcd.

Long-term records in certain residential areas of Seattle (Fig. 4-7) indicate that net winter consumption has increased by only about 10 per cent since 1938. Meter routes from which these records were obtained are in older residential districts where population changes have been negligible and thus permit direct comparison over a long period. The difference between the curve for the older districts and that showing gross annual per capita consumption for the system as a whole may be due to some extent to the static nature of the selected areas. Increased consumption for commercial and industrial purposes, coupled with greater summer irrigation demands in new single-

family dwellings, is believed to account for most of the difference between the two curves.

Industrial Water Consumption. Winter water consumption data were obtained also from three meter routes covering the heavy to medium industrial area extending along the Duwamish Waterway from Harbor Island to Boeing Field. Average daily consumption in this gross area of 975 acres was 3.1 mgd, or 3,200 gpad (gallons per acre per day). Values for the individual routes ranged from 1,840 to 3,700 gpad.

Higher consumption occurs in other seasons of the year, particularly the summer months when large quantities of water are used for gravel washing and cold storage operations. Such operations, however, are not significant with respect to the volume of sewage and industrial waste, as the waste water they produce is suitable for direct discharge to adjacent waterways.

Cost of Water. Costs of water are determined by the costs of transmission, treatment, pumping, storage and distribution. For the Seattle system, the total cost amounted to $122.09 per million gallons in 1956 and to $128.70 in 1955.

Monthly water rates for a normal 3/4-inch residential service are $1.00 for the first 300 cubic feet and 10 cents for each additional 100 cubic feet. As a matter of interest, rates for similar service in San Francisco include a fixed charge of $1.10 for a 3/4-inch service, plus 25.9 cents per 100 cubic feet for the first 3,300 cubic feet.

Power

Two major agencies, the city of Seattle Department of Lighting, and the Puget Sound Power and Light Company, serve the metropolitan area with electric power and light. City Light's service zone covers the area between Puget Sound on the west and Lake Washington on the east and extends from the Snohomish County line on the north to the vicinity of South 160th Street on the south. Puget Sound Power and Light's service area covers most of the remainder of the area, including Renton, Tukwila, and the area east of Lake Washington. This company also serves eight counties in western Washington, excluding areas therein which obtain power from municipal systems and public utility districts.

Prior to 1951 there was a considerable overlapping in the service areas of the two purveyors. Through a purchase agreement in that year, City Light took over all distribution in its present service area and has since maintained a continuing program to eliminate the many duplicate components.

Power Sources. The hydroelectric generating station at Cedar Falls was the first to be constructed by City Light. This station, with an initial capacity of 2,400 kw (kilowatts), was constructed in 1905 as an adjunct to the water department's development on Cedar River. Additions since that time consist of three hydro stations on Skagit River, Gorge, Diablo, and Ross, and two steam stations, Lake Union and Georgetown, both of which are now used only rarely. Upon completion of the Gorge high dam in 1959, the total capacity of these stations will be increased from the present level of 693,000 kw to 750,000 kw.

An application has been submitted by the city to the Federal Power Commission for a license to construct the so-called Boundary project on the Pend Oreille River near the Washington-British Columbia border. In addition, preliminary studies of two more sites on Skagit River, Copper and Thunder Creeks, are now in progress. The Boundary project will produce 540,000 kw, and the Skagit River sites a maximum of 200,000 kw. Completion of these projects will bring the total generating capacity of city owned facilities to nearly 1.5 million kw.

Puget Sound Power and Light develops hydroelectric power at its Dieringer powerhouse south of Auburn, utilizing water diverted from White River. It also has a 70,000 kw steam plant at Renton. Additional hydroelectric plants are located at several other sites in

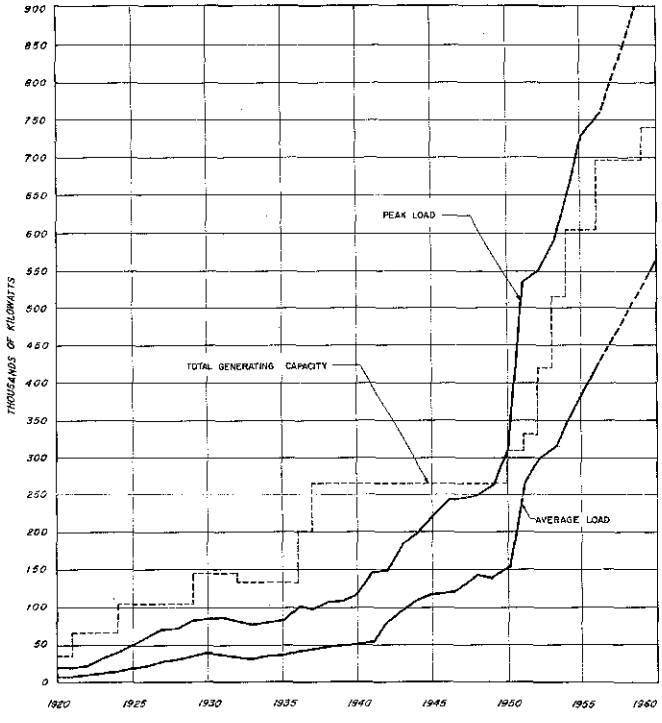


Fig. 4-9. Generating Capacity and Power Consumption, Seattle City Light System

With improvements scheduled for completion by 1960, total generating capacity of the system will have been nearly tripled from the 1932 - 1950 level. Since 1950 the system has utilized power from outside sources to meet peak load requirements. Data from Seattle Department of Lighting.

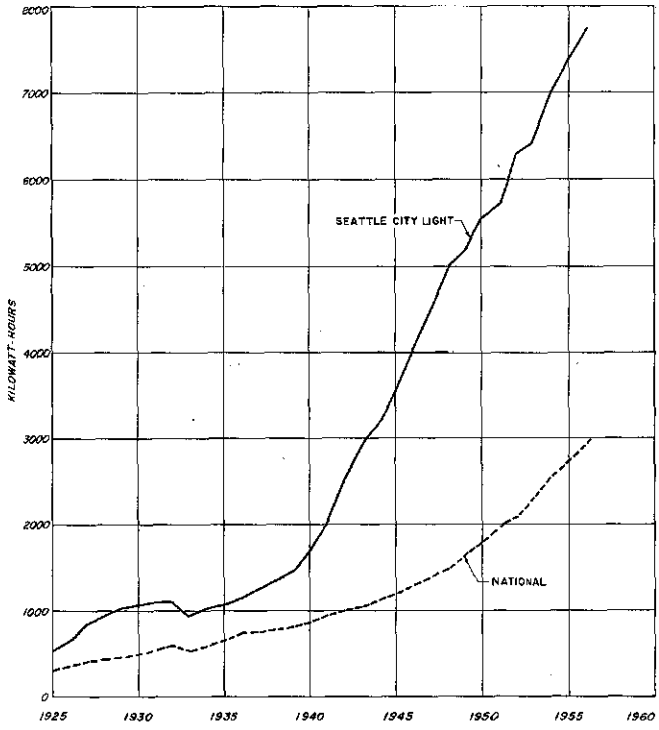


Fig. 4-10. Average Annual Residential Power Consumption

Power consumption for residential use in the Seattle City Light system was 2.6 times as great as in the nation as a whole in 1956. For residential service, costs are approximately one per cent per kilowatt hour. Data from Seattle Department of Lighting.

western Washington, including Rock Island on Columbia River. Total generating capacity, including the steam plant at Renton, is 278,250 kw. Projected additions of over 800,000 kw will bring the total to over 1 million kw when all currently proposed sites are developed and proposed enlargements are completed.

Seattle City Light and Puget Power and Light, along with three other western Washington electric utilities, Tacoma City Light, Chelan County Public Utility District, and Snohomish County Public Utility District No. 1, formed the Puget Sound Utilities Council in 1956. By creating an operating "pool" of electric energy through integrated operation of reservoirs and a flexible system of interchanges, this organization has achieved a saving in output which otherwise would have been lost.

Power and light systems serving the metropolitan area are also a part of the much larger Northwest Power Pool. As such, they have available both surplus and firm power commitments from other members, principally the Bonneville Power Administration which generates over 50 per cent of the power produced in the Pacific Northwest.

By the time all of the feasible hydroelectric resources in the Northwest have been developed, the demand for power is expected to exceed the combined capabilities of all such sources. Eventually, therefore, it will be necessary to turn to steam generation of power, using either conventional or nuclear fuels. Since it is estimated that 15 to 20 years may elapse before this step becomes necessary, it is quite conceivable that technological developments in the use of nuclear fuels will make them economically attractive for power generation.

Power Consumption. Since 1950, peak loads on the City Light system have exceeded generating capacity and have been met by purchases from other sources (Fig. 4-9). Similarly, sales of surplus power have been made to other suppliers at such times at it has been possible to do so.

During 1956 the total electrical input through the City Light system was over 3.7 billion kwh (kilowatt hours), which is equivalent to a continuous load of 424,000 kw. Actual power output from City Light plants averaged 310,000 kw and the balance, 114,000 kw, was supplied through interchanges and purchases from outside sources.

The largest use of power in 1956 was by the Aluminum Company of America, which purchases an average of 30,000 kw for use at its East Wenatchee and Vancouver, Washington alumina reduction plants. This power is supplied by Seattle as part of its purchases from the Pend Oreille County Public Utility District and is transmitted over lines of the Bonneville Power Administration. In 1956, City Light contracted to supply 37,500 kw required for the operation of two 100-ton electric furnaces at the new Seattle plant of the Bethlehem-Pacific Coast Steel Company.

Average residential consumption in Seattle, as in the rest of the nation, has been increasing for many years (Fig. 4-10). Consumption in 1956 was 7,800 kwh, or 2.6 times the national average of 3,000 kwh.

Table 4-20 lists power sales in 1956 according to various types of use. The difference between the total sale of 3.255 million kwh and the total input of 3.7 billion kwh is accounted for by system losses and by power used in the operation of City Light facilities.

Table 4-20. Seattle City Light Power Sales in 1956

Type of use	Power sales	
	million kwh	per cent
Residential	1,537	47
Commercial and industrial	1,450	45
Public street lighting	49	2
Other public agencies	177	5
Railroads and railways	41	1
Other sales	1	
Total sales	3,255	100

Source: "1956 Annual Report", City of Seattle, Department of Lighting.

Table 4-21. Comparative Electric Power Bills

Type of use	Demand, kw	Consumption, kwh	Typical monthly electric bill, dollars			
			Seattle[a]	Portland	San Francisco	Los Angeles[a]
Residential		25	0.75	1.00	1.45	1.00
		100	2.52	3.05	3.64	2.77
		250	4.40	5.25	6.44	4.90
		500	5.14	6.25	9.44	6.65
Commercial power	1.5	150	3.30	4.95	5.90	4.40
	3.0	375	6.83	9.90	12.73	8.90
	6.0	750	12.45	15.40	24.60	16.10
	12.0	1,500	23.70	25.85	44.85	29.50
	30.0	6,000	79.50	85.25	138.00	76.70
Industrial service	75	15,000	172.00	214.00	270.00	164.00
	75	30,000	238.00	313.00	405.00	269.00
	150	30,000	305.00	388.00	490.00	289.00
	150	60,000	437.00	541.00	736.00	457.00
	300	60,000	572.00	703.00	896.00	529.00
	300	120,000	836.00	942.00	1,348.00	857.00
	500	100,000	928.00	904.00	1,443.00	849.00
	500	200,000	1,348.00	1,209.00	2,173.00	1,364.00
	1,000	200,000	1,798.00	1,751.00	2,663.00	1,649.00
	1,000	400,000	2,500.00	2,361.00	4,069.00	2,558.00

Source: "Typical Electric Bills — Cities of 50,000 Population and More", Federal Power Commission, 1956.

Based on rates in effect January 1, 1956.

[a]*Municipally-owned utilities.*

Cost of Power. Charges by Seattle City Light for various classes of electric service are given in Table 4-21. This table also gives comparable charges at Portland, San Francisco and Los Angeles.

Monthly rates in Seattle for single family residences using electricity as the sole means of cooking, water heating and lighting are $5.00 for the first 500 kwh or less, 0.7 cents per kwh for the next 2,000 kwh, and 0.9 cents per kwh for all over 2,500 kwh. During 1956, the average cost of residential power to those having the all-electric rate was slightly under 1 cent per kwh, whereas the cost under the standard residential rate was slightly over 1.5 cents per kwh. It is of interest here to note that the national average residential cost in 1956 was slightly over 2.5 cents per kwh.

Natural Gas

Use of natural gas began in 1956 with the completion of a pipeline from the San Juan Basin in Colorado and New Mexico. This new fuel is distributed locally by the Seattle Gas Company, which formerly served much of Seattle with manufactured gas. Quantities allocated to the company are adequate both for residential and for greatly expanded industrial use. It is expected that the availability of natural gas will lead to the future establishment of many new industrial plants, particularly in the industrial area of the Green-Duwamish Valley where the main transmission line is located.

FUTURE LAND USE

Land use plans serve as a general guide in locating parks, schools and highways, and in determining utility requirements. They serve also as a necessary preliminary step in setting up zones for the various classifications of residential, commercial, industrial and agricultural use.

Studies leading to the preparation and adoption of comprehensive land use plans have, since 1950, been a major activity of the Seattle, King County and Snohomish County Planning Commissions. Although adopted as an official expression of intent, these plans do not have the legal status of zoning and are subject to revision in the light of changing conditions and outlook.

Land use plans applicable to the study area (Fig. 4-11) were developed from a composite map of the entire Everett-Seattle-Tacoma metropolitan complex which was prepared by the Puget Sound Regional Planning Council. In the generalized plan here shown, local retail areas, small parks and other minor details have been omitted, and the proposed Duwamish and lower Green River industrial development has been added.

The sewerage and drainage survey is primarily concerned with two classes of use, namely, residential and local commercial, and industrial and central commercial. It is concerned also with the location of

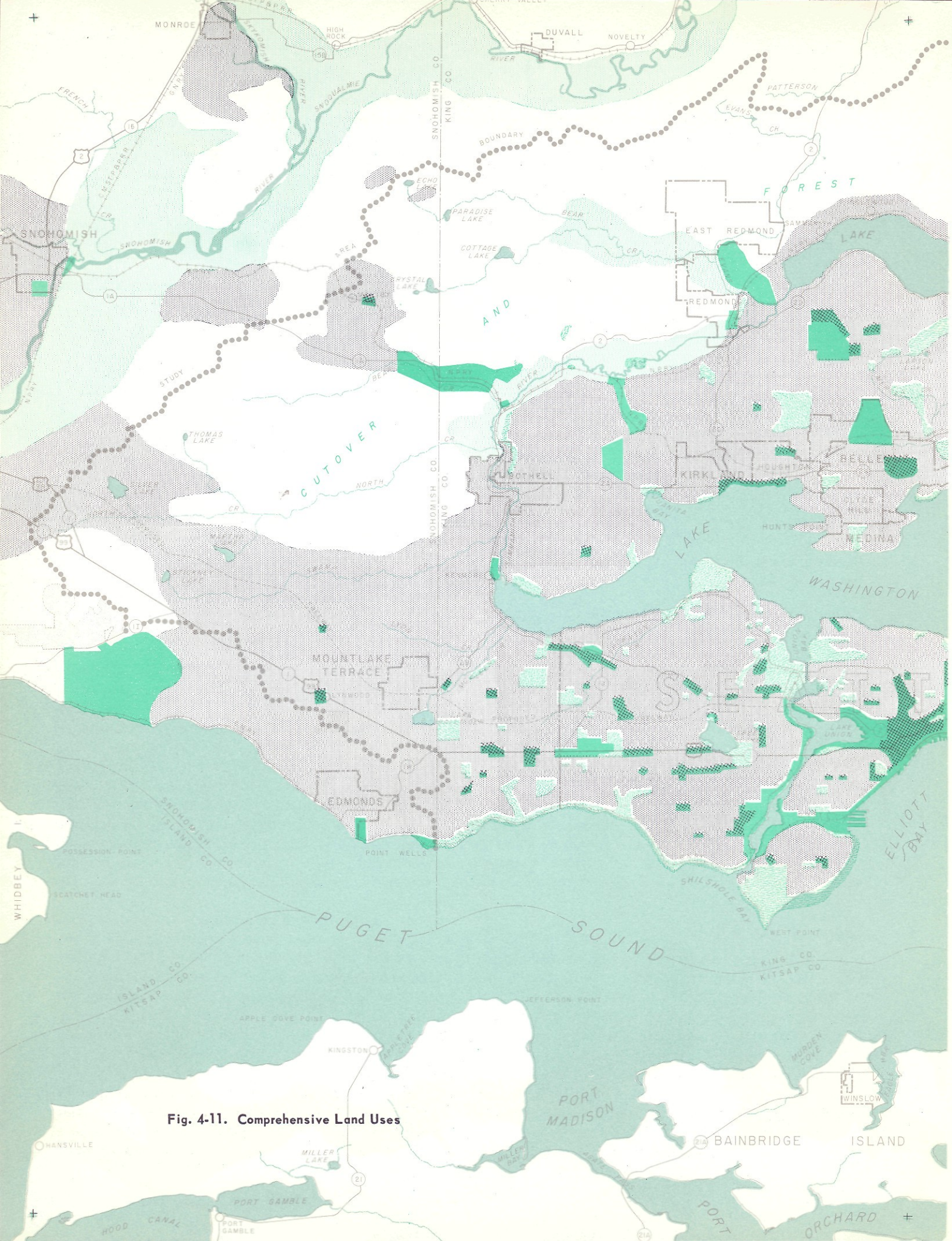
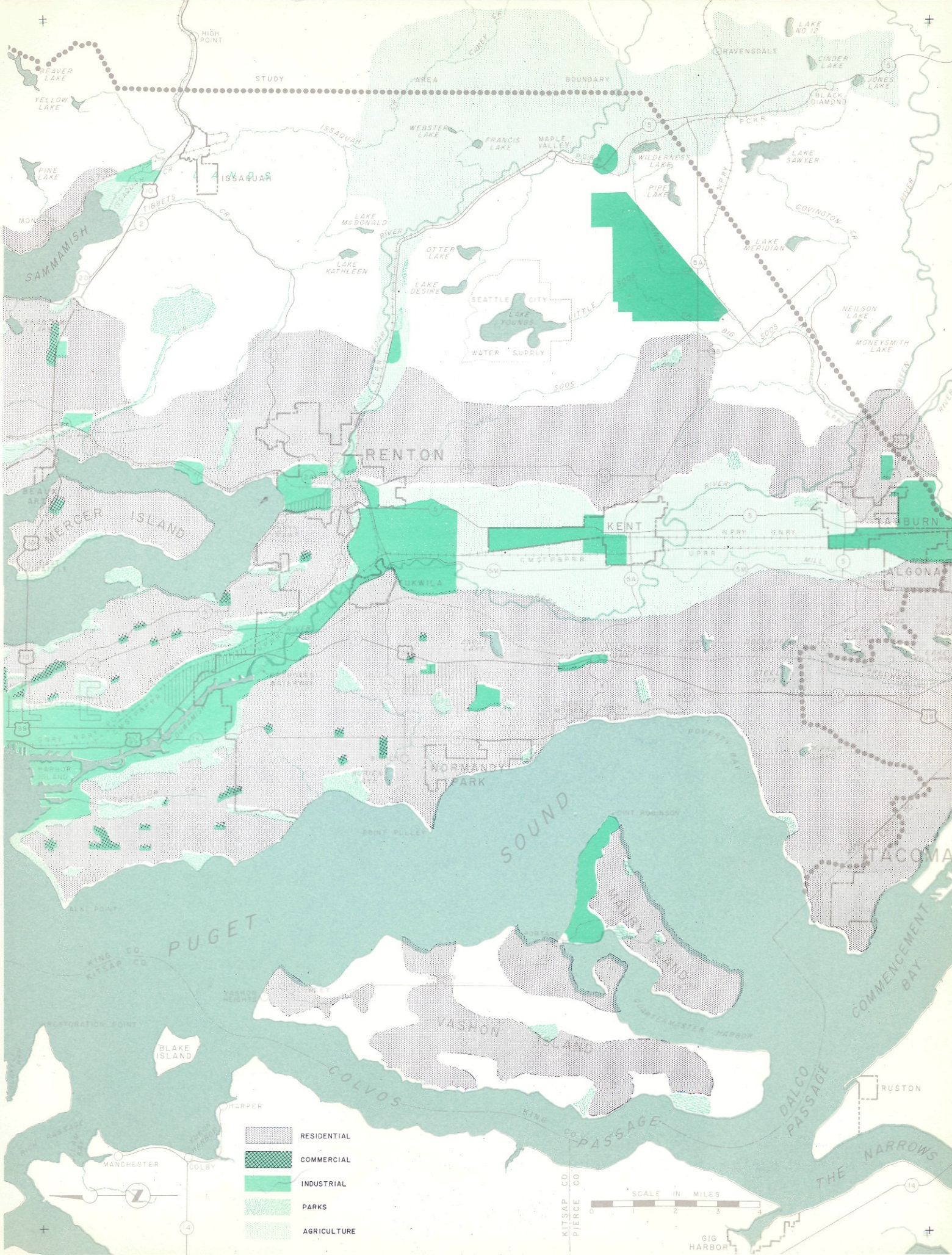


Fig. 4-11. Comprehensive Land Uses


STUDY AREA BOUNDARY
HIGH POINT
BEAVER LAKE
YELLOW LAKE
PINE LAKE
ISSAQUAH
WEBSTER LAKE
FRANCIS LAKE
MAPLE VALLEY
RAVENSDALE
LAKE NO 12
CINDER LAKE
JONES LAKE
BLACK DIAMOND
WILDERNESS LAKE
PIPE LAKE
LAKE SAWYER
COVINGTON CR
LAKE MERIDIAN
NEILSON LAKE
MONEYSMITH LAKE
LAKE MCDONALD
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
SAMMAMISH
RENTON
MERCER ISLAND
KENT
AUBURN
ALGONA
TUKWILA
NORMANDY PARK
TACOMA
PUGET SOUND
VASHON ISLAND
BLAKE ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
RUSTON
MANCHESTER
COLBY
GIG HARBOR
KING CO.
KITSAP CO.
PIERCE CO.
RESIDENTIAL
COMMERCIAL
INDUSTRIAL
PARKS
AGRICULTURE
SCALE IN MILES
0 1 2 3 4

agricultural lands, parks, woodlands, and otherwise undeveloped areas. This is because the lower rates of runoff from such areas must be taken into account in the planning of drainage facilities. Similarly, plans for the development of waterfront parks and beaches must be considered in planning for the disposal of effluents from sewage treatment plants.

Residential and Local Commercial Areas

Except for minor areas of steep or unstable banks and hillsides, all land not specifically reserved for other purposes is considered to be suitable for residential purposes, including attendant schools, local shopping centers and local parks. Areas in Fig. 4-11 for which no use is defined are subject to future residential development. Presently, these areas are largely cut-over lands or second growth woodlands having soils of submarginal agricultural value.

For sewerage and drainage planning, it is necessary to anticipate both the areal extent and the population density of future residential areas. These factors are considered in the next chapter.

Industrial and Commercial Areas

In industrial and central commercial areas, the volume and composition of sewage and the amount of storm runoff are controlled generally by the type of development. This in turn involves many variables, including the nature and number of waste producing industries, the extent of building and paved areas, and employment density.

The central commercial area of downtown Seattle is now well defined. Any expansion will be at the expense of adjoining densely populated apartment house areas, and will have little if any effect on sewerage and drainage planning.

In addition to the downtown area, the term "commercial area" applies to large suburban professional and commercial office developments. Developments of the type described as "industrial park" which may include professional and commercial establishments in addition to warehouses and light industries, are considered herein as industrial areas.

Since industry is a basic factor in the economy of the metropolitan area, its expansion in proportion to the anticipated growth in population must be assumed and future land uses must be planned accordingly. In the absence, therefore, of specific information concerning the types of industries which are likely to develop during the period under consideration, it is essential that a reasonably valid projection be made of the probable location and the areal extent of industrial zones, both light and heavy.

Employment and Manufacturing. Statistics relating to manufacturing employment in King County are given in Table 4-22 and shown in Fig. 4-12. These are for an 18-year period from 1940 through June 1957 and cover total employment, total population, and population in the age group of 20 to 64 years. It will be seen that employment ranged from a minimum of 52,000 in March 1940 to a maximum of 107,000 in July 1944, and that the 20 to 64 year age group provides more than 90 per cent of the labor force. Following the end of the war, employment dropped in 1946 to the pre-war total of 52,000, after which it rose rather steadily and reached a peak of 108,700 in June 1957. For the first six months of 1957, the average was 100,000.

To explain the increase in the past year, it may be pointed out than an unprecedented peace-time increase in aircraft manufacturing employment occurred in the 12-month period July 1956 to June 1957. In that period, employment in the aircraft field rose from 42,900 to 64,000, or almost 50 per cent. During the same 12 months, all other manufacturing employment increased by only 500, while non-manufacturing employment showed a slight drop.

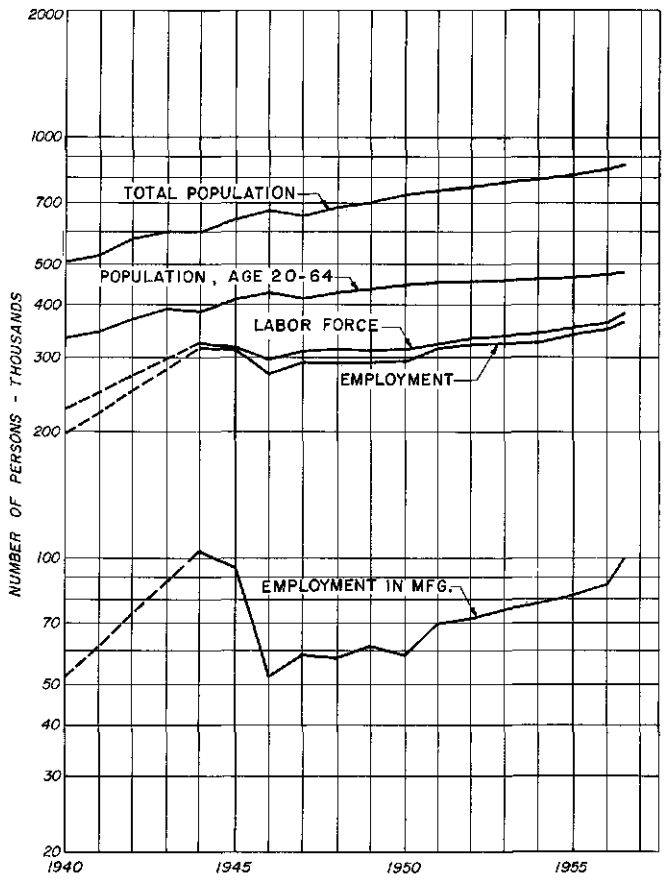


Fig. 4-12. King County Employment and Population, 1940-1957

Labor force and employment figures include nonresidents of King County who actually are employed in the county. Data for 1957 are for the first six months of the year. Data from Washington State Department of Employment Security and Seattle Planning Commission.

Table 4-22. Average Annual Labor Force and Employment in King County, 1940 - 1957

Year	Labor force, thousands	Employment,[a] thousands			Population, thousands	
		Total	All manufacturing	Aircraft manufacturing	Total	Age 20 - 64
1940	227[b]	195[b]	52[b]	6[b]	507	334
1944	324[c]	318[c]	107[c]	39[c]	600	388
1945	319[c]	313[c]	95[c]	35[c]	643	414
1946	299[c]	276[c]	52[c]	10[c]	672	430
1947	312	293	59	13	653	415
1948	314	292	58	15	685	430
1949	313	293	62	24	706	438
1950	314	296	59	20	734	449
1951	327	317	70	23	750	453
1952	335	322	72	29	764	456
1953	340	326	76	33	781	460
1954	347	328	78	37	795	462
1955	357	342	82	38	812	464
1956	367	354	87	43	841	475[e]
1957	378[d]	366[d]	100[d]	58[d]	859	480[e]

Year	Per cent of age 20-64 population				Manufacturing employment	
	Labor force	All employment	Manufacturing employment	Aircraft manufacturing employment	Per cent of all employment	Per cent of total population
1940	68	58	15.6	1.8	28	10.2
1944	84	82	27.6	10.1	34	17.8
1945	77	76	22.9	8.5	30	14.8
1946	70	64	12.1	2.3	19	7.7
1947	75	71	14.2	3.1	20	9.0
1948	73	68	13.5	3.5	20	8.4
1949	72	67	14.1	5.5	21	8.8
1950	70	66	13.1	4.5	20	8.1
1951	72	70	15.5	5.1	22	9.3
1952	73	71	15.8	6.4	22	9.5
1953	74	71	16.5	7.2	23	9.8
1954	75	71	16.9	8.0	24	9.8
1955	77	74	17.7	8.2	24	10.1
1956	77	75	18.3	9.0	25	10.4
1957	79	76	20.8	12.1	27	11.7

Sources: Washington State Employment Security Department, U.S. Bureau of the Census, and Seattle Planning Commission.

[a]*Employees within King County includes some residents of Pierce, Kitsap, and Snohomish Counties.*

[b]*As of March.*

[c]*As of July.*

[d]*Average for first 6 months of 1957 (preliminary report).*

[e]*1953 estimates of age distribution corrected for current high birth rate.*

In August of 1957, Boeing announced that cutbacks and program revisions in defense contracts would necessitate a personnel reduction of 6,000 to 8,000 by the end of 1957. Fortunately, because of the short duration of the high employment condition and because the reduction will be accomplished largely through normal attrition, the impact on the local economy is not expected to be serious. Nevertheless, this incident serves to illustrate one of the problems associated with an economy which depends heavily on a single industry.

In terms of total employment, manufacturing has accounted for 19 to 34 per cent, with 20 to 25 per cent representing the normal peace-time range. Nationally, most metropolitan areas fall within the 20 to 25 per cent range, although 23 to 25 per cent is regarded as an optimum.

Relationships between employment in King County

and both the total and the age 20 to 64 population (Table 4-22 and Fig. 4-12) are obscured by the inclusion of a considerable number of workers who reside in Pierce, Kitsap, and Snohomish Counties. This situation accounts in part for the increase of 64,000 in the labor force between 1950 and 1957, as compared to only 31,000 in the 20 to 64 age group.

Total employment in King County, expressed as a percentage of its population, has been relatively constant since 1948, ranging from 40.3 per cent to 42.8 per cent. In the absence, however, of adequate employment statistics for the Central Puget Sound Region, it can be assumed that the ratio of employment to the total population supported by that employment has declined. This condition is general throughout the country and is due to a decline in the 20 to 64 age group proportion of the total. In King County, the decline was from 66.7 per cent in 1940 to 57.8 per cent in 1955. As stated in the next chapter, this trend is expected to be reversed after 1960.

For the purposes of this report, it is assumed that 60 per cent of the population will be in the age 20 to 64 group, that 65 per cent of these will be gainfully employed, and that employment in manufacturing will not normally exceed 23 per cent of total employment. On that basis, 9 per cent of the population will be employed in manufacturing.

For a projected population of 2,240,000 by the year 2030 (Chapter 5), the corresponding total in manufacturing will be about 200,000.

Areal Extent of Industrial Zones. Land requirements for manufacturing industries differ for various types of operations. Heavy industries such as petroleum refining, steel rolling and fabrication, and the manufacturing of chemicals, paper, and wood products may require one acre for every 2 to 20 employees. Lighter industries, such as aircraft, machine parts, electronic equipment, textile, garment, and food products may have up to 300 employees per acre. These are net manufacturing densities and are based on the actual plant area plus areas required for landscaping, off street parking and loading, interior plant roads and sidings, and open storage. Vacant lands held for plant expansion are not included. Gross industrial density, on the other hand, is determined by dividing manufacturing employment by net manufacturing area plus all vacant land, roads, railroad rights-of-way, and the land occupied by industrial service industries such as repair shops, warehouses, scrap yards, machinery sales and service, trucking concerns, and shipping terminals.

Table 4-23. Summary of Employee Density Study of Recent Manufacturing Industries, Duwamish Industrial District

	Group A[a]	Group B[b]	Group C[c]
Number of manufacturing plants	21	10	7
Net manufacturing area, acres	61.1	51.9	158.8
Number of employees	1,481	1,234	1,561
Net manufacturing density, employees per acre			
Weighted average	24.3	23.8	9.8
Maximum	43.5	43.5	23.2
Minimum	5.5	12.5	2.6

[a]*All new industrial plants constructed in Duwamish industrial district between June 1950 and February 1954.*

[b]*Sample of Group A selected for detailed study.*

[c]*Selected industries, constructed between 1930 and 1954, having adequate off-street parking and of types likely to become established in proposed Duwamish Port Industrial District. Includes proposed foundry in Issaquah expected to require 100 acres.*

Source: Knappen-Tippetts-Abbett-McCarthy, "Development Plan for Duwamish and Lower Green River Valley", 1954.

Land uses in the Duwamish industrial district presently are equally divided between manufacturing and service industries. Based on a detailed study of this area, consultants to the Duwamish and Green River Joint Survey Board[4] report a net manufacturing density of 20 employees per acre and a gross industrial density of 9.5 per acre. These figures exclude the aircraft industry, which at Seattle had a net density of 300 per acre in 1957.

In the case of new establishments, the present trend in both light and heavy industries is toward lower employee densities. Automation, the provision of adequate off-street parking space and loading facilities, and the demand for a pleasant working environment all contribute to this trend. For example, it is assumed[4] that the proposed port development district will have a net manufacturing density of 9.8 employees per acre and a gross industrial district density of 3.5 per acre. These figures are based on a summary of a study of newer industries in the Duwamish Valley (Table 4-23) and provide the following land uses:

	Per cent
Net manufacturing	35
Net service industries	18
Vacant, for future expansion	20
Roads and highways	16
Railroads	9
Utilities and others	2

In the aforementioned study, a gross industrial density of 9.5 is assumed for vacant land within present industrial areas, while a gross density of 3.5 is assumed for dispersed sites. It should be noted, however, that the port development study was concerned primarily with heavy industry and that the projections

[4]*Knappen-Tippetts-Abbett-McCarthy,* Development Plan for the Duwamish and Lower Green River Valley, *1954.*

were limited to 1970, by which time only scattered development of dispersed sites is expected.

For purposes of the sewerage and drainage study, it is believed that a value of 8.0 for the average gross density of all industrial areas by the year 2030 is a reasonable assumption. It is expected that the low value of 3.5 for the Duwamish-Lower Green River project will be offset by the following conditions at other locations.

1. The extremely high density of the aircraft industry.
2. The generally higher density of other light industries.
3. The probable lower ratio of service industries at dispersed sites.
4. The inclusion in the Duwamish district of the principal railroad terminal yards and port terminal facilities.

On the basis of 8.0 persons per gross industrial acre and 200,000 employed in manufacturing, the total industrial area required to serve the projected future population within the study area is 25,000 acres.

Location of Industrial Acres. Locations of present and projected industrial zones within the study area are shown on the land use map (Fig. 4-11). These zones are essentially as depicted in the composite land use plan of the Puget Sound Regional Planning Conference. The principal exception is the industrial area shown in the lower Green River valley south to South 180th Street.

In considering the location of industrial areas, it should be noted that urban planners everywhere are concerned over the high rate at which prime agricultural lands within and adjacent to metropolitan communities are being diverted to other purposes. To avoid this economic loss and the consequent impact on food costs, particularly truck produce, there is a strong movement among planners to preserve as much of the best agricultural land as is reasonably possible.

Locally, the Duwamish-Green-Puyallup valley between Tacoma and Seattle comprises the best agricultural land of the two counties. Since, however, level terrain is the most economical to develop for both industrial and residential purposes, agriculture throughout the valley is being gradually replaced. Eventual extinction can be prevented only through strict zoning by the counties and continued adherence to such zoning in areas annexed to valley cities. Yet economic and political forces arrayed against complete and continued agricultural zoning are of such magnitude that total exclusion of industry on a permanent basis cannot be expected. It is believed, therefore, that the development indicated in Fig. 4-11 represents a reasonable compromise.

In general, the dispersed industrial sites (Fig. 4-11) follow the recommendations of a joint study in 1951 by the King County and Seattle Planning Commission and the Seattle Chamber of Commerce. The extensive heavy industrial area east of Kent, known as the Covington site, will probably develop only if industrialization of the Green River valley is restricted as indicated on the map.

The total area of the industrial zones, both existing and projected, is 24,800 acres. This is adequate for the predicted future population of the study area.

Chapter 5
POPULATION

In the growth of any metropolitan area, the gradual change of land use from rural to urban residential, commercial, and industrial brings with it an inevitable demand for adequate sewerage and storm drainage service. For planning purposes it is essential that the magnitude and extent of this demand be anticipated. Sewage characteristics, both as to quality and strength, are directly related to the size of the contributory population and its sustaining industries. The extent of drainage and sewage collection systems, as well as the location and size of principal trunk and intercepting sewers, pumping stations and treatment and disposal works are affected by the areal distribution of the population.

The long useful life of sewerage and storm drainage facilities, coupled with the relative economy of large conduits as opposed to a multiplicity of smaller conduits, necessitates forecasts of population growth much farther into the future than is normally desirable. For example, local collection drains and sewers are commonly designed for ultimate or saturation development of the area they serve. And similarly, trunk and interceptor drains and sewers are usually designed to meet the demands which will be imposed on them 50 or more years in the future.

Projections of population growth and distribution are required also for the purposes of determining when specific facilities are likely to be needed and of formulating an orderly program of stage construction and financing. The ultimate success, therefore, of any storm drainage or sewerage plan depends upon a searching analysis of the many controlling factors or conditions which affect population growth and urban development.

FACTORS AFFECTING POPULATION GROWTH

Factors involved in projecting population growth of a metropolitan area include:

1. General population trends.
2. Birth and death rate trends.
3. National state migration patterns.
4. Industrial and commercial opportunities.
5. Availability and economy of power, water, fuel, and transportation.
6. Availability of areas suitable for residential and industrial expansion.
7. Desirability of living conditions as reflected by climate, recreational and educational facilities, and social environment.

Each of these factors is to be evaluated in the light of past and present experience. In view, however, of the fact that population growth is subject to the influence of unpredictable future events, all projections referenced to specific years must be regarded at best as probable rather than precise.

General Population Trends

More than 15 years of economic prosperity in the United States have brought about an unexpected upsurge in the national population. As of July 1, 1957, the official estimate was 171 million, representing an increase of more than 13 per cent since 1950. Estimates of growth to the year 1975, made in 1953 by the U. S. Bureau of the Census, were revised upward in 1955 because of continuing high birth rate. If the present trend continues, the expected totals are 228

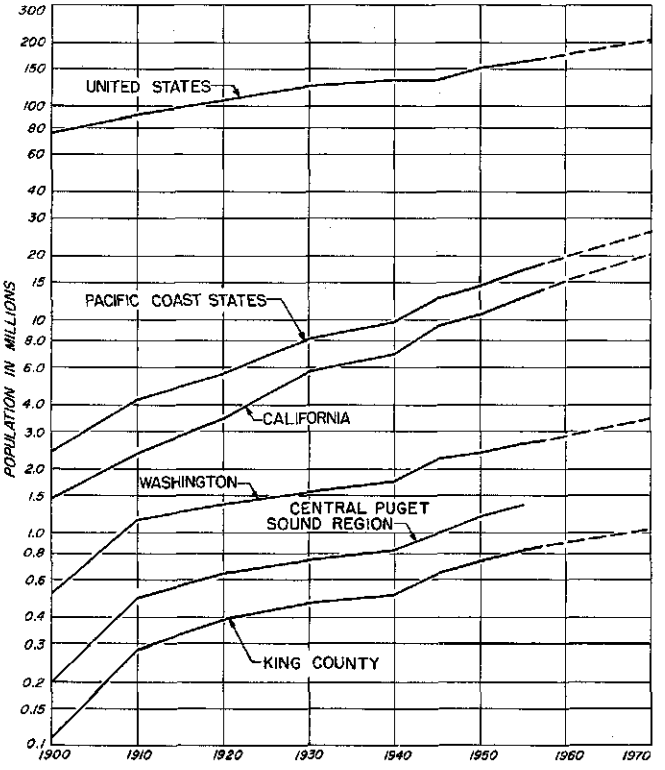


Fig. 5-1. General Population Trends, 1900-1970

For the semi-logarithmic plotting here employed, equal slopes show equal rates of growth, regardless of numerical increase. Since 1910 Washington and the Central Puget Sound Region have grown at a rate intermediate to that of the United States and the three Pacific Coast states. See Table 5-1 for tabulated values and data sources.

million by 1975, 300 million by 1990, and 400 million by 2010. Even if birth rates were to decline by 1975 to the low prewar level, the expected population in that year would be 207 million and would climb to about 300 million in 50 years. In other words, within the design period to be considered in this report, total population of the United States will be at least two times, and possibly three times greater than it was in 1950.

An accelerated rate of urbanization is revealed by recent population statistics. In the period from 1950 to 1955, the standard metropolitan areas of the country, of which there are 168 according to the U.S. Bureau of the Census, had a population increase of over 11.6 million as compared with an increase of 12 million for the country as a whole. Growth of the metropolitan areas is due not only to natural increase but to migration from rural areas and smaller communities. It is apparent, therefore, that the growth of Seattle and its environs is not a local phenomenon but is characteristic of a nation-wide metropolitan trend.

As a result of westward migration, the population of the Pacific Coast states (Fig. 5-1 and Table 5-1)

Table 5-1. Past Populations and Current Projections to 1970

Year	United States		Pacific States		California		Washington	
	Population 1,000	per cent increase	Population 1,000	per cent increase	Population 1,000	per cent increase	Population 1,000	per cent increase
1880	50,156		1,115		865		75	
1890	62,948	25.4	1,888	69.0	1,213	40.5	357	376.
1900	75,995	20.7	2,417	28.0	1,485	22.4	518	45.0
1910	91,972	21.0	4,192	73.5	2,378	60.1	1,142	120.
1920	105,711	14.9	5,567	32.8	3,427	44.1	1,357	18.8
1930	122,775	16.1	8,194	47.2	5,677	65.7	1,563	15.2
1940	131,669	7.2	9,733	18.8	6,907	21.7	1,736	11.1
1945	132,481[a]		12,800[a]		9,344[a]		2,206[a]	
1950	151,132	14.5	14,487	48.8	10,586	53.3	2,379	37.0
1955	165,270[b]		17,253[b]		12,961[b]		2,607[b]	
1957	171,229[b]						2,667[b]	
1960	179,375[c]	18.7	19,986[c]	38.0	15,273[c]	44.3	2,849[c]	19.8
1965	193,491[c]		22,851[c]		17,661[c]		3,123[c]	
1970	209,242[c]	16.7	26,071[c]	30.5	20,296[c]	32.9	3,459[c]	21.4

Year	Central Puget Sound Region[d]		King County		Snohomish County		Seattle	
1880			7					
1890			64	826.			43	
1900	196		110	72.	24		81	88.3
1910	482	146.	285	159.	59	146.	237	193.
1920	634	31.6	389	36.8	68	15.	315	32.9
1930	737	16.3	464	19.1	79	16.	366	16.2
1940	820	11.3	505	8.9	89	13.	368	0.5
1945			643[e]				431[e]	
1950	1,196	45.9	733	45.2	112	26.	468	27.2
1955	1,329		812[e]		131		550[e]	
1957			859[e]				572[e]	
1960			892[e]	21.7	145[f]	30.	579[g]	23.8
1965			953[e]		158[f]		597	
1970			1,018[e]	14.1			609[g]	5.2

Source: U.S. Decenial Census except as noted.

[a]*Statistical Abstract of the United States, 1956.*

[b]*Current estimates of U.S. Bureau of the Census. Estimate for U.S. includes armed forces overseas.*

[c]*U.S. Bureau of the Census, Current Population Reports, Page 25, No. 160. Based on Series 1 illustrative projections, plus allowance (for U.S. projections) for armed forces overseas equal to number overseas in 1957.*

[d]*Snohomish, Kitsap, King, and Pierce Counties.*

[e]*Seattle Planning Commission.*

[f]*Snohomish County Planning Commission, 1954.*

[g]*Seattle Planning Commission, CPR No. 14, rev. May 1956 and adjusted to 1956 city limit.*

Table 5-2. Average Rates of Population Growth, 1910 - 1955

Area	Average annual rate of growth, per cent	Years required to double the population
United States	1.30	54
Washington	1.85	38
United States urban areas	2.12	33
King County	2.36	30
Pacific Coast states	3.20	22
California	3.83	18

has grown, historically, at a faster rate than that of the nation. Although most of this growth has occurred in California, the rate of population increase in Washington has been greater, quite consistently, than that of the United States. Average annual rates of growth since 1910 (Table 5-2) show further that King County has grown slightly faster than the average for all urban areas in the country. For the population to double itself, the time ranges from 54 years for the United States as a whole to 18 years for California. Washington at 38 years and King County at 30 years lie about half-way between these extremes.

The Seattle metropolitan area merges on the south with that of Tacoma and on the north with that of Everett. Bremerton, across Puget Sound to the west, is linked economically with Seattle and will be tied more closely following completion of the planned construction of a bridge across the sound. The four counties of Snohomish, King, Kitsap and Pierce, therefore, form an economic unit which has been termed the Central Puget Sound Region. Population growth of this region, as shown in Table 5-1, has paralleled that of King County.

Since 1910, Seattle has grown at a slower rate than the county. Moreover, the actual numerical gain within the city since 1930 has been less than that outside the city. Population trends since 1950 are shown in Table 5-3 for those portions of King and Snohomish counties, both incorporated and unincorporated, which lie within the sewerage and drainage study area. Of the total of 864,000 estimated to reside within the study area in 1957, 640,000 live in 19 incorporated cities and towns. Changes in population between 1950 and 1957 reflect the effect not only of numerical growth but of incorporation and annexation. The latter, of course, lead to increases in the incorporated areas at the expense of the unincorporated areas.

Annual estimates of population growth of Seattle and the principal suburban areas within King County, as developed by the Seattle Planning Commission for each census tract, show relative changes between the central city as constituted April 1, 1954 and the major suburban divisions. Based largely on data obtained from residential building permits, these studies reveal that the total population increase of 126,000 in King County from 1950 to 1957 consists of 46,000 within Seattle as constituted in April 1954, 66,000 in suburban areas, and 14,000 in other parts of the county. Although the largest numerical increase, 27,000, occurred in suburbs to the south of the city, the rate of growth has been greatest in the areas north of the city and east of Lake Washington.

Birth and Death Rate Trends

Changing economic conditions and social attitudes bring about changes in birth rate which drastically affect the natural increase in population. Based upon the low birth rate in the 1930-40 decade, Thompson

Table 5-3. Population of Political Divisions within the Sewerage and Drainage Study Area

Political division	1950[a]	1957[b]
Within King County		
Seattle	519,664	572,000
Algona	c	1,303
Auburn	6,497	7,900
Beaux Arts	c	325
Bellevue	c	10,500
Bothell	1,019	1,446
Clyde Hill	c	1,654
East Redmond	c	516
Houghton	1,005	2,050
Hunts Point	c	365
Issaquah	955	1,196
Kent	3,278	4,150
Kirkland	4,713	5,550
Medina	c	2,213
Normandy Park	c	2,991
Redmond	573	1,005
Renton	16,039	16,900
Tukwila	800	1,022
Subtotal, incorporated	554,543	633,000[d]
Unincorporated areas	151,000[e]	197,000
Subtotal, study area in King County	706,000[d]	830,000
Within Snohomish County		
Mountlake Terrace	c	7,381
Unincorporated areas	15,000[e]	27,000
Subtotal, study area in Snohomish County	15,000	34,000
Total	721,000	864,000

[a] *U.S. Bureau of the Census, except as noted.*

[b] *Incorporated areas as estimated by Washington State Census Board. Unincorporated areas as estimated by Seattle and King County Planning Commissions and Snohomish County Planning Commission staff.*

[c] *Incorporated since April 1950.*

[d] *Rounded to nearest 1,000.*

[e] *Estimate from census data. In King County this includes 60,000 persons in areas subsequently annexed to City of Seattle.*

and Whelpton[1], in 1942, estimated that the population of the United States would reach a maximum of 160 million by 1990 and that it would decline thereafter. Due, however, to a subsequent upsurge in birth rate, the figure of 160 million was actually reached in 1954. On the other hand, the death rate is relatively stable and can be projected with greater confidence. Trends in the two rates, birth and death, are portrayed in Fig. 5-2 for King County and Washington. Differences between the two sets of curves represent the rate of natural increase, which, in King County, has ranged from minus 0.1 per 1,000 population in 1936 to 17.6 in 1947. Numerically, the natural increase in county population in the 1930-40 decade was only 6,300 as compared with 76,300 during the 1940-50 decade.

Crude birth and death rates, which are those expressed as the number per year per thousand of population, are affected by the age and sex composition of that population. For the curves in Fig. 5-2, the age factor accounts for the generally rising death rate between 1920 and 1936 and in part for its decline since that time. In making population studies, the specific rates of fertility and mortality for each five-year age group provide more reliable bases for estimation of growth than the so-called crude rates.

Because of the low birth rate during the 1930-40 depression years, the percentage of the population now in the age range of 18 to 27 years, which is the most productive, is abnormally low. For that reason, a marked decline in the crude birth rate between 1955 and 1960 has been anticipated. That such a decline has not occurred, or at most has been slight, results from an offsetting increase in the fertility (or more properly, reproductivity) rate for this age range. Significant also is the fact that specific fertility figures (Table 5-4) show a considerably higher rate for Washington than for the nation as a whole. For the years since 1950, this difference may be due in part to an underestimation of the state population, particularly of the increase due to inmigration of people of childbearing age. In any case, the relatively high income now being earned by the younger groups has resulted in marriage and the raising of families at an earlier age than has heretofore been economically practicable.

The current high rate of reproduction is evidenced not only in the younger age groups but to a lesser degree through age 44. Vital statistics reports show a definite trend toward more families of three or four children and a reduction in the number both of small families and very large families. In Washington in 1955, 90 per cent of all births, by order of child in the family, occurred in the first to fourth child group, and the births of second children slightly exceeded those of first children. This trend is evident in all income groups and, coupled with the movement to single-family suburban homes, is the fulfillment of

[1] *Thompson, W. S. and Whelpton, P. K.,* Estimates of Future Population of the United States, 1940 - 2000, *Scripps Foundation for Population Research.*

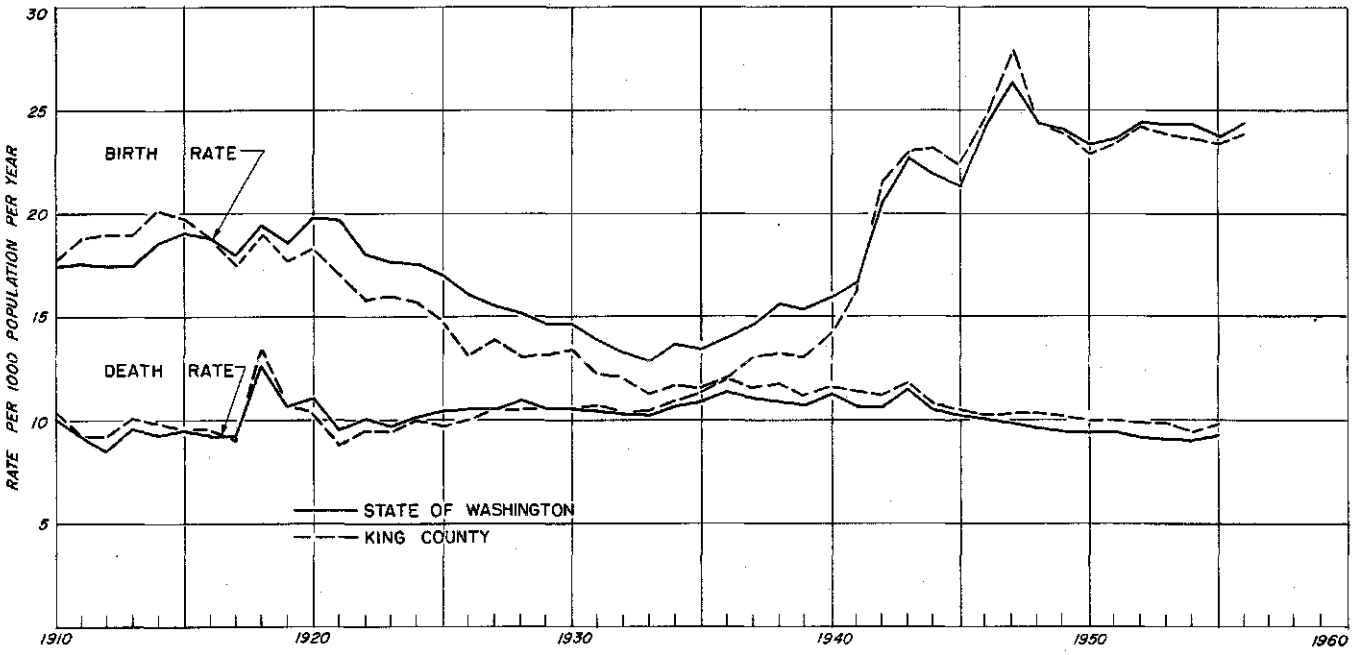


Fig. 5-2. Birth and Death Rates, Washington and King County, 1910-1956

Live birth rates shown have not been adjusted for under-registration. All rates are by place of occurrence for years 1910-1933 and by place of residence since 1933, Natural increase, which is the difference between births and deaths, is greatly affected by changes in birth rate. Sources: Vital Statistics,Summary, Washington State Department of Health; Seattle Planning Commission.

Table 5-4. Specific Age-Fertility Rates for the State and Nation, 1940 - 1956

Year	Annual births per 1,000 women by age of mother							
	Under 15	15 to 19	20 to 24	25 to 29	30 to 34	35 to 39	40 to 44	45 and over
Washington[a]								
1940	b	39	131	125	69	31	9	0.9
1950	b	88	211	164	110	46	13	0.7
1954	0.3	95	268	195	112	53	14	0.8
1955	0.3	93	277	196	110	53	14	0.7
1956	0.3	100	284	204	112	56	15	0.9
United States[c]								
1940	0.7	54	136	123	83	46	16	1.9
1950	1.0	82	197	166	104	53	15	1.2
1952	0.9	85	218	180	113	56	15	1.2
1954	1.0	90	236	188	116	59	16	1.1

[a]*From vital statistics reports, Washington State Department of Health. Values for 1954 - 56 are based on age distribution of female population interpolated from Table 5, Series M3, First Revision of Population Forecasts, Washington State Census Board, January 1956.*

[b]*Included with births to 15-19 year group.*

[c]*From Statistical Abstract of the United States, 1956.*

a long-held ideal of American family life. The current birth rate, therefore, instead of reflecting a postwar reaction of limited duration, may well prevail as long as economic conditions continue to be favorable.

Change in age composition of the population of Washington, resulting largely from the decline and subsequent rise in birth rate, is illustrated by five age distribution graphs in Fig. 5-3. Forward movement of the trough created by the "depression crop" of 1930-40 children is evident in the 15 to 24 year age groups in the graph for 1955. Despite the present high fertility rates of these groups, their numerical disadvantage is expected to result in the secondary trough seen in the graphs for 1965 and 1975. By 1975, the 1945-55 "bumper crop" will be in the period of greatest fertility and may, under favorable economic and social conditions, create the secondary wave of children evident at the left of the graph. In all cases, of course, the projected patterns are subject to many influences, including migration to and from the state.

Migratory Trends

Along with fertility, migration both into and out of a state or community is a major factor in population growth. It is a factor which is difficult to predict, however, because such movements are markedly affected both by local and by national economic conditions. Historically, the growth of Washington has been due largely to migration. Census data show that as of 1950, 42 per cent of the population was native to the state, 50 per cent was born in other states, and 8 per cent was foreign-born.

Fig. 5-4 shows the states of origin of the 1,164,660 persons who had moved to and resided in Washington in 1950. Of these, 613,255 came from the north central states, while 128,420 came from Oregon and California. In contrast, 349,585 natives of Washington resided in other states in 1950. Of this group, however, 212,390 had moved to Oregon and California, representing a net loss to those states of 83,980.

In 1950, the ratio in Washington of American-born inmigrants to outmigrants was 3.3 to 1. Excluding the inter-Pacific state migration, the ratio was 7.5 to 1. Similar data for the census years 1920, 1900, and 1880, also depicted in Fig. 5-4, show the earlier migration pattern and its expansion over the 70-year period.

Whereas the data in Fig. 5-4 reflect the effect of all migration prior to the years indicated, Fig. 5-5 shows that which occurred in two specific periods, 1935 to 1940 and 1940 to 1950. During 1935 to 1940, net inmigration was 16,070 per year and the ratio of inmigration to outmigration was 1.75 to 1. In 1949 to 1950, net inmigration had declined to 2,260 and the ratio was only 1.06 to 1.

Estimates by the Washington State Census Board of net inmigration, based on vital statistics reports and on estimates of population for intercensal years, are presented in Table 5-5. The great influx which occurred during World War II is estimated to have reached a peak of about 128,000 in 1942. This influx centered on the Puget Sound area and was brought about by shipbuilding, aircraft construction, shipping, and military training activities. In 1946, return to a peacetime economy which could not immediately support the new populace resulted in outmigration of an estimated 87,000 persons. During the remainder of the decade, inmigration again took place.

In both the state and King County, and for each decade prior to 1950, population increases from mi-

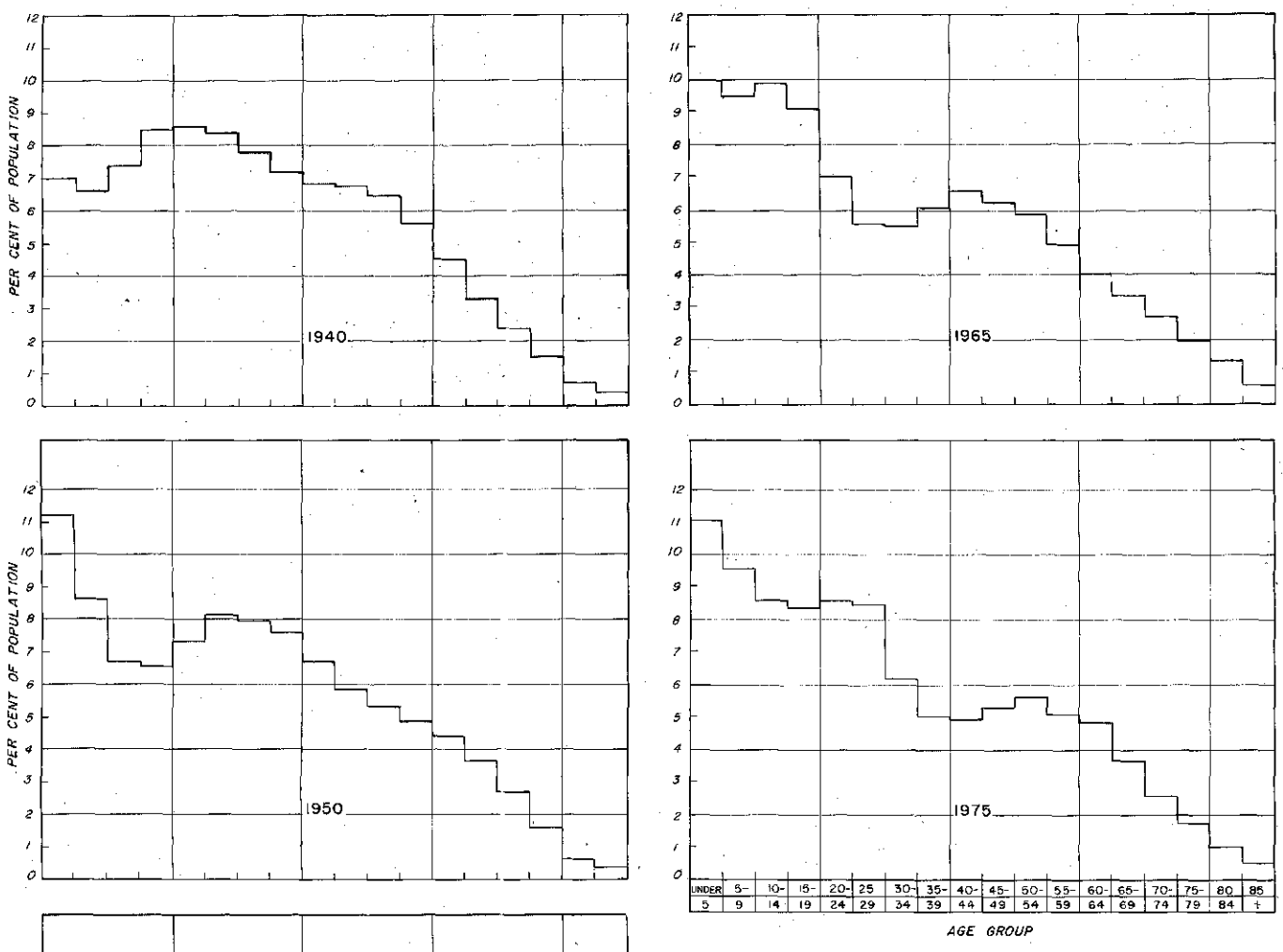


Fig. 5-3. Past and Projected Age Composition of Population, State of Washington, 1940 - 1975

Note forward movement of wave troughs and crests caused by changes in birth rate. Sources: U.S. Bureau of the Census and Washington State Census Board.

gration exceeded substantially those attributable to the birth rate. According to current population estimates and birth and mortality reports, average net annual inmigration to the state was about 9,000 persons per year from 1950 to 1956 inclusive. Inmigration to King County during the same period averaged about 6,000 persons per year. While both figures are subject to considerable error, they are numerically far less than the natural increase for the same periods (Fig. 5-6).

Actually the turning point in the inmigration and natural increase relationship occurred in 1946. Its continuation since then is attributable to enlarged population and high birth rate, as well as to cessation of wartime migration. Net migration to Washington since 1950, though substantial, is but a small fraction of the total westward migration, which is continuing at a rate of about 370,000 persons per year.

In the future, migration to Washington is expected to be an important but not predominant factor in population growth. Providing economic conditions are favorable, it appears likely that migration to the northwest in general and to the Puget Sound Region in particular will increase substantially over that of the current decade. This outlook is based on consideration of such factors as anticipated growth of the whole country, historic and undiminished westward movement, and eventual limitations to growth in California and the southwest imposed by available water supplies.

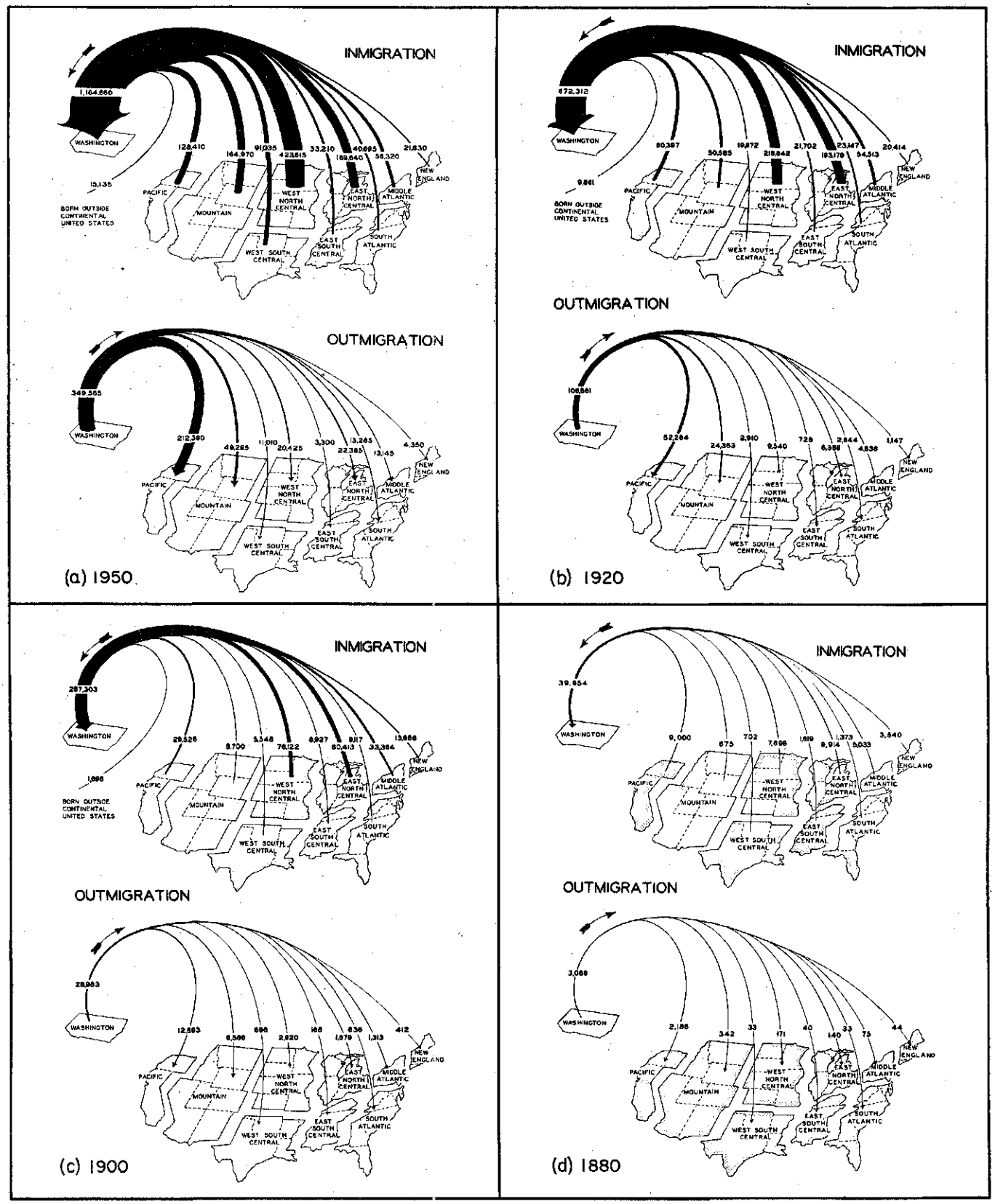


Fig. 5-4. Migration Patterns, State of Washington, 1880 - 1950

Each figure shows the number of native Americans born in other states but residing in Washington in the year shown, and the number born in Washington but residing in other states. Reproduced from *Population Growth and Distribution, State of Washington*, Washington State Census Board, 1955.

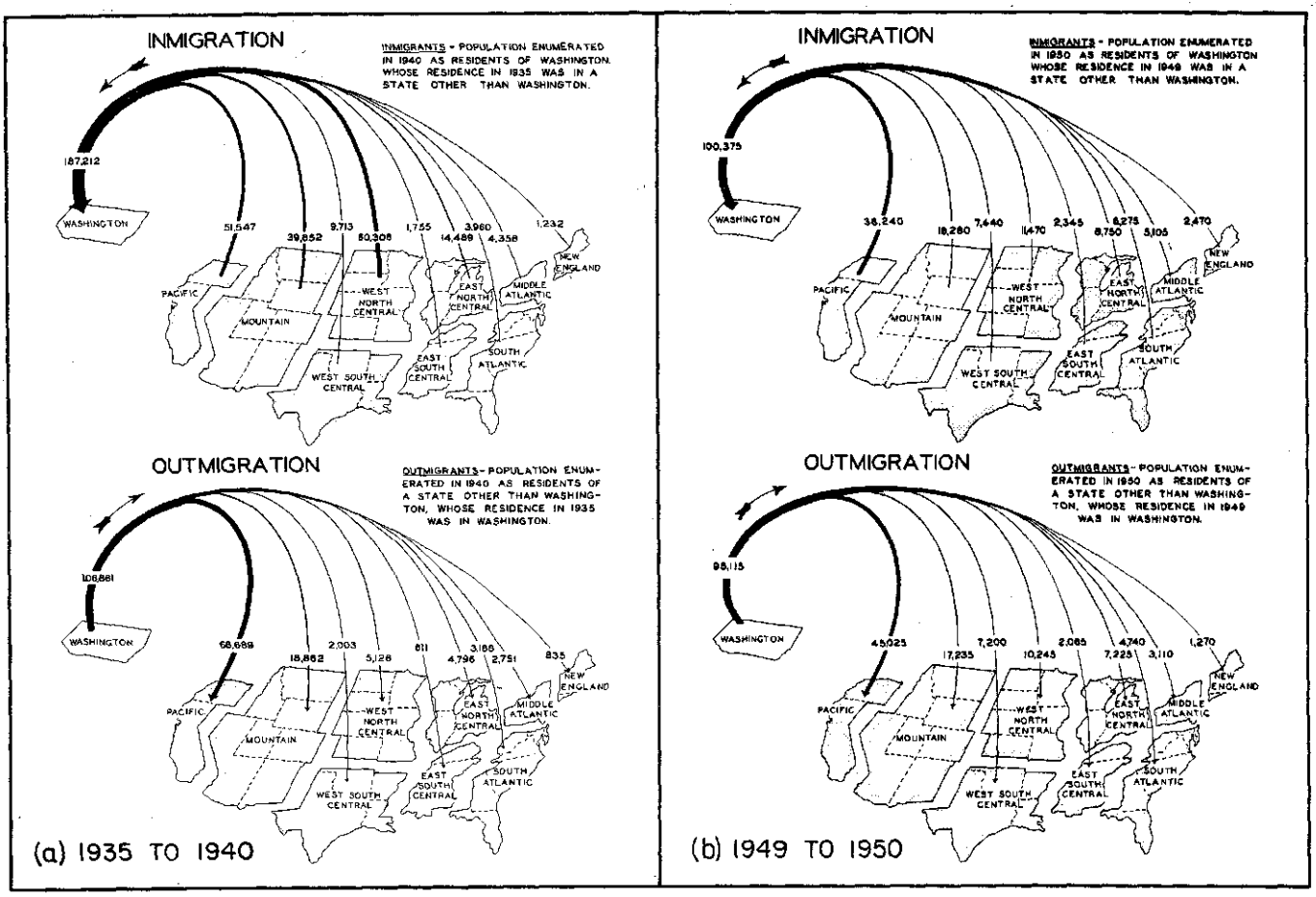


Fig. 5-5. Migration to and from Washington, 1935-1939 and 1949-1950

Each figure shows the migration which occurred during the stated period. Source: *Population Growth and Distribution, State of Washington,* Washington Census Board, 1955.

Industrial and Commercial Opportunities

In the final analysis, future population growth of the metropolitan Seattle area will depend largely upon the opportunity for gainful employment. As evidenced during and immediately after World War II, employment prospects have had an immediate effect on migration to and from the state and the metropolitan area.

Despite the rise in population, there has been an actual decline since 1950 in the number of persons in the 20 to 29 year age group. This has occurred both in the state and in King County, and has been reflected in the number of young persons entering the labor market each year. By 1960, the "wartime crop" will begin to enter the labor market and will reverse the current trend with respect to the proportion of the population in the working-age group. In the event employment opportunities do not match this forthcoming addition to the labor force, population growth due to inmigration would cease and would probably be followed by a period of net outmigration.

As the northwest continues to grow in population, it will support in larger measure the local manufacture of both industrial and consumer goods for the regional market and, in turn, will become less dependent on its forest products and other natural resources. In King County, a broader industrial base will stabilize an economy which now depends too heavily on the aircraft industry. Fortunately, diversified industrial expansion has been taking place in the Puget Sound region and in King County at a slow but apparently increasing pace.

Table 5-5. Estimated Annual Net Migration to Washington, 1910-1949

Period	Average annual net migration
1910-1919	11,500
1920-1929	10,200
1930-1934	6,500
1935-1939	16,000
1940-1949	40,000

Source: Schmid, C. F., Kalbach, W. E., and Miller, V. A., Population and School Enrollment Trends and Forecasts, Washington State Census Board.

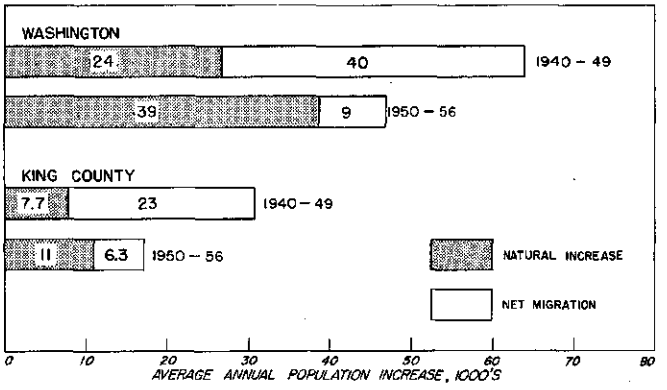


Fig. 5-6. Net Inmigration and Natural Increase Washington and King County, 1940-1956

Inmigration has been the predominant factor in population growth in each decade prior to 1950, whereas natural increase now predominates. The reversal in trend occurred about 1946. Source: U. S. Bureau of the Census, Vital Statistics Summary, Washington State Department of Health; and Seattle Planning Commission.

Utilities and Transportation

The availability of power, water, and fuel in the central Puget Sound region, and the cost thereof compared to other parts of the country, have been discussed in Chapter 4. With the current fuel supply improvement brought about by importation of natural gas, with a continuation of the vigorous program for maintaining an adequate and economical power supply, and with judicious use of its water supply potential, the region can continue to offer industry advantages in utility service. With respect to transportation, the necessity for long hauls is a deterrent to industrial production for the national market. This situation is offset, however, by the favorable location of Puget Sound as harbor for transpacific commerce. Eventual improvement in far-east political and economic conditions would broaden the opportunities for further industrial development.

Physical and Social Environment

The ample availability of suitable sites for industrial and residential development has been described in the preceding chapter. No permanent limitations exist with respect to areas suitable for residential expansion. In the case of industry, however, competition within the region for new industries will tend to limit development to areas which are situated favorably with respect to topography, to marine, rail and highway transport, and to the provision of economical utility services, including sewerage and drainage.

Environmental advantages of the metropolitan Seattle area, including climatic, recreational, educational, and cultural, have been discussed previously. Each of these is significant in the population study to the extent that it attracts industry, encourages inmigration, and retards outmigration.

POPULATION PROJECTION

Many methods may be used for projecting population growth. Among them the principal ones are:

1. A graphical or mathematical projection of the curve of past population growth. One of these, the logarithmic trend method, gives results reasonably comparable to more complex techniques when applied to fast-growing areas in the west.
2. A projection based on relationships between population growth in an area and past growth in other older areas of comparable environment. This method was not used because it fails to take into account external factors related to the time of growth of the areas assumed to be comparable.
3. A projection based upon the trend in the proportion of the area's population to that of the next larger economic or political division. Known as the ratio method, this method requires as a first step a valid projection for the larger division.
4. A determination of the optimum holding capacity, or ultimate population, which an area can contain comfortably, and an estimation of the rate of growth toward that optimum. In the absence of a limiting condition, such as areal extent or water supply, this method is not entirely applicable.
5. A forecast based upon specific assumptions with respect to natural increase and migration. The most accurate of these, the cohort-survival method, takes into account the present age and sex composition of the population, the trends in specific age-fertility rates and age-sex-mortality rates, and the volume and age composition of inmigrants.

High and Low Projections

In utilizing the above methods, it is common practice to prepare a series of projections, each based on specific assumptions as to conditions expected to prevail during the period of projection. Since a considerable spread is generally found between the highest and lowest results obtained, those who use the data must select the projection appropriate to their particular purpose.

For planning sewerage and drainage works, the projection indicating the greatest rate of growth which reasonably may occur should be selected. If actual growth falls below this rate, facilities so designed will simply be adequate for a longer period, and increments scheduled for stage construction will be deferred accordingly. For financing purposes, however, the projection of least growth likely to occur must be considered. This is particularly important for projects involving financing by utility revenue.

Population Studies and Projections Available to the Survey

The Washington State Census Board has prepared cohort-survival projections for the state to 1965,

based on one assumption as to fertility rates and on five different rates of inmigration[2,3]. Stanford Research Institute has used the cohort-survival method to make a single projection to 1975 for each of eleven western states[4]. A similar but less precise procedure, known as the component method, has been used by the Bureau of the Census in preparing projections from 1950 census data[5]. A report on population growth of the northwest states, published in 1952 by the Columbia River Inter-Agency Committee, is based on one of the earlier census bureau projections and is of particular value because it contains revisions allowing for the economic outlook and resources of the region[6].

Population studies and projections made by the Planning Commissions of Seattle and of King and Snohomish counties have been of great value to the sewerage and drainage survey. In 1952, the Seattle Planning Commission prepared a projection for King County based on crude birth and death rates[7]. In the same year it also prepared a projection to 1970 for Seattle, using census tracts based on land occupancy[8]. These were followed in 1953 by a cohort-survival study and three projections for King County to 1970, utilizing low, medium, and high assumptions as to inmigration and the same assumptions as to fertility as were employed by the Washington State Census Board[9]. Early in 1954 the Seattle Planning Commission staff made an informal projection to the year 2000 by extrapolation from the 1970 medium cohort-survival study. Also in that year, projections to 1970 were prepared for the principal city and suburban divisions of King County[10]. These were revised in May 1956 to include development on the west side of Lake Sammamish.

Starting with a base study and report in 1954[11], the Seattle Planning Commission has issued annual reports on current population estimates by census tracts, political subdivisions, and major county divisions. These estimates, based largely on residential construction permit data and on military and institutional reports, provide a reasonably reliable history of population growth and distribution since the 1950 census.

The King County Planning Commission, in 1952 and 1953, prepared a series of comprehensive land use plans for major divisions of the county. These plans, together with information developed by the commission in subsequent special studies, have been used to define the optimum land occupancy pattern for much of the sewerage and drainage study area. In 1954, the Snohomish County Planning Commission made a projection to 1965 for the county and also for an area lying southward from Everett to the county line. In this study, the ratio method was used along with state projections by the Washington State Census Board. Additionally, the Snohomish County Planning Commission conducted an informal census by census tracts in 1954 and has made estimates of near ultimate populations for specific school service areas in the southern part of the county.

In utilizing the above mentioned projections for King and Snohomish counties, it should be noted that all are based either directly or indirectly on the future specific age-fertility rates assumed by the Washington State Census Board in its cohort-survival study. In that study, it was assumed that, after 1950, fertility rates for the 20-24 year and 30 to 34 year age groups would remain substantially the same as in 1950, and that the rates for all other age groups would decline in varying degrees. Resulting crude birth rates were estimated to be 23.9 per 1,000 persons per year for 1950-55, 18.8 for 1955-1960, and 18.1 for 1960-65. In comparison, the apparent birth rate was 24.1 in 1950-55, and since then has increased to 24.5 instead of declining. Fertility rates have increased in even greater degree. If the high rates continue, the state estimate of growth due to natural increase by 1960 will be low by about 70,000 persons and the Seattle Planning Commission estimate for King County will be low by more than 20,000.

For the period 1960-65, assuming that 1956 rates (Table 5-4) continue, fertility rates will be 27 to 63 per cent higher than those projected by the State Census Board for the specific age groups, and the crude birth rate will remain approximately at the current level of 24 per 1,000 persons per year. Beyond

[2] *Schmid, C. F., et al.*, Population and Enrollment Trends and Forecasts, State of Washington, *Washington State Census Board, 1953.*

[3] First Revision of Population Forecasts, State of Washington, 1955 to 1965, *Washington State Census Board, January 3, 1956.*

[4] *Nielson, H. C.*, Population Trends in the United States Through 1975, *Stanford Research Institute, August 1955.*

[5] *Current Population Reports, Series P25, No. 78, 110, 123, and 160. U. S. Bureau of the Census.*

[6] Population Projections for the Pacific Northwest States and Region, 1960 and 1975, *Columbia Basin Inter-Agency Committee, 1952.*

[7] *Current Planning Research No. 8, May 15, 1952, Seattle Planning Commission.*

[8] Population Forecasts for the City of Seattle by Census Tracts, *CPR No. 19, September 15, 1952, Seattle Planning Commission.*

[9] Population Trends and Projections, Seattle Standard Metropolitan Area, 1900 - 1970, *CPR No. 13, November 1, 1953, Seattle Planning Commission.*

[10] Population Trends and Projections, Seattle Metropolitan Area by Major Divisions, 1920 - 1970, *CPR No. 14, April 1, 1954, Seattle Planning Commission.*

[11] Population Trends in the Seattle Standard Metropolitan Area, 1950 - 1954, *CPR No. 15, November 1, 1954, Seattle Planning Commission.*

1965, the crude birth rate would tend to rise as the depression-born generation passes beyond the age of parenthood and is replaced by the larger wartime and postwar generation.

Official state and King County projections for 1960, 1965 and 1970, based on medium inmigration, along with revisions thereto based on the assumption of continued high fertility rate, are given in Table 5-6. For the two assumptions as to fertility, the tabulated figures show a difference in total population of about 6 per cent by 1965 and 11 per cent by 1970. For King County, the population growth due to natural increase between 1960 and 1970 would rise from 76,000 to 171,000. In the absence, therefore, of any indication toward a decline in fertility rates, the official short-range projections must be considered as somewhat conservative.

Procedure Employed in Population Projection

As indicated earlier, population projections to serve the purposes of this report must cover a period equal to or greater than that represented by the longest economic life of any of the facilities likely to be constructed. With that requirement in mind, projections were developed to the year 2030 for the sewerage and drainage study area. As a preliminary step, estimates of saturation or ultimate population density were made for Seattle and for each of the principal suburban divisions of the presently developing metropolitan area. This work was done by the three planning commission staffs in consultation with the survey staff, and takes into account the comprehensive land use plans which have been developed for Seattle and the suburban divisions. For the high projection, it was assumed that, on the average, the 2030 population would be 80 per cent of the saturation total.

Table 5-6. Short-Range Population Projections for Washington and King County

Projection	Population in thousands		
	1960	1965	1970
State of Washington			
Declining fertility[a]	2,832	3,058	
Constant fertility[b]	2,900	3,240	
Difference	68	182	
King County			
Declining fertility[c]	892	953	1,018
Constant fertility[b]	912	1,007	1,133
Difference	20	54	115

[a]*With constant medium inmigration of 10,000 per year, from First Revision of Population Forecasts, State of Washington, 1955 to 1965, Washington State Census Board, January 3, 1956.*

[b]*Above projections revised to 1956 specific age fertility rates for Washington, given in Table 5-4.*

[c]*With medium inmigration of 7,500 per year, 1950 to 1960, and 5,000 per year, 1960 to 1970, from CPR No. 13, Seattle Planning Commission.*

In this phase of the work, the population study divisions were those previously established by the Seattle Planning Commission. In addition to eleven divisions within the city, they included seven divisions covering roughly the area northward to the Snohomish County line, eastward to Lake Sammamish and Renton, and southward to Salt Water State Park.

Populations for the intermediate years, 1980 and 2000, were derived by extrapolation to 2030 of the King County 1970 projection. These were then distributed among the several divisions on the basis of local conditions likely to affect their rates of growth. While such a procedure is applicable to the above noted population study divisions, all of which lie within the presently developing urban pattern, it is of questionable validity in the outlying and largely undeveloped areas in the northeastern, eastern and southern portion of the sewerage and drainage study area. The problem, therefore, was to estimate the additional population likely to reside in the present fringe areas.

For the purpose of the sewerage survey, the procedure used in developing the high projection was to extend the United States population to the year 2030 by the logarithmic trend method, assuming continuation of the 1954-55 fertility rate. With that as a base, the populations of the Pacific states, of Washington, of the Puget Sound region, and of King and Snohomish counties were estimated by the ratio method. The two county populations were then distributed, taking into account the previous determination for the urban divisions which would be approaching saturation limits. In so doing, allowance had to be made for urban development on Vashon Island and in Kitsap County as integral parts of the metropolitan Seattle area. An allowance had to be made also for the metropolitan development of Everett.

For the low projection, the growth of the state was reduced to the lowest rate which has occurred in any prior 30-year period. In addition, the relationship of the low to the high projection was determined for subsequent application to the survey study area.

Projection of State, Region, and County Populations

As a base for local projections, the population of the United States is assumed to climb to 228 million by 1975, 400 million by 2010, and 550 million by 2030. The high rate of increase thus indicated is predicated on a continuation of the 1954-55 fertility rates.

For 1975, the forecasted population of 228 million is taken from the Series AA projection of the U.S.

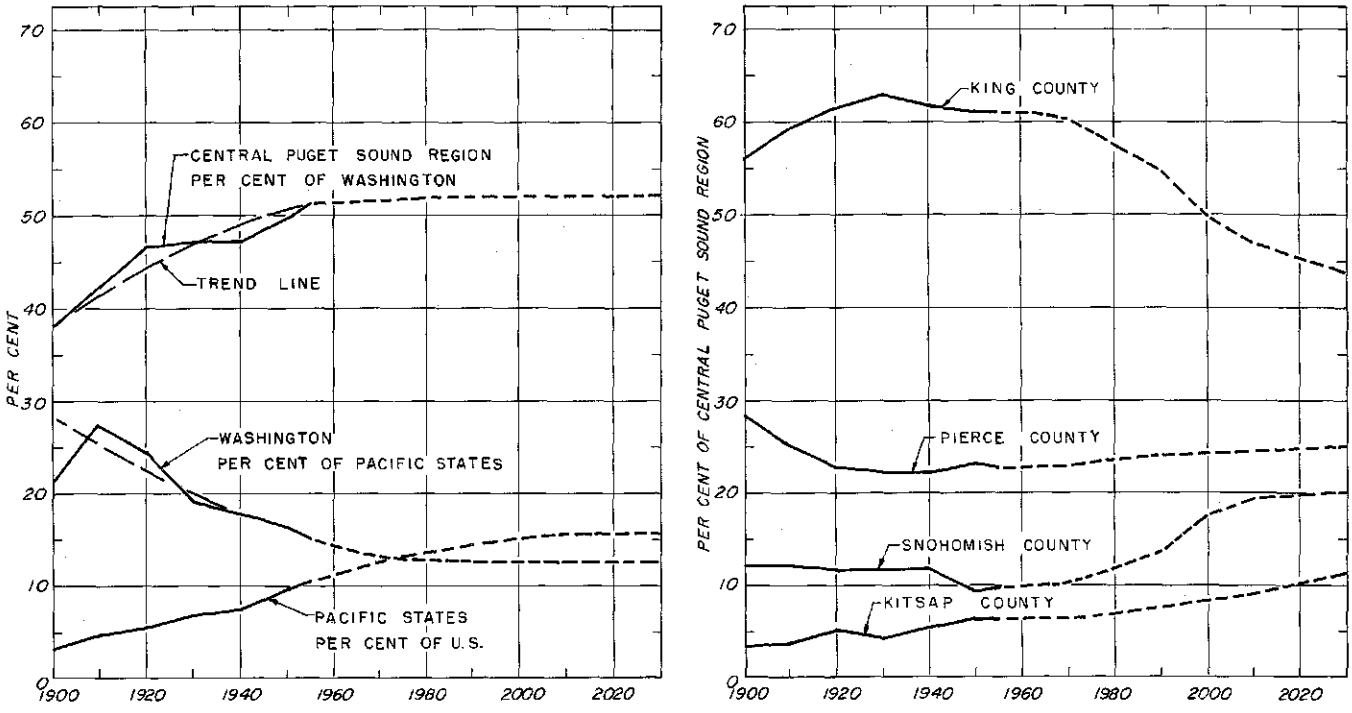


Fig. 5-7. Population Ratios Expressed as Per Cent of Next Larger Division

Bureau of the Census[12]. Figures for the two later years are obtained by extrapolation of the 1975 projection at an average increase of 1.61 per cent per year. Since this rate exceeds all prior rates of national growth between 1900 and 1950, some question may be raised as to the propriety of using it in developing a high projection of reasonable attainment.

Insofar as the projection to 1975 is concerned, the 1.61 per cent per year rate of increase is currently being exceeded and the economic outlook for the next decade[13] is favorable toward its continuation. For the long range outlook, demographers note that revolutionary technological advances in the last 15 years have placed the nation on the threshold of a new era of material well-being for which there is no historical precedent. On that basis, the validity of the high population projection will depend not on the inherent productive capacity of the nation, but on how effectively we meet the economic, social and political problems arising from the predicted advances.

In applying the high projection to the local scene, it was assumed that past and projected trends in population ratios will come to equilibrium in 40 to 50 years. As shown in Fig. 5-7 and Table 5-7, this assumption implies that the ratio of the Pacific states to the nation will end its rise by that time and that the population of those states will grow thereafter at about the same rate as the nation. It also implies that the ratio of Washington to the other Pacific states, which has been declining since 1910, will become constant at about the end of the present century. By combining the two ratios, it was determined that the ratio of the population of Washington to that of the nation should rise from 1.59 per cent in 1955 to 1.94 per cent by the year 2010 and thereafter should remain at that level.

Since 1900 the population of the central Puget Sound region, comprising King, Pierce, Snohomish, and Kitsap counties, has increased from 38 per cent to 51 per cent of that of the state. As the trend line, however, indicates that the rate of increase has been declining, it is assumed that the ratio will stabilize at about 52 per cent. This implies that while the four counties will continue to comprise the principal metropolitan center of the state, other urban centers in Washington, along with the remainder of the Puget Sound region, will grow in the future at a rate equal to the four-county area.

Within the central Puget Sound region, the ratio of population in King County to that of the region has been declining since 1930. Kitsap County has shown an almost continuous rise since 1900, while changes in the ratios for the other two counties have not been consistent. In the absence of established trends suitable for mathematical extrapolation, the curves in Fig. 5-7 were extended on the basis of present conditions and probable future developments.

[12] Revised Projections of the Population of the United States, by Age and Sex: 1960 to 1975, *Current Population Reports, Series P25, No. 123, October 1955, U.S. Bureau of the Census.*

[13] Potential Economic Growth of the United States During the Next Decade, *Committee Staff Report to the Joint Committee on the Economic Report, 83rd Congress, 2nd Session.*

Table 5-7. Population Ratio Expressed as Per Cent of Next Larger Division

Year	Pacific to U.S.	Washington to Pacific	Central Puget Sound Region to Washington	King County	Pierce County	Snohomish County	Kitsap County
				to Central Puget Sound Region			
1900	3.17	21.4	38.0	56.1	28.3	12.2	3.4
1910	4.55	27.4	42.2	59.0	25.1	12.2	3.7
1920	5.37	24.4	46.0	61.4	22.7	10.7	5.2
1930	6.69	19.1	47.2	62.9	22.2	10.7	4.2
1940	7.40	17.8	47.2	61.6	22.2	10.8	5.4
1950	9.57	16.4	49.7	61.0	23.2	9.4	6.4
1955	10.5	15.1	51.2	61.1	22.7	9.8	6.4
1960	11.2[a]	14.3[a]	51.4	61.0	22.7	9.9	6.4
1965	11.9[a]	13.7[a]	51.5	60.8	22.8	10.0	6.4
1970	12.5[a]	13.3[a]	51.6	60.2	22.9	10.4	6.5
1975	13.1	13.1	51.7	59.0	23.4	10.9	6.7
1980	13.5	12.9	51.8	57.6	23.6	11.8	7.0
1990	14.4	12.7	51.9	54.8	23.9	13.7	7.6
2000	15.2	12.6	52.0	50.0	24.2	17.5	8.3
2010	15.5	12.5	52.0	47.0	24.5	19.3	9.2
2020	15.5	12.5	52.0	45.2	24.8	19.8	10.2
2030	15.5	12.5	52.0	43.7	25.0	20.0	11.3

[a]*From Series 1 projection, Illustrative Projections of the Population, by States, 1960, 1965 and 1970. Current Population Reports, Series P-25, No. 160, August 1957.*

Considering the present stages of urban and industrial development in King County, it is assumed that the population of this county will continue to decline in relation to the other three. It is assumed also that Pierce County, which is presently well developed, will have a relative gain at a slow rate which will taper off at the close of the study period. Snohomish County, particularly in the vicinity of Everett and southward, is expected to grow rapidly until the end of the century and at a slower pace thereafter. Kitsap County is expected to grow at a gradually accelerating pace not only because of its own industrial development but also because of its close ties to Seattle.

High Population Projection. High population projections for Washington, for the central Puget Sound region, and for King and Snohomish counties, as listed in Table 5-8, are based on the aforementioned high projection of national population and on the ratios given in Table 5-7. Forecasts for the state (Fig. 5-8) fall below the figures given in Table 5-6 for 1960 and 1965 but show an increased rate of growth after 1970. The slower initial rate derives from the lower fertility rates for the nation as compared to the state (Table 5-4), while the rise after 1970 results from births to the large postwar generation. Beyond 1970, the projected totals for the state are 4,320,000 by 1980, 6,500,000 by 2000, and 10,600,000 by 2030.

For King County, the high projection indicates a population of 1,290,000 by 1980, 1,690,000 by 2000, and 2,400,000 by the year 2030. Up to 1970, as in the case of the state, the projection falls below the short range estimate based on current King County fertility rates (Table 5-4). On the other hand, the 2030 forecast of 2,400,000 exceeds a tentative projection of the Seattle Planning Commission staff by more than forty per cent. This is because the latter assumes a lower initial rate of growth and a continuous decline in the rate to an average of 0.96 per cent per year during the last 30 years.

For Snohomish County, the high projection indicates a population of 264,000 by 1980, 592,000 by 2000, and 1,100,000 by 2030.

Low Population Projection. To obtain a low projection suitable for consideration of financing problems, a forecast of the state population was developed, using an increase of 1.41 per cent per year. This is the least rate of growth the state experienced in any 30-year period.

Based on the 1.41 per cent annual rate, the projected low population is 87.5 per cent of the high projection for 1980, 77 per cent for the year 2000, and 71.5 per cent for 2030. Percentages for intermediate years are given in Table 5-8. By applying these percentages to the high projection for King County, the resulting low projection for the county falls below the Seattle Planning Commission medium projection for all years.

Population Projection Within the Study Area

As outlined previously, population estimates for the sewerage and drainage survey study area were

Table 5-8. Projected State, County, and Study Area Population, 1957 - 2030

Year	Washington			Central Puget Sound Region,[c] 1,000	King County, 1,000	Snohomish County, 1,000	Survey Study Area, 1,000
	High, 1,000	Low,[a] 1,000	Ratio,[b] per cent				
1957	2,670			1,370	859	134	864
1960	2,880	2,850	99.0	1,480	900	147	910
1970	3,480	3,270	94.0	1,800	1,080	187	1,080
1980	4,320	3,770	87.5	2,240	1,290	264	1,260
1990	5,300	4,350	82.0	2,760	1,510	378	1,500
2000	6,500	5,000	77.0	3,380	1,690	592	1,740
2010	7,750	5,730	74.0	4,030	1,900	780	1,920
2020	9,100	6,590	72.5	4,730	2,140	935	2,090
2030	10,600	7,600	71.5	5,500	2,400	1,100	2,240

[a]*Low estimate based on average annual increase of 1.41 per cent per year.*
[b]*Ratio of high estimate to low estimate.*
[c]*Comprising King, Snohomish, Kitsap and Pierce Counties.*

derived primarily on the basis of near-saturation conditions in presently developing divisions of the area. On the other hand, the population within the fringe portions, which also must be included, was derived through distribution of the total populations previously projected for King and Snohomish counties.

Population study divisions within the survey area (Fig. 5-9) include 11 divisions within the city limit of Seattle, 12 others in King County, and that part of Snohomish County within the Lake Washington watershed. With few exceptions, those within King County comprise one or more entire census tracts. In a few cases, census tracts were split because of widely divergent topographic and land use conditions.

Study divisions within the city and seven others (North 2; East 1, 2 and 3; and South 1, 2 and 3) are as defined in 1954 by the Seattle Planning Commission. For these divisions, near-saturation conditions may reasonably be assumed to develop within the design period. Except for minor adjustments, the population and population density forecasts given in Table 5-9 for the year 2030 and intermediate years are the same for each division as those tentatively established by the planning commission staffs.

For the year 2030, the projected populations for Seattle and the seven King County divisions amount to a total of 1,417,000 persons. A similar study by the Snohomish County Planning Commission staff indicates a near-saturation population of 223,000 for the western and central portion of the Lake Washington watershed lying within Snohomish County. For those portions, therefore, of the survey study area which are subject to near-saturation development within the design period, the estimated population totals 1,640,000.

The balance of the study area, comprising about 250 square miles of land area, or 43 per cent of the total, is for the most part not likely to reach near-saturation levels within the design period. To arrive at a reason-

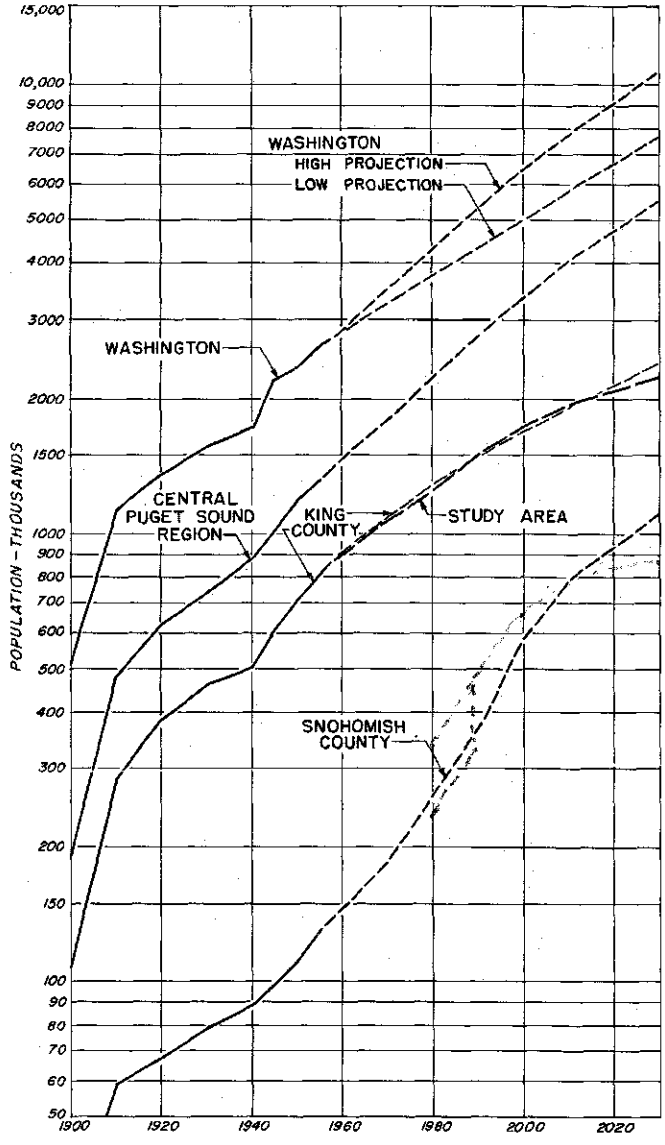


Fig. 5-8. Projected State, County, and Study Area Populations, 1957-2030

WEST SEATTLE has experienced rapid growth and development is estimated now to be about 90 per cent of the ultimate.

able population estimate for this portion requires a careful consideration of factors influencing development both within and outside of the study area. Outside the study area, these factors include:

1. The proposed bridge across Puget Sound via Vashon Island. This link will increase greatly the suburban development on the island, which has an area of 36 square miles and could ultimately accommodate a population of 75,000 to 100,000.

2. A continued urbanization of the Federal Way area. Although it lies within King County, this area is for the most part within the topographic sewerage area of Tacoma and is close to Tacoma's heavy industrial zone. A projected population of 100,000 by 2010[14] for all portions of King County within the sewerage area of Tacoma may well increase to 120,000 by 2030.

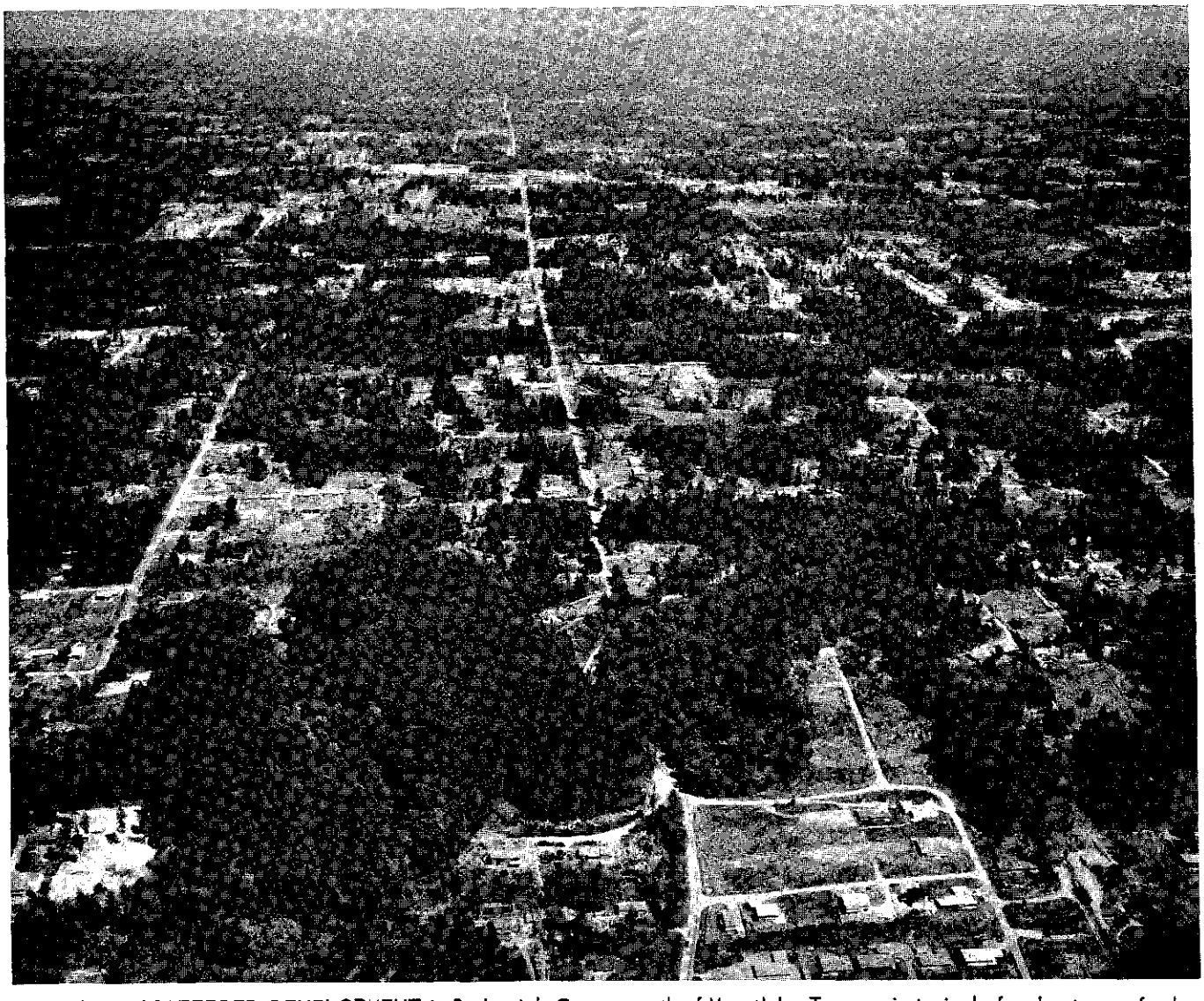
[14] "Metropolitan Tacoma Sewerage and Drainage Survey", *Brown and Caldwell, 1957.*

3. The growth of satellite communities. Communities lying between the survey study area and the Cascades, both existing and yet to be created, eventually will experience vigorous growth. A similar pattern of development has been typical of other metropolitan areas and is strengthened at present by the trend toward dispersal of light industry.

4. A diversification of industry in Snohomish County. Based largely on forest products in the past, the industrial economy of Snohomish County will be bolstered in the future by the oil refineries planned for construction both north and south of Everett. Further diversification and dispersal can be expected and will bring with it a widespread development of new residential areas.

Within the sewerage and drainage study area, factors influencing future development include:

1. The southward movement of the center of heavy



PRESENT SCATTERED DEVELOPMENT *in Snohomish County, north of Mountlake Terrace, is typical of early stages of suburban residential growth.*

industry. The proposed extension of navigable water in the lower Duwamish River and the construction of sites for heavy industry southward to 180th Street, along with continuing industrialization in the vicinity of Kent and Auburn, will shift the center of industrial employment southward. Early stimulation of residential development on both sides of the Green River valley can be expected as a result of this movement. Eventually, either the valley will be completely industrialized, or if agricultural zoning is enforced, the proposed Covington heavy industrial area will be developed. In either case, this will promote residential development in the adjoining or surrounding southeast division.

2. The freeway and bridge program. Construction of the proposed system of freeways and expressways, coupled with a second Lake Washington crossing, not only will facilitate travel within presently congested areas but will extend commuting distances and thus affect residential distribution.

3. A dispersion of light industry. Development of numerous small sites for light industry in the eastern and northern divisions of the study area will affect population growth in these divisions.

With the foregoing factors taken into account, it was estimated that the five study divisions within King County (East 4 and 5, South 4 and 5, and Southeast) will have a population in the year 2030 of about 560,000. This total is expected to be divided among the five divisions on the basis set forth in Table 5-9.

Within Snohomish County, enlargement of the study area eastward to include all of the watershed tributary to Lake Washington added 10,440 acres to the area of 38,400 acres for which the tentative near-saturation population previously had been estimated. Based on the county land-use plan for the watershed, modified

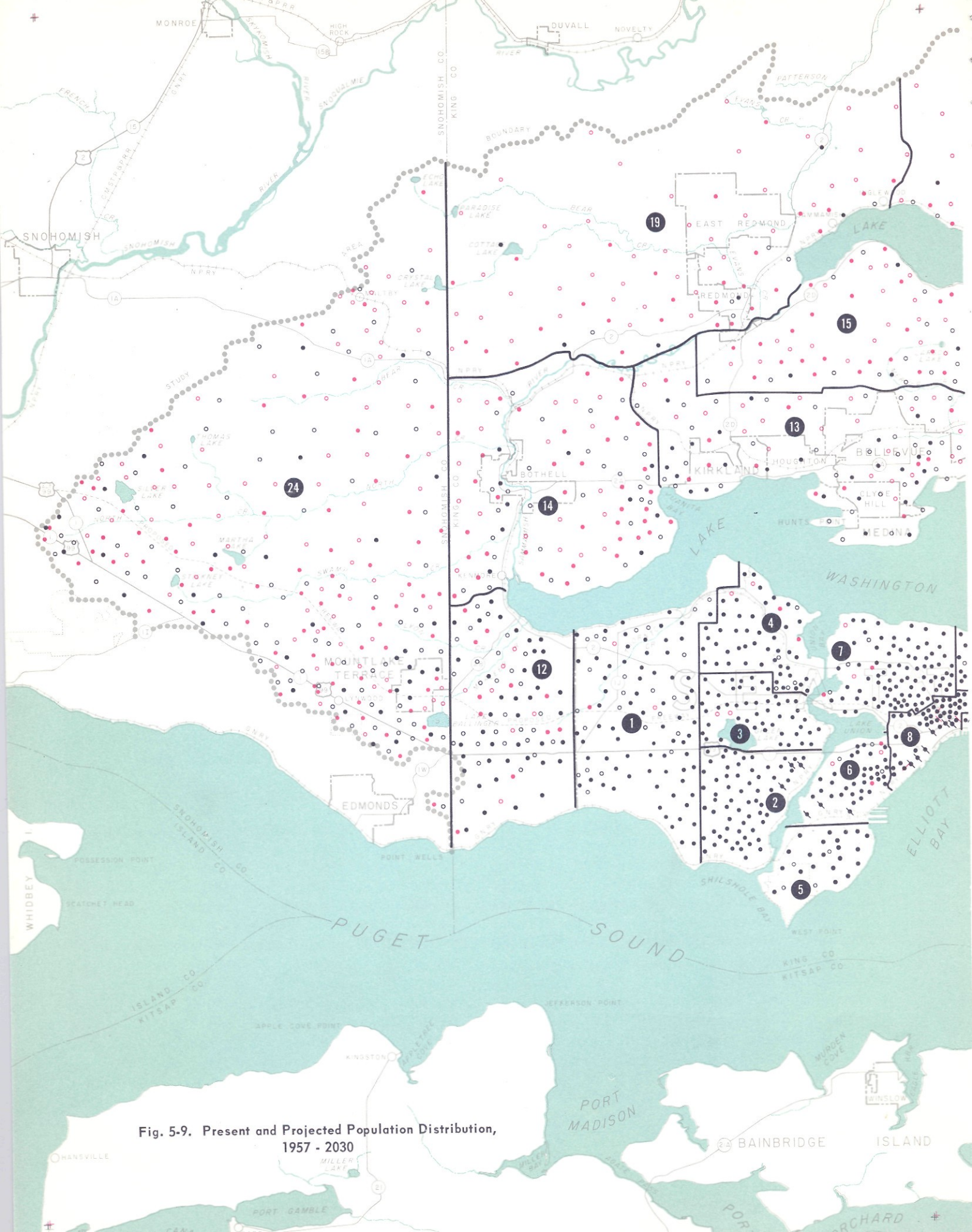


Fig. 5-9. Present and Projected Population Distribution, 1957 - 2030

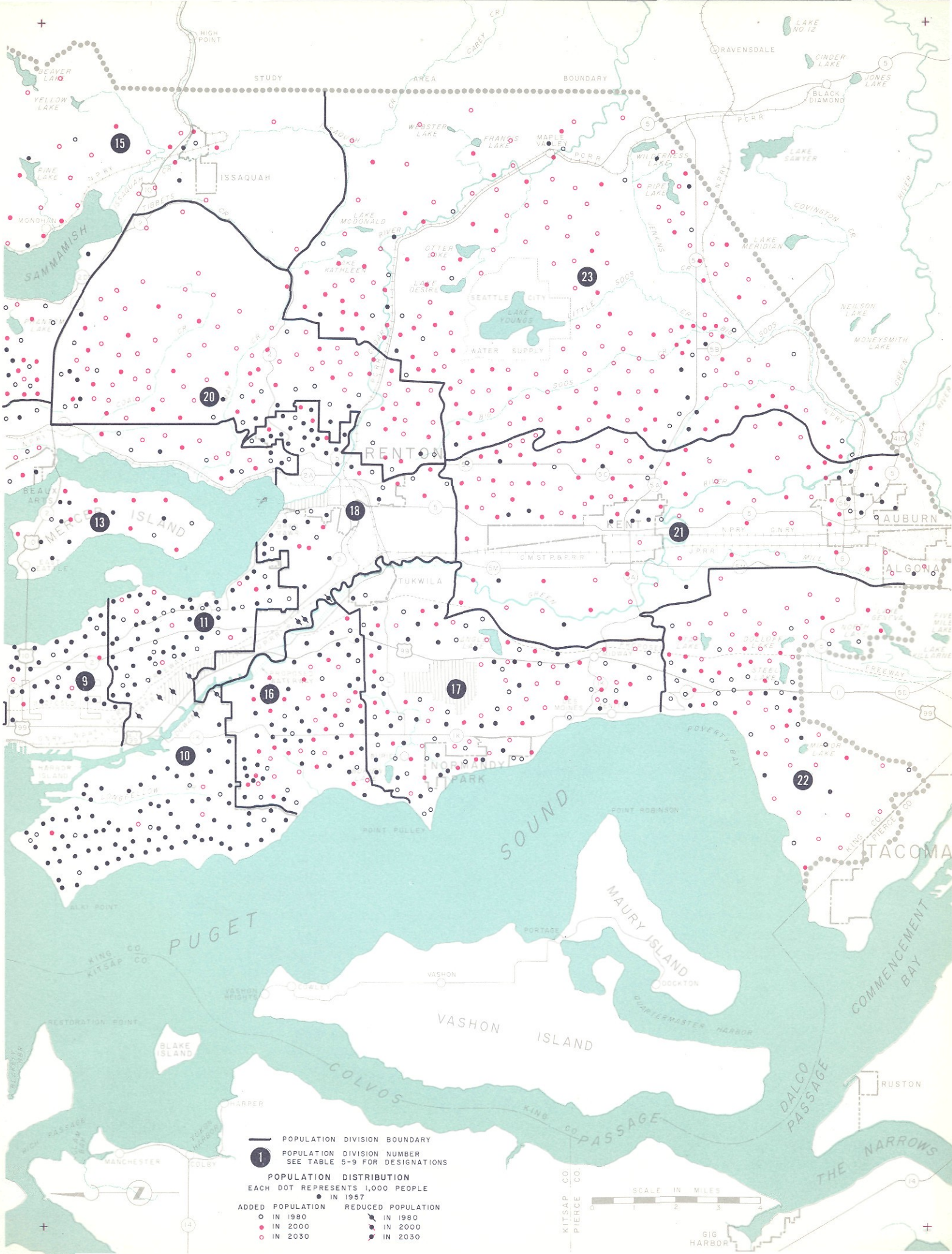

STUDY AREA BOUNDARY
HIGH POINT
BEAVER LAKE
YELLOW LAKE
RAVENSDALE
LAKE NO 12
CINDER LAKE
JONES LAKE
BLACK DIAMOND
WEBSTER LAKE
FRANKS LAKE
MAPLE VALLEY
WILDERNESS LAKE
PIPE LAKE
LAKE SAWYER
COVINGTON CR
LAKE MERIDIAN
NEILSON LAKE
MONEYSMITH LAKE
ISSAQUAH
SAMMAMISH
LAKE MCDONALD
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
RENTON
BEAUX ARTS
MERCER ISLAND
KENT
AUBURN
ALGONA
TUKWILA
NORMANDY PARK
POVERTY BAY
POINT ROBINSON
POINT PULLEY
TACOMA
PUGET SOUND
ALKI POINT
KING CO
KITSAP CO
MAURY ISLAND
PORTAGE
VASHON
DOCKTON
QUARTERMASTER HARBOR
VASHON ISLAND
COMMENCEMENT BAY
RESTORATION POINT
BLAKE ISLAND
COLVOS PASSAGE
DALCO PASSAGE
RUSTON
HARPER
MANCHESTER
COLBY
THE NARROWS
PIERCE CO
KITSAP CO
PIERCE CO
GIG HARBOR
SCALE IN MILES
0 1 2 3 4
POPULATION DIVISION BOUNDARY
POPULATION DIVISION NUMBER SEE TABLE 5-9 FOR DESIGNATIONS
POPULATION DISTRIBUTION
EACH DOT REPRESENTS 1,000 PEOPLE
IN 1957
ADDED POPULATION
IN 1980
IN 2000
IN 2030
REDUCED POPULATION
IN 1980
IN 2000
IN 2030

Table 5-9. Present and Projected Population Distribution of Study Area

Population division[a]	Area, acres[b]	Population, thousands				Density, persons per acre			
		1957	1980	2000	2030	1957	1980	2000	2030
Seattle - 1957 limits									
1. Broadview - Lake City	10,400	72	97	98	100	6.9	9.3	9.4	9.6
2. Ballard - Fremont	3,810	63	60	60	60	16.5	15.7	15.7	15.7
3. Green Lake - Wallingford	2,760	47	47	47	49	17.0	17.0	17.0	17.8
4. Ravenna - View Ridge	3,530	47	51	53	55	13.3	14.4	15.0	15.5
5. Magnolia	2,450	21	26	26	26	8.6	10.6	10.6	10.6
6. Queen Anne	2,450	33	33	35	37	13.5	13.5	14.3	15.1
7. Capitol Hill - Madrona	4,350	82	85	87	89	18.8	19.5	20.0	20.5
8. Downtown	1,470	44	38	36	34	30.0	25.9	24.5	23.1
9. Rainier Valley - Mt. Baker	5,330	38	42	43	44	7.1	7.9	8.1	8.3
10. West Seattle	10,170	80	85	86	89	7.9	8.4	8.5	8.8
11. Columbia - Rainier Beach	6,320	45	59	63	67	7.1	9.3	10.0	10.6
Subtotal - Seattle	53,040	572	623	634	650	10.8	11.8	12.0	12.3
12. North 2	9,690	36	66	78	85	3.1	5.7	6.7	7.3
13. East 1	23,320	39	88	114	146	1.7	3.8	4.9	6.3
14. East 2	16,480	10	40	78	97	0.6	2.4	4.7	5.9
15. East 3	34,080	16	47	106	180	0.5	1.4	3.1	5.3
16. South 1	8,300	39	50	73	87	4.7	6.0	8.8	10.5
17. South 2	15,920	36	72	97	122	2.3	4.5	6.1	7.7
18. South 3	10,000	31	39	50	50	3.1	3.9	5.0	5.0
19. East 4	34,210	6	12	39	82	0.2	0.4	1.1	2.4
20. East 5	21,750	4	16	35	55	0.2	0.7	1.6	2.5
21. South 4	28,850	20	27	76	113	0.7	0.9	2.6	3.9
22. South 5	15,450	7	31	54	91	0.5	2.0	3.5	5.9
23. Southeast	50,330[c]	14	33	117	216	0.3	0.7	2.3	4.3
Subtotal - King County	321,420	830	1,144	1,551	1,974	2.6	3.6	4.8	6.1
24. Snohomish County	48,840	34	113	190	264	0.7	2.3	3.9	5.4
Total	370,260	864	1,257	1,741	2,238	2.3	3.4	4.7	6.0

[a]*See Fig. 5-9 for location of population divisions.*

[b]*Gross area excluding major lakes, waterways, and undeveloped tidelands.*

[c]*Excluding Lake Young reservoir and Seattle Water Department property having total area of 2,560 acres.*

slightly in light of the county projection, the population of the Snohomish County portion of the study area was estimated to reach 264,000 in the year 2030. For the entire study area, therefore, the population forecasts give a total of 2,238,000 persons in 2030 (Table 5-9).

Totals for the intermediate years, 1980 and 2000, are based on the estimated rate of growth within each of the population divisions.

Detailed Distribution of Study Area Population

In developing the population projections, the method used for that purpose required a general distribution of population among the major divisions of the study area. This information, together with that developed from available data on existing distribution and from future land use plans, was utilized in preparing the detailed distribution illustrated by the dot map in Fig. 5-9. In analyzing the land use plans, consideration was given in particular to the locations of industrial zones, recreational areas, and highways.

It will be noted that certain areas within Seattle already have attained their maximum population. In a few, such as the downtown section and certain industrial areas, a decrease is predicted. Within the design period, the net growth within the present limits of Seattle is expected to be 80,000 as compared to 1,370,000 in the remainder of the study area.

Average densities predicted for the several divisions in 2030 range from 2.4 to 20.5 persons per gross acre. For the entire study area, the average density is estimated to be 6.0 persons per gross acre.

Population density, along with land use for commercial and industrial purposes, governs both the volume of sewage and the amount of storm runoff from a particular area. For that reason, the dot map (Fig. 5-9) is used later to obtain both design population and population density values for the individual sewerage and drainage service areas.

PART III

EXISTING SEWERAGE, SEWAGE DISPOSAL AND DRAINAGE

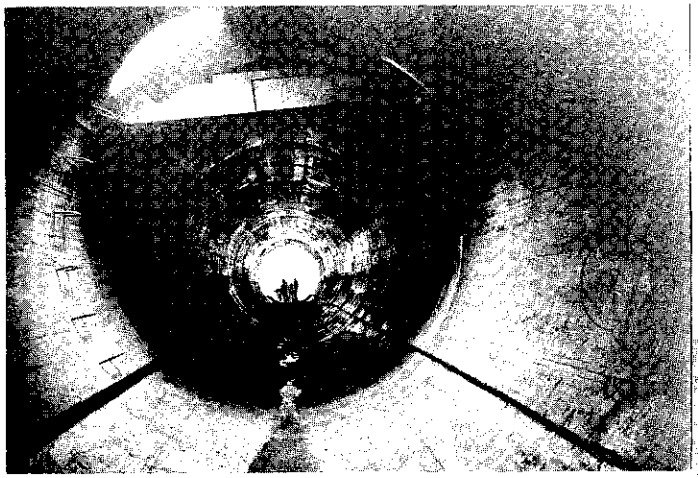
Chapter 6. Existing Sewerage and Drainage Facilities
Chapter 7. Sewage Characteristics
Chapter 8. Environmental and Economic Effects of Sewerage and Drainage Deficiencies

CONSTRUCTION OF NORTH TRUNK SEWER

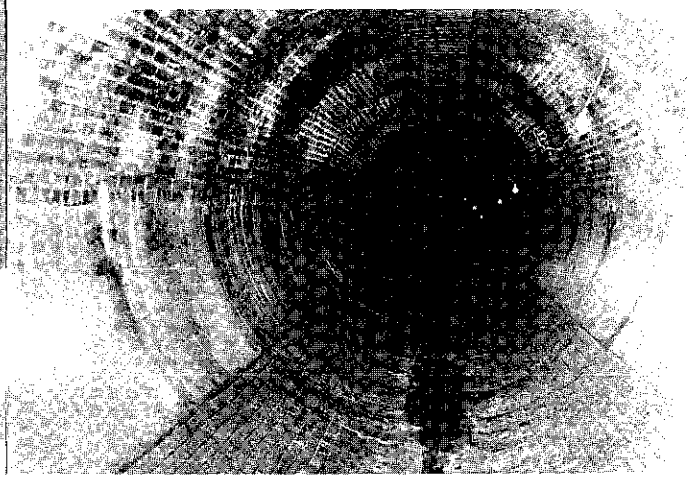
LAYING BRICK ARCH in 144-inch Interbay tunnel section.

← OPEN CUT TRENCH for 144-inch Interbay section constructed in 1912.

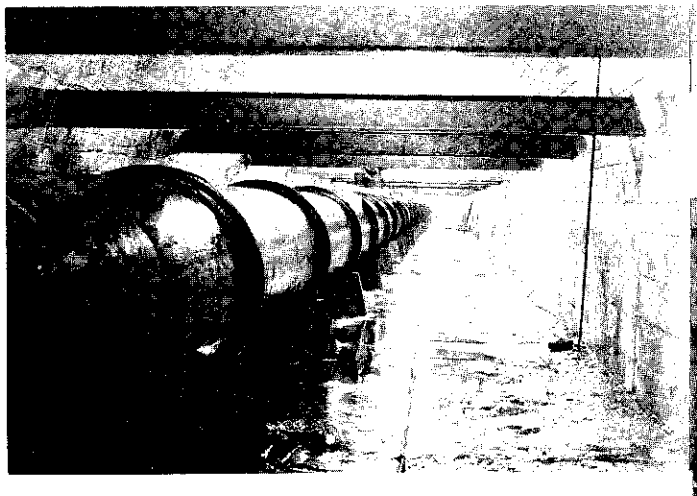
← 144-INCH CONCRETE SECTION constructed in open cut, ready for laying of brick invert and construction of manhole.



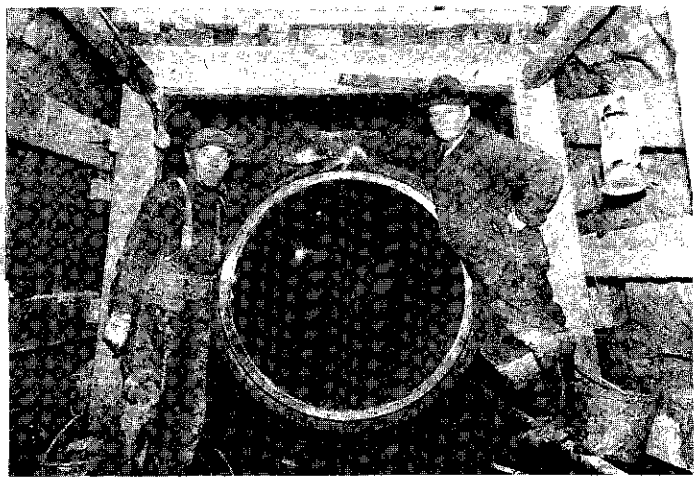
COMPLETED 114-INCH TUNNEL SECTION beneath Montlake Boulevard.



← 40-INCH SIPHON IN TUNNEL under ship canal at 3rd Street West. Subsequently, an additional 60-inch siphon was laid in space at right.



48-INCH CONCRETE PIPE being laid in Laurelhurst tunnel section.



SEATTLE, 1912

Chapter 6
EXISTING SEWERAGE AND DRAINAGE FACILITIES

One of the basic objectives of the present survey is that of determining the extent to which facilities presently in use can be incorporated in a long-range program of sewerage and drainage improvements. Accordingly, all such facilities were appraised and evaluated in terms of their ability to meet both present and future needs.

Information presented in this chapter was derived from questionnaires answered by various sewerage agencies, from reviews of plans and reports, and from field investigations. Information and data concerning sewage volumes and composition, although obtained as part of the study of existing systems, are set forth in Chapter 7 on sewage characteristics.

Responsibility for providing sewerage service within the metropolitan area is divided among 19 cities and 22 sewerage districts (Fig. 6-1). Of these 41 agencies, 15 are presently engaged in the operation of sewerage facilities (Table 6-1). The remainder (Table 6-2) are either (1) in various stages of planning, financing and constructing facilities (2) annexed to operating agencies, or (3) essentially inactive. In addition to the public systems, there are 8 semi-public and private systems which serve military, airport, and industrial establishments, and multiple housing developments (Fig. 6-1 and Table 6-3).

Between them, the various agencies operate and maintain about 1,550 miles of sewers, 75 pumping stations, and 25 sewage treatment works. Of the total length of sewers, about 510 miles are separate sanitary lines, while 1,040 miles are of the combined type carrying both sanitary sewage and storm drainage. Virtually all of the combined sewers are located in Seattle.

The aggregate design capacity of the 25 treatment plants is 28 mgd, which is sufficient to treat only about one-third of the average daily sewage flow generated during dry weather (average DWF). Consequently, close to 50 mgd of sewage and industrial wastes are being discharged without treatment through about 60 independent outfalls. Despite the multiplicity of facilities, only about 70 per cent of the residents of the area are served by public sewers. The remaining residents provide and maintain individual septic tank systems, which are being constructed at a rate of about 6,000 per year.

Information regarding assessed valuation, tax rate, service and other charges, and bonded indebtedness is given in Table 6-4. This table also lists annual costs of operation and maintenance.

Responsibility for providing drainage facilities rests with the cities and, to a limited extent, with local drainage districts and the counties. In addition to Seattle, only four cities and one local district have storm drainage systems worthy of note. Drainage activities of the counties are limited by law to situations directly associated with street and highway construction and drainage. As a result, such facilities as exist in the unincorporated areas are mainly by-products of street and highway drains and serve localized problem areas only.

In the following sections of this chapter, each existing sewerage and drainage system is analyzed in sufficient detail to permit its evaluation in terms of the future sewerage and drainage requirements of the area.

SEWERAGE FACILITIES OUTSIDE SEATTLE

Of the 35 square miles now occupied by cities and sewer districts situated in the metropolitan area outside Seattle, about 20 square miles were sewered as of July 1, 1957. In this outside area, sewerage service is provided by five cities and nine sewer districts (Table 6-1). Sewage treatment plants are operated by each of the five cities and by seven of the nine districts.

Bellevue Sewer District

The Bellevue Sewer District was organized in 1948 to serve a small section of what is now the city of Bellevue. Today, the district encompasses an area of nine square miles, including most of Bellevue, all of four other cities (Beaux Arts, Clyde Hill, Hunts Point and Medina), and adjacent unincorporated areas. Only 1.1 square miles are now sewered and serve a connected population of 4,100. Construction has begun, however, in Utility Local Improvement District (ULID) No. 7, which will serve the northerly section of the district. Construction has begun also in ULID No. 8, which includes Clyde Hill, Hunts Point and Medina. No service is contemplated in the Beaux Arts area until about 1959.

The Bellevue collection system consists of 21 miles of sewers ranging in size from 6 to 16 inches, and four sewage pumping stations, two of which are utilized for flushing an intercepting sewer submerged in

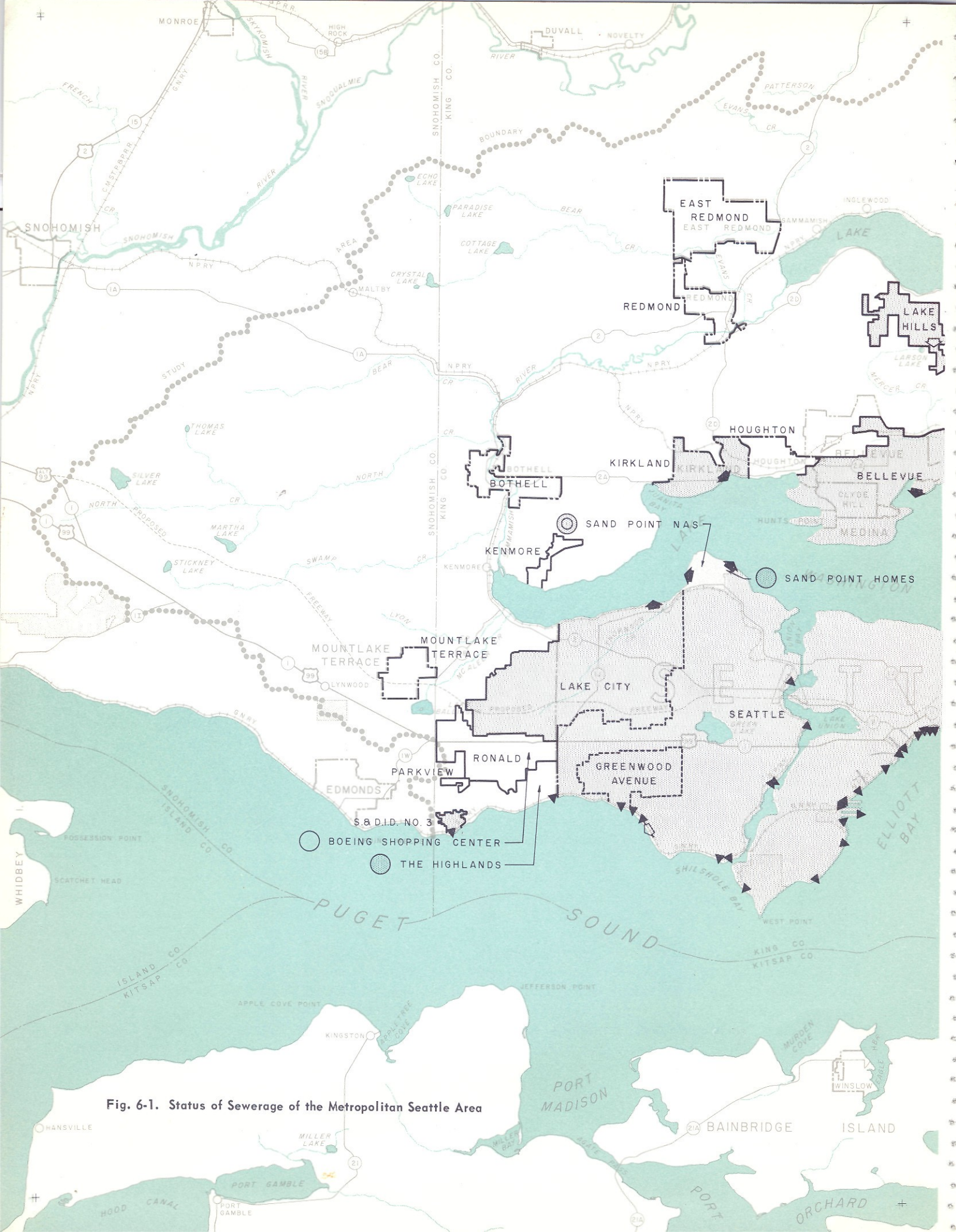


Fig. 6-1. Status of Sewerage of the Metropolitan Seattle Area

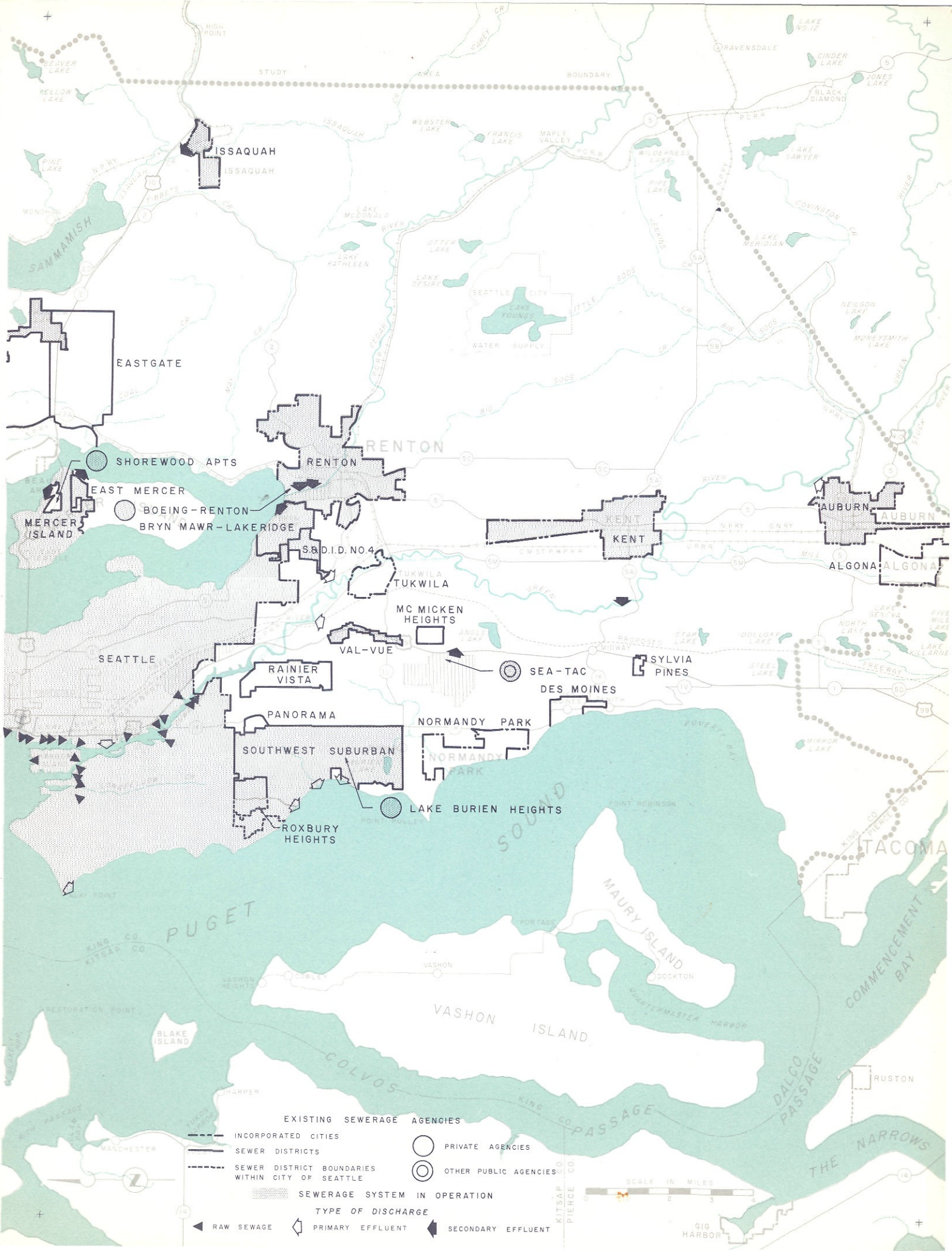

ISSAQUAH
EASTGATE
SHOREWOOD APTS
EAST MERCER
MERCER ISLAND
BOEING-RENTON
BRYN MAWR-LAKERIDGE
RENTON
S.B.D.I.D. NO.4
TUKWILA
MC MICKEN HEIGHTS
VAL-VUE
SEATTLE
RAINIER VISTA
SEA-TAC
SYLVIA PINES
DES MOINES
PANORAMA
NORMANDY PARK
SOUTHWEST SUBURBAN
LAKE BURIEN HEIGHTS
ROXBURY HEIGHTS
KENT
AUBURN
ALGONA
TACOMA
PUGET SOUND
MAURY ISLAND
VASHON ISLAND
COMMENCEMENT BAY
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
RUSTON
GIG HARBOR
BLAKE ISLAND
SAMMAMISH
EXISTING SEWERAGE AGENCIES
INCORPORATED CITIES
SEWER DISTRICTS
SEWER DISTRICT BOUNDARIES WITHIN CITY OF SEATTLE
PRIVATE AGENCIES
OTHER PUBLIC AGENCIES
SEWERAGE SYSTEM IN OPERATION
TYPE OF DISCHARGE
RAW SEWAGE
PRIMARY EFFLUENT
SECONDARY EFFLUENT
SCALE IN MILES
0 1 2 3 4

Table 6-1. Status of Sewerage in Operating Sewerage Agencies

Agency	Date agency formed	Area, square miles		Population	
		Total	Sewered	Total	Connected
Cities and towns					
Auburn	1891	2.1	2.0	8,000	7,000
Issaquah	1913	0.7	0.3	1,100	1,450
Kent	1890	3.0	1.7[a]	4,150	4,000
Kirkland	1905	2.0	2.0[b]	5,750	5,750
Renton	1901	5.0	3.3	16,500	14,800
Seattle[c]	1865	79.5[d]	69.5[e]	561,000	510,000
Sewer districts					
Bellevue[f]	1948	9.0	1.1	[g]	4,100
Bryn Mawr - Lake Ridge	1950	1.1	0.8	[g]	4,500
East Mercer	1952	0.3	0.3	600	500
Lake Hills	1955	1.9	0.3	1,550	1,460
Mercer Island	1953	2.2	0.9	3,000	800
Sewerage and Drainage District No. 3	1939	0.5	0.5[h]	[g]	1,100
Sewerage and Drainage District No. 4	1943	0.4	0.3	2,300	2,400
Southwest Suburban	1945	6.8	3.5	[g]	12,600
Val Vue	1945	0.4	0.2	[g]	1,150
Total		114.9	86.7		571,610

Agency	Sewers			Number of pumping stations	Treatment plants			Discharge to
	Miles	Year first constructed	Diameter range, inches		Type[i]	Year constructed[j]	Capacity, average mgd	
Cities and towns								
Auburn	28	1910	6-30	1	P	1950	1.1	Green River
Issaquah	13	1940	8-15	0	S	1940	0.23	Issaquah Creek
Kent	10	[k]	6-24	1	S	1954	1.7	Green River
Kirkland	32	1942	6-24	5	S	1943-51	1.6	Lake Washington
Renton	36	1910	6-24	1	S	1943-53	2.2	Cedar River
Seattle[c]	1,233[m]	1883	6-144	39[n]	[p]		26.5[q]	[r]
Sewer districts								
Bellevue[f]	21	1953	6-16	4	S	1954-57	0.56	Lake Washington
Bryn Mawr-Lake Ridge	15	1952	6-18	2	S	1952	0.43	Lake Washington
East Mercer	5	1954	8-12	0	S	1955	0.13	Lake Washington
Lake Hills	7	1955	8-18	5	P	1955-57	0.32	Land
Mercer Island	25	1956	6-16	8	[s]			[t]
Sewerage and Drainage District No. 3	7[u]	1941	6-18	0	[v]			Puget Sound
Sewerage and Drainage District No. 4	10	1943	6-18	2	P	1943-44	0.2	Duwamish River
Southwest Suburban	54	1942	8-36	5	P	1956	8.2	Puget Sound
Val Vue	3	1947	6-15	0	P	1955	0.28	Duwamish River
Total	1,526			73			43.45	

[a] *Includes 0.2 square mile outside city limits.*
[b] *1.5 square miles inside city limits; 0.5 square mile outside.*
[c] *Includes the systems of Lake City, Greenwood Avenue, and Roxbury Heights Sewer Districts now operated by Seattle.*
[d] *Exclusive of water areas.*
[e] *Includes 2.5 square miles outside city limits.*
[f] *Includes cities of Beaux Arts, Bellevue, Clyde Hill, Medina and Hunts Point.*
[g] *No reliable estimate available.*
[h] *Also serves "Old Firlands" area.*
[i] *P - primary; S - secondary.*
[j] *Where two years are shown, second indicates major enlargement.*
[k] *Initial construction of modern sewers assumed to be prior to 1930.*
[m] *1,029 miles combined; 204 miles separate sanitary.*
[n] *Does not include several small lift stations serving small areas.*
[p] *Seattle operates five treatment plants and has one under construction; in addition, there are a number of raw sewage outfalls and bypasses; see section on existing Seattle system.*
[q] *Total capacity. Includes Alki Point plant and enlargement of Lake City plant, both under construction.*
[r] *See section on existing Seattle system.*
[s] *Sewage treated at Bellevue plant.*
[t] *See Bellevue.*
[u] *Includes 1.9 miles outside district.*
[v] *Comminuter and chlorination station, inoperative.*

Table 6-2. Status of Sewerage in Non-Operating Sewerage Agencies [a]

Agency	Date formed	Area, square miles	Status
Cities and towns			
Algona	1955	1.4	Inactive.
Beaux Arts	1954	0.1	Annexed to Bellevue Sewer District.
Bellevue	1953	4.0	Served by Bellevue Sewer District.
Bothell	1909	1.6	System financed; plans in preparation.
Clyde Hill	1953	1.0	Annexed to Bellevue Sewer District; sewers under construction.
East Redmond	1956	3.6	Inactive.
Houghton	1947	1.3	Partially served by Kirkland.
Hunts Point	1955	0.3	Annexed to Bellevue Sewer District; sewers under construction.
Medina	1955	1.3	Annexed to Bellevue Sewer District; sewers under construction.
Mountlake Terrace	1954	1.5	Preliminary plans in progress.
Normandy Park	1953	1.6	Inactive.
Redmond	1912	1.3	System financed; construction scheduled 1957-58.
Tukwila	1908	0.7	Inactive.
Sewer districts			
Des Moines	1946	0.5	System financed; plans in progress; construction scheduled spring 1958.
Eastgate	1957	4.0	Preliminary planning scheduled fall 1957.
Greenwood Avenue	1946	2.7	Completely sewered; operated by Seattle.
Kenmore	1955	0.5	Inactive.
Lake City	1946	12.2	Approximately 50 per cent sewered; operated by Seattle.
McMicken Heights	1948	0.3	Inactive.
Panorama	1945	0.2	Inactive.
Parkview	1948	0.1	Inactive.
Rainier Vista	1945	1.2	System financed; plans in preparation, scheduled for construction spring 1958.
Richmond	1949	0.5	System operated by King County Engineer under Sewerage and Drainage District No. 3.
Ronald	1951	1.5	System financed; plans in preparation.
Roxbury Heights	1944	0.8	Approximately 50 per cent sewered; operated by Seattle.
Sylvia Pines	1957	0.1	New district; plans in progress.

[a]*Presently unsewered except as noted.*

the water adjacent to the shoreline of Meydenbauer Bay (Fig. 6-2 and Table 6-5). Complete treatment facilities, with a design capacity of 0.56 mgd, were constructed in 1954 and are now being expanded to a capacity of 2 mgd (Fig. 6-3). The added units are temporary and are to be abandoned when area-wide sewerage service becomes available. Plant effluent is chlorinated and discharged to Lake Washington. Sewage from Mercer Island Sewer District is treated at the Bellevue plant under a contractual arrangement.

Table 6-3. Summary of Private and Semi-Public Sewerage Facilities

System	Facility served	Area served, square miles	Treatment		Discharge to
			Type[a]	Capacity, average mgd	
Boeing-Renton	Aircraft plant	0.25	S	0.5	Cedar River
Boeing Shopping Center	[b]	0.03	P	0.18	Puget Sound
The Highlands	Residential	0.7	None	—	Puget Sound
Lake Burien Heights	Apartments	0.03	P	0.15	Puget Sound
Sand Point Naval Air Station	Military reservation	0.70	S	0.86	Lake Washington
Sand Point Homes	University housing	0.003	S	0.1	Lake Washington
Shorewood	Apartments	0.05	S	0.26	Lake Washington
Seattle-Tacoma International Airport	Airport	[c]	S	0.1	Bow Lake

[a]*P — primary; S — secondary.*

[b]*Constructed to serve proposed shopping center - nonoperating.*

[c]*Serves airport terminal and service buildings.*

Table 6-4. Summary of Financial Information for Sewerage Agencies

Agency	Assessed valuation,[a] $1,000,000	Levy,[b] mills	Service charge,[c] dollars	Connection fee, dollars	Bonds outstanding, $1,000		Annual cost of maintenance and operation, dollars
					General obligation	Revenue	
Sewer districts							
Bellevue	7.8	1.30	3.00	17.00	118.0	1,505.0	29,300
Bryn Mawr - Lake Ridge		None	2.00	17.50	None	295.0	15,230
East Mercer		None	4.00	300.00	None	124.0	5,890
Greenwood	12.2	1.00	2.00[d]	75.00[e]	127.0	1,803.0	[f]
Lake City	42.8	1.00	3.00[g]	35.00[h]	609.0	5,792.0	[f]
Lake Hills		None	3.50	None	None	140.0	4,800
Mercer Island	4.8	2.40	5.00	35.00	150.0	912.0	35,380
Roxbury Heights	1.2	2.30	None[i]	None	17.0	None	[f]
Southwest Suburban	15.2	2.30	3.00	15.00	481.0	3,800.0	84,000[j]
Val Vue		None	4.00	100.00[k]	None	146.0	8,470
Sewerage and Drainage District No. 3		None	0.22[m]	None	None	None	540
Sewerage and Drainage District No. 4		None	22.00[n]	None	None	None	13,240
Cities and towns							
Auburn		None	1.50	None	None	140.0	9,000
Issaquah		None	1.25[p]	0.83[q]	None	None	7,420
Kent		None	0.75	10.00	None	390.0	22,230
Kirkland		None	2.00[r]	50.00	None	283.0[s]	19,000
Renton	18.1	3.05	0.75	10.00	[t]	62.8	19,030
Seattle	637.4	0.003	1.00[u]	None	1,776.0	None	492,130

Based on data submitted by sewerage agencies; five additional agencies, Des Moines, Rainier Vista, Ronald, Redmond and Bothell have issued bonds for proposed facilities but are, as yet, inoperative.

[a] *Assessed valuation as of January 1, 1956, given only if there is levy for sewerage.*

[b] *Portion of general levy designated for sewerage.*

[c] *Monthly, unless otherwise indicated.*

[d] *Per single residence and duplex; more for others. Residents of the district also pay $1.00 per month City of Seattle charge.*

[e] *Per single residence; others, $150.00.*

[f] *Included in Seattle.*

[g] *Sewer bond charge; portion in Seattle also pays $1.00 city service charge; portion outside pays city pro-rata share of annual maintenance and operation cost – $6.54 per household in 1956.*

[h] *Late charge; applies if connected more than one year after service is available.*

[i] *Portion now within Seattle pays $1.00 service charge.*

[j] *Estimated cost for first year of operation.*

[k] *Included in original assessment.*

[m] *Annual per front foot charge.*

[n] *Flat annual charge.*

[p] *Charge is $1.50; $0.25 discount if paid by 10th of month.*

[q] *Per front foot.*

[r] *$3.00 outside city limits.*

[s] *Estimated sewerage portion of $850,000 outstanding sewer and water bonds.*

[t] *Unspecified portion of $589,000 water and sewerage issue.*

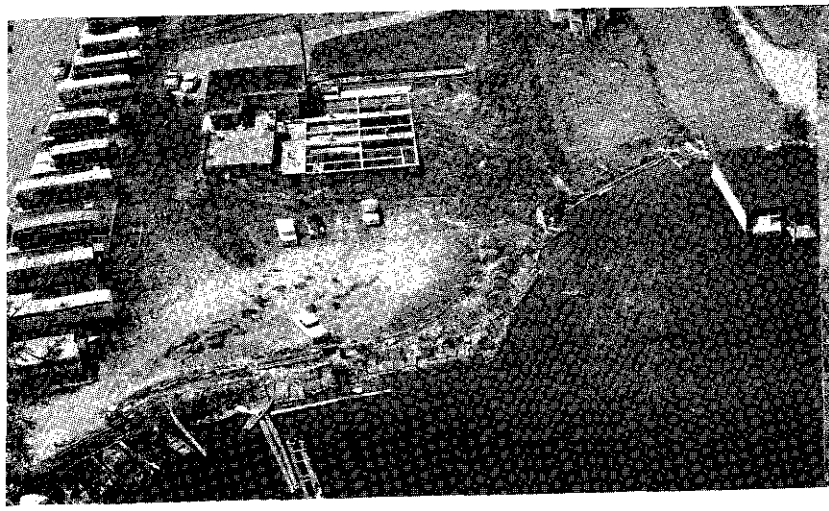
[u] *Minimum charge for 3/4-inch water service, with additional charge of $0.06 per 100 cubic feet for water in excess of 900 cubic feet per month.*

Bryn Mawr-Lake Ridge Sewer District

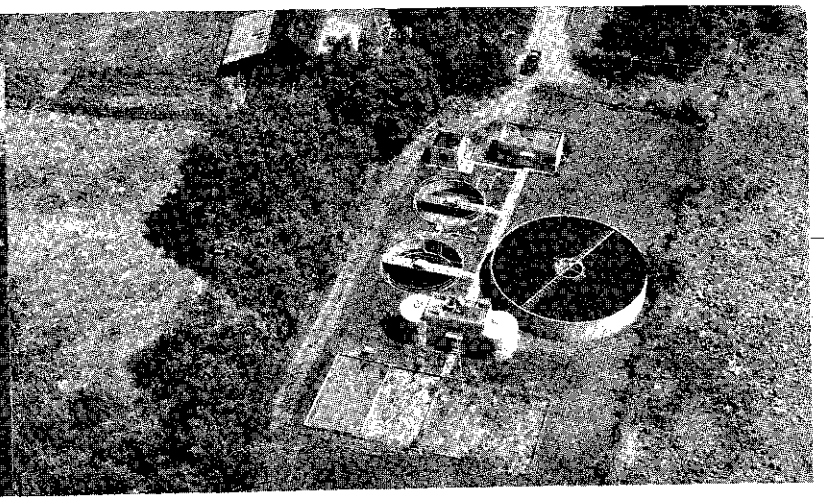
This district, which was formed in 1950 and comprised an area of 1.1 square miles, has boundaries in common with Seattle on the north and with Sewerage and Drainage District No. 4 and the city of Renton on the south. The collection system covers an area of 0.8 of a square mile, and consists of 15 miles of 6-inch to 18-inch sewers and two pumping stations. It serves a connected population of 4,500, as well as part of the Boeing-Renton plant (Fig. 6-4 and Table 6-6). Secondary treatment is provided by a plant which has a design capacity of 0.43 mgd and the effluent, after being disinfected, is discharged to Lake Washington (Fig. 6-5).

East Mercer Sewer District

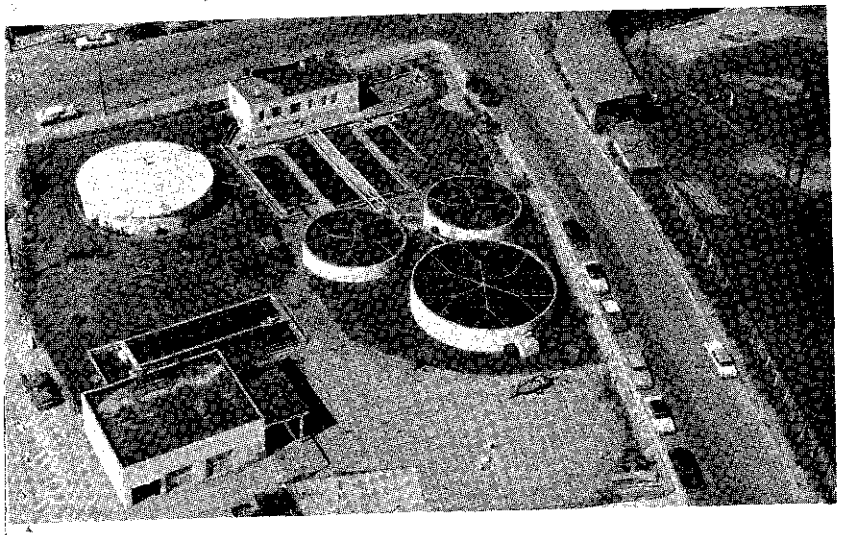
Formed in 1952, the East Mercer Sewer District comprises a 0.3 square mile section of northeasterly Mercer Island. Sewerage facilities were constructed throughout the district in 1954, and now serve a connected population of 500. The system is composed of 5.2 miles of 8-inch to 12-inch sewers (Fig. 6-2 and Table 6-7), and a secondary type sewage treatment plant with a design capacity of 0.13 mgd (Fig. 6-6). Plant effluent is chlorinated and discharged to Lake Washington.



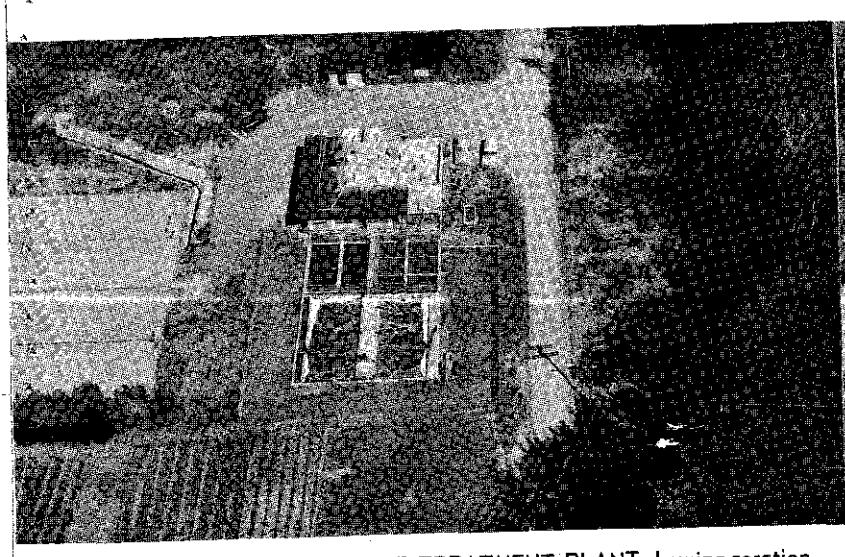
BRYN MAWR SEWAGE TREATMENT PLANT showing digesters and control building at left and aeration and sedimentation tanks at right. Effluent from this activated sludge plant is discharged into Lake Washington. Digested sludge is removed from site by tank truck.



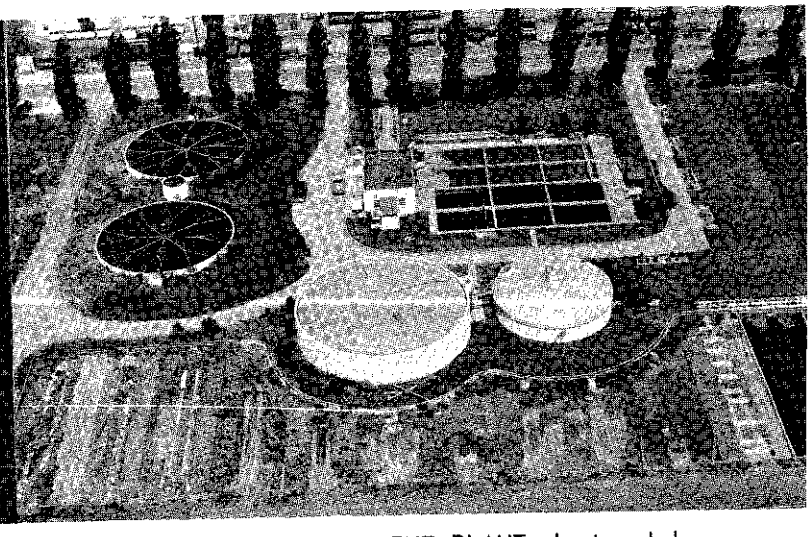
ISSAQUAH SEWAGE TREATMENT PLANT provides secondary treatment for a connected population of 1,100. Picture shows, from front to rear, sludge drying beds, digesters, trickling filter (right), secondary and primary clarifier and control buildings. Effluent is discharged into Issaquah Creek.



KIRKLAND SEWAGE TREATMENT PLANT provides secondary treatment of sewage from a connected population of 5,750, serving, in addition to Kirkland itself, the northerly section of Houghton and unincorporated areas east of the city. Plant effluent is discharged to Lake Washington via a trunk storm drain situated on the street beyond control building, upper right. Final sedimentation tank and sludge conditioning building are at lower left. Primary sedimentation tanks are located between control building and trickling filters with digester at left.



BELLEVUE SEWAGE TREATMENT PLANT showing aeration units in foreground, sedimentation tanks and control building with sludge drying beds at left, and sewer district headquarters in rear. Effluent from this activated sludge plant is discharged to Lake Washington. Plant additions which will quadruple the capacity are now being constructed.



RENTON SEWAGE TREATMENT PLANT showing sludge drying beds in foreground, digesters (center), trickling filters (left rear), two primary and two secondary rectangular sedimentation tanks (right rear), and control building. Note difference in clarity of liquid in primary tanks as compared with that in secondary tanks.

SUBURBAN SEWAGE TREATMENT PLANTS DISCHARGING WITHIN THE LAKE WASHINGTON DRAINAGE BASIN

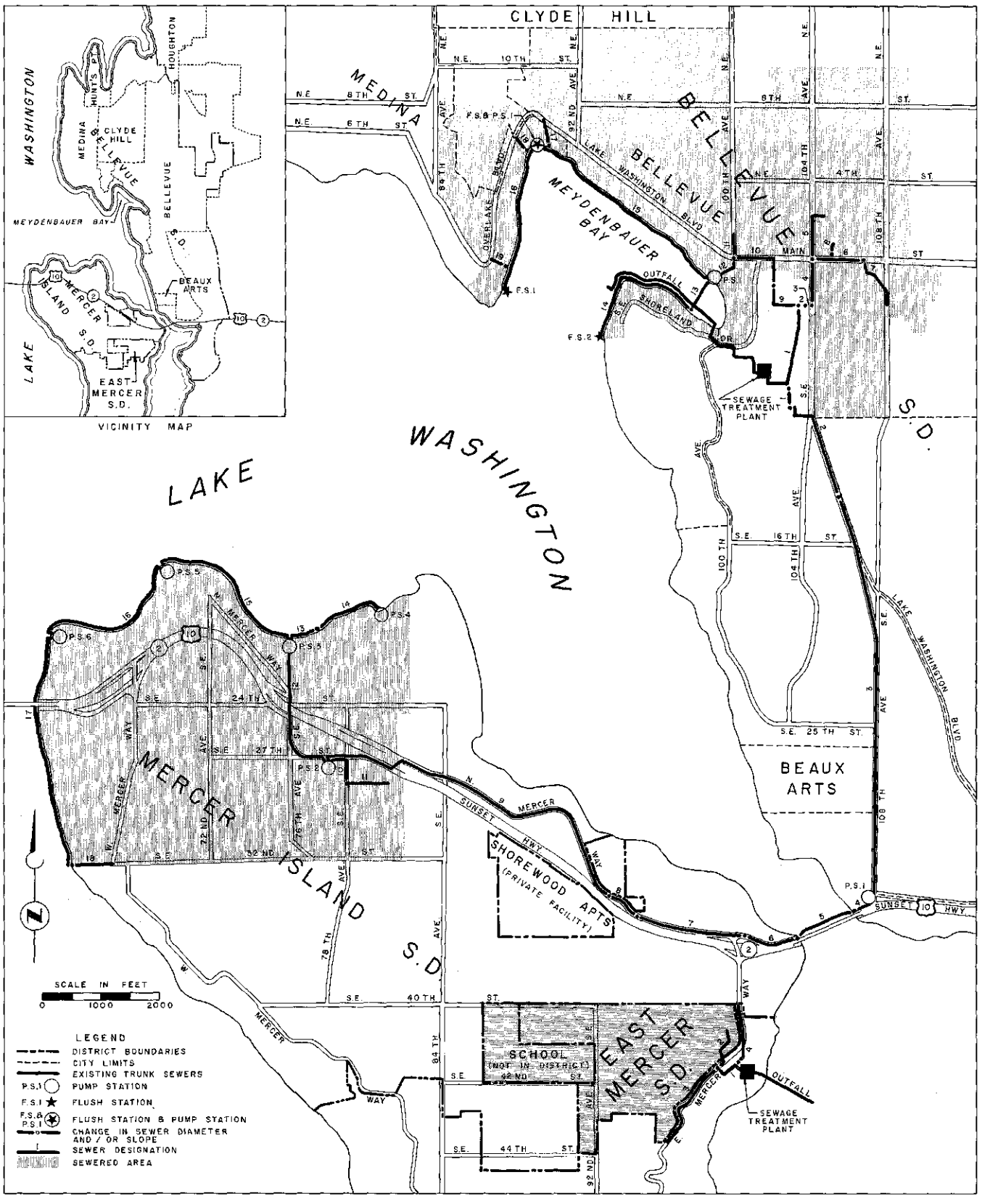


Fig. 6-2. Sewered Areas and Principal Sewerage Facilities Bellevue, East Mercer, and Mercer Island Sewer Districts

Table 6-5. Description of Principal Sewerage Facilities, Bellevue Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	1,880 ft of 14-in. CA pressure section	
2	170 ft of 12-in. CA pressure section	
3	100 ft of 12-in. conc at 6.49%	5.8
4	790 ft of 15-in. conc at 0.20%	1.8
5	890 ft of 10-in. conc at 0.22%	0.7
6	1,180 ft of 10-in. and 12-in. conc and RC at 3.09%	2.5
7	900 ft of 10-in. CI and conc at 0.40%	0.9
8	240 ft of 10-in. conc at 8.00%	1.3
9	410 ft of 12-in. CA at 2.85%	3.9
10	1,480 ft of 12-in. and 15-in. CA and conc at 0.80%	3.7
11	370 ft of 8-in. conc at 6.35%	2.0
12	640 ft of 8-in. CA force main	
13	750 ft of 8-in. CI inverted siphon	
14	2,060 ft of 8-in. CA submerged pressure line	
15	3,780 ft of 10-in. CA submerged pressure line	
16	2,720 ft of 8-in. CA submerged pressure line	
17	360 ft of 8-in. CA and conc at 7.10%	2.1
18	390 ft of 8-in. CA and conc at 1.20%	0.9
19	330 ft of 8-in. CA and conc at 3.0%	1.3
Outfall	3,200 ft of 16-in. CA and CI; discharge to Lake Washington	

[a]*See Fig. 6-2 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. CA signifies cement-asbestos pipe; conc, concrete; RC, reinforced concrete; CI, cast iron.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

Lake Hills Sewer District

Sewers were first constructed in the Lake Hills area in 1955 as part of a private development. The sewer district was formed the same year and now comprises an area of about 1.9 square miles. The collection system is being rapidly extended, and now consists of about 7 miles of 8-inch to 18-inch sewers and 5 sewage pumping stations (Fig. 6-7 and Table 6-8).

A primary type treatment plant, the design capacity of which was doubled in 1957 to its present level of 0.32 mgd, provides treatment for the flow from a connected population of about 1,460 (Fig. 6-8). Plant effluent is sprayed over a well isolated section of land by means of a sprinkler type irrigation system and is absorbed by the soil. The treatment facilities, however, are temporary in nature and are to be abandoned when the area becomes sufficiently developed to permit a permanent solution to its sewerage problems.

Mercer Island Sewer District

This district, formed in 1953, comprises an area of 2.2 square miles, of which 0.9 square mile is sewered. Constructed in 1956, the sewerage facilities consist of 25.1 miles of 6-inch to 16-inch sewers and 8 pumping stations, and serve a connected population of over 800 (Fig. 6-2 and Table 6-9). The principal intercepting sewer is submerged in Lake Washington and extends along the entire shoreline of the district. Because it was impractical to lay this interceptor to grade, it was constructed as a force main and several flushing stations were provided to purge it with lake water for removal of settled solids.

It was planned originally to construct a sewage treatment plant and outfall at the north end of Mercer Island. This idea was abandoned, however, and a siphon was laid under the east channel of the lake to convey sewage to the mainland for treatment at the Bellevue plant.

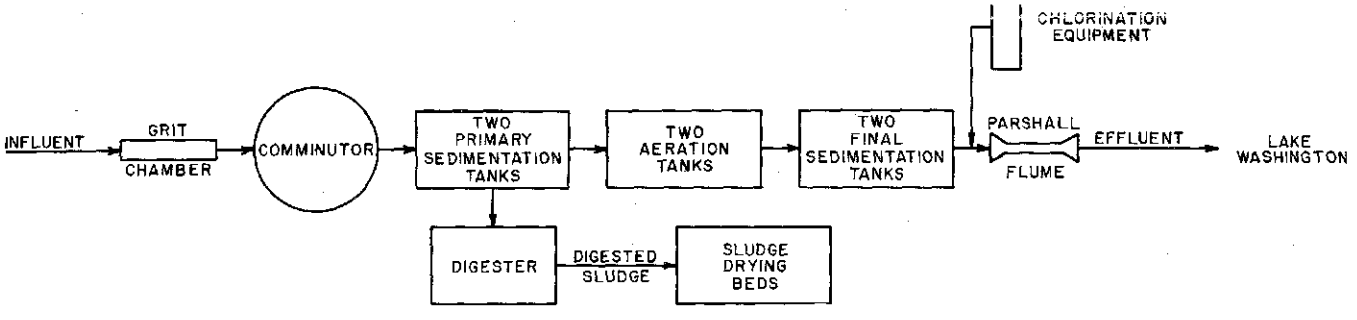


Fig. 6-3. Flow Diagram - Bellevue Sewage Treatment Plant

Complete treatment is provided for sewage from the Bellevue and Mercer Island Sewer districts in this activated sludge plant, the average daily capacity of which is presently being increased from 0.56 mgd to 2.0 mgd by the addition of temporary facilities.

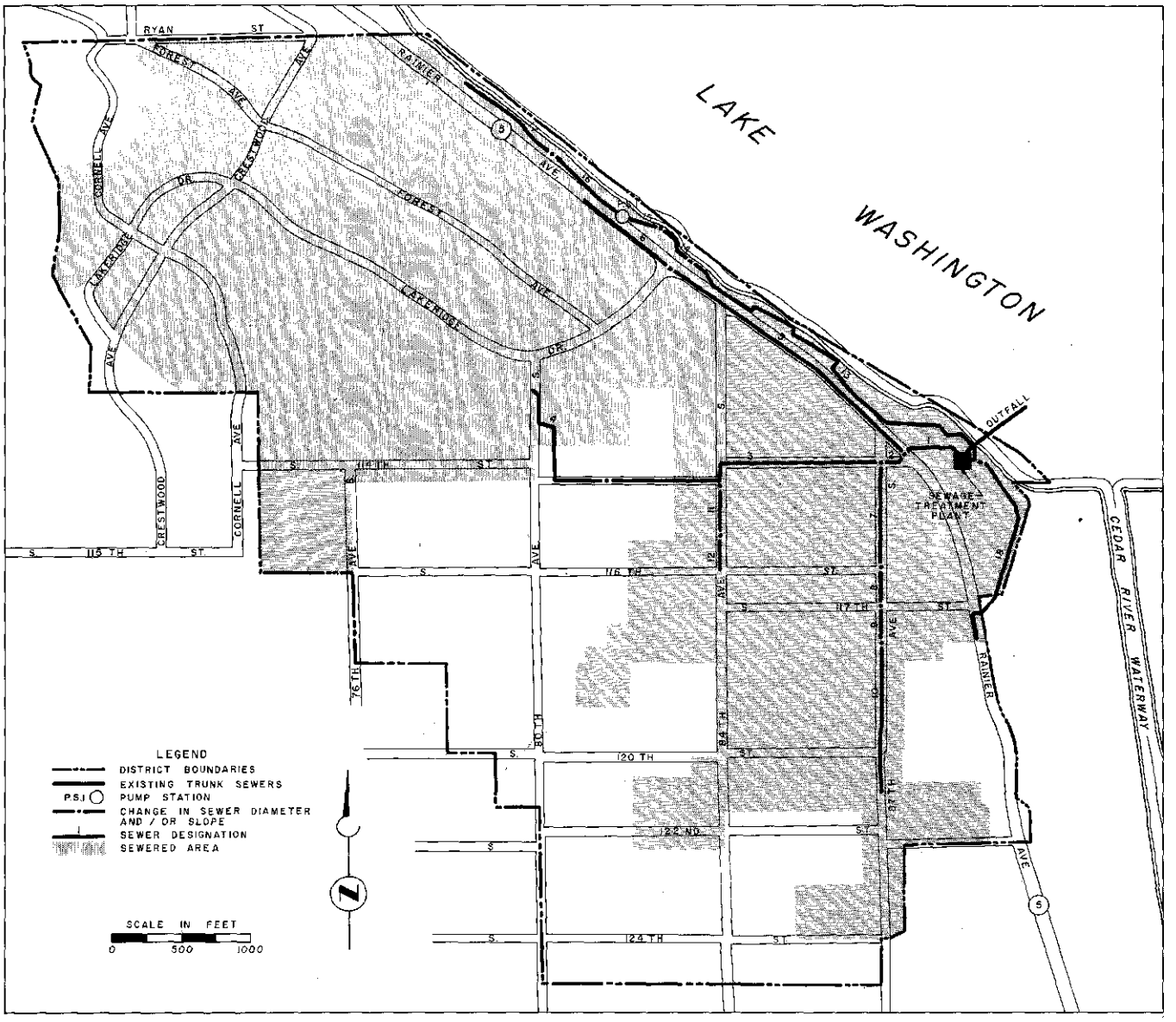


Fig. 6-4. Sewered Area and Principal Sewerage Facilities
Bryn Mawr - Lake Ridge Sewer District

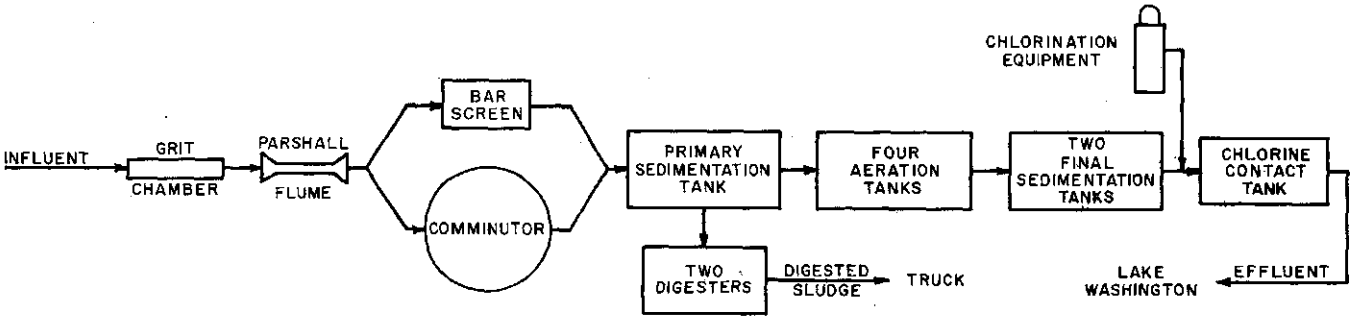


Fig. 6-5. Flow Diagram — Bryn Mawr - Lake Ridge Sewage Treatment Plant

Designed for an average daily capacity of 0.43 mgd, this activated sludge plant provides complete treatment for sewage of the Bryn Mawr - Lake Ridge District and, in addition, serves part of the Boeing plant at Renton.

Table 6-6. Description of Principal Sewerage Facilities, Bryn Mawr - Lake Ridge Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	480 ft of 18-in. PS and conc at 0.31%	3.8
2	190 ft of 10-in. PS at 9.62%	4.4
3	2,060 ft of 8-in. PS at 1.79%	1.1
4	1,140 ft of 8-in. and 10-in. PS at 0.30%	0.8
5	2,190 ft of 12-in. PS at 0.30%	1.3
6	800 ft of 10-in. PS at 0.40%	0.9
7	800 ft of 10-in. PS at 5.93%	3.5
8	260 ft of 10-in. PS at 3.33%	2.6
9	260 ft of 10-in. PS at 0.77%	1.3
10	1,030 ft of 10-in. PS at 0.26%	0.7
11	410 ft of 8-in. PS at 5.13%	1.8
12	220 ft of 8-in. PS at 1.08%	0.8
13	2,760 ft of 12-in. CI and PS at 0.20%	1.0
14	410 ft of 12-in. PS at 0.10%	0.7
15	310 ft of 10-in. CI force main	
16	570 ft of 10-in. CI and conc at 0.23%	0.7
17	810 ft of 10-in. conc at 0.45%	1.0
18	1,560 ft of 8-in. PS at 0.35%	0.5
Outfall	Land section, 50 ft of 12-in. CI; submerged section, 470 ft of 12-in. CI; discharge to Lake Washington	

[a]*See Fig. 6-4 for location.*

[b]*Where size and slope varies within section, limiting or average slope is shown. PS signifies pipe sewer (local designation); CI, cast iron; conc, concrete.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

Roxbury Heights Sewer District

Roxbury Heights Sewer District, formed in 1944, was the first district created in the state under the existing sewer district law. It occupies an area of 0.75 square miles, of which about 0.4 square miles is sewered (Fig. 6-9 and Table 6-10).

Table 6-7. Description of Principal Sewerage Facilities, East Mercer Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	540 ft of 12-in. PS	
2	750 ft of 10-in. PS	
3	1,620 ft of 8-in. PS	
4	1,090 ft of 8-in. PS	
Outfall	Land section, 330 ft of 12-in. PS; submerged section, 1,020 ft of 12-in. CI; discharge to Lake Washington	

[a]*See Fig. 6-2 for location.*

[b]*Slopes unknown; no plans available. PS signifies pipe sewer (local designation); CI, cast iron.*

[c]*Capacity unknown; see footnote b.*

All of the sewered portion of the district was annexed to the city of Seattle in 1956. Since January 1, 1957, the collection system and treatment plant (Fig. 6-10) have been operated by city forces under an informal agreement. A plan is now being formulated, however, whereby the city would continue to operate the collection system but the treatment plant would be abandoned and the sewage conveyed to the Southwest Suburban plant for treatment.

Southwest Suburban Sewer District

Formerly called the White Center Sewer District, this agency was created in 1945 to serve a federal housing project. Subsequent to the war, a large surrounding area was annexed to the original district and the present name was adopted.

As now constituted, the Southwest Suburban District has an area of 6.8 square miles, about one-half sewered, and a connected population of 12,600. Although the initial system for the housing project was constructed in 1942, most of the district's 54 miles of 8-inch to 36-inch sewers and 5 pumping stations

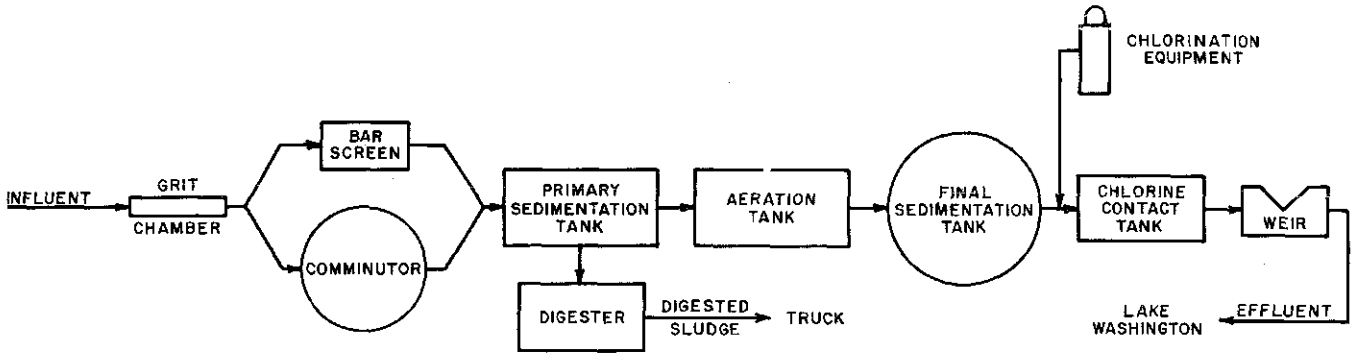


Fig. 6-6. Flow Diagram - East Mercer Sewage Treatment Plant

Activated sludge treatment is provided by the East Mercer Sewer District in this plant which has design capacity for an average daily flow of 0.13 mgd.

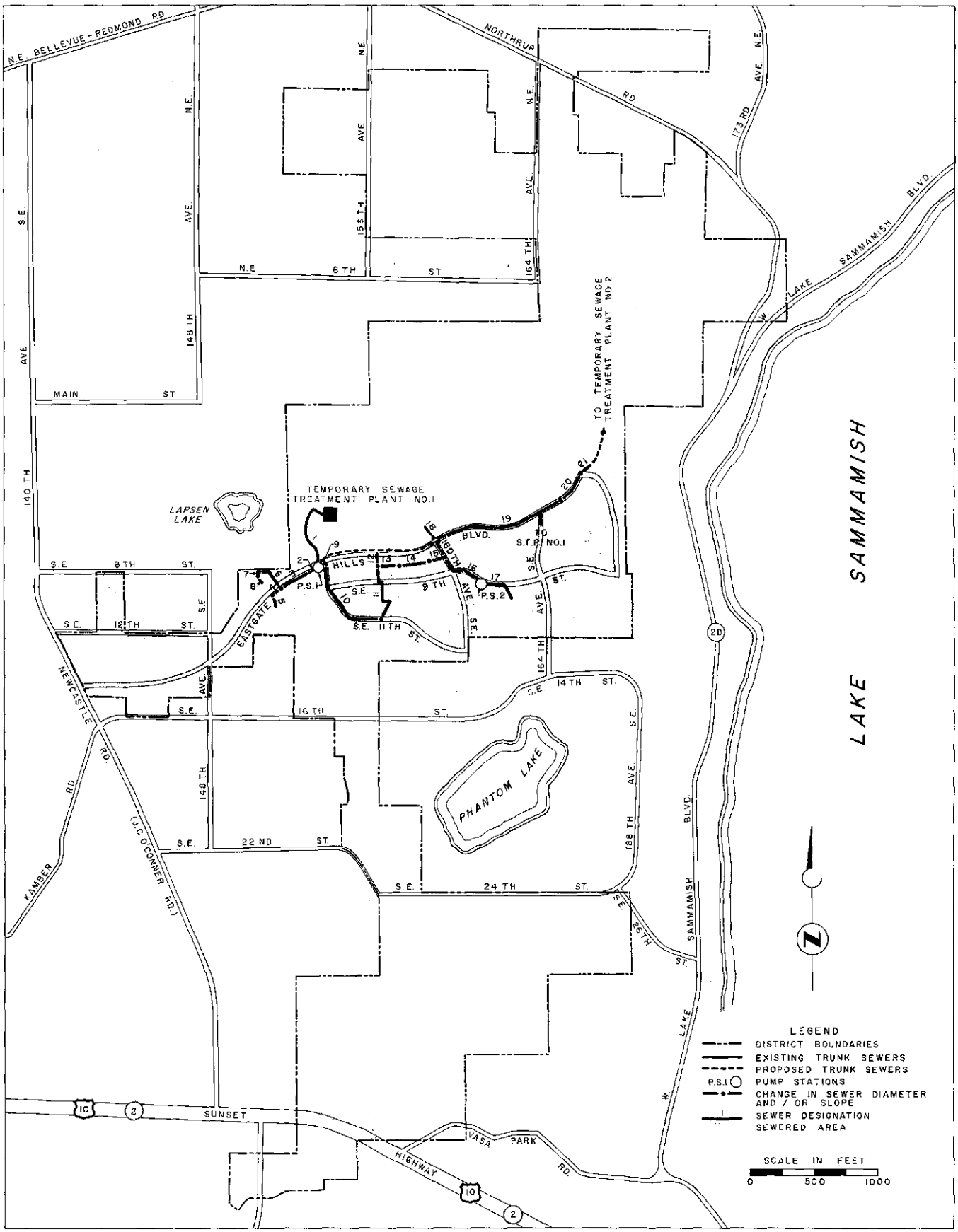


Fig. 6-7. Sewered Area and Principal Sewerage Facilities, Lake Hills Sewer District

Table 6-8. Description of Principal Sewerage Facilities, Lake Hills Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	1,140 ft of 6-in. aluminum pressure line	
2	210 ft of 12-in. conc at 1.17%	2.5
3	510 ft of 12-in. St at 1.17%	2.5
4	170 ft of 12-in. conc at 1.17%	2.5
5	230 ft of 10-in. conc at 0.28	0.8
6	480 ft of 12-in. conc at 0.40%	1.4
7	70 ft of 8-in. conc at 5.22%	1.8
8	170 ft of 8-in. conc at 1.58%	1.0
9	40 ft of 12-in. VC at 1.02%	2.3
10	1,480 ft of 12-in. VC at 0.26%	1.2
11	1,050 ft of 12-in. VC at 0.25%	1.1
12	200 ft of 10-in. VC at 0.42%	0.9
13	320 ft of 8-in. VC at 3.66%	1.5
14	440 ft of 8-in. VC at 5.98%	1.9
15	280 ft of 8-in. VC at 1.36%	0.9
16	830 ft of 6-in. CA force main	
17	600 ft of 10-in VC at 0.28%	0.8
18	230 ft of 15-in. conc at 0.46%	2.8
19	2,120 ft of 18-in. conc at 0.40%	4.3
20	510 ft of 18-in. conc at 0.40%	4.3
21	160 ft of 18-in. conc at 0.50%	4.8

[a]*See Fig. 6-7 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. Conc signifies concrete pipe; St, steel; VC, vitrified clay; CA, cement-asbestos.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

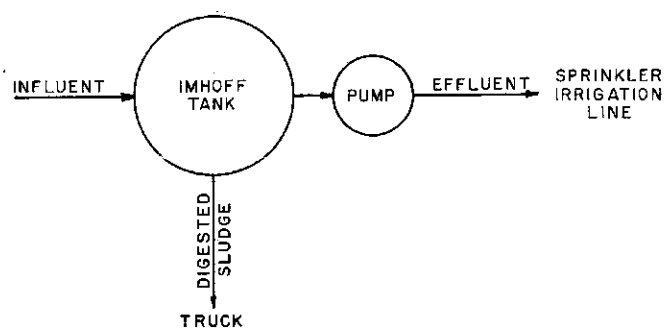


Fig. 6-8. Flow Diagram - Lake Hills Sewage Treatment Plant

Designed to serve temporarily pending full development of the district, an Imhoff tank of wood construction provides primary treatment capacity for an average daily flow of 0.16 mgd. Effluent is sprinkled over the ground surface and absorbed by the soil. An additional Imhoff tank, not shown here, was added during the course of the survey, doubling the plant capacity to 0.32 mgd.

Table 6-9. Description of Principal Sewerage Facilities, Mercer Island Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	410 ft of 12-in. CA at 3.37%	4.2
2	2,020 ft of 10-in. and 15-in. conc gravity and pressure line at 1.16%	4.4
3	7,070 ft of 8-in. and 10-in. CA force main	
4	380 ft of 16-in. CA force main	
5	1,240 ft of 16-in. CI force main	
6	790 ft of 16-in. CA force main	
7	1,980 ft of 18-in. conc at 0.22%	3.2
8	530 ft of 12-in. conc at 0.30%	1.3
9	6,350 ft of 12-in. CA force main	
10	810 ft of 12-in. conc at 0.22%	1.1
11	750 ft of 10-in. conc at 0.30%	0.8
12	2,650 ft of 10-in. CA force main	
13	400 ft of 12-in. CA force main	
14	1,300 ft of 10-in. CA submerged pressure line	
15	2,850 ft of 12-in. CA submerged pressure line	
16	2,730 ft of 12-in. CA submerged pressure line	
17	4,180 ft of 10-in. CA submerged pressure line	
18	340 ft of 10-in. conc at 2.33%	2.4

[a]*See Fig. 6-2 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. CA signifies cement-asbestos pipe; conc, concrete; CI, cast iron.*

[c]*Capacities calculated using Manning's formula, "n" equals 0.013.*

were constructed during the past several years (Fig. 6-9 and Table 6-11). An outstanding feature of the collection system is that all the new sewers, including house connections, are equipped with rubber gasket pipe joints to reduce infiltration of ground water. The original housing project, however, has combined sewers which must be separated to obtain maximum benefits from the new system. A separation program is planned for early construction.

A primary type sewage treatment plant was completed in 1957 (Fig. 6-11). With a design capacity of 8.2 mgd, this plant is the largest now operating in the metropolitan area and is capable of serving the entire Salmon Creek basin as well as adjacent areas along the shore of Puget Sound. Effluent is chlorinated and discharged to Puget Sound.

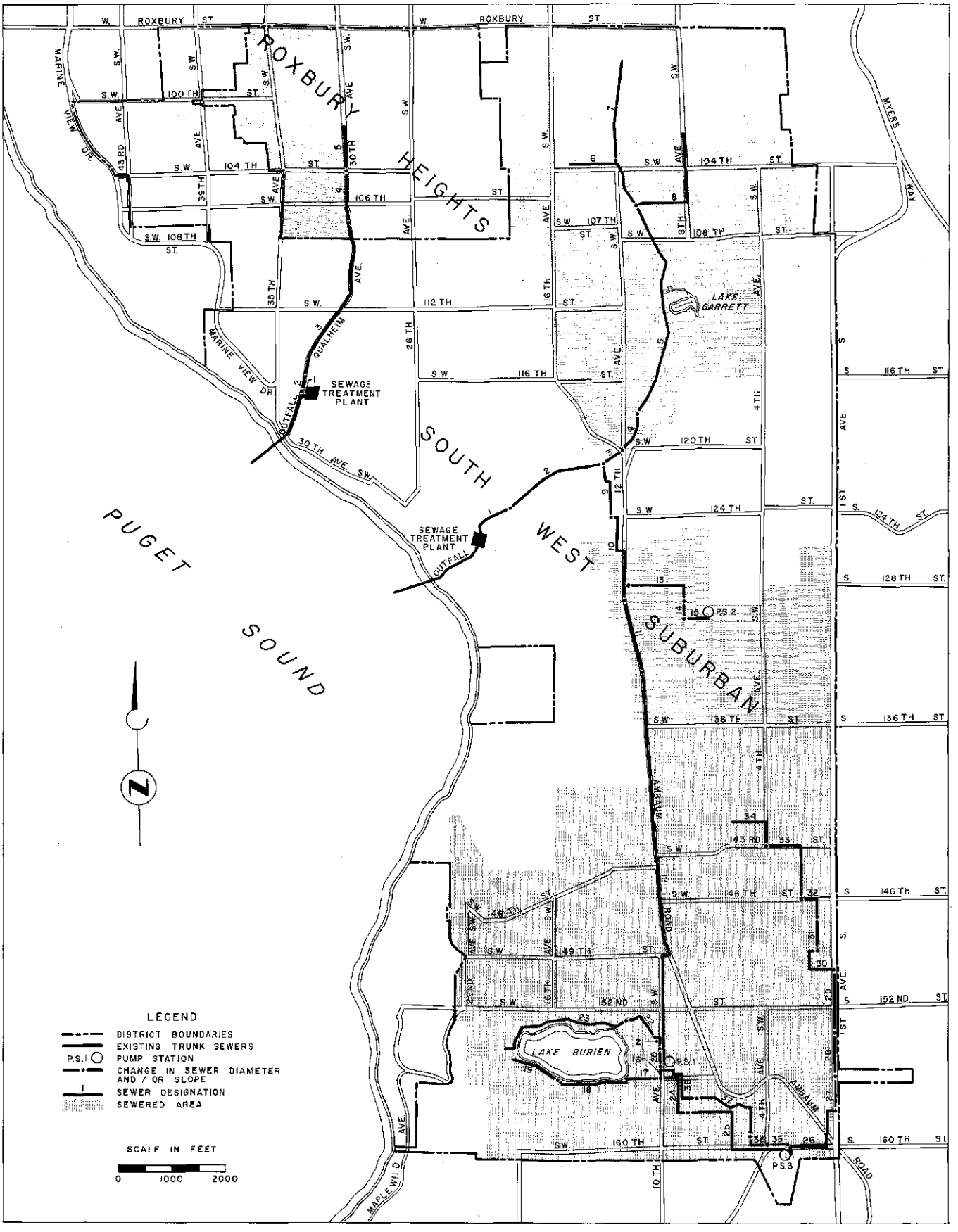


Fig. 6-9. Sewered Areas and Principal Sewerage Facilities, Southwest Suburban and Roxbury Heights Sewer District

Table 6-10. Description of Principal Sewerage Facilities, Roxbury Heights Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	60 ft of 15-in. conc at 0.2%	1.8
2	280 ft of 15-in. conc at 1.7%	5.4
3	3,140 ft of 15-in. conc at 5.75%	9.0
4	1,000 ft of 18-in. conc at 0.5%	4.8
5	830 ft of 15-in. conc at 0.5%	2.9
Outfall	Land section, 900 ft of 15-in. conc; submerged section, 880 ft of 14-in. CI	

[a]*See Fig. 6-9 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. Conc signifies concrete pipe; CI, cast iron.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

Val-Vue Sewer District

The Val-Vue Sewer District, one of the smallest operating districts, was formed in 1945 and covers an area of only 0.37 square miles. Approximately half of the district is served by 3.2 miles of 6-inch to 15-inch sewers (Fig. 6-12 and Table 6-12), and by a primary type treatment plant with a design capacity of 0.28 mgd (Fig. 6-13). Effluent is chlorinated prior to discharge to the Duwamish River.

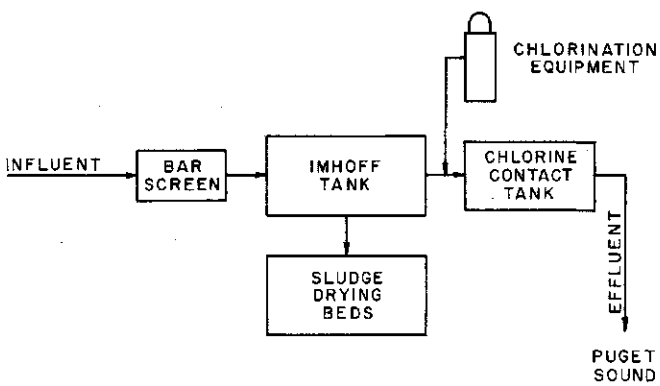


Fig. 6-10. Roxbury Heights Sewage Treatment Plant

Since the annexation of sewered section of the Roxbury Heights Sewer District to the city of Seattle in 1956, this primary treatment plant with a design capacity of 0.15 mgd has been operated by city forces.

Footnotes for Table 6-11 →

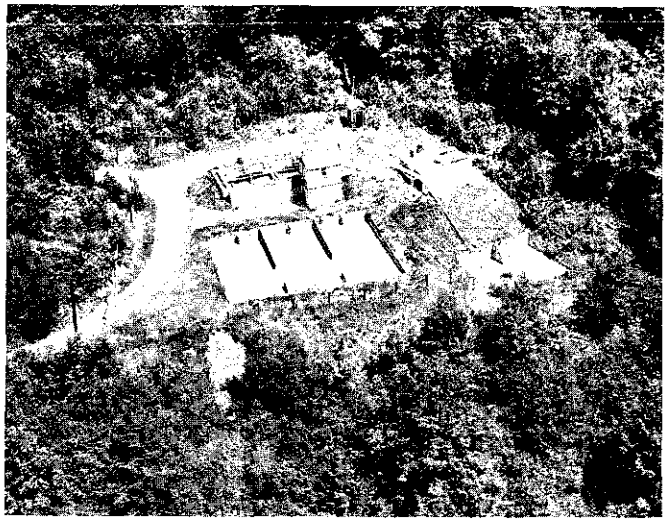
[a]*See Fig. 6-9 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. Conc signifies concrete pipe; CA, cement-asbestos; PS, pipe sewer (local designation); RC, reinforced concrete; St, steel.*

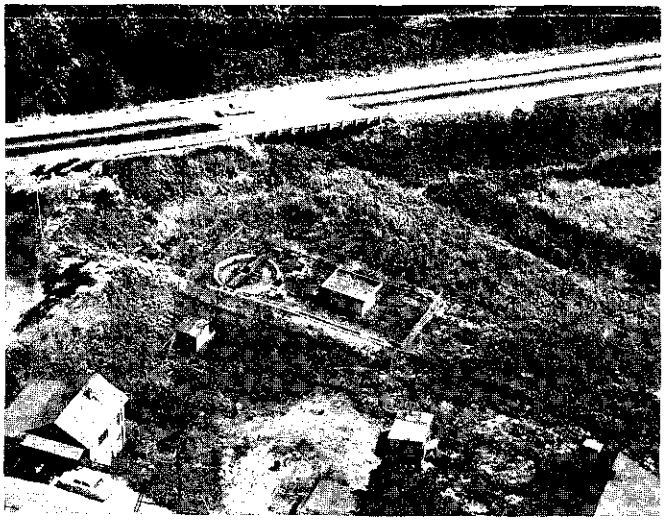
[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

Table 6-11. Description of Principal Sewerage Facilities, Southwest Suburban Sewer District

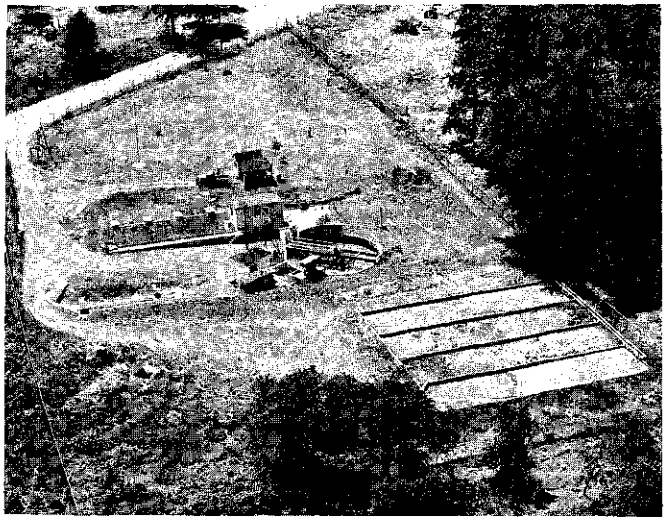
Facility[a]	Description[b]	Capacity,[c] mgd
1	790 ft of 36-in. conc at 1.53%	53
2	1,990 ft of 30-in. conc at 5.20%	60
3	460 ft of 18-in. conc at 6.30%	17
4	900 ft of 21-in. conc at 0.67%	8.3
5	4,240 ft of 24-in. conc at 0.20%	6.5
6	1,780 ft of 18-in. conc at 0.27%	3.5
7	1,920 ft of 12-in. conc at 0.30%	1.3
8	2,270 ft of 12-in. conc at 0.50%	1.6
9	1,080 ft of 24-in. conc at 5.00%	33
10	1,900 ft of 24-in. and 30-in. conc at 0.30%	14
11	1,360 ft of 18-in. conc at 3.68%	13
12	7,800 ft of 16-in. CA force main	
13	1,420 ft of 15-in. and 16-in. conc and CA at 0.33%	2.4
14	350 ft of 15-in. conc at 1.00%	4.2
15	450 ft of 8-in. CA force main	
16	280 ft of 24-in. conc at 0.51%	10
17	680 ft of 15-in. conc at 0.30%	2.3
18	1,320 ft of 14-in. CA at 0.31%	1.9
19	960 ft of 12-in. and 14-in. CA and PS at 0.30%	1.3
20	420 ft of 18-in. conc at 0.55%	5.1
21	320 ft of 18-in. conc at 0.88%	6.4
22	740 ft of 18-in. conc at 0.30%	3.8
23	2,590 ft of 12-in. conc and CA at 0.32%	1.3
24	1,130 ft of 18-in. conc at 0.53%	5.0
25	2,400 ft of 12-in. CA force main	
26	1,820 ft of 18-in. and 24-in. conc at 0.30%	8.0
27	650 ft of 18-in. conc at 1.50%	8.3
28	830 ft of 18-in. conc at 5.71%	16
29	1,150 ft of 18-in. conc at 0.62%	5.4
30	960 ft of 15-in. and 18-in. conc at 1.02%	4.2
31	650 ft of 15-in. conc at 1.95%	5.8
32	670 ft of 15-in. conc at 3.77%	8.0
33	1,680 ft of 15-in. conc at 0.34%	2.4
34	1,100 ft of 15-in. conc at 0.22%	1.9
35	330 ft of 24-in. conc at 0.96%	14
36	470 ft of 24-in. conc at 5.40% (24-in. overflow from this section to Miller Creek)	34
37	1,700 ft of 24-in. conc at 1.00%	15
38	1,030 ft of 24-in. conc at 0.40%	9.2
Outfall	Land section, 1,060 ft of 36-in. RC; submerged section, 640 ft of 36-in St; discharge to Puget Sound	



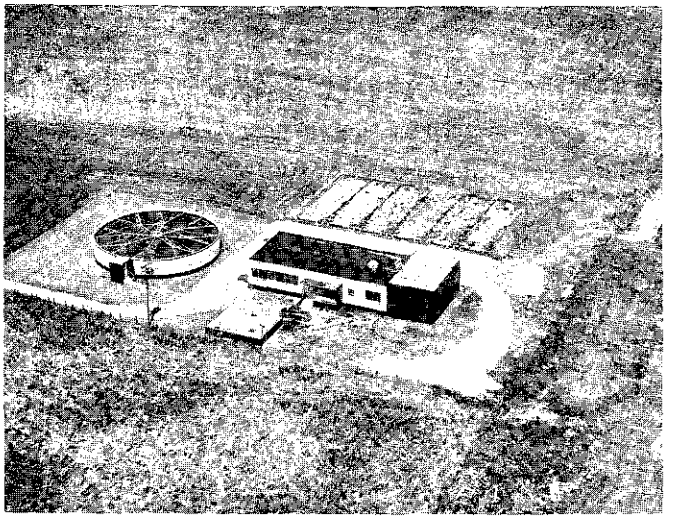
SEELYE SEWAGE TREATMENT PLANT, showing sludge drying beds in foreground, and Imhoff tanks at left and right of pump and chlorinator house.



VAL-VUE SEWAGE TREATMENT PLANT provides primary treatment by means of clarifier-digester unit at left. Steel pressure influent line enters inlet structures from right.



AUBURN SEWAGE TREATMENT PLANT, showing sludge drying beds in right foreground, inlet structures at left, circular primary clarifier and control building. Digesters are underground behind control building.



KENT SEWAGE TREATMENT PLANT provides primary treatment except during canning season. Sedimentation tanks are housed in building adjacent to digesters at right. Trickling filters are shown at left, sludge drying beds in rear.

GREEN-DUWAMISH VALLEY SEWAGE TREATMENT PLANTS

At present, the connected population totals 1,150. This number, however, will be increased substantially with the addition of facilities soon to be constructed for a new ULID.

Sewerage and Drainage Improvement District No. 3

Commonly known as Richmond Beach, this district serves its own half square mile area plus about 40 acres outside its boundaries, including the "Old Firlands" area. Sewerage facilities, originally constructed under a federal grant, have been extended periodically and now comprise about 7.2 miles of sewers ranging in size from 6-inch to 18-inch (Fig. 6-14 and Table 6-13). Parts of the system are utilized as combined sewers, a number of catch basins being connected. Originally, sewage was comminuted and chlorinated but is now discharged into Puget Sound without treatment.

Because the district was formed and continues to operate under drainage law, the King County Commissioners act as district commissioners and operation is supervised by the county engineer. In 1950, the residents of the area formed the Richmond Sewer District, but, to date, the district commissioners have not taken action to acquire and operate the system.

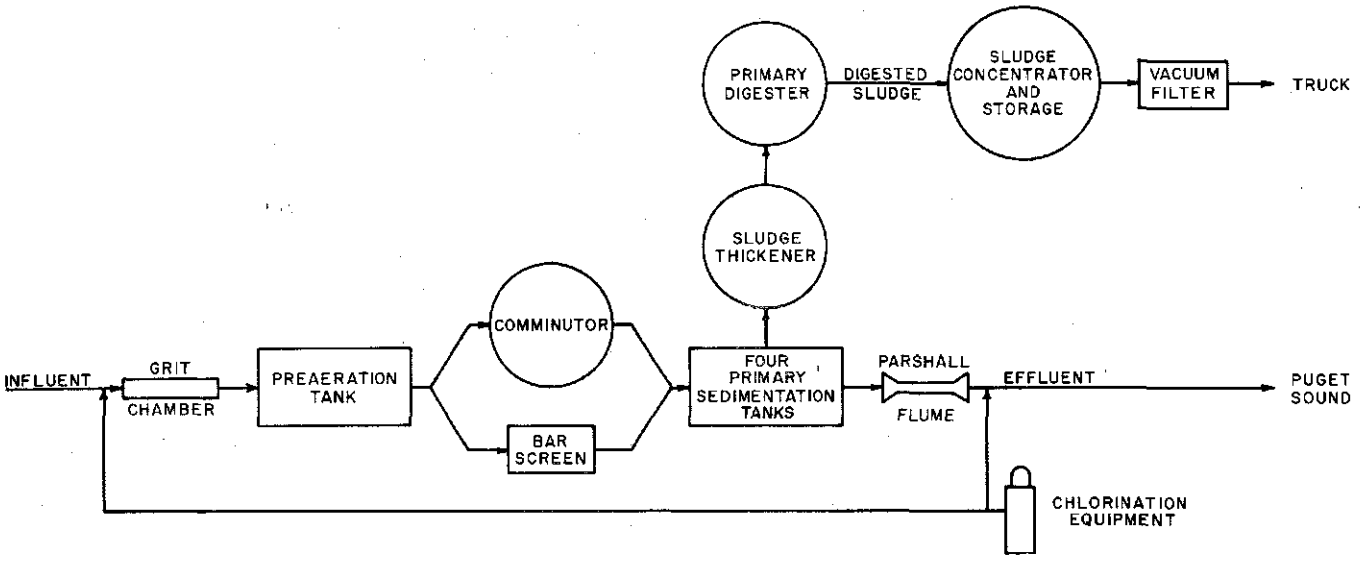


Fig. 6-11. Flow Diagram - Southwest Suburban Sewage Treatment Plant

This primary treatment plant with an average daily capacity of 8.2 mgd was completed in 1957. It is the largest treatment plant in the metropolitan area at the present time and has sufficient capacity to serve its natural tributary area when fully developed.

Sewerage and Drainage District No. 4

This district, usually referred to as Seelye, was formed in 1943 under drainage law and also is administered and operated by King County. It now comprises an area of 0.37 square miles, essentially all of which is sewered.

Initially constructed to serve a private housing development, the collection system has been extended and improved and now consists of 9.5 miles of 6-inch to 18-inch sewers, and 2 pumping stations (Fig. 6-15 and Table 6-14). Treatment is obtained in an Imhoff-tank type of primary treatment plant with a design capacity of 0.20 mgd (Fig. 6-16). Chlorinated effluent is discharged to the Duwamish River.

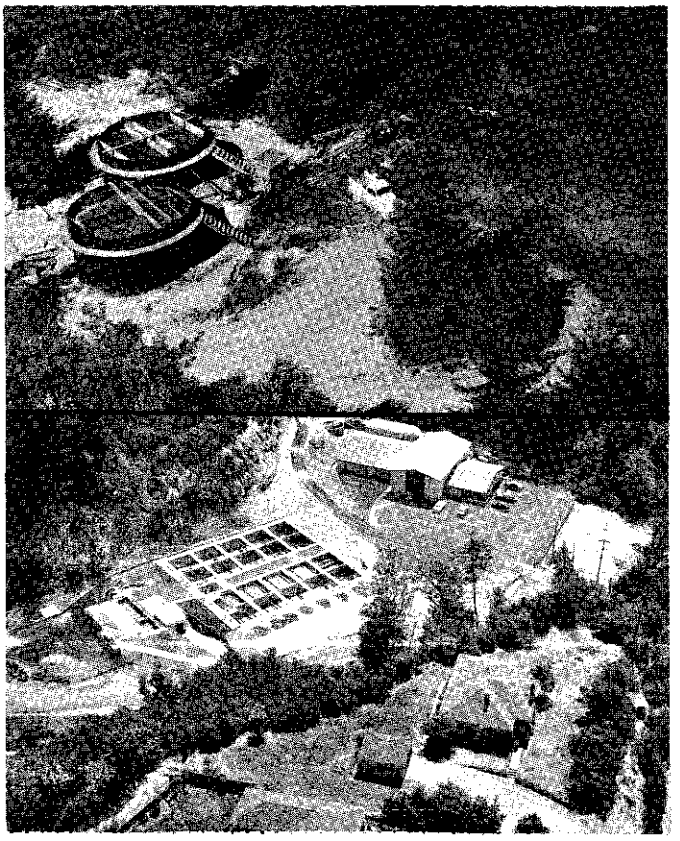
Table 6-12. Description of Principal Sewerage Facilities, Val-Vue Sewer District

Facility[a]	Description[b]	Capacity,[c] mgd
1	370 ft of 12-in. St pressure line	
2	1,030 ft of 15-in. conc at 4.00%	8.3
3	1,410 ft of 12-in. conc at 5.80%	5.6
4	3,070 ft of 12-in. conc at 2.80%	3.8
Outfall	Land section, 470 ft of 24-in. conc; submerged section, 70 ft of 24-in. conc; discharge to Duwamish River	

[a]*See Fig. 6-12 for location.*

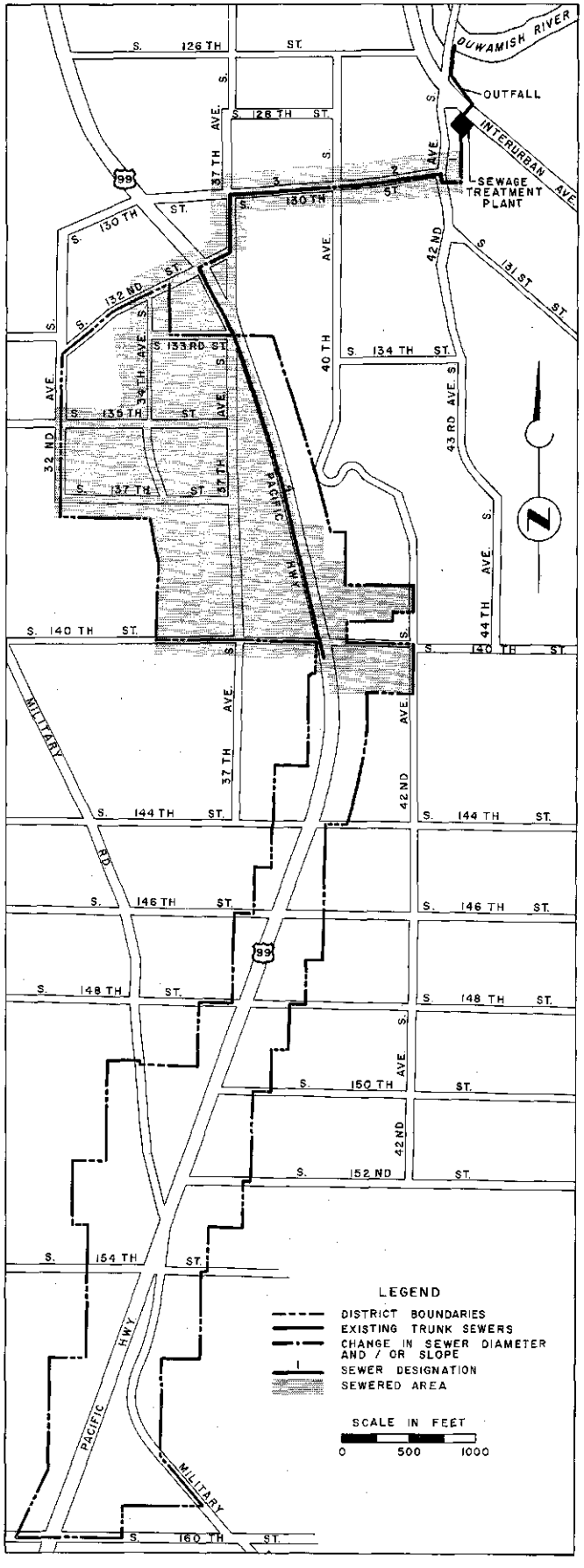
[b]*Where slope varies within section, limiting or average slope is shown. St signifies steel pipe; conc, concrete pipe.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*



(Above) LAKE HILLS treatment plant utilizes temporary wooden Imhoff tanks for primary treatment pending full development of the area. Effluent is sprayed over the surrounding wooded area.

(Below) SOUTHWEST SUBURBAN treatment plant provides primary treatment and has capacity to serve the entire Salmon Creek basin. Inlet and grit removal structures and primary sedimentation tanks, foreground; building at rear houses digesters, metering, pumping, sludge conditioning equipment, maintenance shop, office and laboratory. Note proximity of residences on surrounding bluffs.



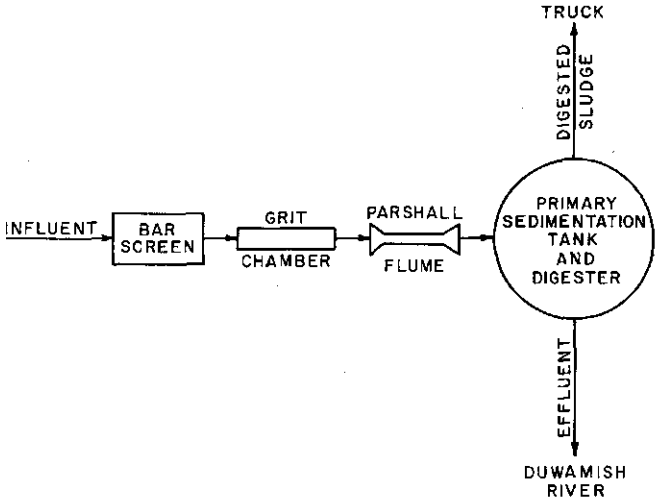


Fig. 6-13. Flow Diagram – Val-Vue Sewage Treatment Plant

The Val-Vue treatment plant, designed for an average flow of 0.28 mgd, provides primary treatment for the Val-Vue Sewer District situated adjacent to Highway 99 a short distance west of Tukwila.

Table 6-13. Description of Principal Sewerage Facilities, Sewerage and Drainage Improvement District No. 3

Facility[a]	Description[b]	Capacity,[c] mgd
1	1,310 ft of 15-in. PS at 1.00%	4.2
2	900 ft of 8-in. PS at 8.54%	2.3
3	570 ft of 15-in. PS at 3.88% (section overflows to 21-in. storm overflow to Puget Sound)	8.1
4	200 ft of 15-in. PS at 6.81%	11
5	460 ft of 12-in. PS at 2.81%	3.8
6	830 ft of 15-in. PS at 8.49%	12
7	30 ft of 18-in. PS at 0.49%	4.8
8	260 ft of 18-in. PS at 0.29%	3.6
9	320 ft of 15-in. PS at 0.71%	3.5
10	340 ft of 12-in. PS at 0.03%	0.4
11	320 ft of 10-in. PS at 0.03%	0.3
12	10,160 ft of 8-in. PS, slope unknown	
Outfall	Length, size and material unknown; discharge to Puget Sound	

All combined sewers except No. 12, which is separate sanitary.

[a]*See Fig. 6-14 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. PS signifies pipe sewer (local designation).*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

Fig. 6-12. Sewered Area and Principal Sewerage Facilities Val-Vue Sewer District

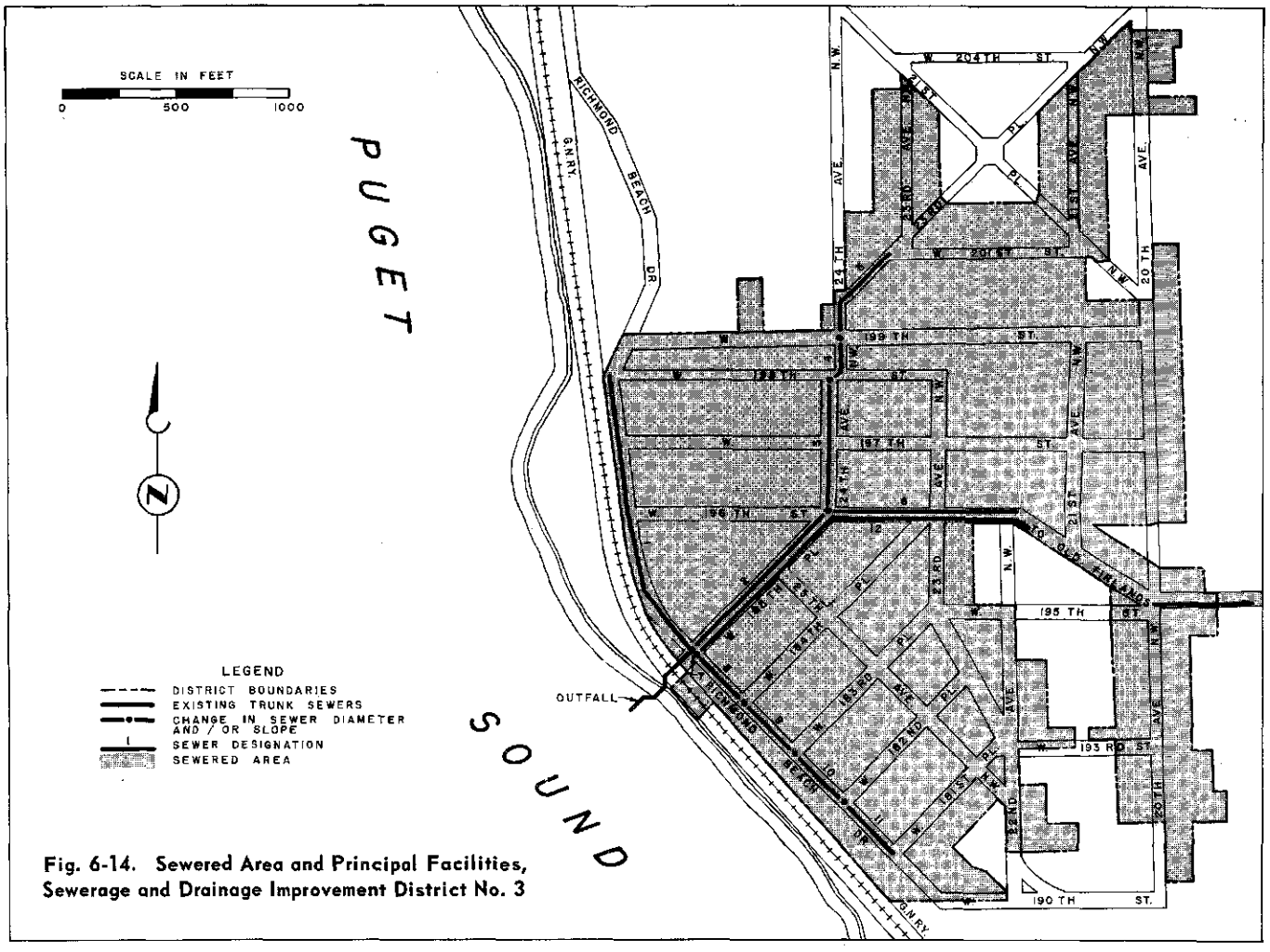


Fig. 6-14. Sewered Area and Principal Facilities, Sewerage and Drainage Improvement District No. 3

Table 6-14. Description of Principal Sewerage Facilities, Sewerage and Drainage District No. 4

Facility[a]	Description[b]	Capacity,[c] mgd	Facility[a]	Description[b]	Capacity,[c] mgd
1	160 ft of 8-in. pipe at 37.60%	4.8	14	370 ft of 8-in. pipe at 3.13%	1.4
2	390 ft of 8-in. pipe at 0.63%	0.6	15	340 ft of 8-in. pipe at 3.23%	1.4
3	150 ft of 8-in. pipe at 4.13%	1.6	16	230 ft of 8-in. pipe at 0.50%	0.6
4	300 ft of 8-in. pipe at 4.64%	1.7	17	530 ft of 8-in. pipe at 0.63%	0.6
5	160 ft of 8-in. pipe at 5.72%	1.9	18	270 ft of 8-in. pipe at 1.33%	0.9
6	190 ft of 8-in. pipe at 10.32%	2.8	19	300 ft of 8-in. pipe at 4.40%	1.6
7	240 ft of 8-in. pipe at 6.92%	2.1	20	190 ft of 8-in. pipe at 5.44%	1.8
8	220 ft of 8-in. pipe at 2.70%	1.3	21	230 ft of 8-in. pipe at 8.90%	2.4
9	360 ft of 8-in. pipe at 0.63%	0.6	22	2,790 ft of 6-in. CA force main	
10	400 ft of 8-in. pipe at 1.90%	1.1	23	140 ft of 15-in. conc at 0.56%	3.1
11	280 ft of 8-in. pipe at 0.81%	0.7	24	500 ft of 12-in. PS at 0.50%	1.6
12	190 ft of 8-in. pipe at 3.12%	1.4	25	840 ft of 8-in. PS at 3.20%	1.4
13	190 ft of 8-in. pipe at 9.00%	2.4	Outfall	Length unknown, 8-in. conc; discharge to Duwamish River	

[a]*See Fig. 6-15 for location.*
[b]*Where slope varies within section, limiting or average slope is shown. Except as indicated, material unknown; conc signifies concrete pipe; PS, pipe sewer (local designation); CA, cement-asbestos.*
[c]*Capacity calculated using Mannings formula, "n" equals 0.013.*

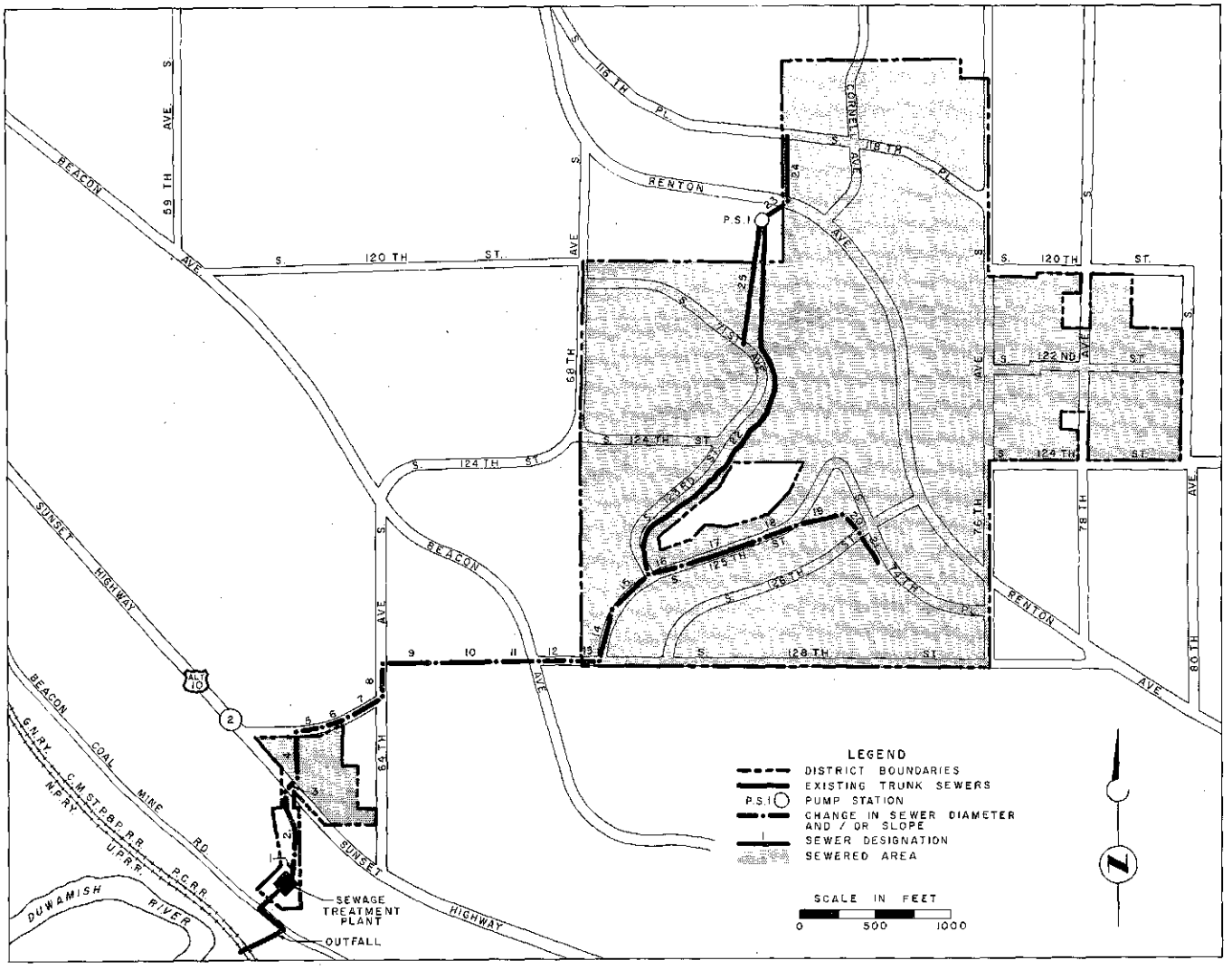


Fig. 6-15. Sewered Area and Principal Sewerage Facilities Sewerage and Drainage District No. 4

City of Auburn

Auburn was incorporated in 1891 and now occupies an area of 2.1 square miles, almost entirely sewered. Of its present population of 8,000, about 7,000 are connected to the sewerage system. The first sewers were constructed in 1910 and were designed to collect sanitary sewage only. Subsequently, however, combined sewers were used in newer sections of the city and served until 1950 when they were replaced by separate sanitary sewers (Fig. 6-17 and Table 6-15).

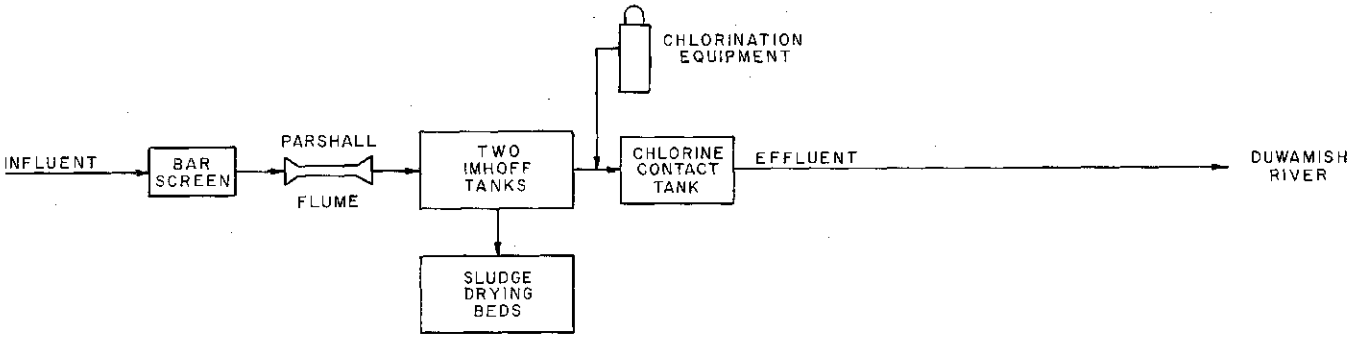


Fig. 6-16. Flow Diagram - Sewerage and Drainage District No. 4 Sewage Treatment Plant

Primary treatment for the small community of Seelye is provided by two Imhoff tanks with a combined average daily capacity of 0.20 mgd. The plant and sewer system are operated by Sewerage and Drainage District No. 4.

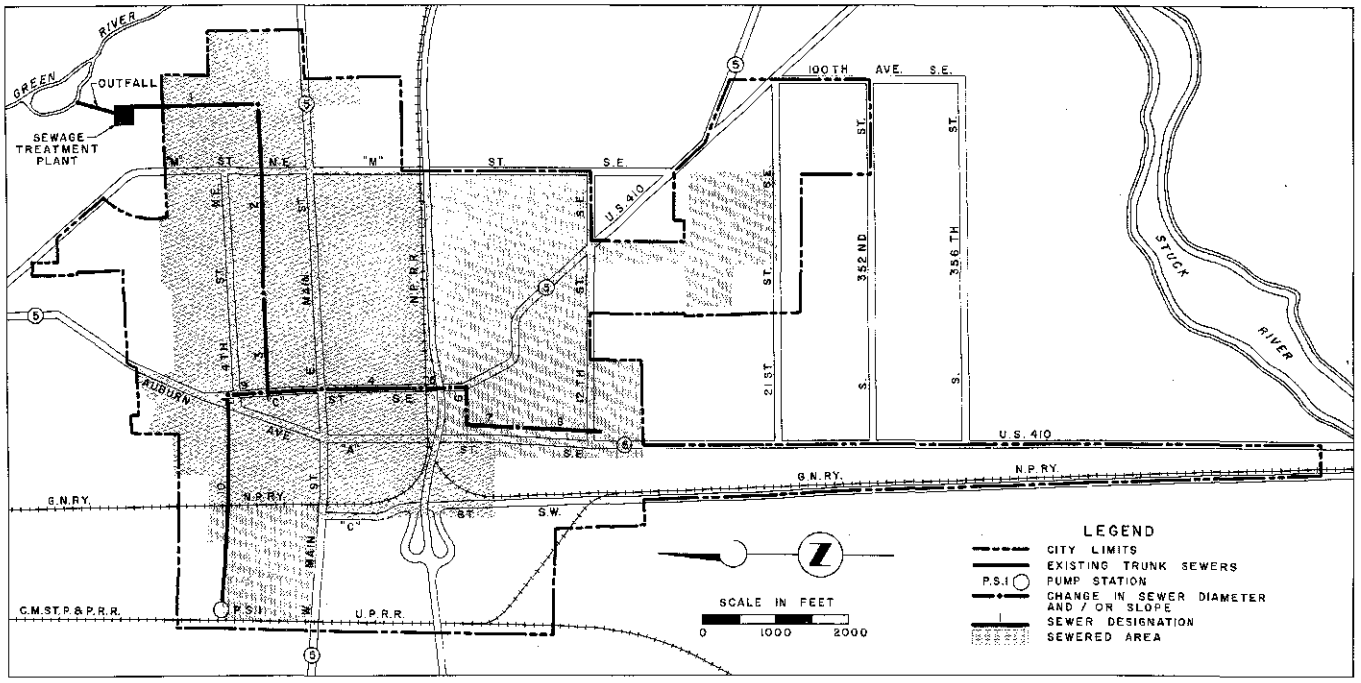


Fig. 6-17. Sewered Area and Principal Sewerage Facilities, City of Auburn

Septic tanks were the sole means of sewage treatment prior to 1950. In that year, a primary sedimentation tank with a design capacity of 1.1 mgd was constructed, and the septic tanks were converted to sludge digestion and sludge storage units (Fig. 6-18). Chlorinated effluent from the treatment plants is discharged to the Green River. Effluent pumping is necessary during high stages of the river.

City of Issaquah

Incorporated in 1913, the city of Issaquah now comprises an area of about 0.7 square miles, one-half of which is sewered. Essentially all of its population, which was 1,196 in 1957, are connected to the sewer system.

Sewerage construction began in 1940. At present, the system consists of 13 miles of sewers ranging in size from 8 inches to 15 inches (Fig. 6-19 and Table 6-16), and a secondary type sewage treatment plant with a design capacity of 0.23 mgd (Fig. 6-20). Plant effluent is chlorinated and discharged to Issaquah Creek.

City of Kent

Incorporated in 1890, Kent is the second oldest city in the metropolitan area. It has a present area of 3 square miles and a population of 4,150,

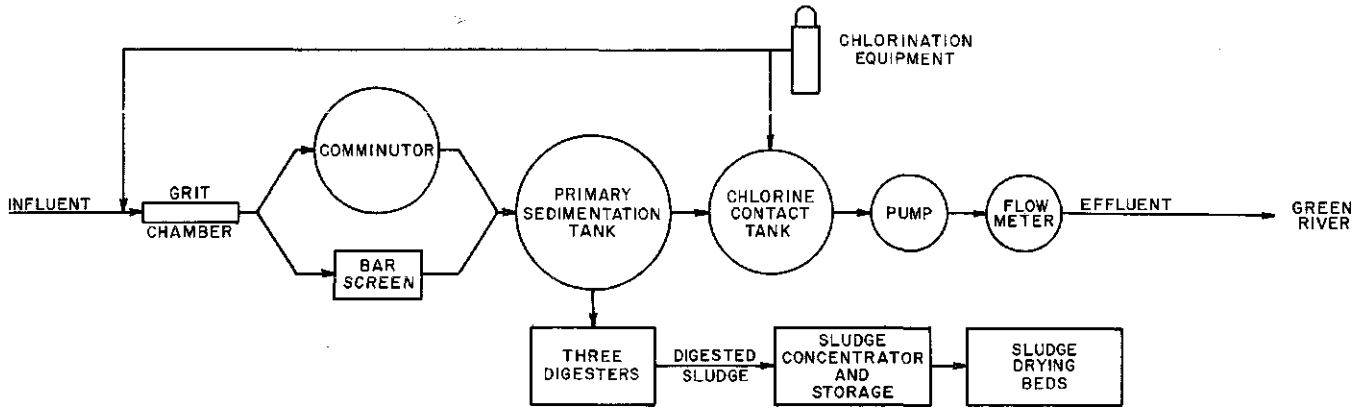


Fig. 6-18. Flow Diagram - Auburn Sewage Treatment Plant

The city of Auburn provides primary treatment in this plant which employs a circular clarifier having capacity for an average daily flow of 1.1 mgd. The sludge digester and storage units were initially designed as septic tanks but were converted when the plant was enlarged in 1950.

Table 6-15. Description of Principal Sewerage Facilities, City of Auburn

Facility[a]	Description[b]	Capacity,[c] mgd
1	1,570 ft of 30-in. pipe at 0.06%	6.5
2	2,570 ft of 24-in. pipe at 0.17%	6.0
3	2,670 ft of 30-in. pipe at 0.13%	9.5
4	2,360 ft of 24-in. pipe at 0.24%	7.2
5	420 ft of 12-in. PS at 0.25%	1.1
6	600 ft of 12-in. PS at 0.50%	1.6
7	690 ft of 18-in. pipe at 0.30%	3.7
8	1,160 ft of 15-in. pipe at 0.30%	2.3
9	560 ft of 18-in. PS at 0.17%	2.8
10	3,000 ft of 12-in. CI force main	
Outfall	Length, size and material unknown; discharge to Green River.	

[a]*See Fig. 6-17 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. Except as indicated, material unknown; PS signifies pipe sewer (local designation); CI, cast iron.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

of which about 4,000 are connected to the sewerage system.

Kent has approximately 10 miles of 6-inch to 24-inch sewers (Fig. 6-21 and Table 6-17), one pumping station, and a sewage treatment plant with a design capacity of 1.7 mgd. Treatment normally consists of primary sedimentation and effluent chlorination (Fig. 6-22). In the canning season, however, a trickling filter equipped with a forced draft blower is used after primary sedimentation to accommodate the additional load from two canneries. Effluent containing filter humus is chlorinated and discharged to the Green River.

Table 6-16. Description of Principal Sewerage Facilities, City of Issaquah

Facility[a]	Description[b]	Capacity,[c] mgd
1	160 ft of 15-in. VC, slope unknown	
2	920 ft of 12-in. VC at 0.40%	1.4
3	1,770 ft of 10-in. VC at 0.40%	0.9
4	840 ft of 12-in. VC at 0.40%	1.4
5	1,010 ft of 12-in. VC at 0.55%	1.7
6	1,000 ft of 10-in. VC at 0.40%	0.9
Outfall	Length unknown, 15-in. VC; discharge to Issaquah Creek.	

[a]*See Fig. 6-19 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. VC signifies vitrified clay pipe.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

City of Kirkland

Incorporated in 1905, the city of Kirkland has an area of 2.0 square miles and had a population in 1955 of 5,750. Within the corporate limits, the present sewered area is about 1.5 square miles but the city also serves about 0.5 square miles of surrounding area, including the northerly part of the city of Houghton.

Sewers were first constructed in Kirkland in 1942. Today, the collection system serves a connected popu-

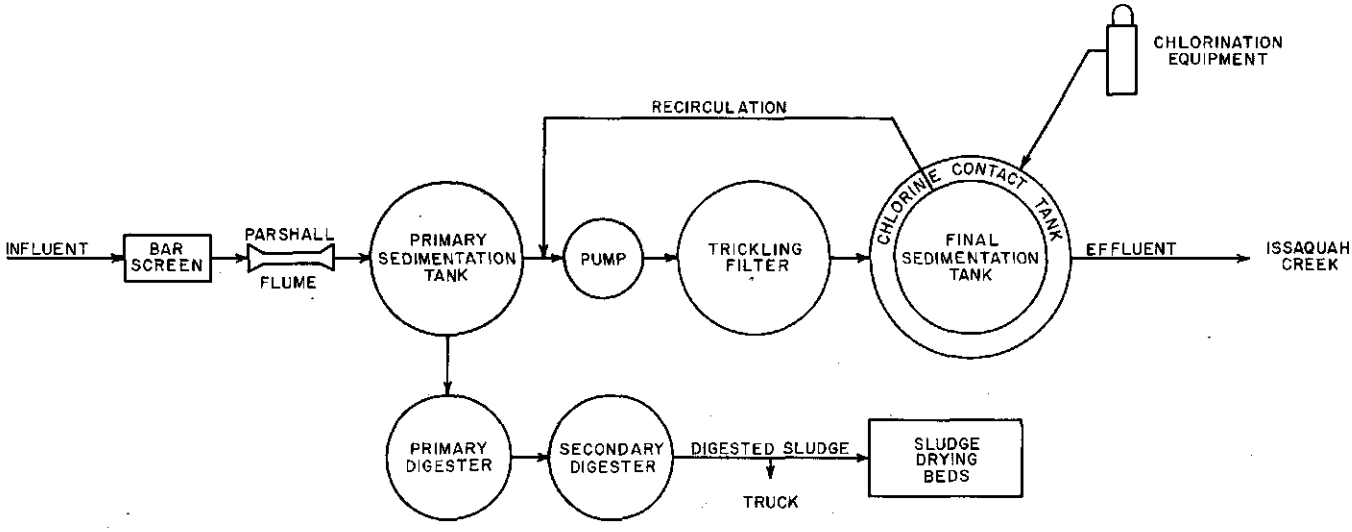


Fig. 6-20. Flow Diagram - Issaquah Sewage Treatment Plant

This sewage treatment plant with design capacity for an average daily flow of 0.23 mgd provides secondary treatment for sewage from the city of Issaquah.

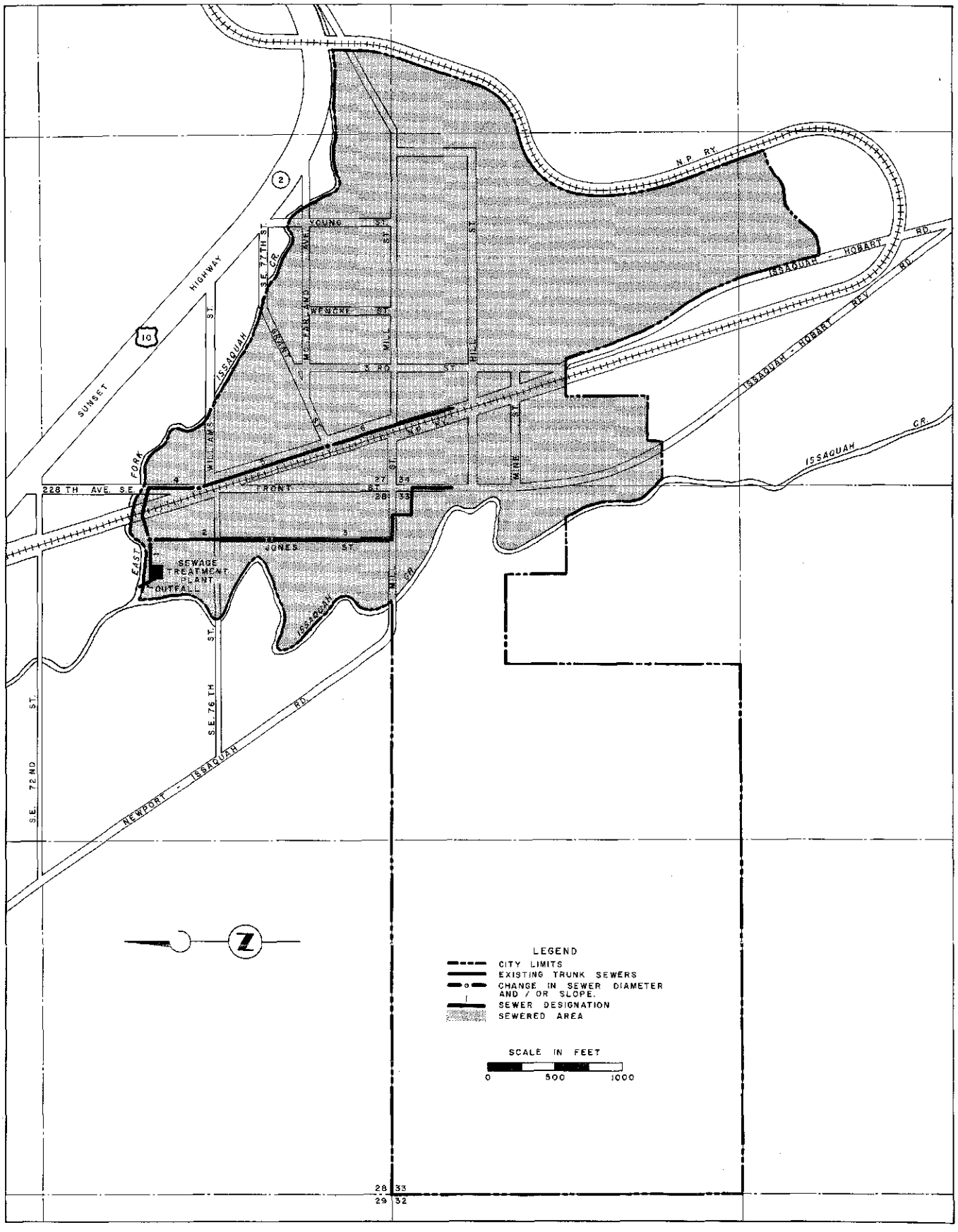


Fig. 6-19. Sewered Area and Principal Sewerage Facilities, City of Issaquah

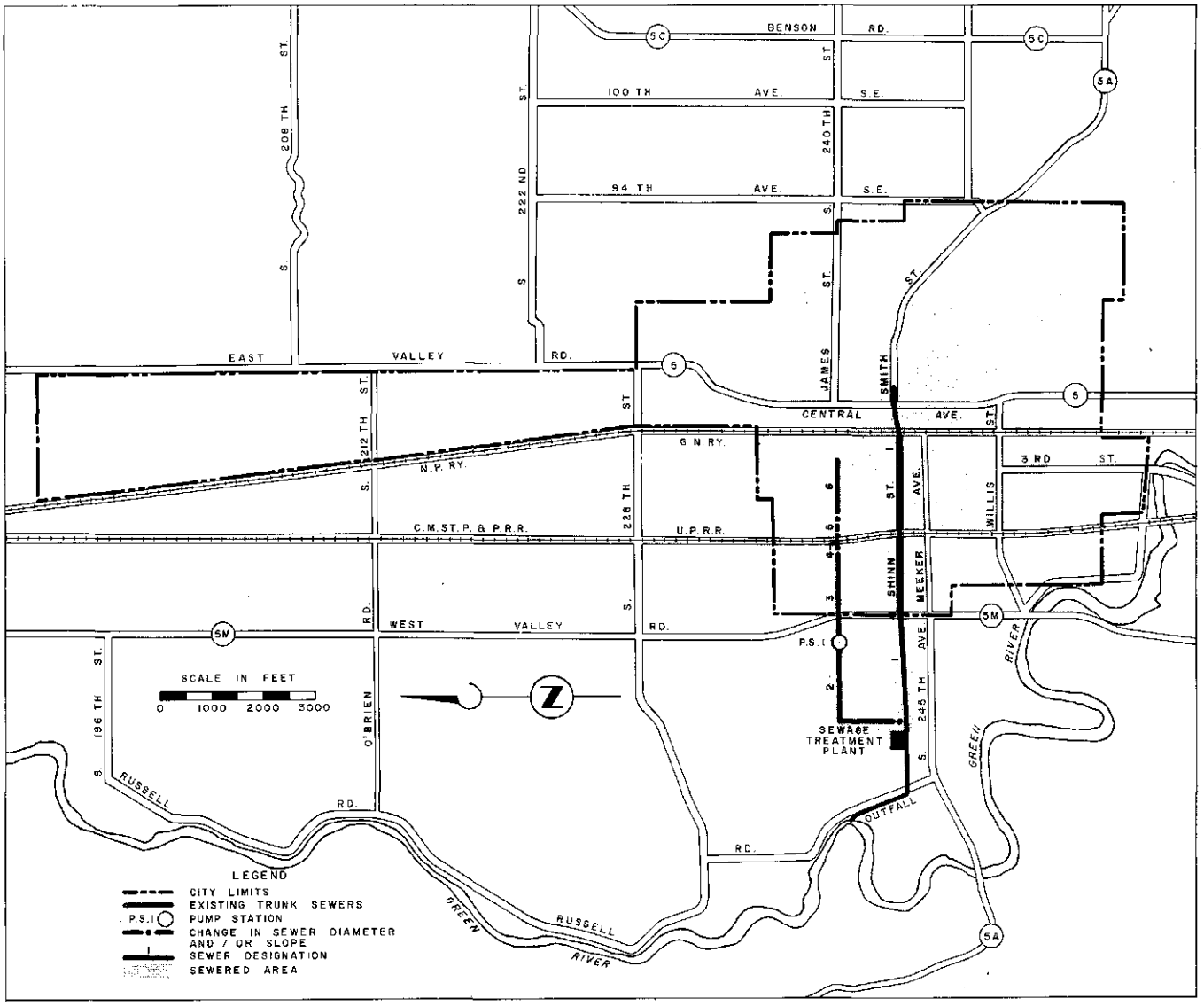


Fig. 6-21. Sewered Area and Principal Sewerage Facilities City of Kent

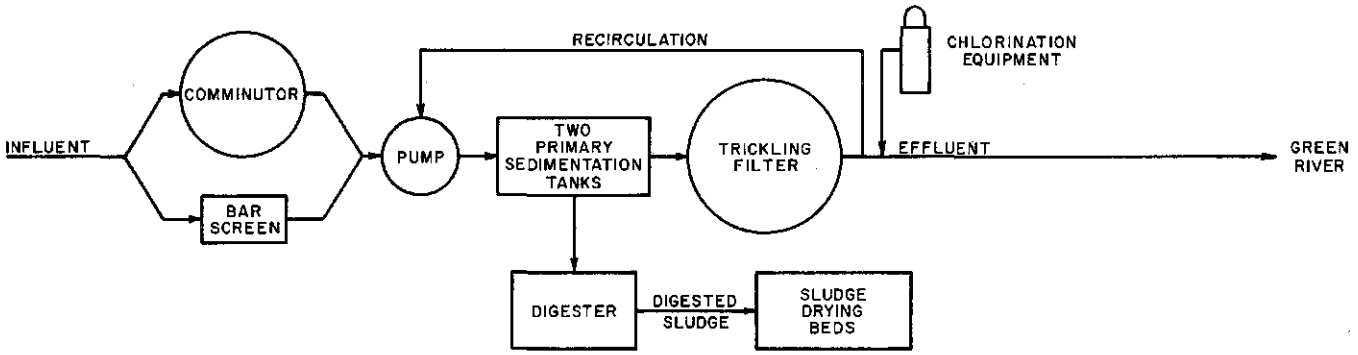


Fig. 6-22. Flow Diagram - Kent Sewage Treatment Plant

Normally operated as a primary treatment plant, the trickling filter is used during the canning season to provide for the extra loading from several canneries. The sedimentation tanks have design capacity for an average daily flow of 1.7 mgd.

Table 6-17. Description of Principal Sewerage Facilities, City of Kent

Facility[a]	Description[b]	Capacity,[c] mgd
1	6,640 ft of 24-in. VC at 0.08%	4.2
2	2,950 ft of 12-in. CA force main	
3	1,830 ft of 18-in. RC at 0.22%	3.2
4	220 ft of 18-in. RC at 0.36%	4.1
5	330 ft of 15-in. RC at 0.30%	2.3
6	1,170 ft of 15-in. RC at 0.20%	1.9
Outfall	Length unknown, 30-in. conc; discharge to Green River.	

[a]*See Fig. 6-21 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. VC signifies vitrified clay pipe; RC, reinforced concrete; CA, cement-asbestos; conc, concrete.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

lation of about 5,750 and consists of 5 pumping stations and 32 miles of sewers ranging in size from 6 inches to 20 inches (Fig. 6-23 and Table 6-18). Treatment facilities, initially constructed in 1943 and enlarged in 1951, share a downtown site with the City Hall and provide two stage secondary treatment during the summer season. During the wet season it is operated as a conventional trickling filter plant and, when so operated, has capacity for an average flow of 0.35 mgd (Fig. 6-24). Chlorinated effluent is discharged to Lake Washington through a trunk storm sewer adjacent to the plant site.

City of Renton

Renton, with a population of 16,500, is the second largest city in the metropolitan area. It was incorporated in 1901 and constructed its first sewers in 1910. Sewerage service is provided to a connected population of about 14,800 through a collection system which covers two-thirds of the city area of 5 square miles. This system has 1 pumping station and a total of 36 miles of 6-inch to 20-inch sewers (Fig. 6-25 and Table 6-19).

Treatment is obtained by means of a secondary type plant, which was constructed in 1943 and enlarged in 1953 to its present design capacity of 2.2 mgd (Fig. 6-26). Plant effluent, after chlorination, is discharged near the mouth of the Cedar River.

Private and Semi-Public Systems

Five privately owned developments, two publicly

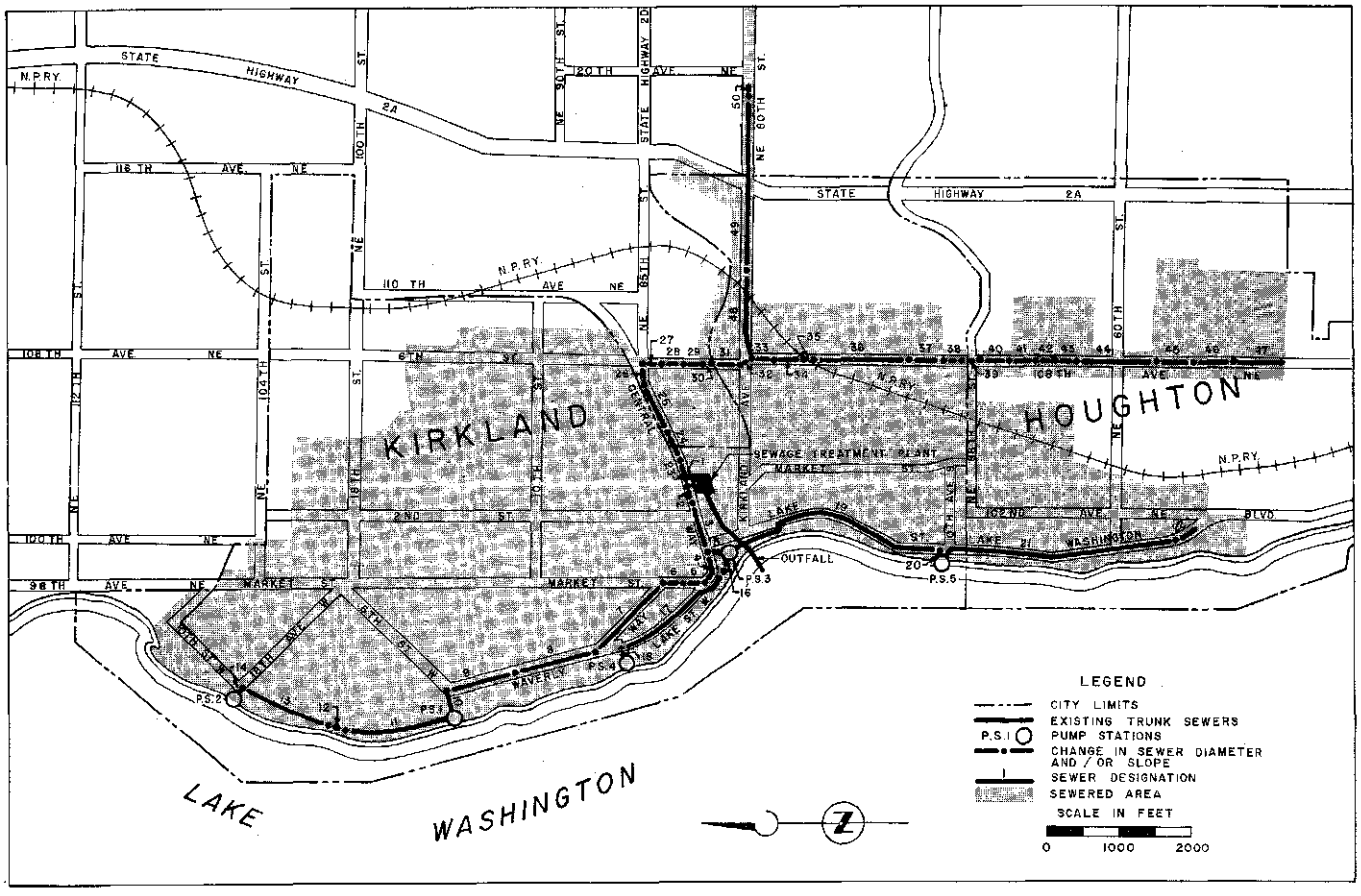


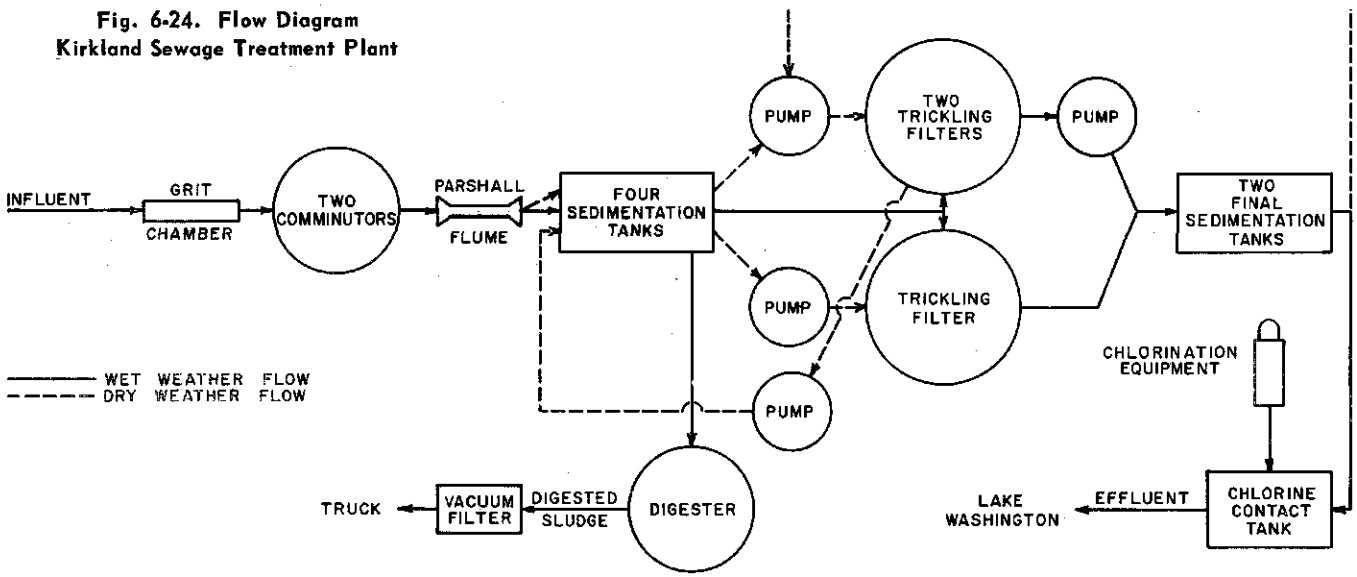
Fig. 6-23. Sewered Area and Principal Sewerage Facilities, City of Kirkland

Table 6-18. Description of Principal Sewerage Facilities, City of Kirkland

Facility[a]	Description[b]	Capacity,[c] mgd	Facility[a]	Description[b]	Capacity,[c] mgd
1	60 ft of 24-in. conc at 0.10%	4.6	27	250 ft of 12-in. conc at 1.00%	2.3
2	260 ft of 18-in. conc at 0.13%	2.4	28	300 ft of 12-in. conc at 2.00%	3.2
3	690 ft of 15-in. conc at 0.18%	1.8	29	280 ft of 12-in. conc at 9.50%	7.1
4	440 ft of 12-in. conc at 0.25%	1.1	30	70 ft of 12-in. conc at 1.00%	2.3
5	320 ft of 8-in. conc at 5.00%	1.8	31	400 ft of 12-in. conc at 6.60%	5.8
6	300 ft of 8-in. conc at 6.20%	2.0	32	140 ft of 12-in. conc at 5.70%	5.4
7	1,340 ft of 10-in. conc at 0.60%	1.1	33	300 ft of 12-in. conc at 5.00%	5.1
8	1,140 ft of 10-in. conc at 0.50%	1.0	34	450 ft of 12-in. conc at 1.00%	2.3
9	1,000 ft of 10-in. conc at 0.30%	0.8	35	170 ft of 12-in. conc at 6.40%	5.8
10	300 ft of 6-in. CI force main		36	1,240 ft of 10-in. conc at 2.00%	2.0
11	1,420 ft of 12-in. CA at 0.22%	1.1	37	440 ft of 10-in. conc at 1.00%	1.4
12	220 ft of 12-in. conc at 0.22%	1.1	38	260 ft of 8-in. conc at 0.60%	0.6
13	1,380 ft of 12-in. CA at 0.22%	1.1	39	250 ft of 8-in. conc at 1.70%	1.0
14	20 ft of 4-in. CI force main		40	400 ft of 8-in. conc at 2.50%	1.2
15	190 ft of 8-in. CA force main		41	380 ft of 8-in. conc at 6.70%	2.0
16	270 ft of 12-in. conc at 0.43%	1.5	42	270 ft of 8-in. conc at 4.30%	1.6
17	1,950 ft of 12-in. conc at 0.25%	1.1	43	260 ft of 8-in. conc at 1.50%	1.0
18	130 ft of 4-in. CI force main		44	1,400 ft of 8-in. VC at 1.20%	0.9
19	3,250 ft of 12-in. conc at 0.25%	1.1	45	530 ft of 8-in. VC at 1.10%	0.8
20	30 ft of 8-in. CI force main		46	540 ft of 8-in. VC at 0.96%	0.8
21	3,690 ft of 12-in. conc at 0.25%	1.1	47	830 ft of 8-in. VC at 0.60%	0.6
22	310 ft of 12-in. conc at 1.90%	3.1	48	1,250 ft of 10-in. conc, slope unknown	
23	350 ft of 18-in. conc at 0.15%	2.6	49	2,470 ft of 8-in. conc, slope unknown	
24	410 ft of 15-in. conc at 0.18%	1.7	50	50 ft of 10-in. conc, slope unknown	
25	730 ft of 12-in. conc at 2.66%	3.7	Outfall	Plant effluent discharges to storm sewer, thence to Lake Washington	
26	330 ft of 12-in. conc at 1.80%	3.0			

[a]*See Fig. 6-23 for location.* [b]*Where slope varies within section, limiting or average slope is shown. Conc signifies concrete pipe; CI, cast iron; CA, cement-asbestos; VC, vitrified clay.* [c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

Fig. 6-24. Flow Diagram Kirkland Sewage Treatment Plant



owned installations, and one industry are still served by independent sewerage systems. Undoubtedly, some of these will be connected to the systems of nearby cities and districts as the metropolitan area continues to develop.

Boeing-Renton. Sanitary sewage from the Boeing Airplane Company operation at Renton is collected and conveyed to a secondary type treatment plant. Chlorinated effluent from this plant, which has a design capacity of 0.5 mgd, is discharged to Cedar River waterway several thousand feet above the point where the waterway enters Lake Washington. Site drainage and some diluted industrial wastes are discharged to Lake Washington through separate outfalls. Concentrated toxic and other processing wastes are hauled

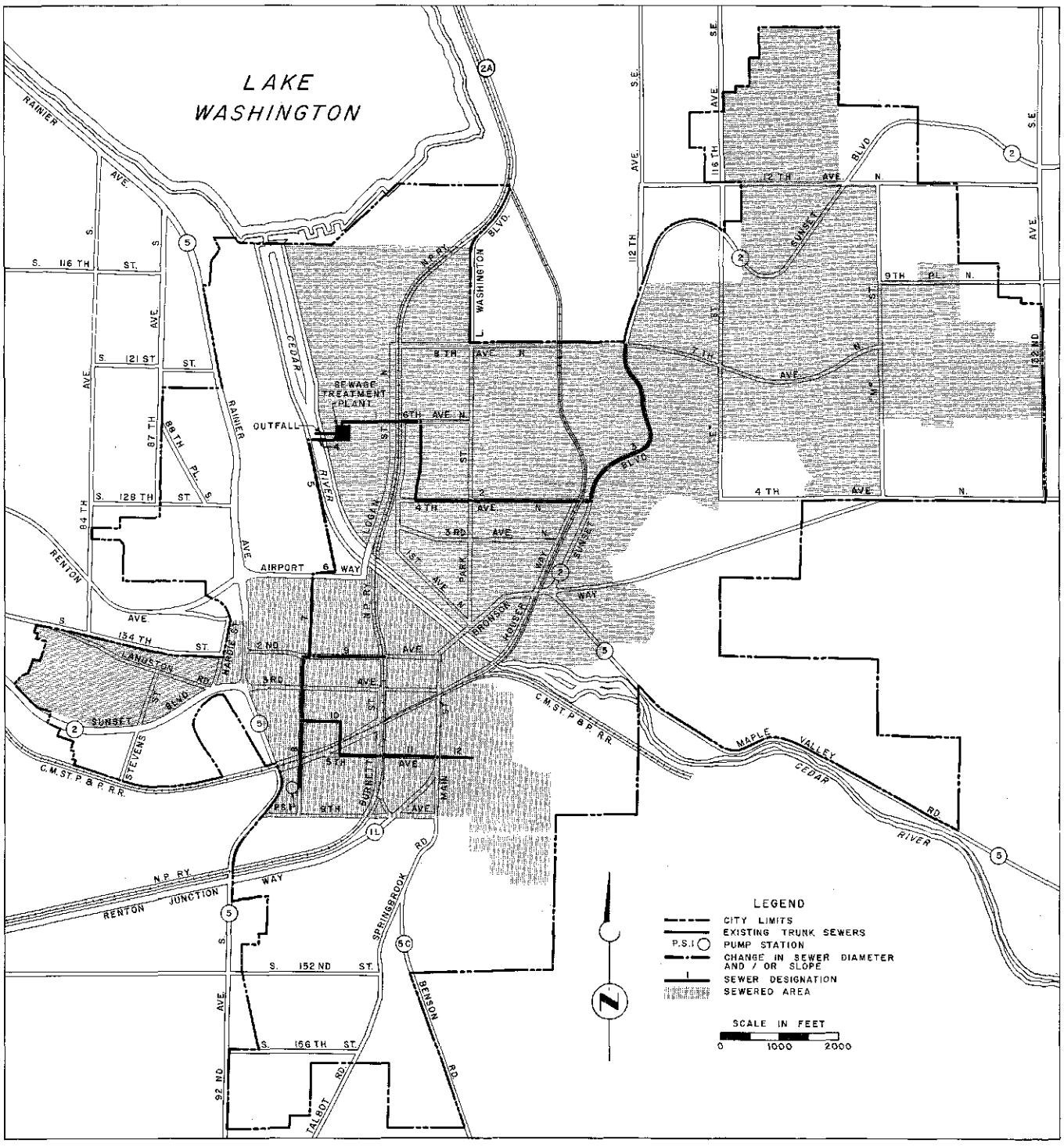


Fig. 6-25. Sewered Area and Principal Sewerage Facilities, City of Renton

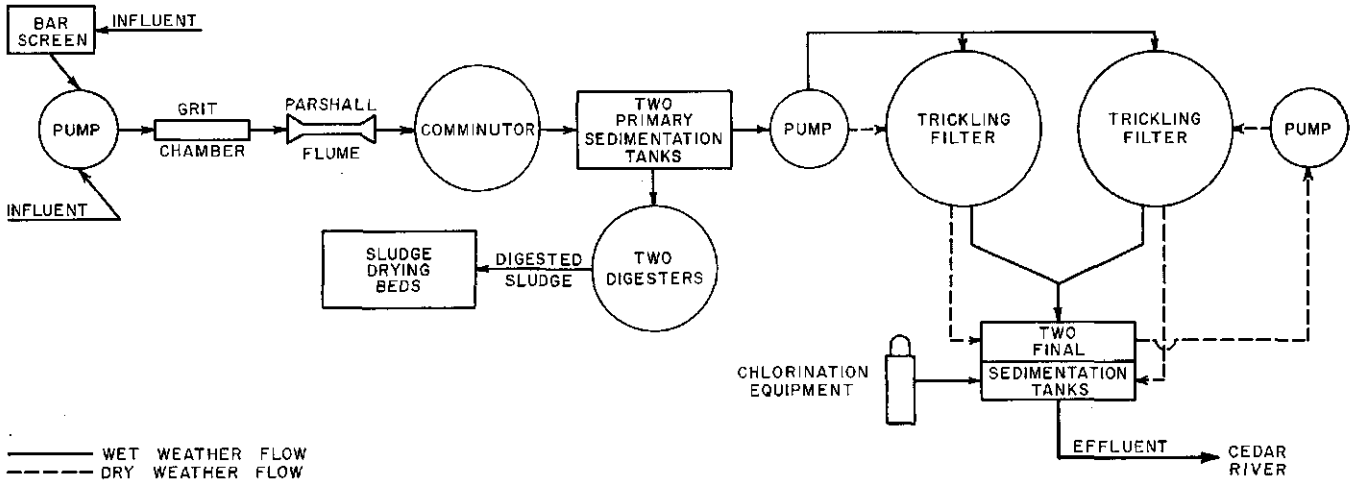


Fig. 6-26. Flow Diagram - Renton Sewage Treatment Plant

Additional treatment is provided for dry season flows by converting one of the final sedimentation tanks and one of the trickling filters to second stage treatment units, as shown. Wet weather flows are given standard trickling filter type secondary treatment. The average daily capacity of the primary sedimentation tanks, based on two hours detention, is 2.2 mgd.

from the site by tank truck for disposal elsewhere.

Boeing Shopping Center. Designed to serve a proposed shopping center in the vicinity of Aurora Avenue and 160th Street, this system consists of a single 12-inch sewer extending to Puget Sound, and of an Imhoff tank type of primary sewage treatment plant with a design capacity of about 0.18 mgd. These facilities are not in use, however, as the shopping center has not yet been developed.

Table 6-19. Description of Principal Sewerage Facilities, City of Renton

Facility[a]	Description[b]	Capacity,[c] mgd
1	1,430 ft of 24-in. conc at 0.20%	6.5
2	4,320 ft of 24-in. conc at 0.23%	7.0
3	3,420 ft of 12-in. conc at 3.88%	4.5
4	520 ft of 24-in. conc at 1.00%; includes siphon under Cedar River; 24-in. overflow ahead of siphon	14.7
5	1,500 ft of 24-in. conc at 0.12%	5.1
6	1,210 ft of 24-in. conc at 0.09%[d]	4.4
7	2,600 ft of 24-in. conc at 0.12%	5.0
8	1,120 ft of 8-in. CI force main	
9	1,130 ft of 18-in. conc at 0.70%	5.7
10	1,930 ft of 18-in. conc at 0.20%	3.1
11	900 ft of 15-in. conc at 0.30%	2.3
12	570 ft of 10-in. conc at 19.00%	13.8
Outfall	Land section, 260 ft of 24-in. conc; submerged section, length unknown, 24-in. conc; discharge to Cedar River	

[a]*See Fig. 6-25 for location.*

[b]*Where slope varies within section, limiting or average slope is shown. Conc signifies concrete pipe; CI, cast iron.*

[c]*Capacity calculated using Manning's formula, "n" equals 0.013.*

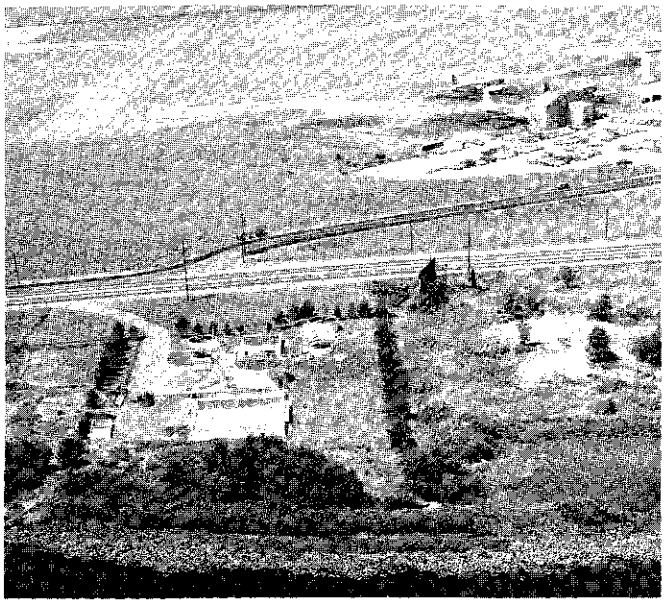
[d]*Distance scaled; slope calculated.*

The Highlands. A privately owned sewer system serves this residential development which is situated along Puget Sound in the vicinity of the Seattle Golf and Country Club. No treatment is provided for the sewage which is discharged to Puget Sound at several points.

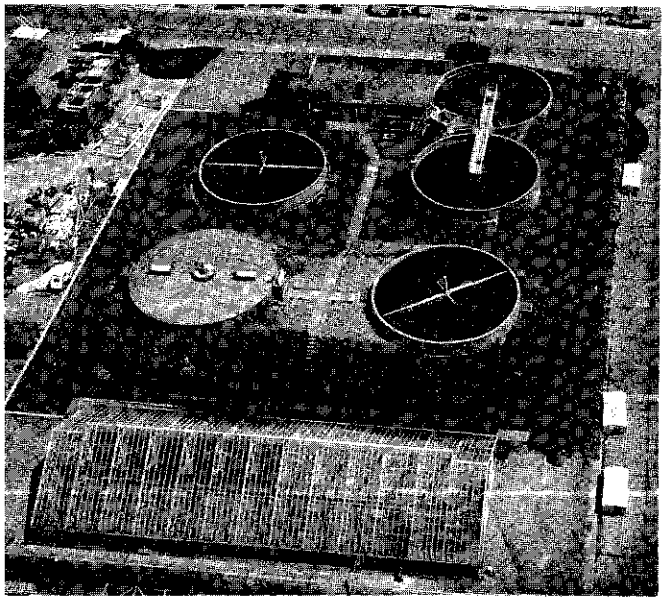
Lake Burien Heights. Sewerage facilities serving this private housing development, located at SW 136th Street and Ambaum Boulevard SW, consist of local collecting sewers and an Imhoff tank type of primary treatment with a design capacity of 0.15 mgd. Chlorinated plant effluent is discharged into Puget Sound.

Sand Point Naval Air Station. This station operates its own sewerage system, which includes a trickling filter type secondary sewage treatment plant with a design capacity of 0.86 mgd. Chlorinated effluent is discharged to Lake Washington.

Sand Point Homes. Originally built for war housing, this multiple dwelling development, located in the vicinity of 60th Street NE and 65th Street, is now owned and operated by the University of Washington. Sewerage facilities, also owned and operated by the University, consist of local collecting sewers and an activated sludge type secondary treatment plant with a design capacity of 0.1 mgd. Plant effluent is discharged to Lake Washington after being chlorinated.



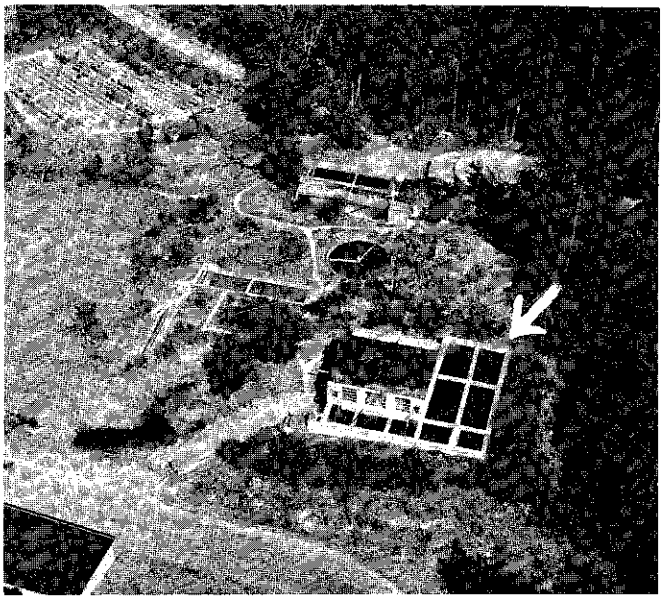
SEATTLE-TACOMA AIRPORT TREATMENT PLANT with a design capacity of 0.10 mgd serves the Seattle-Tacoma Airport. Bio-filters provide secondary treatment and plant effluent is chlorinated and discharged into Bow Lake. Digested sludge is dried in open air drying beds in foreground.



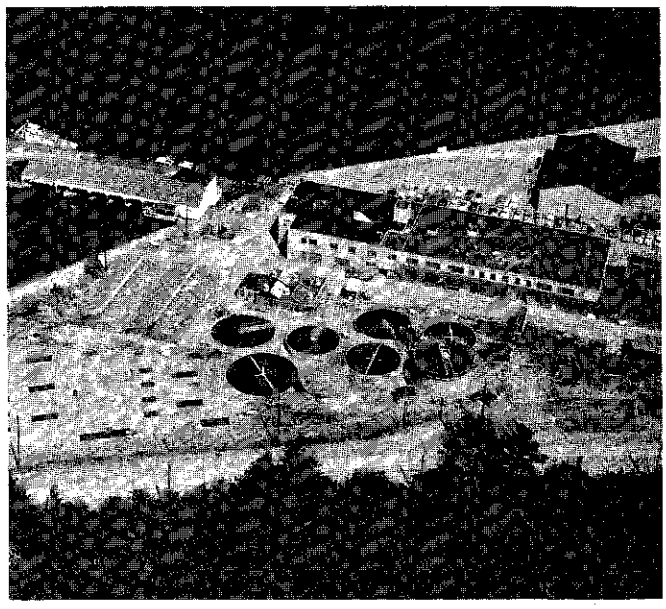
BOEING-RENTON SEWAGE TREATMENT PLANT provides trickling filter type secondary treatment for sanitary wastes generated at the vast Boeing Airplane Company plant at Renton. Note glass covered sludge drying beds in foreground.

PRIVATE AND SEMI-PUBLIC SEWAGE TREATMENT PLANTS

SHOREWOOD APARTMENTS SEWAGE TREATMENT PLANT shares a site with the water filtration plant (arrow) at Shorewood Apartments on Mercer Island. Primary sedimentation (rectangular tank) is followed by roughing filter and activated sludge treatment. Digested sludge is used as a soil conditioner on apartment grounds.



SAND POINT NAVAL AIR STATION SEWAGE TREATMENT PLANT employs bio-filters for secondary treatment of sewage from the station. Digested sludge is air dried in the glass covered drying beds at left.



Shorewood Apartments. This private housing development, situated along the northerly shoreline of Mercer Island, has its own sewer system and an activated sludge type secondary sewage treatment plant with a capacity of 0.26 mgd. Plant effluent is chlorinated prior to discharge to Lake Washington. The Shorewood development is bounded on three sides by the Mercer Island Sewer District.

Seattle-Tacoma Airport. Sewerage facilities serving the Seattle-Tacoma Airport are operated by the Port of Seattle. Sewage is treated in a secondary type plant having a design capacity of 0.1 mgd and plant effluent is chlorinated and discharged to Bow Lake.

Operation and Maintenance

With only minor exceptions, operation and maintenance of the various treatment works and pumping stations are excellent. This, of course, is a credit to plant operators and their assistants and reflects a high degree of interest in their assigned duties. Much of the good work can be attributed no doubt to an interchange of ideas at meetings of plant operators, and to attendance at short courses in plant operation and maintenance. The latter are given biennially under the joint sponsorship of the University of Washington, the Pollution Control Commission, the State Department of Public Health, the Association of Washington Cities, and the Pacific Northwest Sewage and Industrial Wastes Association.

On the other hand, sewer maintenance practices vary widely in scope and thoroughness. Many of the agencies have no regularly assigned maintenance personnel, with the result that such work is confined either to emergency needs or, at best, to occasional cleaning and repair jobs during slack periods in other assigned duties. A few agencies, however, conduct cleaning programs on a schedule designed to cover their systems about once each year.

STORM DRAINAGE FACILITIES OUTSIDE SEATTLE

A brief description of existing storm drainage facilities outside Seattle is given in Table 6-20.

Taken as a whole, the facilities there listed are of little or no significance from the standpoint of comprehensive storm drainage planning. Only Auburn, Kent, Kirkland and Renton have constructed drains which locally serve substantial areas. Most of the other cities have done nothing about this problem and only a few have constructed minor drains to relieve critical areas.

Numerous isolated tracts in unincorporated portions of the metropolitan area are served by storm drains.

Table 6-20. Storm Drainage Systems Outside of the City of Seattle

Place	Description of system
Auburn	A separate storm drainage system was constructed in 1950. Some catch basins and numerous roof drains are connected to the sanitary sewers. Trunk storm drains are insufficient in size to serve more than city areas.
Bellevue	Except for several large culverts under streets and other obstructions, there are no significant facilities. Several 24-inch storm drains designed to serve isolated areas are under construction. A comprehensive storm drainage system is being designed.
Bothell	One 12-inch to 36-inch drain intercepts storm drainage from a number of laterals which serve the westerly section of the city. The remainder of the city has no system. The existing system is reported to have some sanitary sewer connections.
Houghton	Several minor storm drains serve part of the business section. Two war housing projects, one of which has been razed, have local systems.
Kent	Two storm drains, a 12-inch and an 18-inch, serve part of the downtown area and other isolated sections of the city. Considerable street drainage and nearly all roof drainage from the downtown area is discharged into sanitary sewers.
Kirkland	Separate storm drains, ranging in size from 12 inches to 60 inches, convey storm runoff from a considerable section of the city to Lake Washington. Effluent from the sewage treatment works discharges to one of the storm drain trunks. Numerous catch basins and roof drains are connected to the sanitary sewers.
Lake Hills	All developed sections are served by a separate storm drainage system. Area has comprehensive storm drainage plan. Trunks range from 12 inches to 42 inches in diameter.
Mountlake Terrace	One 15-inch storm drain serves school grounds. Remainder of city has no storm drainage facilities.
Redmond	Existing drainage facilities consist of a single 12-inch drain. A storm drainage system is scheduled for construction in 1958.
Richmond Beach	Served by partially combined sewer system. See section on existing sewerage.

For the most part, however, these were constructed for the purpose of relieving localized flooding problems, mainly in connection with street and highway drainage.

SEWERAGE AND DRAINAGE FACILITIES OF THE CITY OF SEATTLE

Seattle lies across the natural outlets of the Green-Duwamish and Lake Washington drainage basins, which together comprise about 90 per cent of the metropolitan area. Its sewerage system, besides being by far the largest and most complex of those under consideration, occupies a strategic location with respect to long-term sewerage planning on a metropolitan basis. This situation, coupled with problems brought about by major deficiencies in present facilities, necessitated an extensive study of the Seattle system.

General Features of the Seattle System

A general layout of the Seattle sewerage and drainage systems, together with their service areas and points of disposal, is shown in Fig. 6-27. In all, about 67 square miles of the land area within the city, which totals 79.5 square miles, are presently sewered. An outside area of 2.5 square miles also is served by the city. Within the city, the connected population is roughly 510,000, as compared to an estimated total population of 560,000.

According to available records, the collection system consists of 1,029 miles of combined sewers, 204 miles of separate sanitary sewers, 7 miles of storm drains, and 39 pumping stations. About one-fifth of the estimated total sewage and industrial waste flow of 65 mgd now being generated receives varying degrees of treatment at 5 sewage treatment plants. The balance of 52 mgd is discharged as raw sewage through about 50 outfalls scattered along Puget Sound, Elliott Bay, Duwamish River, Lake Union and the Ship Canal. In addition, a number of industries discharge wastes independently to Duwamish River, Lake Union and the Ship Canal. Enlargements to the existing Lake City plant and construction of a new treatment plant at Alki Point, which are both in progress, will increase the total treatment capacity by about 15 mgd.

As stated in Chapter 2, early collection systems followed natural drainage channels and discharged raw sewage to the nearest watercourses. But as the city developed, its natural topography was altered to some extent by filling of low areas and by major regrading for street construction. Further, as pollution of Lake Union and Lake Washington increased and more adequate points of disposal became a necessity, intercepting sewers and tunnels were constructed across natural drainage boundaries in order to convey the sewage to salt water. Consequently, most of the area which comprises the city up to 1954 is served by four major and numerous minor collection systems. With one exception, all of these discharge raw sewage directly to Puget Sound. Sewage from the Henderson-East Marginal Way system, which is the exception, is delivered to the Diagonal Avenue plant for treatment and disposal.

In 1954, the north side annexation brought in four sewer districts, each of which had previously constructed its own separate sanitary sewer system. Of these, the Blue Ridge, North Beach and Greenwood systems lie in the Puget Sound basin and discharge to the sound, while the Lake City systems drains toward and discharges to Lake Washington. Annexation of the Roxbury Heights area in 1956 brought still another separate sanitary sewer system into the city. Four of these five districts have sewage treatment plants but none of them has storm drainage facilities.

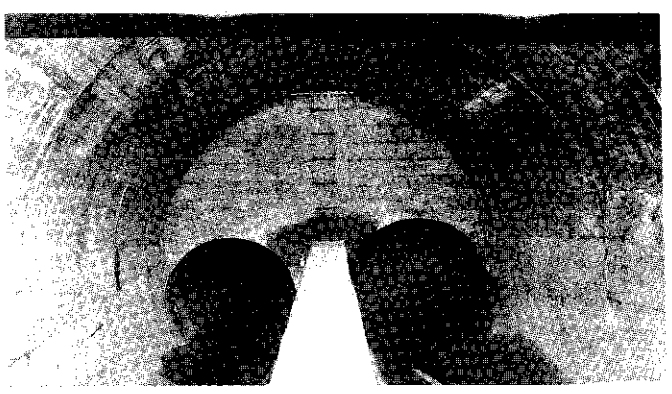
Principal Sewerage Facilities

Principal sewerage facilities are shown and the areas they serve are described in Figs. 6-28 through 6-35. Component sections of the various sewers are designated by numbers and each section is described under its corresponding number in Table 6-21. Flow diagrams for the five sewage treatment plants are shown in Figs. 6-36 through 6-40. For convenience in reference, Table 6-21 and Figs. 6-28 through 6-40 are located at the end of this chapter.

Local Effects of Combined Sewers

At Seattle, local effects stemming from the use of combined sewers are manifested both by frequent overflows from overflow and bypass structures and by flooding of streets and basements. Flooding occurs during even moderate storms and is due to overload conditions in the collection system. In some cases, overloading leads also to direct damage of sewers and related structures.

The frequencies with which overflow and overload conditions develop, as well as their nature and extent, depend on a number of different factors. Of these, the most important is the rainfall intensity used in



SIDE WEIR overflow to Ship Canal

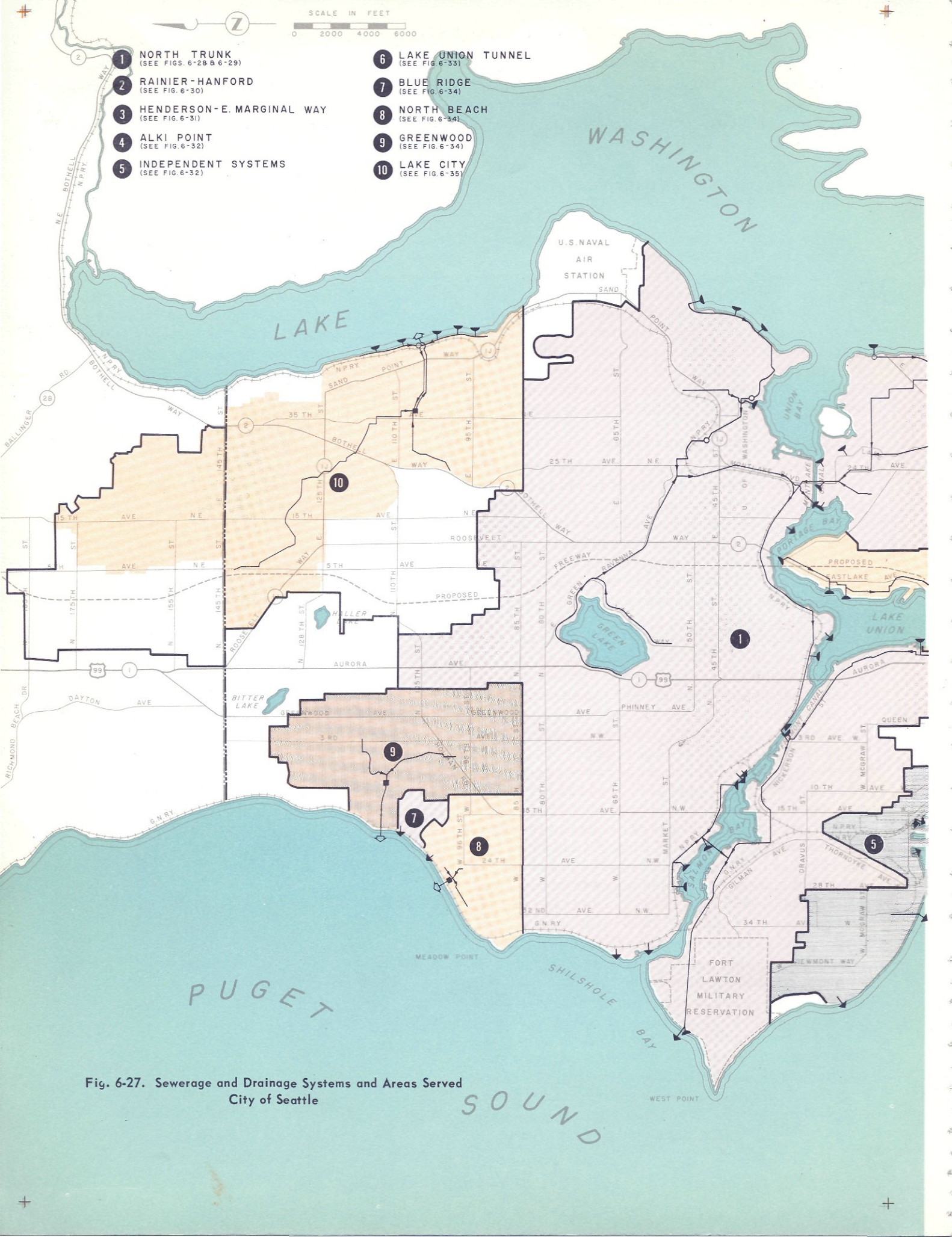
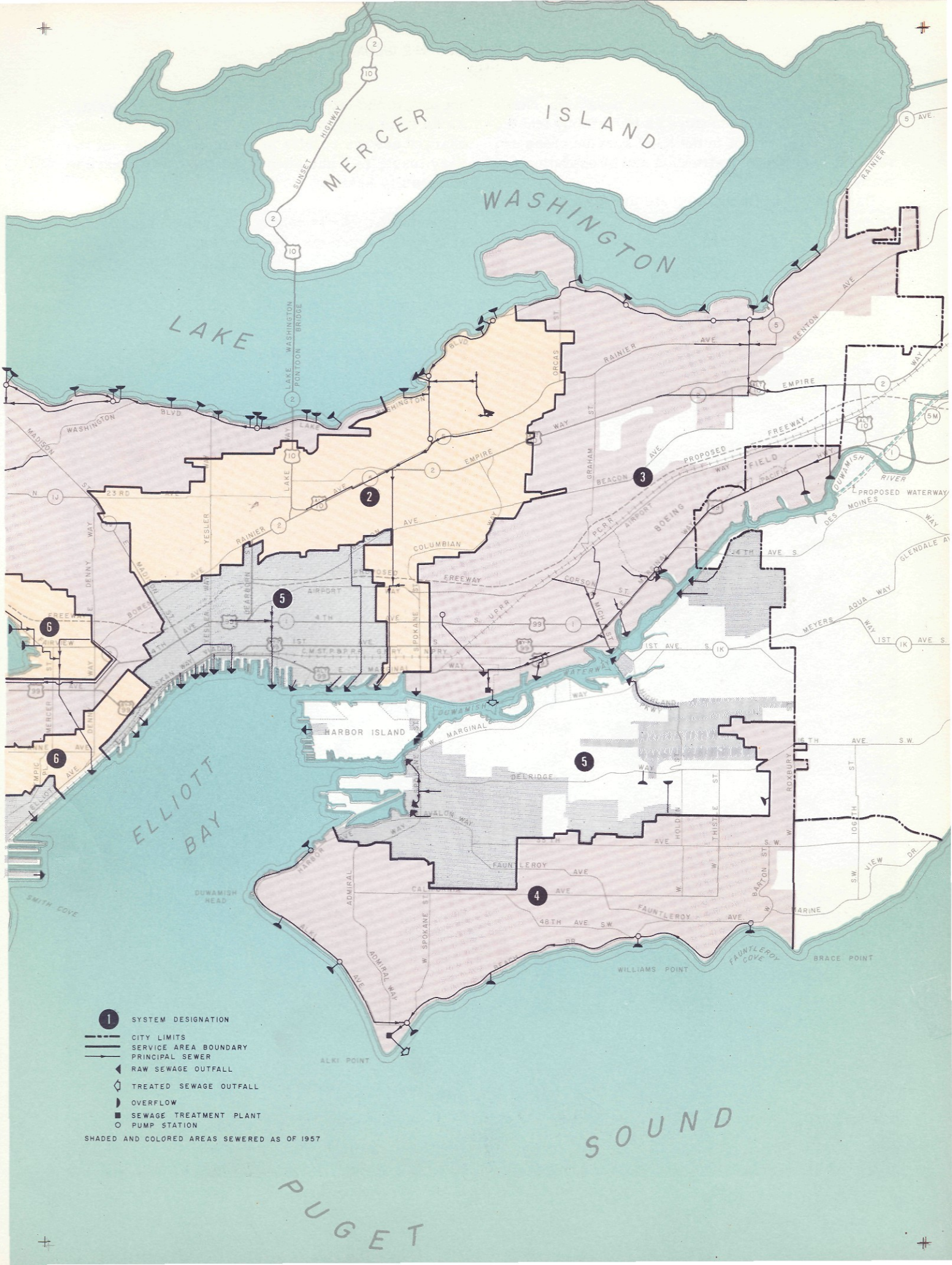


Fig. 6-27. Sewerage and Drainage Systems and Areas Served
City of Seattle

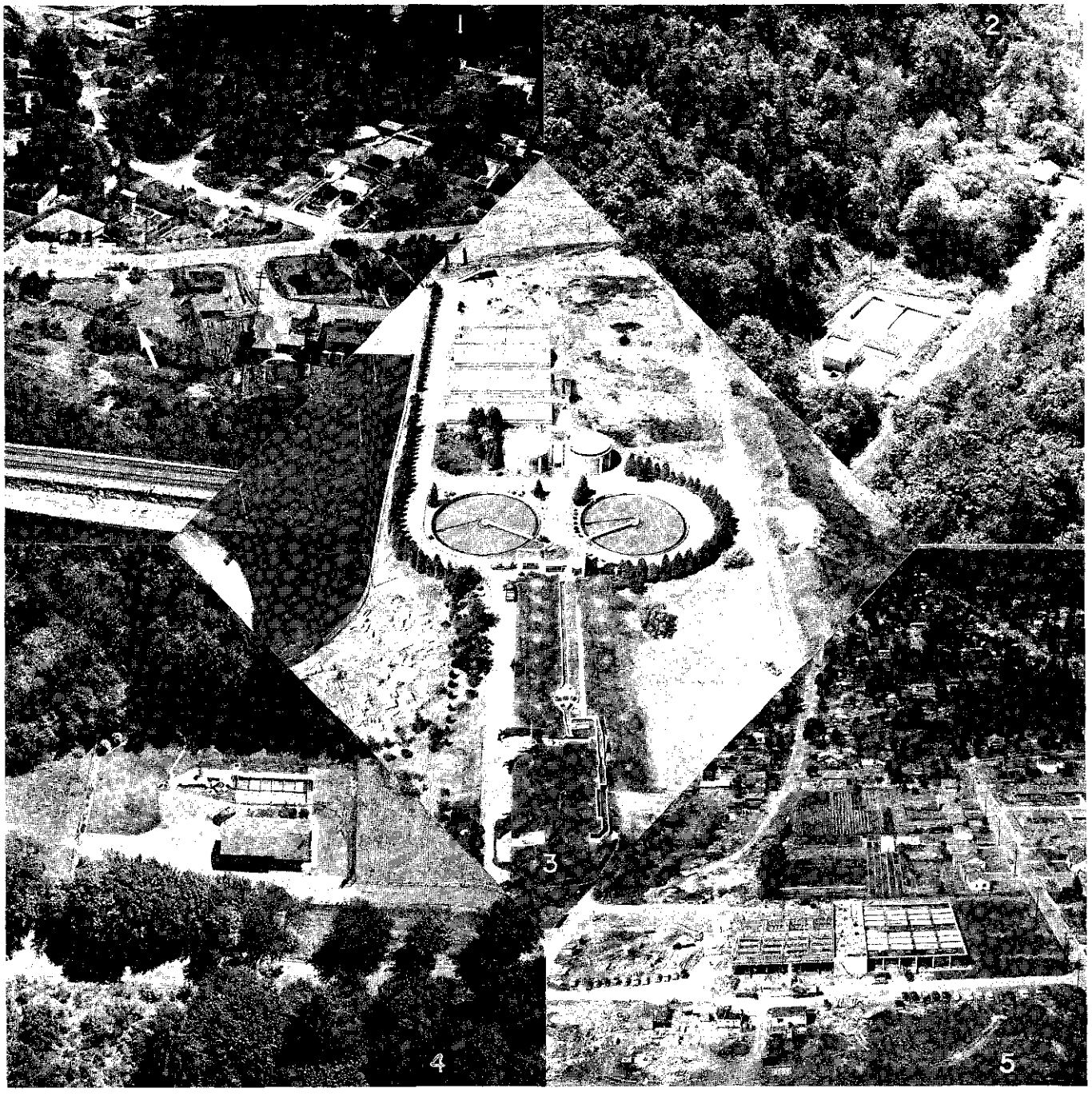

MERCER ISLAND
LAKE WASHINGTON
ELLIOTT BAY
PUGET SOUND
HARBOR ISLAND
DUWAMISH HEAD
SMITH COVE
ALKI POINT
WILLIAMS POINT
FAUNTLEROY COVE
BRACE POINT
SUNSET HIGHWAY
LAKE WASHINGTON PONTOON BRIDGE
PROPOSED FREEWAY
PROPOSED WATERWAY
DES MOINES
DUWAMISH RIVER
BOEING FIELD
AIRPORT
SEAFIRST
1 SYSTEM DESIGNATION
CITY LIMITS
SERVICE AREA BOUNDARY
PRINCIPAL SEWER
RAW SEWAGE OUTFALL
TREATED SEWAGE OUTFALL
OVERFLOW
SEWAGE TREATMENT PLANT
PUMP STATION
SHADED AND COLORED AREAS SEWERED AS OF 1957

determining the necessary sewer capacity. Other factors involve such matters as topography and the extent of development in the local service areas and the quality both of construction and of operation and maintenance.

Damage to public and private property resulting from the backing-up of water in surcharged sewers has been a matter of continual concern and expense not only to the city but to many individual residents. Aside from property damage, overflows from surcharged sewers results in a situation which, at the very least, is a nuisance and a potential hazard to community health.

Overflows. As is common practice in the design of combined systems, Seattle sewers are provided

SEWAGE TREATMENT PLANTS SERVING SEATTLE



with overflow and bypass structures whereby wet weather flows in excess of intercepting sewer and pumping station capacities are diverted to convenient points of disposal. Although the majority of these structures, which may be either overflow manholes or side weirs, are incorporated in trunk lines leading to intercepting sewers, many are located also along the interceptors themselves.

Theoretically, overflow structures in use at Seattle are designed to function when the flow in the sewer reaches from five to nine times the estimated sanitary flow. Since, however, the overflow weir crests generally are set at a height approximately equal to the quarter depth of the approach sewer, overflowing usually begins when the ratio of storm flow to sanitary flow reaches two to one, or even less.

Overflows are frequent, even during summer months, occurring at most structures on an average of about 40 times during the period from May through September. On the other hand, because flows in excess of trunk or interceptor capacity are generally of short duration, the actual volume of sanitary sewage thus discharged is relatively small.

Further information concerning overflow conditions is presented in subsequent chapters. Chapters 8 and 10 discuss their effects on receiving waters; Chapter 13 presents an analysis of their frequency and volume as related to the size of intercepting sewers; and Chapter 15 discusses the overflow problem as it affects future sewerage planning and design.

Overloaded Sewers. Until recently, the design of combined sewers in Seattle has been based on a uniform runoff of 15 cubic feet per minute per acre. For many of the drainage areas, this amount of runoff can be expected to occur with storms having a recurrence interval of only two years. It follows, therefore, that overloading of the systems in these areas is likely to occur at a similar frequency.

As emergencies have arisen, various steps have been taken to prevent or alleviate conditions resulting from the backing up of excess flows. Systems originally intended to function independently have been interconnected; weirs have been installed in manholes to divert part of the flow from one sewer to another; sewers tributary to overloaded sections have been plugged and the flow diverted; emergency bypasses have been constructed; and relief sewers have been installed to provide additional capacity in the most troublesome areas. As a result of these modifications, some of which have been completed without any record being made of their nature and location, it is difficult to make the hydraulic analyses which are required for the planning of trunk and interceptor improvements. In fact, conditions in some areas are such that an analysis of the present facilities is practically impossible.

Additional information concerning overloaded sewers will be found in two of the later chapters. Chapter 8 includes a detailed discussion of their detrimental effects, while Chapter 18 presents an analysis of the problem as it affects future planning and design.

Operation and Maintenance

Maintenance of the Seattle sewerage system is performed on a variable schedule based both on past experience and on the results of an annual inspection program. As part of the latter, all sewers up to 15 inches in diameter are candled for evidence of obstructions or damage. Inspection of most of the large sewers, however, is limited to sections in the vicinity of points of entry. This is because of the long distances between manholes, the depth of sewage flow, and the inaccessibility of tunnel sections.

←

(1) NORTH BEACH SEWAGE TREATMENT PLANT, a 0.43 mgd primary treatment plant, is hidden beneath this landscaped site (arrow). The only plant structure visible to nearby residents is the small building which houses the access stairway at center of the triangular site. Note the two residences immediately adjacent to the property line at right of site. Effluent is discharged to Puget Sound offshore from the beach shown in foreground.

(2) ROXBURY HEIGHTS SEWAGE TREATMENT PLANT, originally constructed by the Roxbury Heights Sewer District, has been operated by Seattle since annexation of the Roxbury area in 1956. Primary effluent from the Imhoff tanks shown here is discharged to Puget Sound offshore from the beach shown in background.

(3) DIAGONAL AVENUE TREATMENT PLANT is the only plant in the area treating combined sewage. The circular clarifiers, center, provide primary sedimentation for an average daily flow of 8.0 mgd. Wet weather flow to the plant is limited to this rate by an upstream flow regulator, the excess being bypassed. Note the glass covered sludge drying beds behind digesters and the effluent discharge into Duwamish River at top of picture near end of row of trees.

(4) GREENWOOD AVENUE SEWAGE TREATMENT PLANT, located near the mouth of Piper Creek in Carkeek Park, is designed to remove floatable materials from sewage. Effluent is chlorinated and discharged into Puget Sound opposite the public bathing beach. Agitation of sewage by air to promote floatation is noticeable in forward compartment. Floatable materials are removed from the stilling chamber in rear by spray jets. Building in foreground houses blower and other plant equipment.

(5) LAKE CITY SEWAGE TREATMENT PLANT during construction of additions in 1957. Section at right houses the original 2.5 mgd activated sludge plant. Digesters are at center and new units which will bring plant capacity to 10.0 mgd are at left. Effluent is discharged to Lake Washington through a tunnel under a high ridge which prevents natural flow from the site to the lake by gravity.

Certain sewers, especially those with steep grades, have required little or no maintenance, whereas others require flushing, dragging or root removal from one to four times every year. Since troublesome sections are generally known from past experience, regular cleaning programs are scheduled accordingly. Maintenance schedules for other sections are developed annually on the basis of conditions observed during the annual inspection.

Sand traps, which are located ahead of each of the siphons, are inspected frequently and normally require cleaning only twice a year. Material removed at such times is inspected for pieces of broken pipe or brick which might indicate breakage in tributary sewers. Debris is pumped from catch basins on an average of three to four times a year.

A crew of four to eleven men is regularly assigned to sewer maintenance duties in each of five maintenance districts, and is augmented as necessary to cope with seasonal and emergency conditions. In addition, a city-wide crew of twelve men is regularly assigned to maintenance of the storm water catch basins. This crew also is augmented for emergency work during and following storms. Mechanical equipment includes fourteen trucks, five of which are equipped with winches and dragging equipment; four trailer-mounted cleaning and root cutting devices; and seven eductors for catch basin cleaning.

During the annual inspection, and also while performing routine maintenance duties, a special effort is made to detect cracks, breaks, and other signs of deterioration in sewer lines and structures. Failures requiring immediate attention are either replaced or repaired on an emergency basis, whereas other repair and replacement work is scheduled as manpower becomes available. Cracked sewers normally are not replaced until there is evidence of progressive failure. Cracks are estimated to have developed in about 15 per cent of the smaller sewers now in service and, of course, are far more numerous in older systems.

Principal maintenance problems are the sand and grit deposits which accumulate in sewers with flat grades, and the frequent backups which occur in many sections of the city during periods of moderate to heavy rainfall. In sewers which cannot be cleaned by conventional equipment, or which are inaccessible, accumulated deposits of sand and grit may account for some of the backups.

Pumping stations are operated and maintained by two crews of two men, each crew working under a machinist who in turn reports to the foreman of sewage treatment plants. Normally, each station is visited three times weekly for the purpose of washing down wet wells, cleaning floats, changing pumping record charts, recording total pumpage, greasing and oiling equipment, alternating manually operated pumps, checking equipment for signs of failure, and general cleaning. Conditions which cannot be corrected by the maintenance crews are reported to the machinist, who either makes the necessary repairs in the field or calls upon the general machine shop or the department electrician for assistance.

Sewage treatment plants are operated and maintained by a regular crew of twelve operators and laborers under the supervision of a foreman. Each of the five treatment plants is attended eight hours a day and is normally unattended for sixteen hours. At present, operators assigned to the Diagonal Avenue plant also operate and maintain the Roxbury Heights plant on a part-time basis. When required, the services of a machinist and an electrician are available, as is additional help during emergencies. Two painters, regularly assigned to the city shop, do part-time painting and waterproofing at pumping stations and treatment plants.

Table 6-21. Description of Principal Sewerage Facilities, City of Seattle

Facility[a]	Description[b,c]	Capacity,[d] mgd
North Trunk System – Interbay District		
1	1,120 ft of 48-in. CI submarine outfall to water depth of 40 ft	80[e]
2	8,000 ft of 144-in. BR and BI at 0.033%, includes 3,380 ft of tunnel under Fort Lawton	273
3	4,040 ft of 144-in. BR and BI at 0.032%	269
4	1,640 ft of 138-in. BI at 0.035%	290
5	4,550 ft of 138-in. BR and BI at 0.033%	244
North Trunk System – Ballard District		
6	1,290 ft of twin 36-in. wood stave inverted siphon; discharge through 450 ft of 72-in. RC at 0.028% to sewer 3 at 24th Avenue West	29[f]
7	Sandcatcher, overflow and siphon structure, includes 120 ft of 36-in. overflow outfall	.....
8	960 ft of 66-in. RC at 0.035%	40
9	2,080 ft of 54-in. RC at 0.04%	26
10	2,130 ft of 42-in. RC at 0.10%	21
11	1,360 ft of 36-in. RC at 0.08%	12
North Trunk System – Central District		
12	50 ft of 60-in. BI at 3.53%, sandcatcher, 30 ft long side weir overflow, and 400 ft of 39-in by 60-in. rectangular overflow outfall. Overflow starts when flow reaches 30 mgd. Weir capacity, 145 mgd; overflow outfall capacity 110 mgd with water surface at weir crest	.....
13	2,550 ft of 72-in. BI at 0.19% and 120 ft of 78-in. BI at 0.24%	120
14	2,500 ft of 60-in. BR tunnel at 0.13 - 0.14% and 250 ft of 66-in. BR tunnel at 0.17%	54
15	980 ft of 54-in. BR tunnel at 0.18%	46
16	2,920 ft of 48-in. BR tunnel at 0.20 - 0.26%	38
17	30-ft long side weir overflow and 460 ft of 42-in. and 48-in. overflow outfall. Overflow starts when flow reaches 32 mgd. Weir capacity, greater than 130 mgd	.....
18	2,550 ft of 84-in. BI at 0.21%, includes 200 ft of tunnel	162
19	210 ft of parallel 24-in. and 66-in. conc inverted siphons under Broad Street underpass	116
20	3,530 ft of 60-in. BR at 0.61% and 390 ft of 66-in. BR at 0.38%	113
21	360 ft of 48-in. BR at 1.60% and 640 ft of 42-in. BR at 4.21%	100
22	240 ft of 48-in. BR at 1.03%	83
23	330 ft of 42-in. BR at 1.0%	56
North Trunk System – Lake Union District		
24	360 ft of parallel 48-in. and 60-in. CI inverted siphons in concrete tunnel; discharge through 150 ft of 108-in. BI to sewer 5	125
25	460 ft of 108-in. BI at 0.073%	225
26	Sandcatcher and emergency overflow	.....
27	4,800 ft of 108-in. BI at 0.074 - 0.077%	221
28	4,760 ft of 108-in. BR and BI at 0.087%	230
29	3,210 ft of 108-in. BR at 0.068%	182
30	1,960 ft of 108-in. BR at 0.065%	178
North Trunk System – Green Lake District		
31	80 ft of 96-in. BI at 0.074%; discharge to sewer 30	160
32	50 ft long side weir overflow and 1,260 ft of 84-in. overflow outfall. Overflow starts when flow reaches 120 mgd. Weir capacity, greater than 375 mgd	.....
33	6,880 ft of 138-in. BR and BI at 0.16%	535

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Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
34	3,560 ft of 72-in. BR tunnel at 2.563% and 130 ft of 126-in. BR tunnel	375
35	Sandcatcher	.....
36	3,420 ft of 90-in. BR and BI at 0.45%	288
37	2,200 ft of 90-in. BI at 0.15%	195
38	1,760 ft of 90-in. BI at 0.11% and 290 ft of 90-in. BI at 0.085%	160
39	320 ft of 72-in. BI at 0.11%	90
40	2,170 ft of 54-in. BI at 0.14 - 0.16%	47
41	590 ft of 48-in. BI at 0.14 - 0.16%	34
42	660 ft of 42-in. BI at 0.16 - 0.26%	26
43	800 ft of 36-in. BR at 0.24 - 0.28%	16
44	2,940 ft of 54-in. BI at 0.22%	59
45	3,000 ft of 42-in. BI at 0.20%	29
46	Overflow weir from Green Lake to sewer 38, includes 170 ft of 24-in. connecting pipe. 3-ft weir with crest at elevation 160.5 mean sea level datum	.....
47	Overflow weir from Green Lake to sewer 44, includes 210 ft of 24-in. connecting pipe. 3-ft weir with crest at elevation 160.5 mean sea level datum	.....
48	Overflow weir from Green Lake to sewer 44, includes 160 ft of 24-in. connecting pipe. 2-ft weir with crest at elevation 160.5 mean sea level datum	.....
49	930 ft of 96-in. BR at 0.10%	160
North Trunk System — Laurelhurst District		
50	1,730 ft of 42-in. RC at 0.08% and 300 ft of 36-in. BR at 0.85%; discharge to sewer 49	18
PS-1	Pumping station, includes 80 ft of 20-in. CI force main. Contains 2 pumps; capacities 5.2 mgd at 39 ft total head and 5.4 mgd at 39 ft total head	.....
51	2,410 ft of 42-in. RC at 0.08%	18
52	730 ft of 30-in. conc at 0.4%	17
53	610 ft of 30-in. conc at 0.24%	13
PS-2	Pumping station, includes 40 ft of 20-in. CI force main and 230 ft of 36-in. emergency overflow and bypass line. Contains 2 pumps; capacities 4.2 mgd at 22 ft total head and 4.8 mgd at 21 ft total head	.....
54	840 ft of 48-in. RC at 0.07%	24
55	1,200 ft of 48-in. RC at 0.045%	20
56	1,660 ft of 42-in. RC at 0.06 - 0.072%, includes 1,500 ft of tunnel	17
57	740 ft of 42-in. RC at 0.05%	14
58	Stormwater overflow and 430 ft of 24-in. conc and 30-in. wood stave outfall	.....
59	1,600 ft of 30-in. conc at 0.09%	7.8
60	2,450 ft of 30-in. conc at 0.07%	7.0
61	60 ft of 18-in. conc at 1.72%, 20-ft long side weir overflow, and 980 ft of 36-in. conc and wood stave overflow outfall. Overflow starts when flow reaches 4 mgd. Weir capacity, 60 mgd	.....
62	4,540 ft of 36-in. to 60-in. at 0.25 - 4.5%. Capacity range 70 - 90 mgd	.....
North Trunk System — Lake Washington District		
63	1,090 ft of 48-in. BR at 0.15%; discharge to sewer 30	31
64	510 ft of 48-in. BR inverted siphon under Lake Washington Ship Canal	24
65	Sandcatcher, 40-ft long side weir overflow, and 220 ft of 60-in. overflow outfall. Overflow starts when flow reaches 30 mgd. Weir capacity greater than 260 mgd	.....
66	770 ft of 114-in. BR at 0.123%	280

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Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
67	1,280 ft of 66-in. BI at 0.227%	100
68	2,770 ft of 90-in. BI at 0.173%	200
69	1,540 ft of 72-in. BR at 0.2%	120
70	4,260 ft of 60-in. BR and BI at 0.5%	115
71	1,210 ft of 66-in. BI at 0.16%	85
72	1,760 ft of 60-in. BR and BI at 0.186%	72
73	850 ft of 54-in. BR at 0.18 - 0.22%	46
74	1,940 ft of 54-in. BR at 0.15 - 0.16%	42
75	5,360 ft of 48-in. BR and BI at 0.25%	40
76	1,360 ft of 32-in. by 48-in. BR oviform at 0.54%	28
77	1,190 ft of 24-in. by 36-in. BR oviform at 2.09%	26
PS-3	Pumping station, discharges to sewer 75 through 1,010 ft of twin 12-in. CI force mains. Contains 2 pumps; capacities 1.2 mgd at 46 ft total head and 2.7 mgd at 46 ft total head. Emergency overflow and stormwater bypass from adjacent manhole through 410 ft and 730 ft of parallel 20-in. CI outfalls. The longer outfall also serves as overflow from tributary sewer	.....
78	2,930 ft of 15-in. conc at 0.17%	1.7
PS-4	Pumping station, includes 120 ft of 10-in. CI force main. Contains 2 pumps; capacities 1.2 mgd at 13 ft total head and 1.2 mgd at 14 ft total head. Emergency overflow and stormwater bypass from adjacent manhole through 420 ft of 24-in. CI outfall	.....
79	4,240 ft of 18-in. conc at 0.13 - 0.14%; discharge to PS-3	2.6
PS-5	Pumping station, discharges to sewer 77 through 300 ft of 12-in. CI force main. Contains 2 pumps; capacities 0.95 mgd at 81 ft total head and 1.7 mgd at 85 ft total head. Emergency overflow and stormwater bypass from adjacent manhole through 300 ft of 24-in. CI outfall	.....
80	1,960 ft of 12-in. at 0.22%	1.1
81	370 ft of 24-in. at 0.10%	4.6
82	3,960 ft of 21-in. at 0.12%	3.6
83	3,440 ft of 18-in. at 0.14%	2.6
PS-6	Pumping station, includes 60 ft of 10-in. CI force main. Contains one pump having a capacity of 1.3 mgd at 18 ft total head. Emergency overflow and stormwater bypass from adjacent manhole through 16-in. CI outfall. Overflow from tributary sewer through 230 ft of 20-in. CI outfall	.....
84	3,140 ft of 15-in. at 0.17%	1.7
85	1,840 ft of 8-in. at 0.35%	0.5
Rainier - Hanford System		
86	5,390 ft of 100-in. by 150-in. BI at 0.075%. Discharges through 48-in. CI submarine outfall at low flows and over spillway at shore line at high flows	308
87	720 ft of 48-in. BI at 0.10%	29
88	6,070 ft of 108-in. BI at 0.4%, includes 5,600 ft of tunnel	510
89	440 ft of 102-in. BI at0.286 - 0.30%	365
90	2,310 ft of 102-in. BI at 0.21 - 0.26%	315
91	1,400 ft of 75-in. BR at 0.55%, discharges to sewer 90. Formerly discharged to sewer 199, which has been abandoned	195
92	850 ft of 72-in. BR at 0.55%	175
93	2,480 ft of 66-in. BI at 0.28 - 0.32%	102
94	3,740 ft of 60-in. BI at 0.31 - 0.32%	95
PS-7	Pumping station, discharges to sewer 93 through 50 ft of parallel 10-in. and 20-in. force mains. Contains 3 pumps; capacities 1.0 mgd at 30 ft total head, 4.7 mgd at 32 ft total head, and 5.0 mgd at 32 ft total head	.....

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Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
95	3,520 ft of 42-in. BR at 0.10%, includes 1,800 ft of tunnel	16
96	Sandcatcher	.....
97	1,270 ft of 18-in. at 0.14 - 0.69%	2.6
98	1,290 ft of 72-in. BI at 0.074 - 0.084%, includes 200 ft of 72-in. RC and 36-in. CI overflow outfall	74
99	1,290 ft of 72-in. BI at 0.055% - 0.065%	64
100	430 ft of 48-in. BI at 0.12%	32
101	1,650 ft of 48-in. BI at 0.10 - 0.12%	29
102	630 ft of 42-in. BI at 0.13 - 0.14%	24
103	400 ft of 15-in. conc at 0.68 - 1.5%, includes 30-in. overflow to creek on 38th Avenue between Conover Way and Alaska Street	3.5
PS-8	Pumping station, discharges to facility 96 through 100 ft of 10-in. CI force main, includes 430 ft of 16-in. CI emergency overflow and stormwater bypass outfall. Contains 2 pumps; capacities 1.2 mgd at 15-ft total head and 1.5 mgd at 16 ft total head	.....
104	1,440 ft of 21-in. conc at 0.10%	3.2
105	320 ft of 15-in. conc at 0.42%	2.7
106	3,260 ft of 15-in. conc at 0.17 - 0.18%	1.7
PS-9	Pumping station, includes 50 ft of 8-in. CI force main. Contains one pump having a capacity of 0.58 mgd at 14 ft total head	.....
Henderson - East Marginal Way System		
STP	Sewage treatment plant, primary type. Design capacity 8 mgd at 2 hours detention. Discharges through 30-in. outfall to Duwamish Waterway	.....
107	3,300 ft of 60-in. RC with VC liner at 0.055%	39
108	3,700 ft of 60-in. RC with VC liner at 0.05%	37
109	Flow regulator and overflow, designed to divert 6.5 mgd from sewer 147 to sewer 108, includes 30 ft of 24-in. connecting pipe	.....
110	4,320 ft of 42-in. RC at 0.12%; discharges to sewer 147	23
PS-10	Pumping station, includes short length of 20-in. force main. Contains 2 pumps; capacities 2.4 mgd at 18 ft total head and 2.6 mgd at 18 ft total head. Emergency overflow and stormwater bypass from adjacent manhole through 350 ft of 36-in. conc outfall	.....
111	9,200 ft of 42-in. RC at 0.10%	21
112	20-ft long side weir overflow and 1,490 ft of 42-in. conc overflow outfall at 0.2 - 3.2%. Weir crest set at crown of interceptor	.....
113	1,080 ft of 42-in. RC at 0.10%	21
114	Side weir overflow, 20 ft of twin 24-in. CI at 0.178%, 75 ft of 36-in. conc at 0.178%, 60 ft of 42-in. conc at 0.10% and 1,560 ft of 84-in. conc overflow outfall. Overflow starts when flow reaches 19 mgd. Weir capacity, greater than 110 mgd	.....
115	680 ft of 84-in. RC at 0.108%	133
116	3,090 ft of 72-in. RC at 0.103%	87
117	650 ft of 60-in. RC at 0.21%	76
118	680 ft of 60-in. RC at 0.15%	64
119	2,250 ft of 60-in. RC at 0.09 - 0.10%	53
120	2,080 ft of 42-in. RC at 0.20 - 0.23% and 48-in. BR at 0.12 - 0.13% and 250 ft of parallel 30-in. VC at 0.35% and 24-in. VC at 0.87%	30
121	Junction chamber receiving overflows from facilities 134 and 136 and 1,300 ft of 84-in. RC overflow outfall	.....
122	1,230 ft of 72-in. BI at 0.55 - 0.65% and 60 ft of 48-in. RC at 2.22%, overflow; discharges to facility 121	200

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Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
123	860 ft of 66-in. BI at 0.65 - 0.7%, overflow	173
124	490 ft of 60-in. BR at 1.0%, overflow	145
PS-11	Pumping station, discharges to sewer 120 through 1,290 ft of twin 15-in. CI force mains. Contains 2 pumps; capacities 2.4 mgd at 20 ft total head and 2.8 mgd at 21 ft total head. Emergency overflow and stormwater bypass from adjacent manhole with discharge to sewer 121	.....
125	1,900 ft of 21-in. conc at 0.16 - 0.2%	4.1
PS-12	Pumping station, includes 10 ft of 10-in. CI force main. Contains 2 pumps; capacities 1.5 mgd at 16 ft total head and 1.6 mgd at 18 ft total head	.....
126	3,230 ft of 21-in. conc at 0.14 - 0.15%	3.8
127	1,800 ft of 18-in. conc at 0.14%	2.6
PS-13	Pumping station, includes 10 ft of 8-in. CI force main. Contains 2 pumps; capacities 1.0 mgd at 18 ft total head and 1.1 mgd at 18 ft total head. Emergency overflow and stormwater bypass from adjacent manhole through 350 ft of 16-in. wood stave outfall. Tributary sewer also overflows to outfall	.....
128	2,810 ft of 15-in. conc at 0.17%	1.7
129	Overflow manhole and 680 ft of 24-in. conc and wood stave outfall	.....
130	2,540 ft of 24-in. conc at 0.1% and 330 ft of 30-in. at 0.64%	4.6
131	4,420 ft of 18-in. conc at 0.19%, discharges to PS-11	3.0
132	Overflow manhole and 90 ft of 15-in. and 16-in. outfall	.....
133	1,220 ft of 15-in. VC at 0.45%	2.8
134	20 ft long side weir overflow, discharges to sewer 121. Overflow starts when flow reaches 3 mgd. Weir capacity, greater than 77 mgd	.....
135	1,220 ft of 48-in. BI at 0.7%	77
136	30 ft long side weir overflow, discharges to sewer 121. Overflow starts when flow reaches 4 mgd. Weir capacity greater than 93 mgd	.....
137	1,570 ft of 78-in. BI at 0.075%	93
138	1,410 ft of 78-in. BI at 0.055 - 0.066%	85
139	760 ft of 72-in. BI at 0.14%	100
140	30 ft long side weir overflow, discharges to sewer 124. Sanitary flow discharges to sewer 119 through 18-in. sewer. Overflow starts when flow reaches 17 mgd. Weir capacity greater than 145 mgd	.....
141	2,910 ft of 60-in. BR and BI at 0.9 - 1.62%	155
142	2,040 ft of 60-in. BI at 0.5 - 0.65%	130
143	2,260 ft of 24-in. RC at 0.10 - 0.18%	4.6
144	1,170 ft of 30-in. RC at 0.12 - 0.18%	9.0
145	450 ft of 24-in. RC at 0.07 - 0.36%	5.0
146	1,110 ft of 70-in. by 102-in. BI outfall at 0.075%, from facility 109	110
147	2,110 ft of 65-in. by 96-in. BI at 0.052 - 0.067%	83
148	2,870 ft of 50-in. by 78-in. BI at 0.075 - 0.10%	48
149	740 ft of 50-in. by 78-in., BI at 0.10%	53
150	1,740 ft of 42-in. BI at 0.11%	22
151	860 ft of 50-in. by 78-in. BI outfall at 0.085%, from facility 152	49
152	Flow regulator and overflow, designed to divert 1.5 mgd from sewer 153 to sewer 107	.....
153	330 ft of 40-in. by 66-in. BI at 0.11%	31
154	510 ft of 30-in. conc at 0.13%	10
155	230 ft of 48-in. RC culvert, discharges to slough	.....

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Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
156	1,020 ft of 42-in. steel at 0.04%	13
157	350 ft of roadside ditch	.....
158	250 ft of 84-in. RC at 0.033%	74
159	1,900 ft of 60-in. RC at 0.039 - 0.053%	33
160	670 ft of 54-in. RC at 0.058 - 0.089%	31
161	1,150 ft of 48-in. RC at 0.079 - 0.084%	27
PS-14	Pumping station, discharges to sewer 160 through 1,140 ft of 18-in. conc force main and 480 ft of 24-in. conc at 0.15 - 0.23%. Contains 3 pumps; capacities 4.3 mgd at 28 ft total head, 4.8 mgd at 27 ft total head, and 4.8 mgd at 28 ft total head	.....
West Seattle Systems		
STP	Sewage treatment plant, primary type. Design capacity of 60 mgd wet weather flow at 30 minutes detention. Discharges to water depth of 85 ft in Puget Sound through 1,400 ft of 42-in. RC outfall	.....
PS-15	Pumping station, discharges to STP through 1,200 ft of parallel 24-in. and 42-in. force mains. Emergency overflow and stormwater bypass through 1,100 ft of 66-in. outfall	60
162	1,410 ft of 30-in. RC at 0.45%	18
PS-16	Pumping station, includes 4,440 ft of 24-in. force main and 560 ft of 60-in. overflow outfall	17
163	6,890 ft of 42-in. RC at 0.052%	15
PS-17	Pumping station, includes 2,600 ft of 21-in. force main and 480 ft of 42-in. overflow outfall	9.0
164	4,850 ft of 54-in. RC at 0.11%	42
165	2,000 ft of 54-in. pressure sewer	.....
166	4,740 ft of 36-in. pressure sewer	.....
PS-18	Pumping station, includes 1,400 ft of twin 27-in. force mains and 800 ft of 72-in. overflow outfall	39
167	840 ft of 42-in. RC at 0.25%	33
PS-19	Pumping station, includes 6,250 ft of 30-in. force main and 620 ft of 60-in. overflow outfall	23
168	1,540 ft of 42-in. conc at 0.11% and 70 ft of 24-in. conc and CI outfall	24
169	1,540 ft of 42-in. conc at 0.11% and 160 ft of 24-in. conc and CI outfall	24
170	2,840 ft of 18-in. conc at 1.8 - 16.1%. Outfall data unavailable	22
171	1,180 ft of 30-in. conc at 0.07% and 17-in. by 20-in. wood box outfall	7.0
172	860 ft of 24-in. at 0.19 - 0.448%	6.5
173	1,080 ft of 30-in. conc at 0.095%	8.0
174	Overflow, 700 ft of 24-in. BR	.....
175	700 ft of 24-in. conc at 3.78% and septic tank. Discharge from septic tank to Longfellow Creek through 30-in. and 36-in. culvert	28
176	1,450 ft of 12-in. conc at 5.5 - 22%, stormwater overflow from pumping station at West Webster and 28th SW to Longfellow Creek	8.5
177	650 ft of 30-in. conc at 3.9 - 10.6% and 14-in. wood stave outfall	65
Lake Union Tunnel System		
178	5,600 ft of 72-in. BR at 0.08%, includes 5,300 ft of tunnel. Discharges through 48-in. CI submarine outfall at low flows and through 60-in. RC outfall at shore line at high flows	66
179	330 ft of 32-in. by 48-in. BR oviform at 3.02%	69
180	1,540 ft of 28-in. by 42-in. BR oviform at 2.74 - 8.77%	44
181	830 ft of 28-in. by 42-in. BR oviform at 2.22%. Overflow manhole on sewer 23 diverts up to 10 mgd to sewer 181	40
182	1,580 ft of 24-in. by 36-in. BR oviform at 0.15%	7.0
183	1,830 ft of 21-in. and 24-in. VC at 0.15%	4.0

Continued on next page

Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
184	2,490 ft of 24-in. by 36-in. BR and 21-in. VC at 0.15%	4.0
185	1,240 ft of 24-in. by 36-in. BR oviform and 24-in VC at 0.15%	5.7
186	1,370 ft of 21-in VC at 0.15 - 0.18%	4.0
187	1,350 ft of 15-in. and 18-in. VC at 0.18 - 0.4%	2.9
188	1,000 ft of 21-in. and 24-in. VC at 0.33 - 1.47%	8.5
189	790 ft of 18-in. VC at 0.19%	3.0
190	1,150 ft of 15-in. VC at 0.25 - 0.4%	2.1
191	620 ft of 12-in. VC at 0.4%	1.5
Elliott Bay Independent Systems		
192	800 ft of 96-in. BI at 0.12%. Discharges through 180 ft of 20-in. CI submarine outfall at low flow and over spillway at shore line at high flow	200
193	1,640 ft of 90-in. BI at 0.12 - 0.15%	180
194	1,020 ft of 90-in. BI at 0.10 - 0.11%	160
195	760 ft of 84-in. BI at 0.13%	150
196	1,130 ft of 84-in. BI at 0.10%	130
197	500 ft of 72-in. BI at 0.14%	100
198	410 ft of 54-in. BI at 1.0%	125
199	4,450 ft of 48-in. by 72-in. BR oviform abandoned tunnel at 0.286%. Tunnel gated at east portal. Reportedly receives flow from west slope of Beacon Hill through drop manholes	70
200	880 ft of 72-in. BI at 0.175%. Discharges through 54-in. RC submarine outfall at low flow and through 72-in. outfall at shore line at high flow	110
201	490 ft of 66-in. BR at 0.177%	77
202	960 ft of 72-in. BI at 0.15%	105
203	770 ft of 60-in. BI at 0.13%	60
204	2,560 ft of 48-in. BI at 0.13%	33
205	1,430 ft of 48-in. BR and BI at 0.33 - 0.38%. Outfall data unavailable	48
206	610 ft of 32-in. by 48-in. BR oviform at 0.33%	22
207	320 ft of 28-in. by 42-in. BR oviform at 0.32%	15
208	920 ft of 22-in. by 33-in. BR oviform at 0.66%	11
209	1,140 ft of 42-in. RC at 5.0 - 5.2%. Discharges through 760 ft of 16-in submarine outfall at low flow and through 24-in. outfall at shore line at high flow	147
North Beach System		
STP	Sewage treatment plant, primary type. Design capacity 0.43 mgd at 2 hours detention. Discharges through 1,200 ft of outfall, including 1,000 ft submarine section, to Puget Sound	.....
210	770 ft of 18-in. conc at 1.65 - 3.26%	8.8
211	1,010 ft of 15-in. conc at 5.53 - 8.6%	13
212	200 ft of 24-in. conc at 1.03%	15
213	1,510 ft of 18-in. conc at 1.71 - 6.63%	9.0
Blue Ridge System		
214	1,050 ft of 10-in. CI outfall	.....
Greenwood Avenue System		
STP	Sewage treatment plant, primary type, designed to remove grease and floatable matter only. Discharges through 3,200 ft of outfall, including 1,200 ft submarine section, to Puget Sound	.....
215	500 ft of 24-in. conc at 1.59%	18

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Table 6-21. Continued

Facility[a]	Description[b,c]	Capacity,[d] mgd
216	3,000 ft of 12-in. conc at 3.3 - 20.8%	5.0
217	310 ft of 21-in. conc at 2.52%	16
218	2,640 ft of 15-in. and 18-in. conc at 2.04 - 27.6%	9.6
219	1,430 ft of 21-in. conc at 0.28 - 0.37%	5.4
Lake City System		
STP	Sewage treatment plant, secondary type. Discharges to Lake Washington through 2,440 ft of 72-in. RC at 0.3% and 1,410 ft of 90-in. horseshoe tunnel at 0.3%. Discharge through 30-in. submarine outfall at low flow and through 42-in. and 48-in. submarine outfalls at high flow. Diversion structure at treatment plant provides for overflowing Thornton Creek to outfall at high creek levels	10
220	1,650 ft of 36-in. conc at 0.45%, includes 70 ft inverted siphon across Thornton Creek	29
221	2,670 ft of 36-in. conc at 0.90 - 2.66%	25
222	960 ft of 42-in. conc at 0.3%	36
223	1,220 ft of 30-in. conc at 2.12%	38
224	2,610 ft of 30-in. conc at 1.5 - 1.9%	36
225	2,810 ft of 27-in. and 30-in. conc at 1.74 - 13.5%	26
226	1,330 ft of 30-in. conc at 0.8%	24
227	1,940 ft of 27-in. and 30-in. conc at 0.6 - 2.4%	21
228	6,090 ft of 30-in. at 0.5 - 3.6%	19
229	12,000 ft of 8-in. to 12-in. lake front gravity interceptor with intermediate lift stations	.....
PS-20	Pumping station, discharges to STP through 4,200 ft of 14-in. force main. Contains 3 pumps; capacities 1.0 mgd at 38 ft total head, 1.1 mgd at 37 ft total head, and 1.1 mgd at 40 ft total head	.....

[a]*See Figs. 6-29 to 6-37 for location of facilities.*

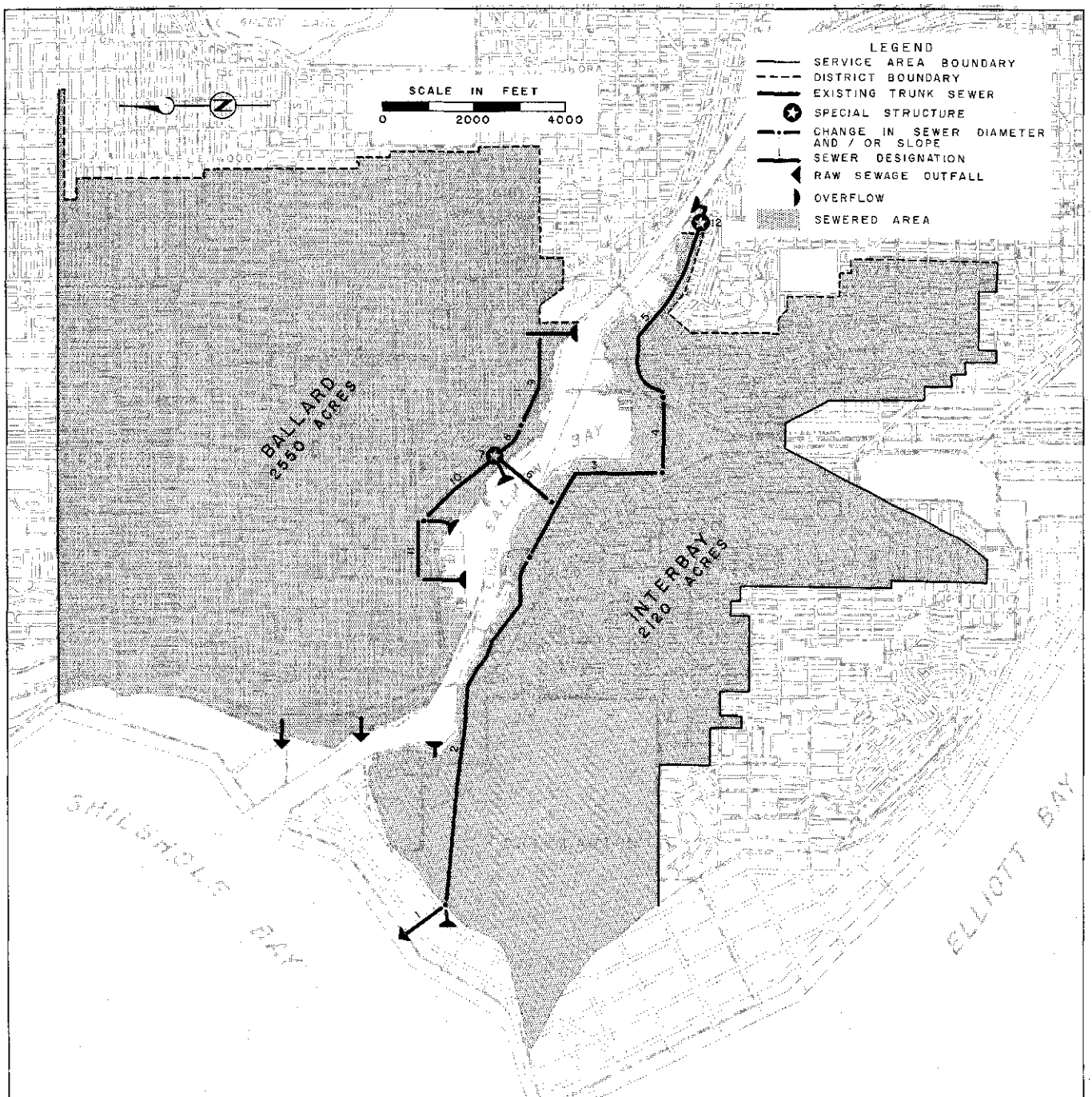
[b]*BR, brick; n equals 0.017 for 42-in. and smaller, n equals 0.015 over 42-in. Conc, concrete; n equals 0.013. Bl, concrete with brick invert; n equals 0.013. RC, reinforced concrete; n equals 0.013. VC, vitrified clay; n equals 0.013. CI, cast iron; c equals 100. CMP, corrugated metal pipe; n equals 0.017.*

[c]*All lengths rounded off to nearest 10 feet.*

[d]*Flowing full. Where size, slope or type of material varies within sections, capacity determined either for limiting condition or for minimum surcharging where physical conditions permit. For cross sections other than circular, capacity calculated for equivalent circular section.*

[e]*At mean sea level with approach sewer flowing full.*

[f]*With sewer 3 flowing full.*

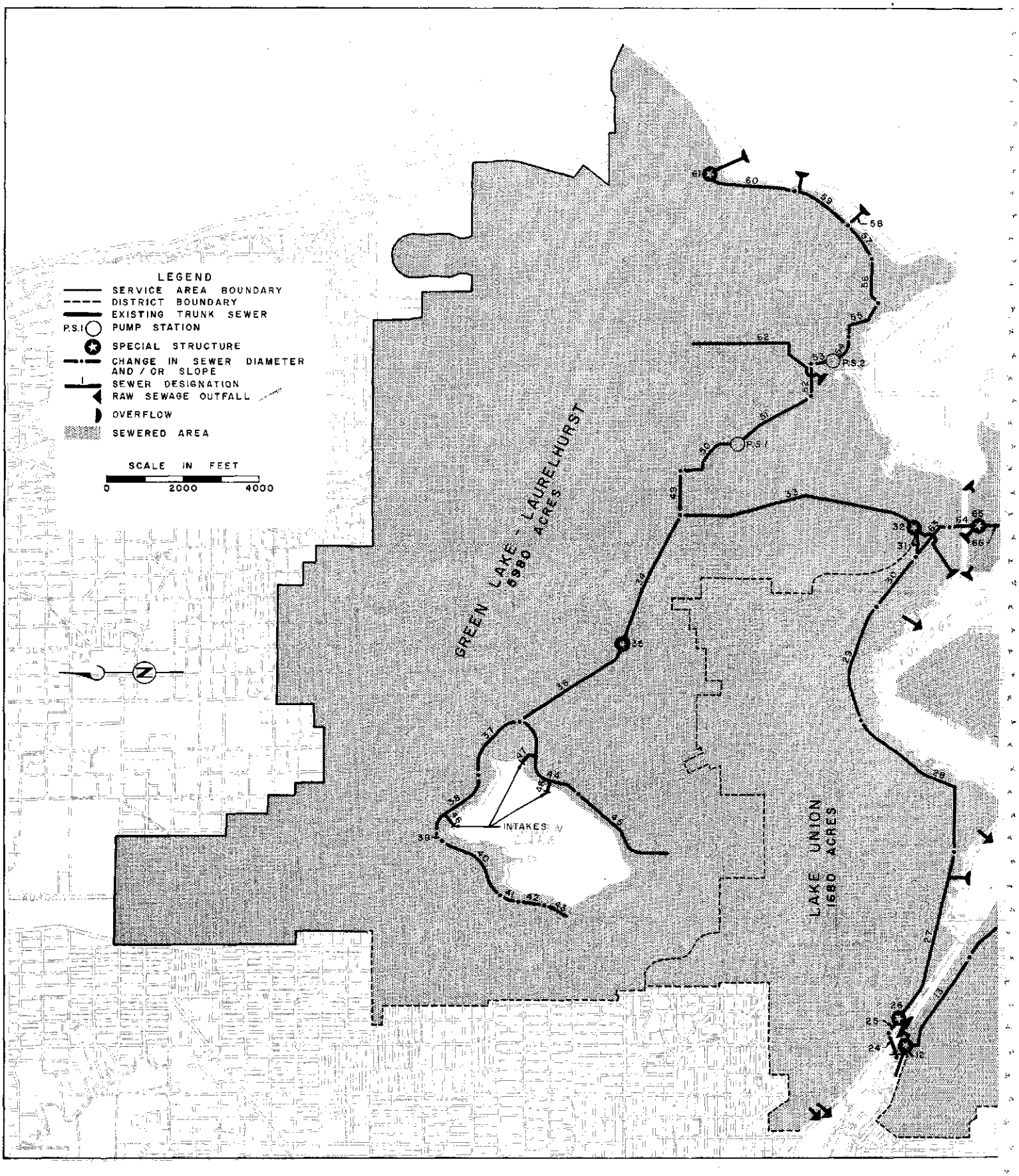


Trunk system terminates with a tunnel 200 feet in depth under Fort Lawton (No. 2) and an outfall extending 1,200 feet into Shilshole Bay to a depth of 40 feet (No. 1). Flows in excess of the outfall capacity are discharged to the beach over a weir in the end of the tunnel. Lateral sewers serving the Interbay district discharge directly to the trunk, while flow from the Ballard district is intercepted along the north side of the waterway and conveyed to the trunk through twin siphons (No. 6).

Flow from a small area along the west side of Ballard is discharged to Puget Sound through independent outfalls. A waterfront interceptor, soon to be constructed, will divert this flow to a point below the Government Locks where a siphon will convey it beneath the waterway to the North Trunk.

The North Trunk system also serves an additional 12,410 acres to the east and south (Fig. 6-29). All numbered sections are described in Table 6-21.

Fig. 6-28. North Trunk System - Interbay and Ballard Districts


LEGEND
SERVICE AREA BOUNDARY
DISTRICT BOUNDARY
EXISTING TRUNK SEWER
P.S.1 PUMP STATION
SPECIAL STRUCTURE
CHANGE IN SEWER DIAMETER AND / OR SLOPE
SEWER DESIGNATION
RAW SEWAGE OUTFALL
OVERFLOW
SEWERED AREA
SCALE IN FEET
0 2000 4000
GREEN LAKE - LAURELHURST
6980 ACRES
LAKE UNION
1680 ACRES
INTAKES
P.S.1
P.S.2

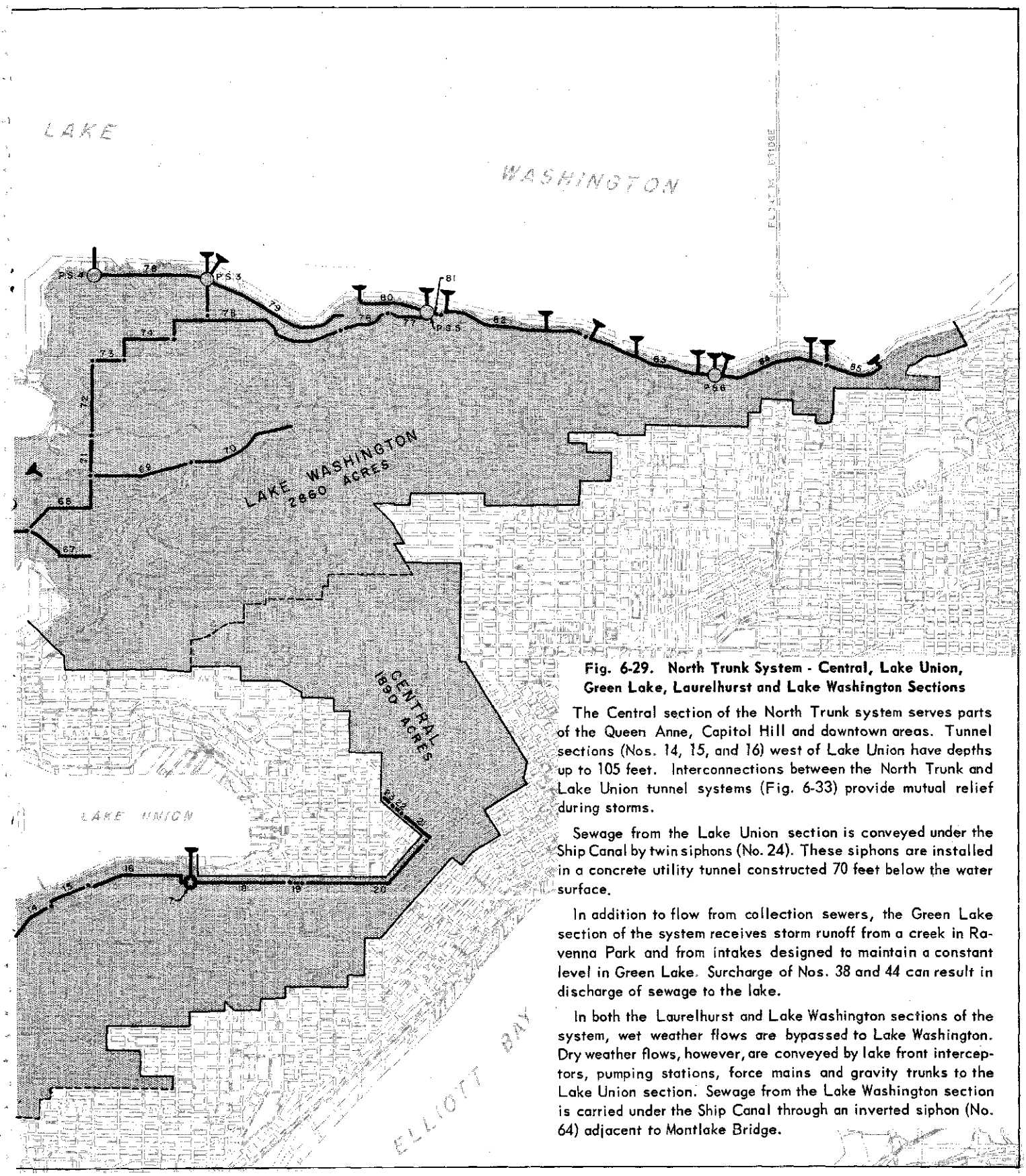


Fig. 6-29. North Trunk System - Central, Lake Union, Green Lake, Laurelhurst and Lake Washington Sections

The Central section of the North Trunk system serves parts of the Queen Anne, Capitol Hill and downtown areas. Tunnel sections (Nos. 14, 15, and 16) west of Lake Union have depths up to 105 feet. Interconnections between the North Trunk and Lake Union tunnel systems (Fig. 6-33) provide mutual relief during storms.

Sewage from the Lake Union section is conveyed under the Ship Canal by twin siphons (No. 24). These siphons are installed in a concrete utility tunnel constructed 70 feet below the water surface.

In addition to flow from collection sewers, the Green Lake section of the system receives storm runoff from a creek in Ravenna Park and from intakes designed to maintain a constant level in Green Lake. Surcharge of Nos. 38 and 44 can result in discharge of sewage to the lake.

In both the Laurelhurst and Lake Washington sections of the system, wet weather flows are bypassed to Lake Washington. Dry weather flows, however, are conveyed by lake front interceptors, pumping stations, force mains and gravity trunks to the Lake Union section. Sewage from the Lake Washington section is carried under the Ship Canal through an inverted siphon (No. 64) adjacent to Montlake Bridge.

This system diverts sewage and storm runoff from the Lake Washington watershed to the East Waterway of Duwamish River. Dry weather flow is intercepted along the lake front and pumped (PS 8) to a tunnel (part of No. 95) under Charlestown Street. From here it is conveyed by gravity to PS 7 which lifts it into the Rainier Avenue trunk.

Flows from the Rainier valley area converge at Hanford Street and are conveyed through a tunnel under Beacon Hill (part of No. 88) and thence to the waterway. Prior to construction of the Lake Washington interceptor and the Hanford tunnel, the flow from Rainier Avenue was diverted through a tunnel under Bayview Street (see arrow). This tunnel was abandoned because of insufficient capacity and concern regarding its possible collapse. Should separation of the Rainier-Hanford system become necessary, the Bayview tunnel could be repaired and utilized as a storm drain.

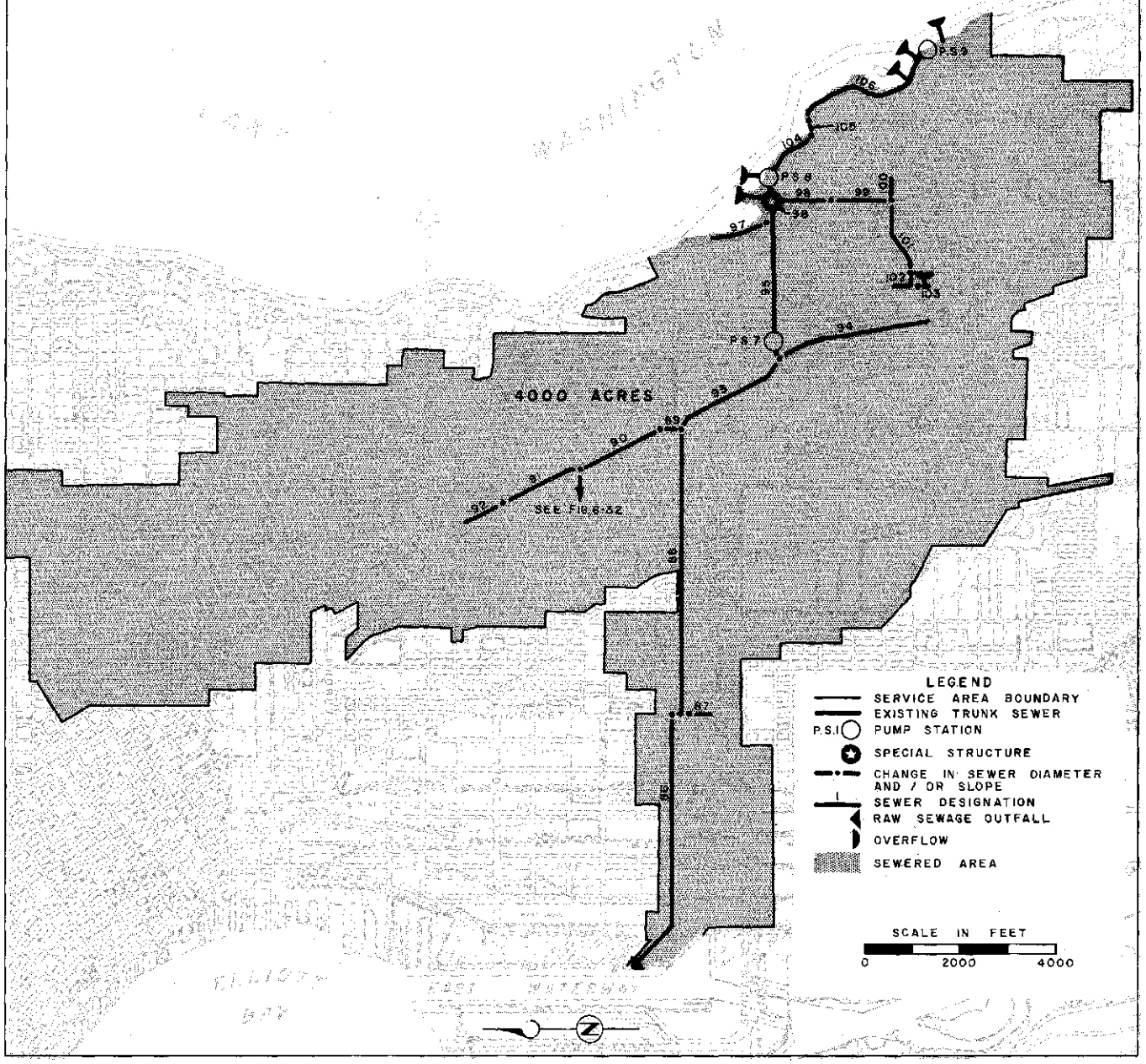


Fig. 6-30. Rainier - Hanford System

Dry weather flows are conveyed from the Lake Washington watershed around the south end of Beacon Hill to East Marginal Way. Flows from parts of the heavily industrialized Duwamish valley, including 470 acres outside the city, are intercepted along East Marginal Way and discharged into the Michigan Street trunk sewer (No. 147). Here a regulating station (No. 109) diverts up to 6.5 mgd through Nos. 108 and 107 to the Diagonal Avenue treatment plant. Since sections of the intercepting sewer have only about 10 per cent of the capacity of tributary trunks, up to 90 per cent of the wet weather flow is bypassed at various locations. Three small systems, as well as a number of industries, discharge directly to the Duwamish.

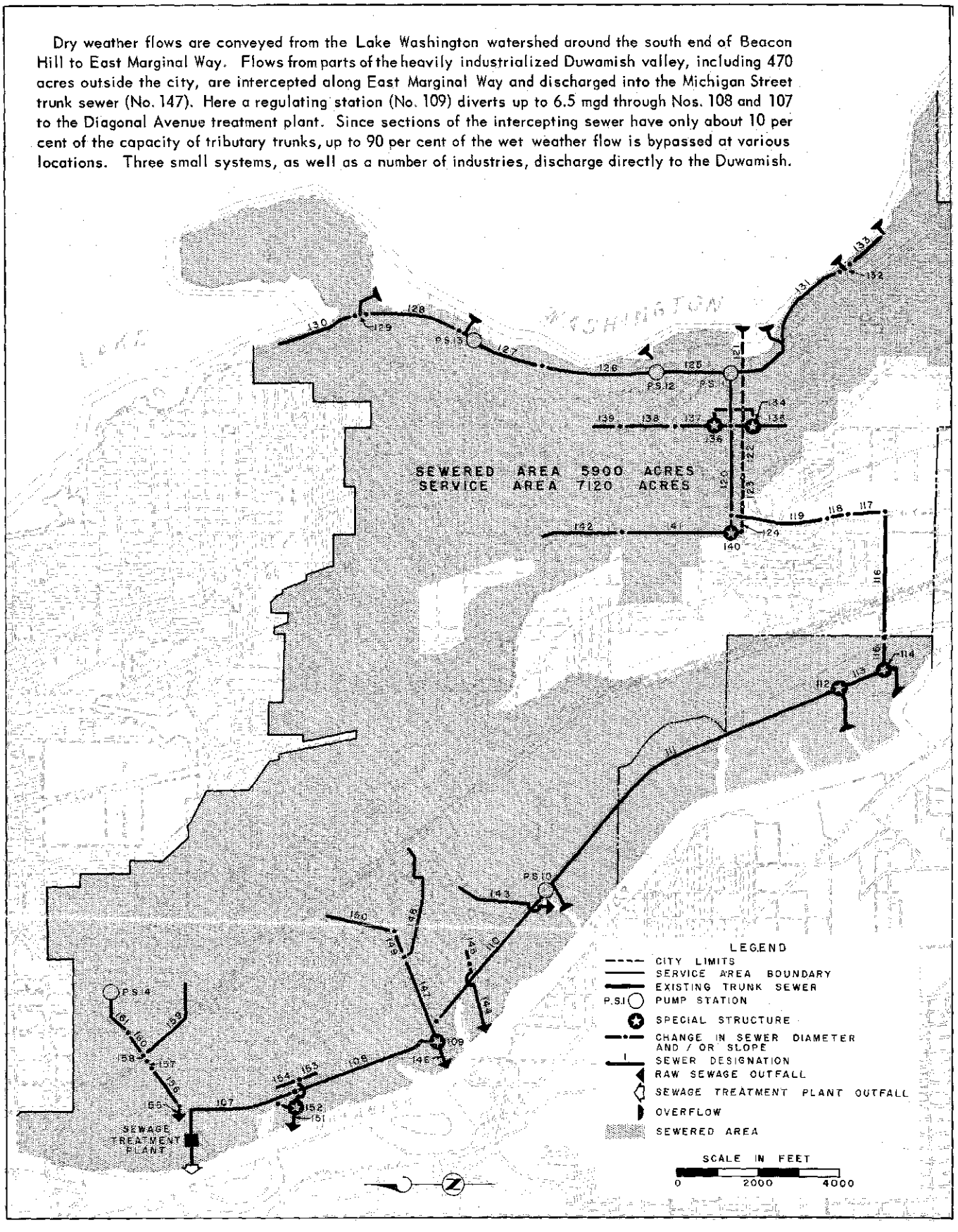


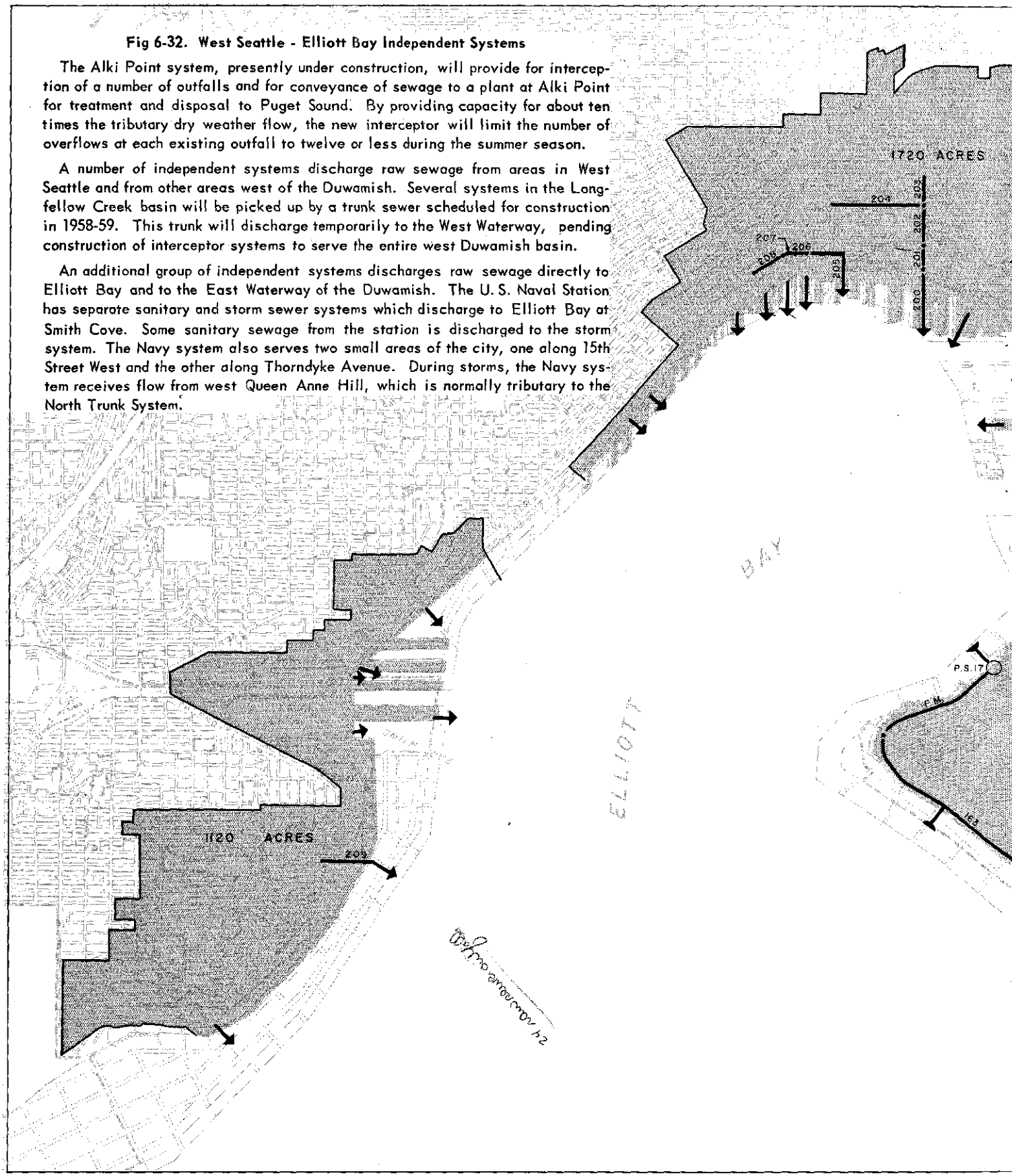
Fig. 6-31. Henderson - East Marginal Way System

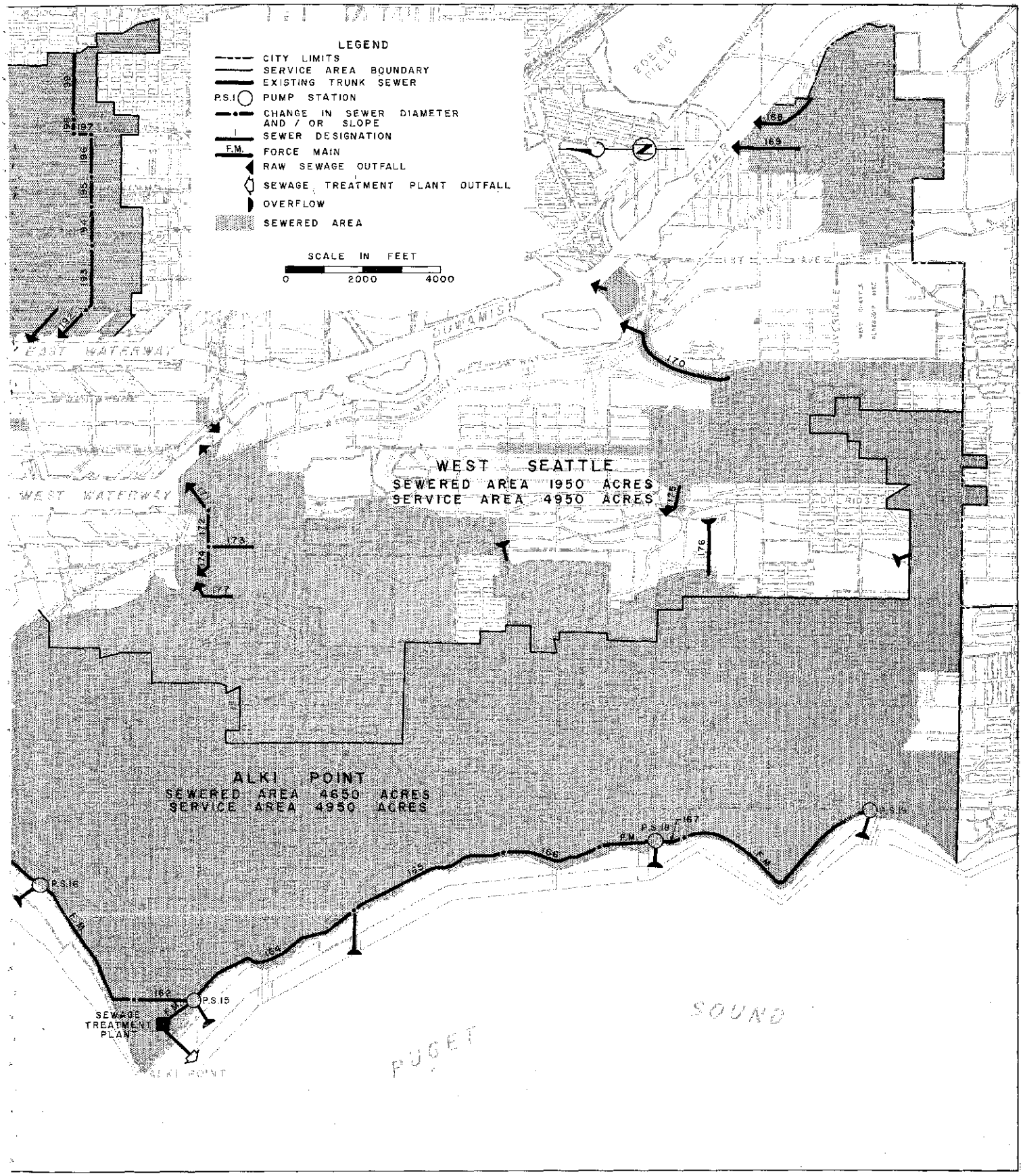
Fig 6-32. West Seattle - Elliott Bay Independent Systems

The Alki Point system, presently under construction, will provide for interception of a number of outfalls and for conveyance of sewage to a plant at Alki Point for treatment and disposal to Puget Sound. By providing capacity for about ten times the tributary dry weather flow, the new interceptor will limit the number of overflows at each existing outfall to twelve or less during the summer season.

A number of independent systems discharge raw sewage from areas in West Seattle and from other areas west of the Duwamish. Several systems in the Longfellow Creek basin will be picked up by a trunk sewer scheduled for construction in 1958-59. This trunk will discharge temporarily to the West Waterway, pending construction of interceptor systems to serve the entire west Duwamish basin.

An additional group of independent systems discharges raw sewage directly to Elliott Bay and to the East Waterway of the Duwamish. The U. S. Naval Station has separate sanitary and storm sewer systems which discharge to Elliott Bay at Smith Cove. Some sanitary sewage from the station is discharged to the storm system. The Navy system also serves two small areas of the city, one along 15th Street West and the other along Thorndyke Avenue. During storms, the Navy system receives flow from west Queen Anne Hill, which is normally tributary to the North Trunk System.




LEGEND
CITY LIMITS
SERVICE AREA BOUNDARY
EXISTING TRUNK SEWER
P.S.1 PUMP STATION
CHANGE IN SEWER DIAMETER AND / OR SLOPE
SEWER DESIGNATION
F.M. FORCE MAIN
RAW SEWAGE OUTFALL
SEWAGE TREATMENT PLANT OUTFALL
OVERFLOW
SEWERED AREA
SCALE IN FEET
0 2000 4000
BOEING FIELD
DUWAMISH
EAST WATERWAY
WEST WATERWAY
WEST SEATTLE
SEWERED AREA 1950 ACRES
SERVICE AREA 4950 ACRES
ALKI POINT
SEWERED AREA 4650 ACRES
SERVICE AREA 4950 ACRES
P.S.16
P.S.15
P.S.18
P.S.19
F.M.
SEWAGE TREATMENT PLANT
PUGET SOUND
ALKI POINT
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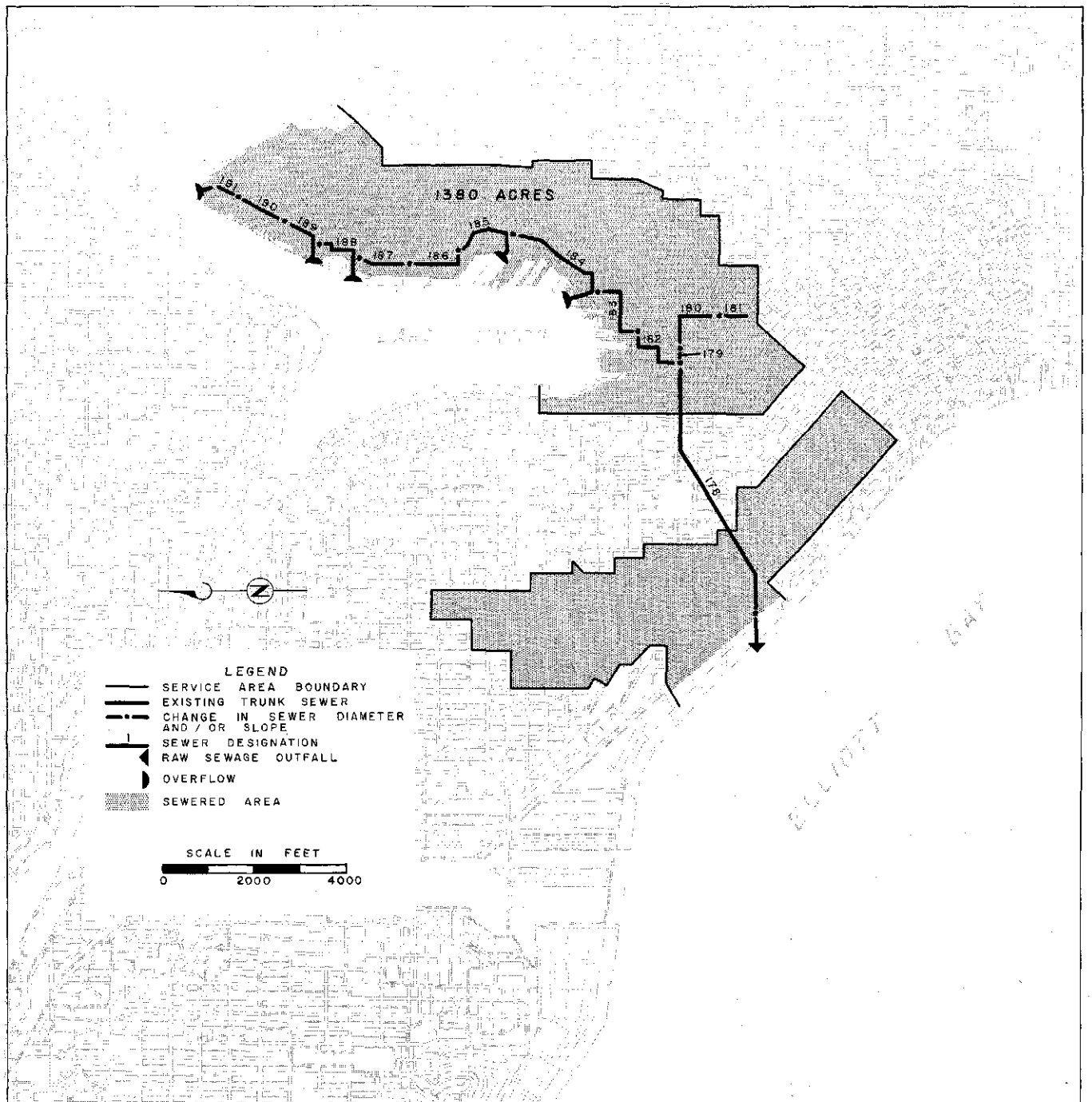
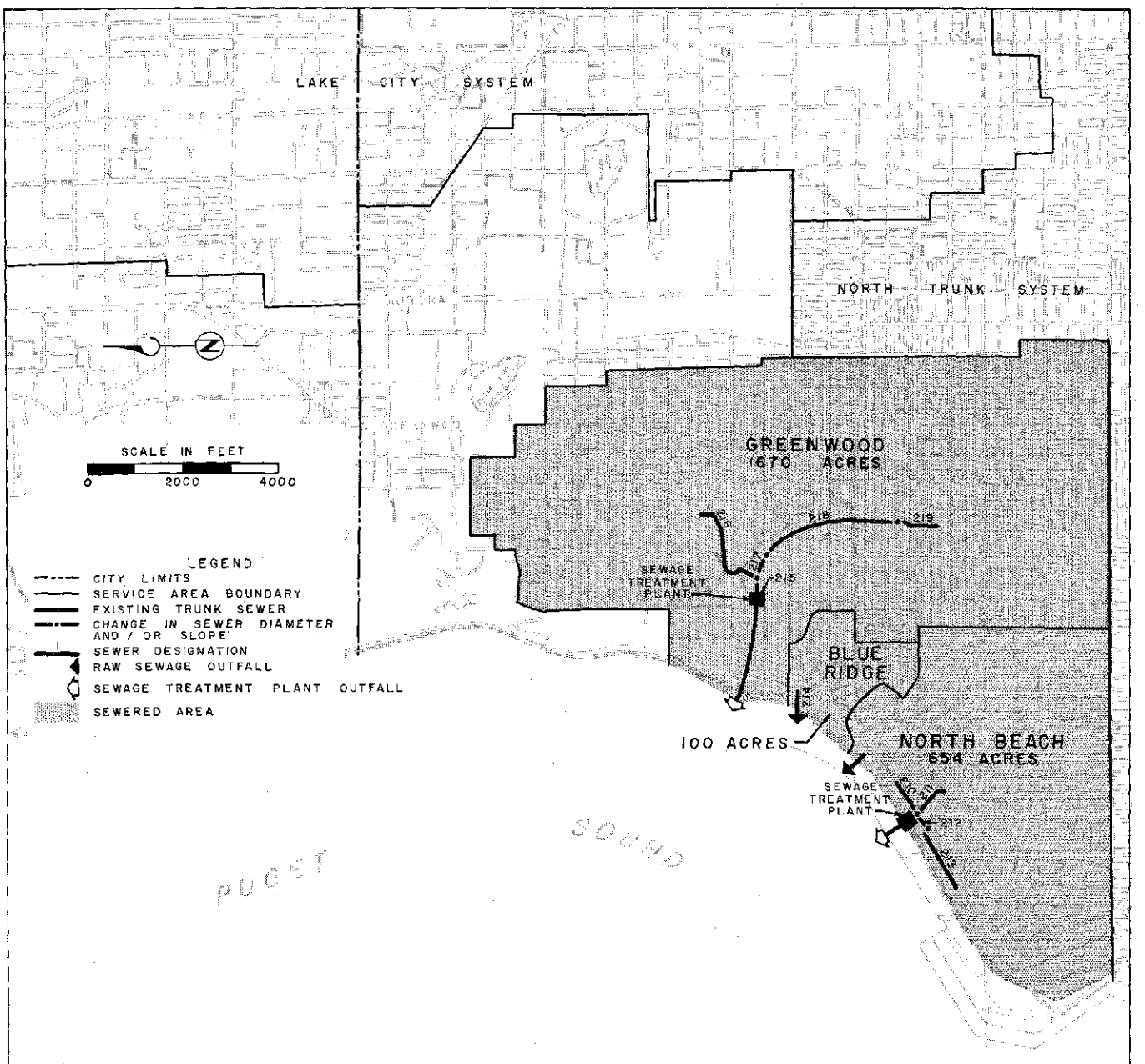


This system was constructed in 1895 to eliminate raw sewage discharges into Lake Union and was the first major sewer project in Seattle. Initially, it served south Queen Anne Hill, Capitol Hill and a large part of the downtown area, but it soon became grossly overloaded and relief was provided by construction of the central section of the North Trunk sewer (Fig. 6-29). At present, the primary function of the Lake Union tunnel system is to divert sewage from the low areas around the lake and from a small downtown section and to convey it to Elliott Bay through the tunnel (No. 178). This system also serves an area lying along the west slope of the divide between Lake Union and Elliott Bay. In the downtown area, there are interconnections with the central section of the North Trunk system (Fig. 6-29).

Frequent backups of sewage are reported to occur at the south end of Lake Union and in the vicinity of the east end of the tunnel.

Fig. 6-33. Lake Union Tunnel System



Developed originally as separate sewer districts, these systems were taken over by Seattle in 1954. All three systems have separate sanitary sewers.

Since their bonds had already been redeemed, the North Beach and Blue Ridge districts were dissolved following annexation to Seattle. The Greenwood Avenue Sewer District continues to operate, but only for the purpose of retiring its bonded indebtedness. District officials estimate this will be accomplished about 1965.

Sewage from the North Beach area is given primary treatment, whereas the Greenwood plant provides only for grease and scum removal. Raw sewage from the Blue Ridge District formerly was discharged through three outfalls. Two of these have been intercepted and the tributary flow, except for that from about 10 homes in low areas, is now conveyed to the North Beach treatment plant. The northeasterly portion of the original district is presently served by the Greenwood Avenue system. These changes have reduced the size of the original Blue Ridge District to about 100 acres, from which sewage is still discharged to Puget Sound without treatment (No. 214).

Each of the three systems is subject to considerable ground water infiltration and to direct inflow from roof drains.

Fig. 6-34. Greenwood Avenue, North Beach and Blue Ridge Systems

The Lake City Sewer District was formed in 1946 to provide sewerage service for a large area northeast of Seattle. In 1954, most of the district area was annexed to Seattle. Since then, the city has operated the sewer system, including the portion outside the present city boundary.

Sewage is conveyed to the treatment plant by two principal gravity trunks, which are intended ultimately to serve the entire Thornton Creek basin, and by an intercepting sewer and a series of pumping stations along the lake front. Treatment plant effluent is conveyed to Lake Washington through a tunnel.

Although the Lake City system is designed for sanitary sewage only, excessive infiltration of ground water has resulted in flows which, even during dry weather, consistently exceed the treatment plant design capacity of 2.5 mgd. Peak rates up to 5 times the average dry weather flow are experienced during storms. Plant enlargements now under construction will provide a design capacity of 10.5 mgd and a hydraulic, or peak rate of flow, capacity of 25 mgd.

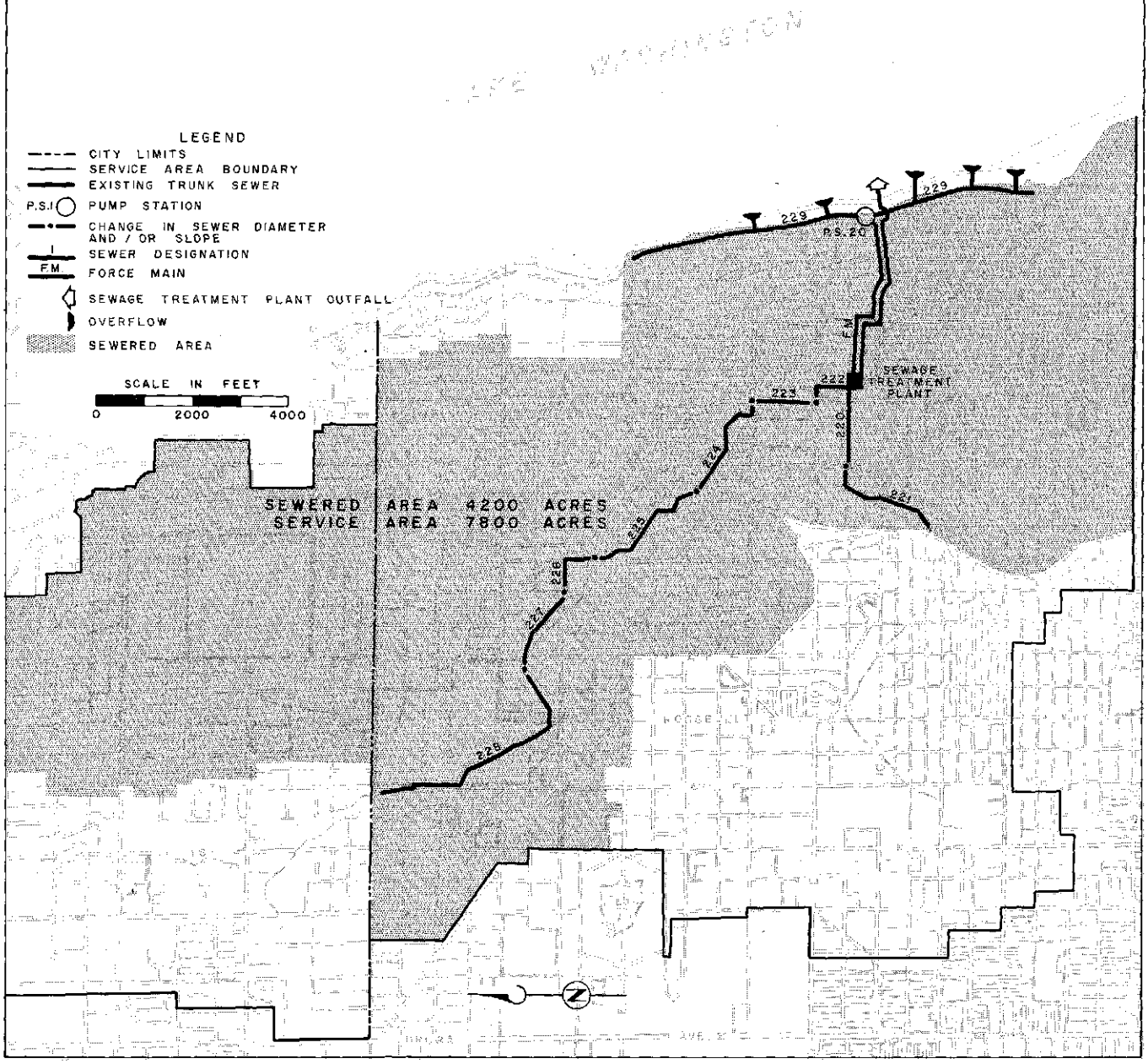


Fig. 6-35. Lake City System

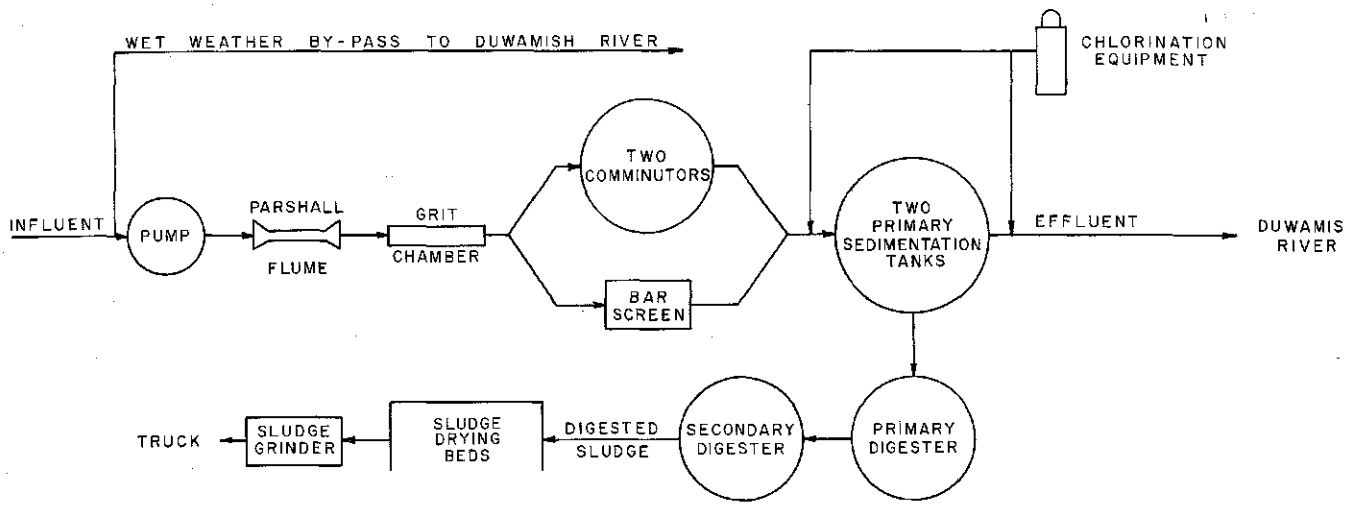


Fig. 6-36. Flow Diagram - Diagonal Avenue Sewage Treatment Plant

The Diagonal Avenue plant provides primary treatment for combined sewage collected by the Henderson - East Marginal Way system of the city of Seattle. The two circular clarifiers have combined capacity for an average flow of 8 mgd. Flow into the plant is limited to the plant capacity by an upstream regulating station which bypasses the excess to Duwamish River during storms. The sludge beds are glass covered in "greenhouse" fashion and dried digested sludge is pulverized and utilized as a soil conditioner at city parks.

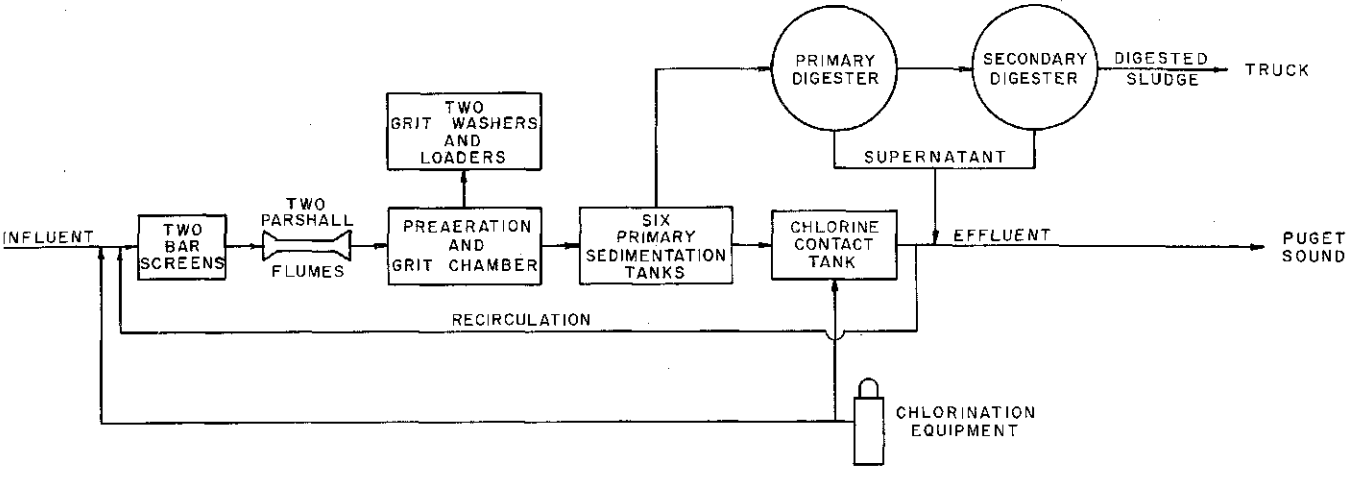


Fig. 6-37. Flow Diagram - Alki Point Sewage Treatment Plant

Presently being constructed, the Alki Point plant will provide primary treatment for West Seattle sewage. The sedimentation tanks are designed to provide 30 minutes detention for storm flows of 60 mgd. During dry weather, the average daily flow is expected to be only 7.5 mgd, and recirculation will be utilized to maintain a detention period of about 5 hours. Because the plant is situated in a residential area, all plant units will be completely housed.

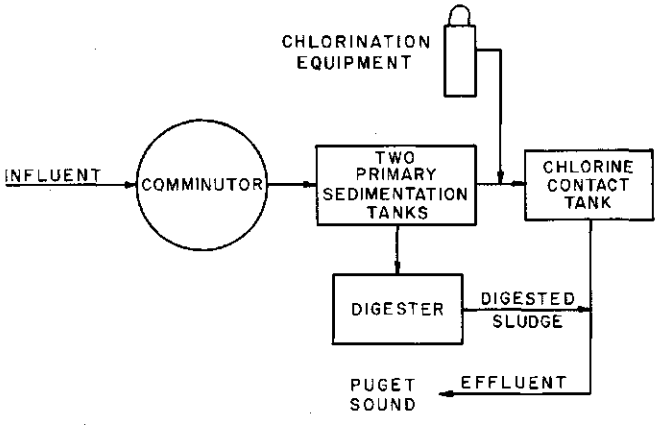


Fig. 6-38. Flow Diagram - North Beach Sewage Treatment Plant

This primary treatment facility is located in a highly developed residential area and for this reason was constructed below the ground surface. A reinforced concrete roof supports a landscaped ground cover. The plant has primary sedimentation capacity for an average daily flow of 0.43 mgd. Digested sludge is discharged to the plant outfall which terminates in 40 feet of water, 1,000 feet offshore.

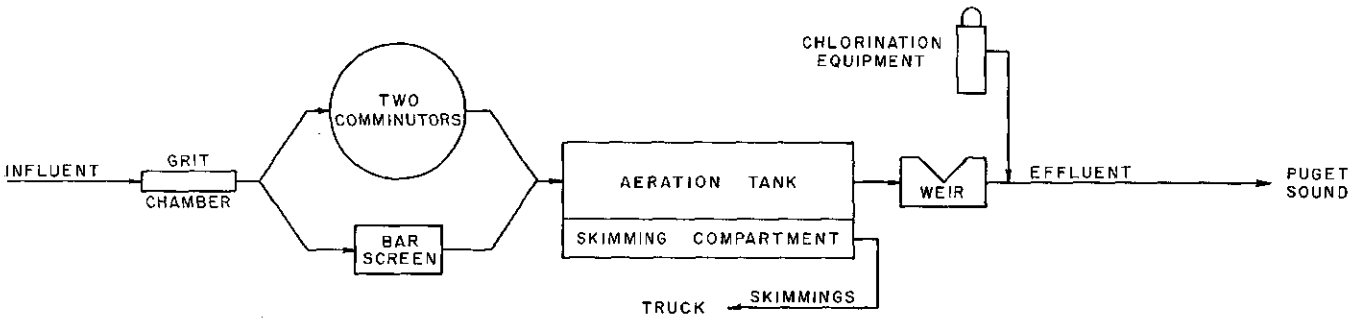


Fig. 6-39. Flow Diagram - Greenwood Avenue Sewage Treatment Plant

Situated in Carkeek Park near the mouth of Piper Creek, this plant serves to remove scum and floatable substances from sewage generated in the Greenwood Avenue Sewer District. The plant was constructed by the district prior to annexation of the area to the city of Seattle in 1954 and, since that time, it has been operated by city forces. Chlorination of the essentially raw sewage effluent does not reduce bacterial densities sufficiently to maintain satisfactory conditions in the public bathing area which extends both north and south of the outfall.

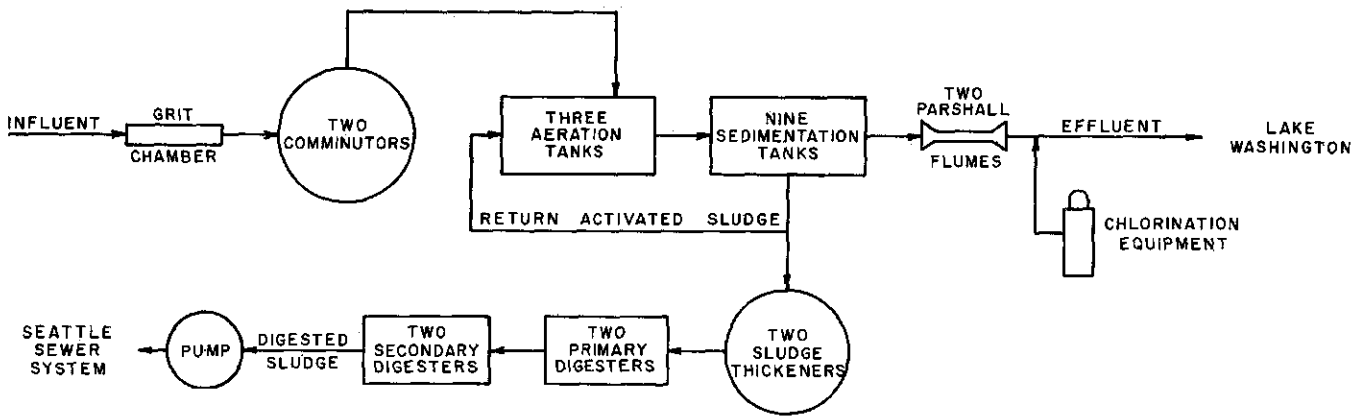


Fig. 6-40. Flow Diagram - Lake City Sewage Treatment Plant

The Lake City sewage treatment plant was constructed by the sewer district prior to annexation of Lake City to Seattle in 1954. City forces have operated the plant since that time. Originally designed for an average daily flow of 2.5 mgd, it soon became grossly overloaded and its capacity is now being increased to 10.0 mgd, which will make it the largest plant in the metropolitan area. As part of this expansion, the standard activated sludge treatment process now being utilized is being modified as shown above.

Chapter 7
SEWAGE CHARACTERISTICS

As a prerequisite to the planning and design of sewerage works, it is necessary to determine the characteristics of the sewage, namely, its volume, its strength, and its composition. Characteristics with respect to volume comprise hourly, daily, and seasonal variations in flow, and are important in that they determine the capacities of collection sewers, pumping stations, treatment units and outfalls. Characteristics with respect to strength and composition, primarily as measured by the suspended solids and biochemical oxygen demand (BOD) determinations, are important in that they exert a controlling influence on the degree of treatment which must be provided in order to produce an effluent of acceptable quality. In other words, a study of the sewage characteristics of a particular community or metropolitan area enables development of the unit quantity and unit load factors which are required for design purposes.

NATURE AND SCOPE OF SEWAGE STUDIES

Field studies undertaken during the present survey included the collection of data pertaining to (1) sewage volume, strength and composition; (2) the location and extent of storm water infiltration; and (3) the type, volume and effect of industrial wastes. Infiltration was studied in detail, with particular reference to the separate sanitary sewers at Lake City where this problem has been a matter of considerable concern. In the course of the field work, all available water supply and sewage treatment plant reports and records were reviewed, and information concerning sewage

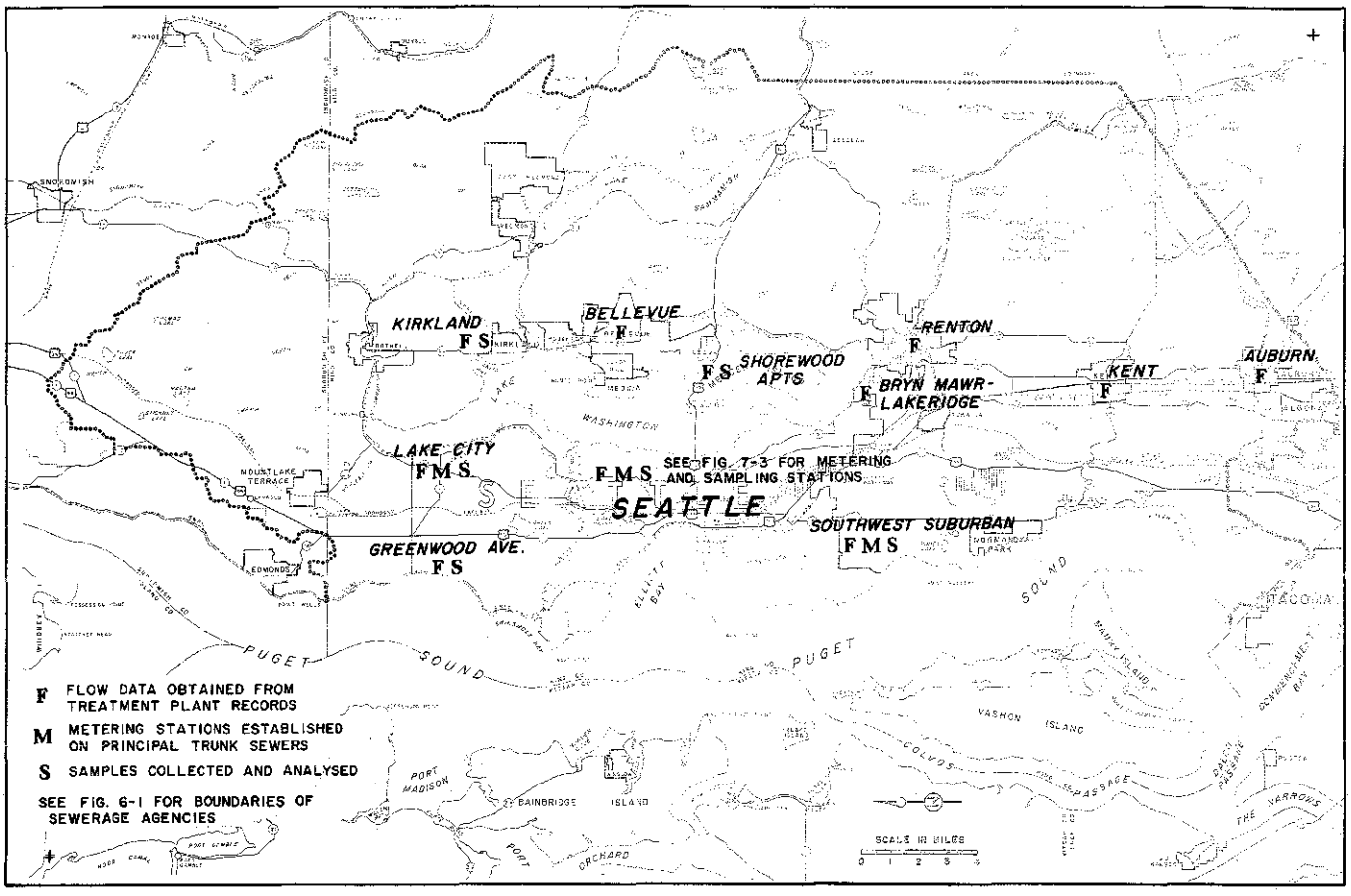
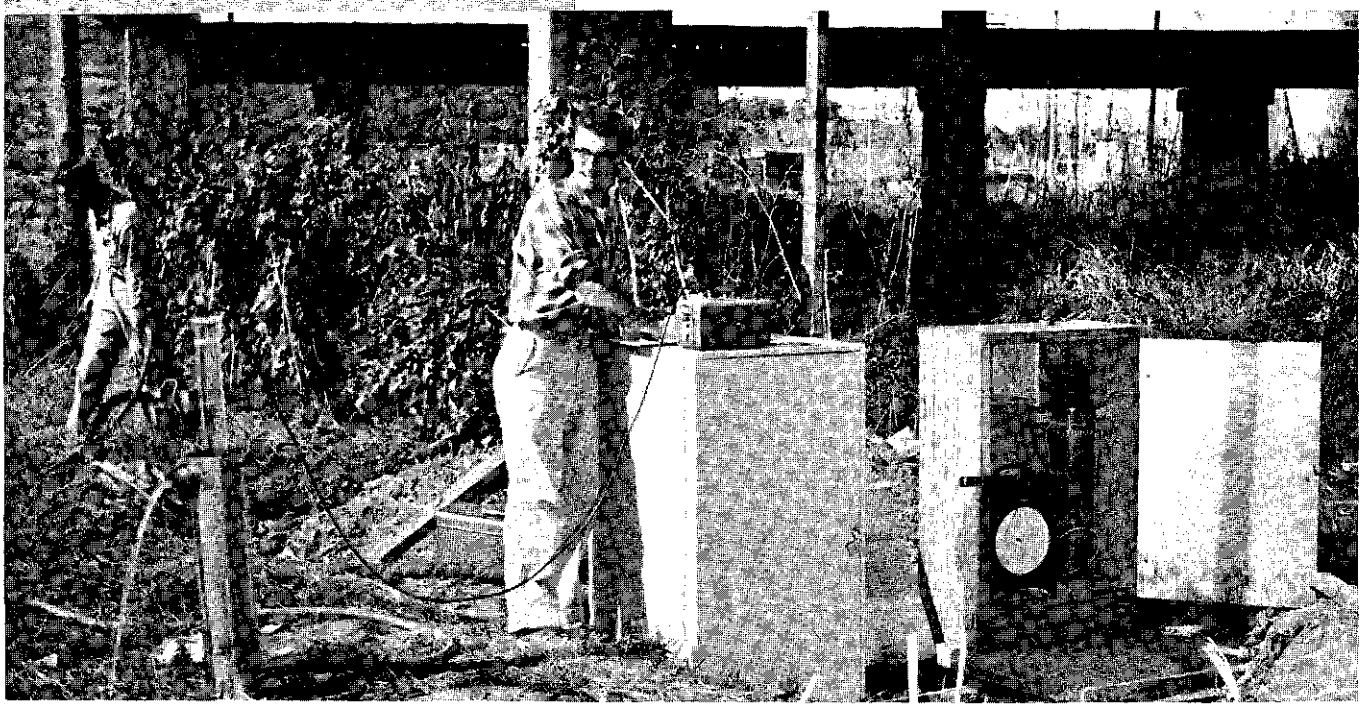


Fig. 7-1. Sources of Data on Sewage Characteristics

Records for existing sewage treatment plants and from metering stations established during the survey provided information on sewage flow rates. Raw sewage samples from a number of the treatment plants and metering stations were collected and analyzed to determine sewage strength and composition.

TOTAL COUNT METHOD, as used for calibration of the metering station on the North Trunk sewer, begins (left photograph) with the pouring of a small quantity of isotope into an upstream manhole. During this operation, the radioactivity level is monitored by means of an ionization rate meter. At the metering manhole downstream (lower photograph) a small quantity of sewage is pumped through a cylinder in which the Geiger-Mueller tubes are suspended. The total number of counts is recorded on a portable scaler shown on top of cabinet in center. On the right is the depth recorder and nitrogen gas cylinder, and in the center is the cabinet housing the vacuum type sewage sampler. See Appendix B for detailed description of total count method.

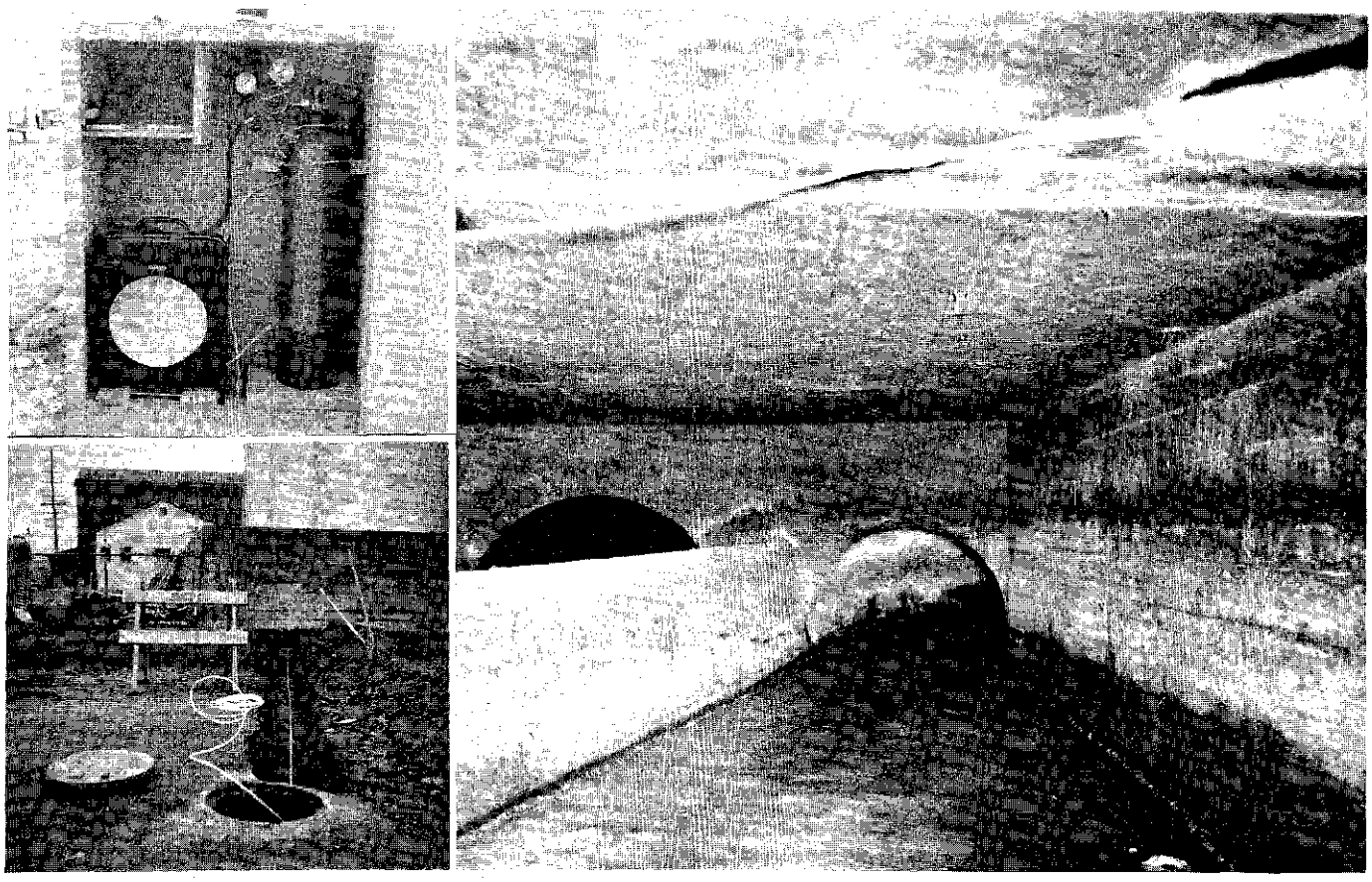


characteristics was developed by flow measurement and by the collection and analysis of representative samples.

Sewage Volume

Volumetrically, sewage can be considered to have two components. Of these, the first consists of sanitary sewage and industrial wastes, while the second consists of underground and surface water which enters sewers through joints and other openings. Depending upon the means and time of access, the latter component is termed either infiltration or storm inflow.

Since storm water inflow is limited generally to



METERING OF SEWAGE FLOW involved installation of a gas bubbler and sensitive pressure recorder to determine the depth of flow. Calibration curves relating depth to rate of flow were developed either by dilution of a radioactive isotope (the total count method herein described) or by calculations based on the physical and hydraulic characteristics of the pipe. On the Ballard trunk, the metering station was in a 116-inch wide by 69-inch high concrete box section with a side overflow weir (right photograph). At this station, the nitrogen gas cylinder and recorder were located off the edge of the roadway (left photographs). Flexible polyethylene tubing, protected by conduit in the roadway, was used between the cylinder and the invert of the sewer.

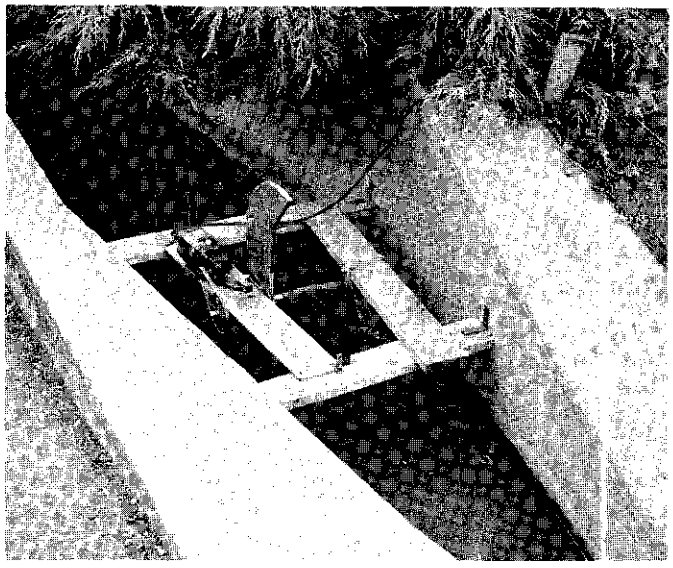
periods of actual rainfall, it is desirable to consider sewage volume in terms of both dry weather and wet weather flow. Dry weather flow (DWF) consists of sanitary sewage, industrial waste contributions, and infiltration from ground water. Wet weather flow (WWF) consists of the same components supplemented by storm water inflow. Dry weather flow determines, for the most part, the normal loading imposed on the major units of a sewage treatment plant, whereas wet weather flow determines the hydraulic capacity required not only for treatment purposes but for collection and conveyance.

Ground water infiltration varies seasonally with ground water level. This condition is significant in that it has a direct bearing on dry weather flows, both winter and summer. For example, if the ground water level is relatively high during the winter or wet season, the dry weather flow, meaning the flow at times when no rain has fallen, is correspondingly high and affects design requirements accordingly.

In the case of systems with separate sewers, records of flow measurements were available at sewage treatment plants serving those systems. In the case of combined sewers, only limited information was obtainable and came from records of flow measurements at the Diagonal Avenue treatment plant.

Of an estimated 75 mgd of sewage generated within the metropolitan area, only 15 mgd is metered regularly at the 25 sewage treatment plants. Additional flow data for the combined systems of Seattle, plus information on upstream infiltration and storm inflow to separate systems, were obtained by establishing metering points at seven locations and by recording the time of pump operation at two pumping stations (Fig. 7-1). In addition, minimum flow rates were estimated at 50 locations by gaging the depth of flow. Insofar as practicable, each of the seven metering points was selected so as to measure flows from the largest tributary area which could be obtained without including any upstream overflow or bypass structure.

Because of the high velocities which prevail in combined trunks during storm conditions, and also because of the wide range of flows, it generally was not practicable to construct a weir or flume within the



DIPPER TYPE SAMPLER was utilized at sampling locations where variation in depth of flow was relatively small. Type of weir or of other control sections immediately downstream determines shape of dipper required to collect quantity of sample proportional to rate of flow. Installation above is in the upstream converging section of the parshall flume at the city of Kirkland sewage treatment plant. Short length of hose discharges sample, which is collected as dipper revolves, to container suspended under board at right.

sewer selected as a measuring point. As a consequence, the sewer itself was utilized as a metering section at all but two of the seven locations. Straight sections having a constant grade were selected and were calibrated over a range of flows by means of the recently developed "total count" method. In this method a small, measured quantity of radioactive isotope is added upstream and the total count of gamma rays emitted by this material in passing the metering point is utilized in determining the rate of flow. A more detailed description of the isotope technique, together with individual calibration curves, is given in Appendix B.

At all metering stations, the depth of flow was measured continuously by means of a gas bubbler and sensitive pressure recorder. In each case, the bubbler tubing was carried in a conduit from the recorder to the invert of the sewer. A cylinder of compressed nitrogen provided the gas supply necessary to operate the bubbler.

Sewage flows, both total daily and instantaneous, are expressed as million gallons per day (mgd). Unit quantities for design purposes are expressed as gallons per capita per day (gpcd) in the case of sanitary sewage, and as gallons per acre per day (gpad) in the case of infiltration, storm inflow and industrial waste.

During those periods when lawn and garden irrigation is at a minimum, water consumption and sanitary sewage flows are approximately equal, and sewage flow in excess of water consumption is a measure of infiltration or storm inflow. Hence, metered records of water consumption serve as a check on sanitary sewage flows and on the variations therein which take place both seasonally and on a long term basis.

Sewage Strength and Composition

Because of the scarcity locally of adequate data on sewage strength and composition, it was necessary to undertake an extensive program of sampling and analysis. As a part of that program, raw sewage was sampled for periods ranging from four to eight days at each of five sewage treatment plants and at two trunk sewer metering stations (Fig. 7-1). Composite samples obtained each day were refrigerated during collection and were analyzed immediately thereafter at the sanitary engineering laboratory of the University of Washington. Laboratory analyses included determinations of hydrogen ion concentration; BOD before and after settling; and of total, suspended, settleable, and volatile solids. Sewage temperatures, and in some cases dissolved oxygen content, were determined in the field. In addition, the relative freshness of the sewage was observed and information was obtained concerning the quantity of grit carried in combined sewers.

In conducting the sampling program, one of the problems was that of collecting composite samples which would be properly representative of the varying conditions which occur in the course of a 24-hour period. An example of the manner in which variations take place in 24 hours is shown in Fig. 7-2. The two curves in this figure are based on flow meas-

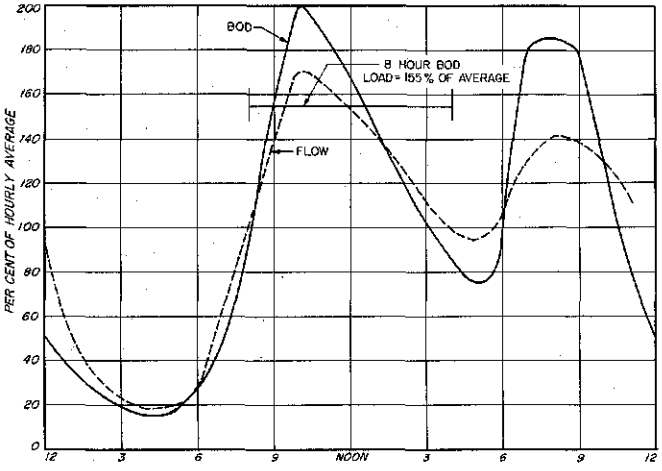
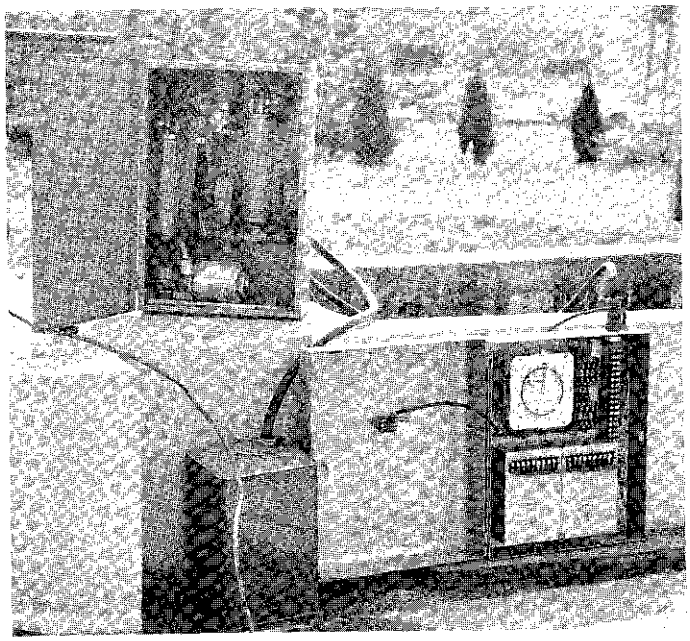
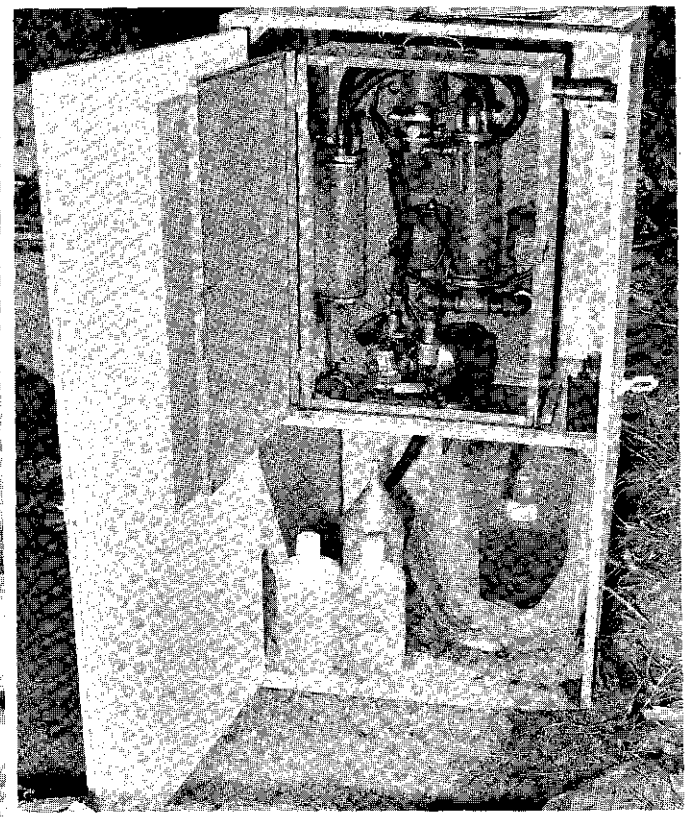
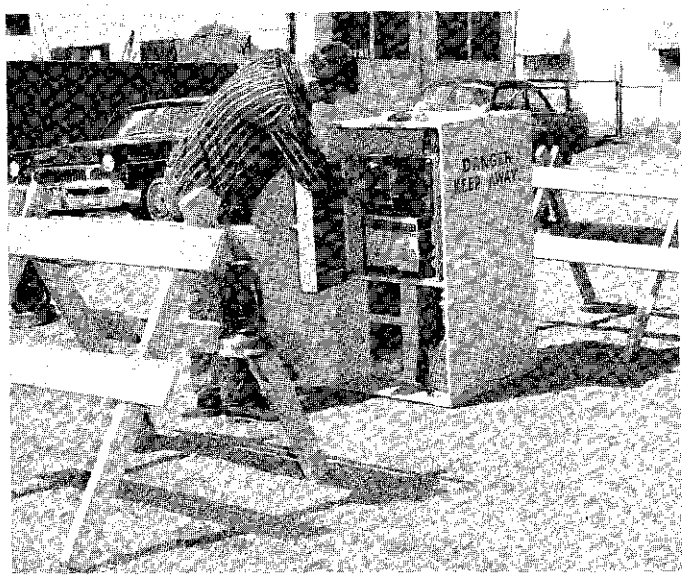


Fig. 7-2. Typical Hourly Variation in Flow and BOD at Greenwood Sewage Treatment Plant

Variations are typical of conditions in separate sanitary sewerage systems serving residential areas. Both the rate of flow and the strength of sewage drop to a minimum during the early morning hours, at which time ground water infiltration is predominant. Based on analyses by Engineering Testing Laboratory of city of Seattle. Samples collected August 12, 1954.







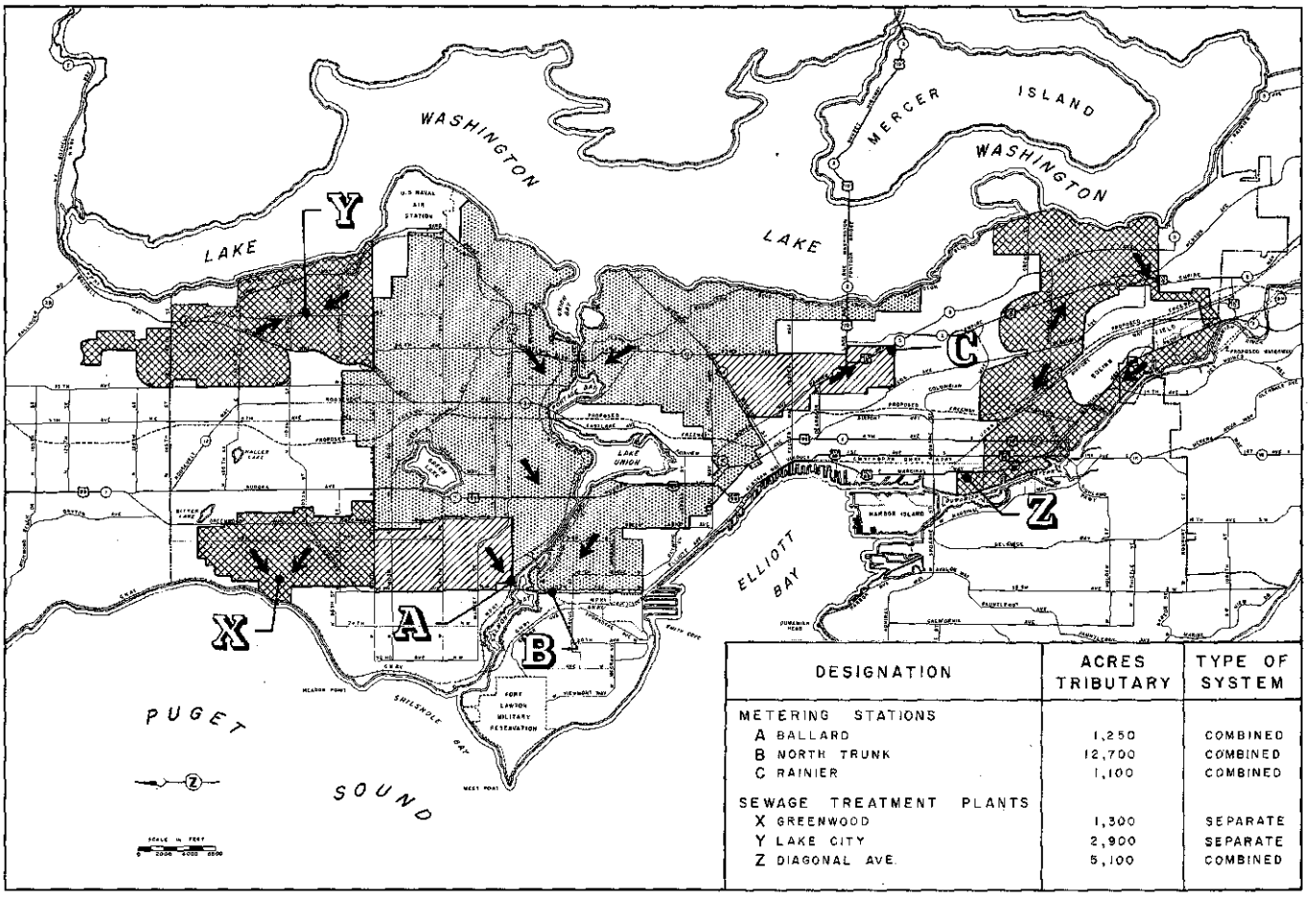
VACUUM PUMP SEWAGE SAMPLER. In above view vacuum pump is seen at the bottom of the metal case under the moisture trap at the left and the sample receiver at the right. In operation, the timer actuates the pump, which in turn draws sewage upward through a vacuum hose extending into the flow being sampled. Short probe control in receiver operates solenoid valves (1) to close line from sewer, (2) to close vacuum line from pump and open to exhaust, and (3) to open drain line to sample storage container. Sewage then drains to container until level falls below the longer probe. At that time, valve on line to container closes and valve on line to sewer opens and allows remainder of sewage in receiver to flow back to the sewer. Sample container in lower right of cabinet is 5-gallon bottle enclosed in plastic bag to eliminate condensation of atmospheric moisture and is refrigerated with dry ice.

urements and analyses of grab samples collected at the Greenwood sewage treatment plant and are typical of dry weather conditions in a small residential area served by separate sanitary sewers. Variations indicated by the curves are naturally more extreme than those which take place in the large trunk lines of the city of Seattle.

Composite samples were collected by means of automatic sampling devices, the function of which is to remove small portions of the flowing sewage at constant intervals ranging from five to twenty minutes. Two types of samplers were used during the survey, one equipped with an automatic dipper and the other with a vacuum pump, and both were operated in conjunction with appropriate devices for flow metering and recording.

Automatic samplers of the dipper type were used only at sewage treatment plants and were installed ahead of metering flumes or weirs in the inlet channels. This sampler is so designed that each dip removes an amount which is directly proportional to the actual flow.

At the metering stations in combined sewers, the dipper machine could not be used because variations in the depth of flow between dry and wet weather quite possibly would exceed the operating range of the dipper. For that reason, a new type of sampler was designed and built which can be mounted over or adjacent to a manhole. Actuated by a timer, this sampler is equipped with a vacuum pump which withdraws a measured portion of sewage at regular intervals through a 1-inch hose and discharges it to a sample bottle. To obtain a reasonably composite sample, the amount collected over periods ranging from eight



DESIGNATION	ACRES TRIBUTARY	TYPE OF SYSTEM
METERING STATIONS		
A BALLARD	1,250	COMBINED
B NORTH TRUNK	12,700	COMBINED
C RAINIER	1,100	COMBINED
SEWAGE TREATMENT PLANTS		
X GREENWOOD	1,300	SEPARATE
Y LAKE CITY	2,900	SEPARATE
Z DIAGONAL AVE.	5,100	COMBINED

Fig. 7-3. Area Tributary to Metering Stations and Major Treatment Plants in Seattle

All metering stations established on Seattle's combined sewers, with the exception of that on the North Trunk, were so located that no overflows occurred upstream from the metering point. Flow in the North Trunk was metered during the late summer months of 1957 when rainfall and overflows were relatively infrequent.

to twelve hours is proportioned manually to the total flow recorded during each such period. While less precise than the dipper machine, the new machine nevertheless enabled the collection of sufficiently representative samples at each of the trunk sewer metering stations.

Measurements of sewage strength and composition are expressed herein in terms both of concentration and of total loading. Concentration is reported as parts per million by weight (ppm), whereas total loading is reported as pounds per day. Unit quantities are expressed as pounds per capita per day (ppcd).

Infiltration

Infiltration and storm inflow require separate consideration because they are the cause of what has become a serious problem to many sewerage systems in the metropolitan area. For example, the Lake City system, which is typical of almost all of them, is subject continuously during winter months to a high rate of infiltration and to a five-fold increase in inflow at the sewage treatment plant during periods of moderate to heavy rainfall.

Industrial Waste

The volume, strength and composition of industrial wastes are not related either to the resident population or to the areal extent of a sewerage service area. They are determined instead by the number, type and magnitude of industrial operations. Insofar as volume is concerned, it is possible for design purposes to use a reasonable allowance per gross acre which is based on experience elsewhere and on local records of water consumption. Strength and composition, on the other hand, are of lesser importance in long range planning and can be reckoned with either by regulating ordinances or by the incremental addition of treatment facilities.

SEWAGE VOLUME AND COMPOSITION

As stated earlier, information regarding sewage characteristics was obtained by flow measurements

Table 7-1. Metering Stations on Combined Sewers in Seattle

Station designation	Location[a]	Sewer size, diameter, inches	Tributary area, acres	Population[b]	Type of area
North Trunk	Thorndyke and Emerson	138	12,700	195,000	Residential, commercial and heavy industrial
Rainier	Rainier and Bayview	102	1,100	22,000	Multiple dwelling and light industrial
Ballard	11 NW and W 45	69 x 116 box	1,250	24,500	Residential, commercial with few industries
Total metered			15,050	241,500	

[a]*See Fig. 7-3.*

[b]*Seattle City Planning Commission estimate, 1955.*

and laboratory analyses and by reviewing operation records and reports. Necessary data were developed both for representative locations in the Seattle system and for other systems requiring separate appraisal.

City of Seattle

Operating records of the Lake City, Greenwood, and Diagonal Avenue sewage treatment plants provided needed information concerning flow conditions in both separate and combined sewer systems. Routine analyses of influent samples, while providing valuable background information, were not considered to be sufficiently representative for the purposes of the survey. This is because single grab samples rather than composite samples were used in making the analyses.

Sewage from the remainder of Seattle, which in dry weather comprises about 90 per cent of all that originating in the metropolitan area, is discharged to Puget Sound without treatment. To study conditions in that area and also to develop essential information concerning sewage characteristics, metering and sampling stations were established at three locations in principal trunk sewers (Fig. 7-3 and Table 7-1).

Lake City System. Situated at East 107th Street and 38th Avenue NE, the Lake City sewage treatment plant serves a tributary area of 2,900 acres with approximately 250 miles of separate sanitary sewers, including house connections. Development of this area, as reflected by the number of permits for new house connections, has been taking place at a rapid rate since 1953 (Fig. 7-4). At the end of 1957, approximately 8,000 permits had been issued, representing a connected population of 25,000.

Wet weather flows during the period from January 1 through January 7, 1956 generally exceeded the maximum capacity of the meter and reached an estimated peak of 14 mgd on January 6 (Fig. 7-5). Dry weather flows, as metered from July 16 through July 22, 1956, averaged close to 3.0 mgd, as compared to a treatment plant design capacity of 2.5 mgd. During this period the per capita rate averaged 120 gpd. The latter value is considerably higher than might be expected in a predominantly residential area such as Lake City where winter water use amounts to 53 gpcd.

A serious situation with regard to storm inflow is evidenced by the excessive wet weather flow. Similarly, excessive ground water infiltration is evidenced by the high per capita flow and the minimum flow rate of 2 mgd.

Samples were taken at the plant for eight days in the period from July 17 through July 31, 1957 (Table 7-2). Average flow during this period was 3.0 mgd. As computed from the analysis results, the per capita BOD contribution averaged 0.14 pounds per day and is considered normal for sewage from a residential area. For suspended solids, the per capita value averaged 0.21 pounds per day. This is somewhat higher than normal and may be attributed partially to the high grit and sand content of trench drainage from sewer construction projects.

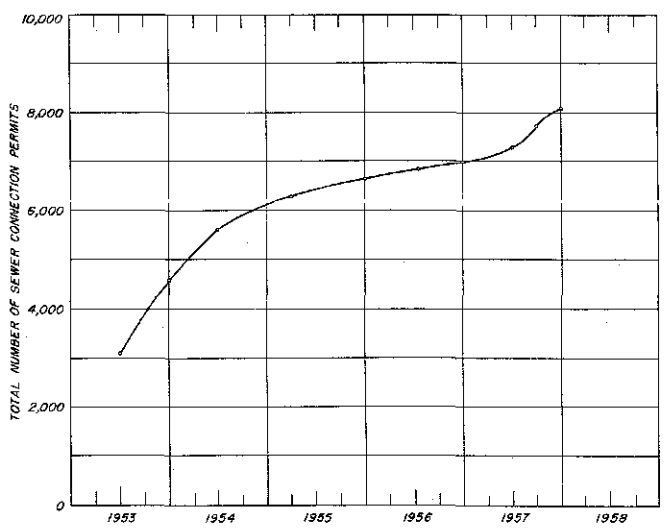


Fig. 7-4. Sewer Connections in Lake City Sewer District

Based on total number of house connection permits issued by the Lake City Sewer District and by Seattle since annexation.

Table 7-2. Sewage Characteristics at Sampling Stations in Seattle

Date	Day	Average flow, mgd	BOD			Solids							
			Raw		Settled[a]	Total		Suspended			Settleable		
			ppm	lb/day	ppm	ppm	Volatile ppm	ppm	lb/day	Volatile ppm	ml/l	ppm	Volatile ppm
Lake City Plant, July 1957													
17	Wed	3.1	128	3,300	79	510	250	258	6,650	162	7.5	162	120
18	Thur	3.1	135	3,480	89	446	222	158	4,070	120	6	92	67
19	Fri	3.1	123	3,180	88	472	354	336	8,650	228	7.5	204	136
24	Wed	3.0	169	4,240	103	446	229	–	–	–	7	–	–
25	Thur	2.9	174	4,200	100	634	372	216	5,220	184	7	172	126
26	Fri	3.1	120	3,100	100	450	278	166	4,300	152	8	136	102
30	Tue	3.0	119	2,980	77	392	206	144	3,600	116	7	76	66
31	Wed	3.0	143	3,580	79	426	250	182	4,550	156	8	116	108
Weighted average				3,500					5,290				
Diagonal Avenue Plant, August 1957													
13	Tue	4.8	151	6,050	60	408	382	186	7,450	156	6.5	140	116
14	Wed	4.7	108	4,220	64	466	–	196	7,660	175	7	62	49
16	Fri	4.8	96	3,840	65	492	–	156	6,250	130	5	94	82
19	Mon	4.5	101	3,780	68	378	–	240	9,000	195	5	170	156
Weighted average				4,470					7,590				
Greenwood Plant[b], July 1954													
8	Thur	1.5	124	1,550	–	–	–	109	1,360	92	–	–	–
9	Fri	1.5	108	1,350	–	–	–	130	1,625	110	–	–	–
14	Wed	1.5	103	1,290	–	–	–	163	2,040	137	–	–	–
15	Thur	1.5	112	1,400	–	–	–	146	1,820	120	–	–	–
16	Fri	1.5	126	1,580	–	–	–	162	2,030	141	–	–	–
19	Mon	1.5	84	1,050	–	–	–	127	1,590	103	–	–	–
20	Tue	1.4	99	1,160	–	–	–	116	1,450	97	–	–	–
21	Wed	1.4	105	1,230	–	–	–	151	1,760	117	–	–	–
Weighted average				1,330					1,710				
North Trunk Sewer, October 1957													
4	Fri	39	100	32,500	63	316	238	106	34,400	104	4	102	102
5	Sat	39	83	27,000	61	224	218	106	34,400	84	3	90	74
7[c]	Mon	48	98	39,200	65	364	234	208	83,000	148	5	148	110
8[c]	Tue	52	85	36,800	65	322	282	116	50,400	100	4	58	40
9	Wed	43	99	35,400	61	344	220	124	44,400	118	4.5	60	60
10	Thur	36	94	28,200	59	316	284	146	43,900	134	5.0	104	92
11	Fri	38	86	27,200	52	292	260	120	38,000	114	4	84	82
Weighted average				32,400					47,000				
Ballard Trunk Sewer, August and September 1957													
22	Thur	1.7[d]	126	1,780	102	390	224	110	1,560	60	5	50	40
23	Fri	1.7	170	2,400	–	376	–	140	1,990	100	5	64	60
26	Mon	1.7	133	1,880	101	466	294	170	2,420	142	5	76	68
28	Wed	1.7	94	1,330	75	312	270	116	1,650	–	5	66	–
29	Thur	1.7	120	1,700	80	506	330	192	2,720	179	7	128	116
30	Fri	1.7	154	2,180	98	378	254	136	1,930	91	6	76	37
3	Tue	1.7	248	3,500	–	524	400	224	3,180	196	8	154	86
4	Wed	1.7	167	2,360	117	406	270	124	1,760	116	7	82	75
Weighted average				2,140					2,150				

All analyses performed on raw sewage samples. See Fig. 7-3 for location of sampling points. Temperature and pH during sampling period: Lake City plant, 15° C, 7.3; Diagonal Avenue plant 19° C, 7.3; Greenwood plant, no temperature, 7.1; North Trunk, 20° C, 7.0; Ballard Trunk, 22° C, 7.3.

[a] *One hour settling period.*

[b] *From analyses performed by Seattle Engineering Department laboratories and converted to 24-hour composite values per Fig. 7-2.*

[c] *Occasional showers on this date.*

[d] *Flow metered on initial day of sampling and similar average assumed for remainder of sampling period.*

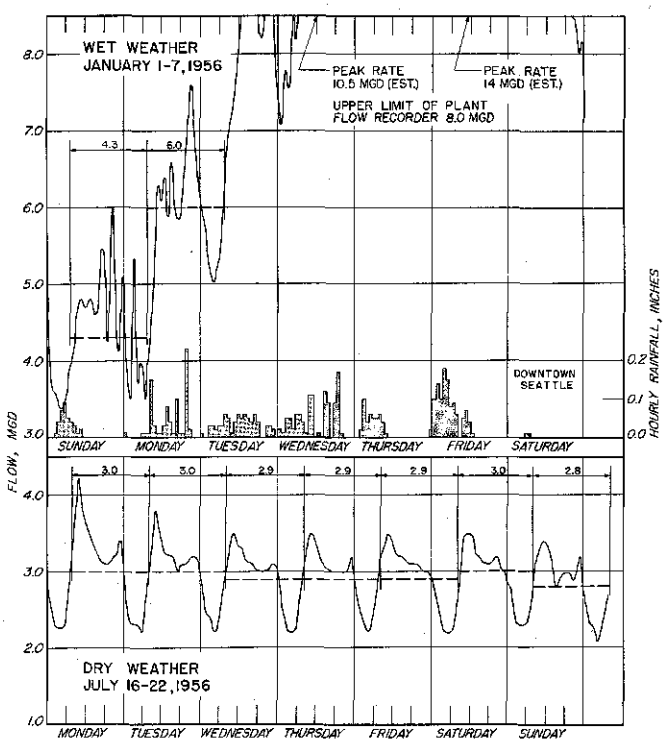


Fig. 7-5. Hourly Variation in Flow at Lake City Sewage Treatment Plant

Estimates by operating personnel provide the only data on flows in excess of 8.0 mgd, the meter chart capacity. It is reasonable to assume that the estimated maximum rate of 14 mgd on January 6, 1956, was exceeded on December 21, 1955 when the treatment plant was flooded. Flows below 8.0 mgd are from plant meter records.

Analyses of 8-hour composite samples, as made by the treatment plant operators during 1955-56, show BOD values ranging from a low of 130 ppm in January to a high of 317 ppm in August. Suspended solids values ranged from 72 ppm in January to 364 ppm in August. Raw sewage temperatures ranged from 35° F in the winter to 61° F in the summer. During the same period, plant effluent temperatures ranged from 40° F in the winter to 63° F in the summer.

Greenwood System. The Greenwood sewage treatment plant is situated at Carkeek Park and serves a tributary area of 1,320 acres. Based on the number of house connections, which at present totals 5,300, the plant serves a connected population of 17,000.

Records of dry and wet weather flow, as obtained from meter readings at the treatment plant, indicate that the Greenwood system is subject to considerable infiltration and storm inflow during wet weather periods (Fig. 7-6). This condition is reflected by an average flow at such times of 3.4 mgd, as compared to a dry weather average of 1.4 mgd. The latter represents a per capita flow of 82 gpd. In contrast, per capita water consumption during the winter averages 62 gpd.

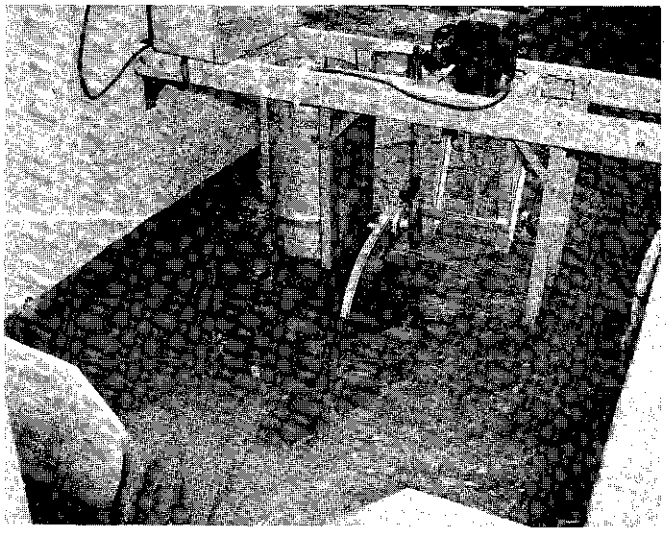
Information regarding BOD and suspended solids loadings was developed from analysis results reported by the engineering department of the city of Seattle[1]. These results, which were for 6-hour composite samples, were adjusted to a 24-hour basis by relating them to the BOD data plotted in Fig. 7-2. As calculated from the 24-hour values (Table 7-2), the per capita contributions are 0.08 pounds per day for BOD and 0.11 pounds per day for suspended solids. The latter is close to normal but the BOD value is low, even for strictly domestic sewage.

Diagonal Avenue System. Situated at Diagonal Avenue and East Marginal Way, the Diagonal Avenue treatment plant is the only one in the metropolitan area which receives sewage from a system of combined sewers. The tributary area, which includes a large industrial district, occupies approximately 5,100 acres (Fig. 6-31) and has an estimated resident population of 30,000.

Average dry weather flow during a two week period (Fig. 7-7) was 4.3 mgd or 143 gpcd. This per capita rate is high and reflects in part the effect of industrial waste contributions. Wet weather input is limited by two upstream regulators which divert all flow above 6.5 mgd to adjacent receiving waters.

Composite samples were collected over a 14-day period from August 6 to August 19, 1957, using for the first time the new sampler described previously in this chapter. Since several complications developed

[1] *Report by Engineering Testing Laboratory of the city of Seattle, September 1, 1954.*



DIPPER TYPE SAMPLER in operation at the influent works of the Lake City treatment plant. In foreground is the V-notch weir which provides necessary upstream flow depth. Sample is discharged to container in pipe cylinder to left of dipper.

in this initial operation, analysis results herein reported are limited to those for samples collected during the second week from August 13 through August 19. Despite the presence of industrial wastes, the analysis result (Table 7-2) show fairly low concentrations for both BOD and suspended solids. In contrast, per capita loadings are somewhat high and reflect the contributions from industrial sources.

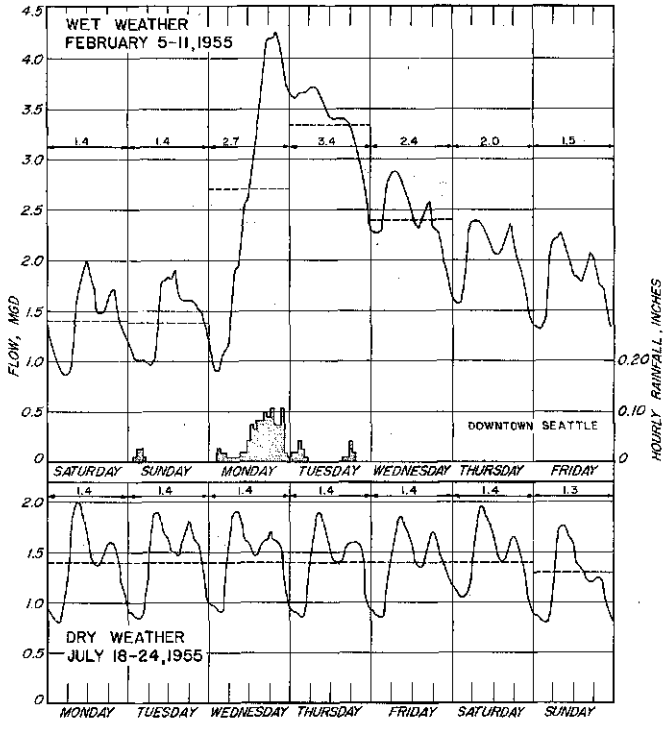


Fig. 7-6. Hourly Variation in Flow at Greenwood Sewage Treatment Plant

The increase in the minimum rate of flow during storm periods and its decline the next few days to dry season levels indicates that roof and foundation drains are the principal sources of wet weather inflow. Based on flow studies made by city of Seattle.

North Trunk System. This is the largest system in Seattle. Flow from a major portion of the system was metered in a manhole at Thorndyke Avenue and Emerson Street during dry weather from September 15 to October 15, 1957 (Fig. 7-8). The tributary area of the metered portion was approximately 12,700 acres (Fig. 7-3) with an estimated contributary population of 195,000. Dry weather flow averaged 35 mgd. This is equivalent to a per capita average of 180 gpd, on which basis it is far in excess of the general average for the metropolitan area.

The high per capita rate is due to a combination of conditions consisting of (1) overflows from Green Lake and from Seattle Water Department reservoirs; (2) direct inflow from surface streams and springs which have been diverted to the sewer system; and (3) ground water infiltration. Quite possibly, some of the infiltration is taking place in the area along Ravenna Boulevard where the recent cave-in occurred.

Sewage was sampled at the metering station manhole for seven days from Ocober 4 through October 11, 1957. All samples were taken by means of the vacuum sampler and were composited manually every eight to twelve hours in proportion to the metered flow. As calculated from the analysis results (Table 7-2), the per capita loadings amount to 0.17 pounds per day for BOD and 0.24 pounds per day for suspended solids. These values are somewhat high,

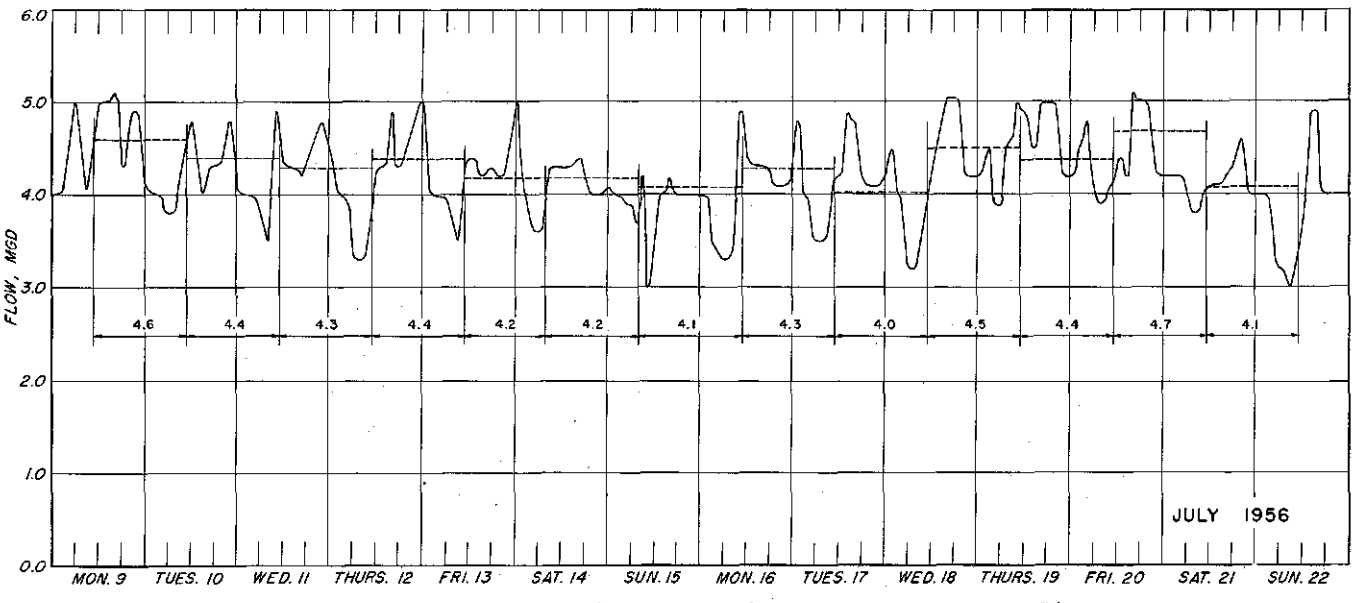


Fig. 7-7. Hourly Variation in Flow at Diagonal Avenue Sewage Treatment Plant

Because of the combined collection system and the predominantly industrial character of the tributary area, sewage flows average considerably higher in dry weather than would be expected from a residential area served by separate sanitary sewers. Based on plant meter records.

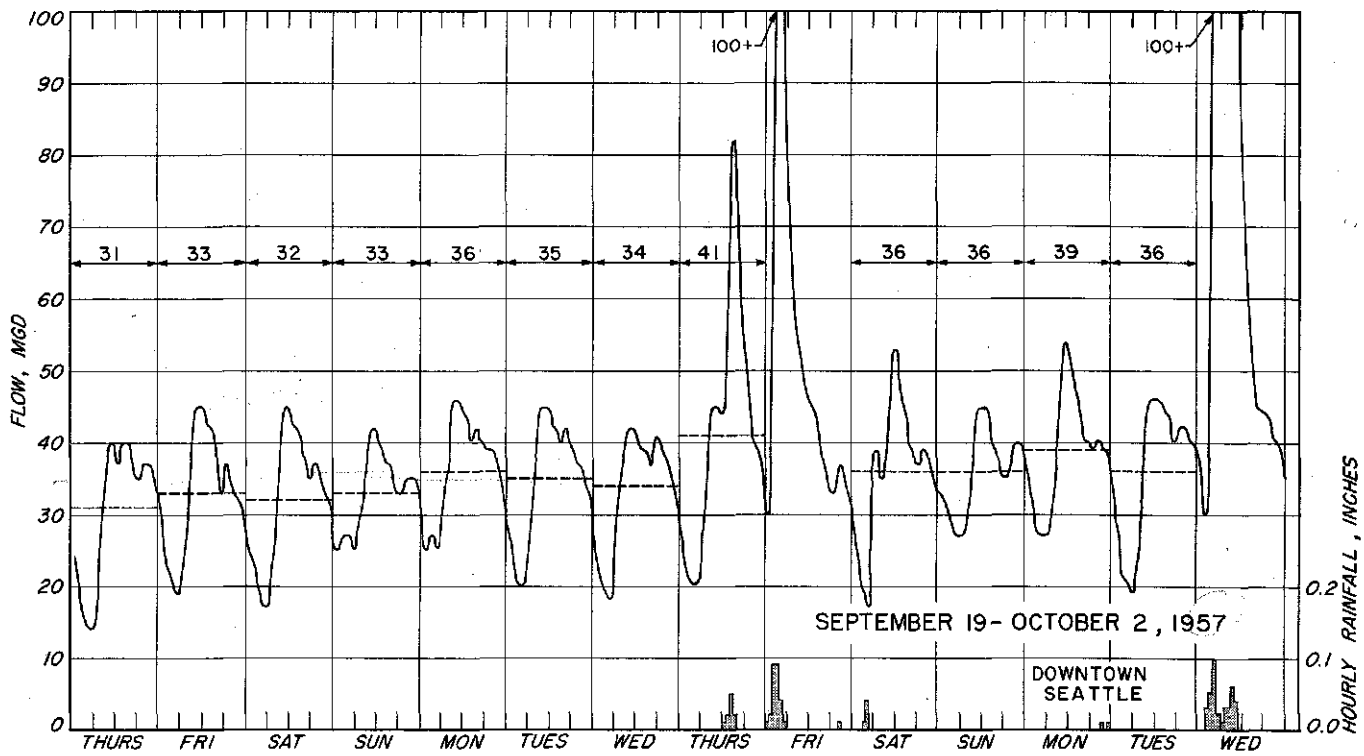


Fig. 7-8. Hourly Variation in Flow at North Trunk Sewer Metering Station

Sewage flow from an estimated population of 195,000 was metered at this station. For dry weather conditions, the average per capita rate of 180 gallons per day is the highest of those observed at all metering locations during the course of the survey.

apparently reflecting the effect of contributions from the industrial area along the Lake Washington Ship Canal. In terms of concentration, however, the North Trunk samples are the weakest of all of those collected in Seattle. In other words, the per capita loadings are high but the strength is low because of the extreme dilution brought about by the above mentioned conditions.

Ballard System. Sewage from approximately 1,300 acres of the Ballard area (Fig. 7-3) was metered near a storm water inflow to Lake Washington Ship Canal. Average dry season flow was 1.7 mgd (Fig. 7-9), while wet season flow averaged almost 5 mgd when rain was not actually falling.

Sewage samples were collected at the metering station on seven days during a period from August 22 to September 4, 1957 (Table 7-2). Average BOD and suspended solids values are each 0.09 ppcd.

Rainier Avenue System. In this system, a metering station was established at Rainier Avenue and Bayview Street. At that point (Fig. 7-3), the tributary area is approximately 1,100 acres, with an estimated population of 22,000.

Wet season flows, as metered from February 16 through February 22, 1957, were slightly lower than

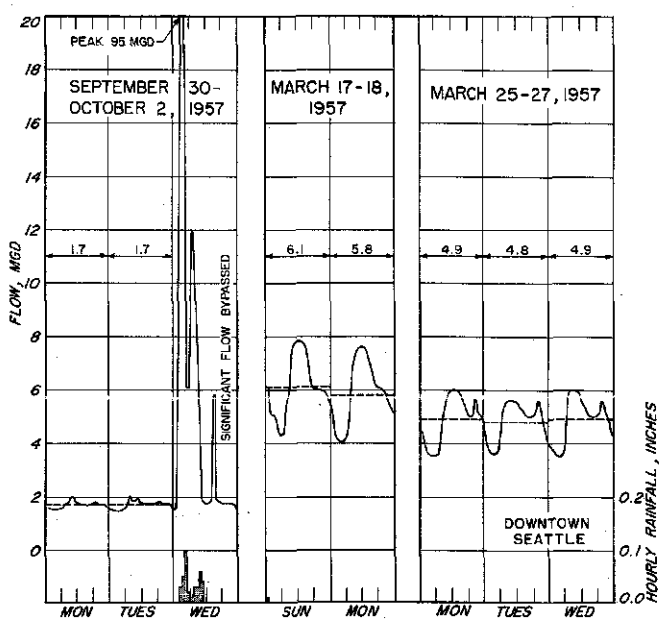


Fig. 7-9. Hourly Variation in Flow at Ballard Trunk Sewer Metering Station

Even in the absence of rainfall, wet season flow in this combined sewer averaged between three and four times the dry season flow. Hourly variations for the first two days shown are of doubtful accuracy because of a shallow depth of flow and the effect of bottom deposits of grit.

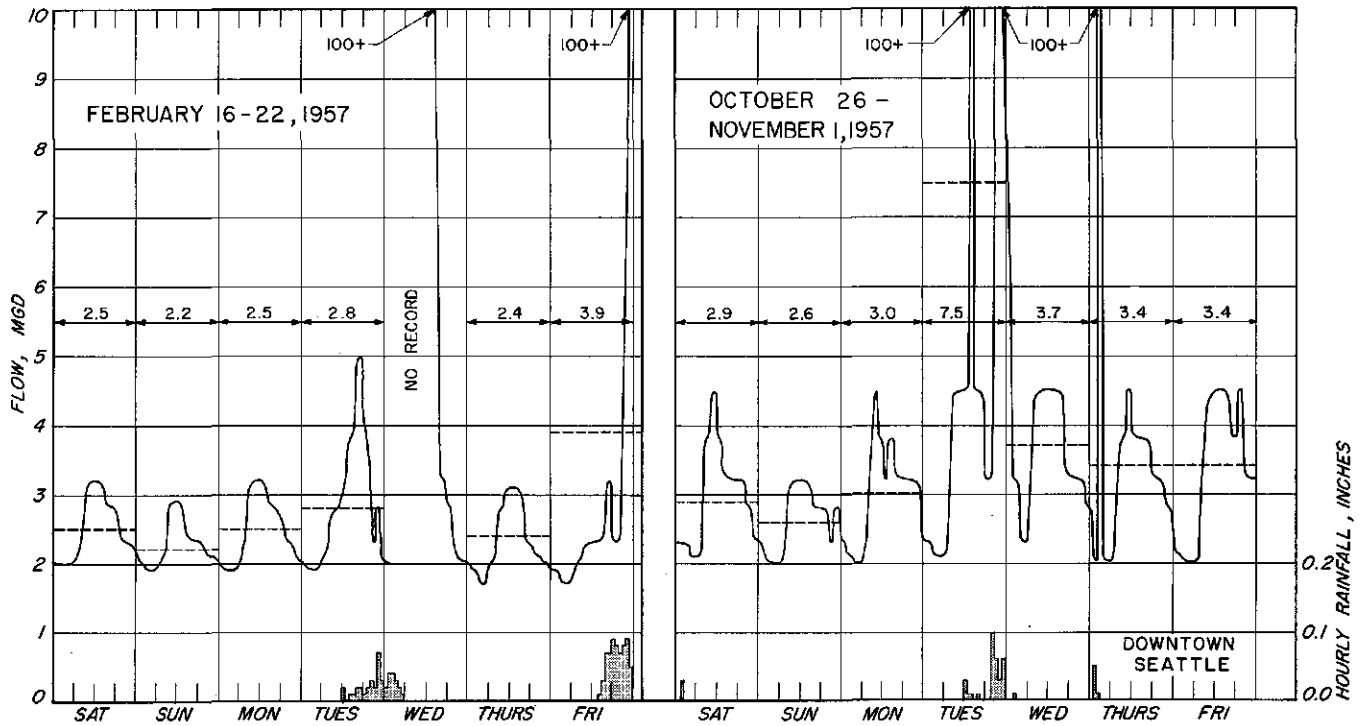


Fig. 7-10. Hourly Variation in Flow at Rainier Avenue Trunk Sewer Metering Station

Peak rates of flow in this combined sewer exceeded 100 mgd even with rainfalls as low as 0.05 inches per hour. The similarity between minimum flows during the two periods indicates a relatively stable ground water level.

those metered near the end of the dry season (Fig. 7-10). Minimum flows during both periods were almost identical, thus indicating a similar ground water level and a similar rate of infiltration. Per capita flow during dry weather averaged 120 gpd.

City of Auburn

Records of sewage flow at the Auburn treatment plant were obtained from meter charts and were plotted (Fig. 7-11) for both a dry season period and wet season period. Dry season flows, as recorded from July 15 to July 21, 1956, averaged 0.6 mgd, or 85 gpcd. Wet weather flow, as recorded from January 1 to January 7, 1956, averaged 1.7 mgd, or 240 gpcd. By comparison, water consumption during the winter amounted to 57 gpcd.

Bellevue Sewer District

Flows at the Bellevue sewage treatment plant also were plotted for typical dry and wet season periods (Fig. 7-12). Dry season flow, as recorded from July 15 through July 21, 1956, averaged 0.14 mgd, representing a per capita flow of 34 gpd. Wet weather flow, as recorded from January 1 through January 7, 1956, averaged 0.8 mgd, and apparently is limited only by the hydraulic capacity either of the plant influent sewer or the inlet works. Winter water consumption in the Bellevue-Medina area averages 57 gpcd.

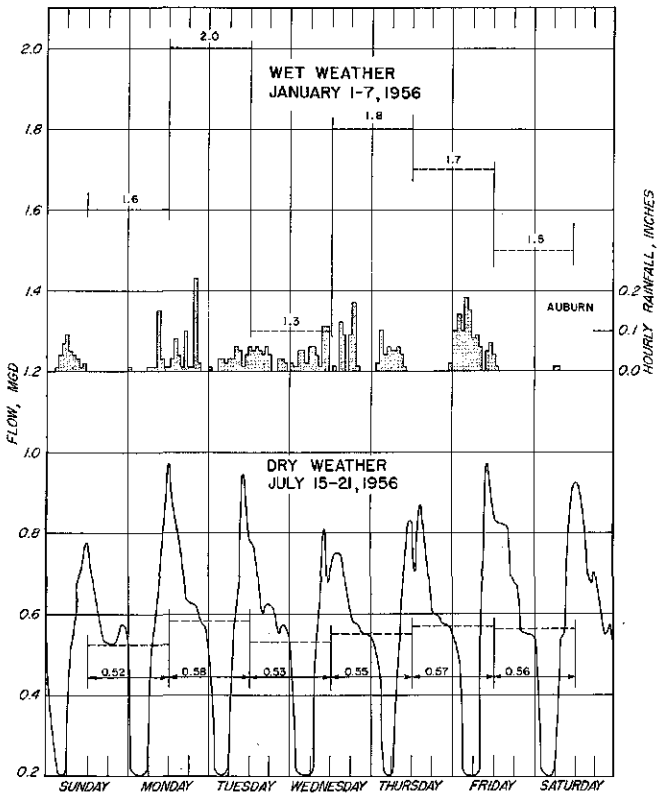


Fig. 7-11. Variation in Flow, Auburn Sewage Treatment Plant

Hourly rates not shown for January as the recorder is inaccurate at high rates of flow. Records of total daily flow during wet weather believed to be reliable. Based on plant meter records.

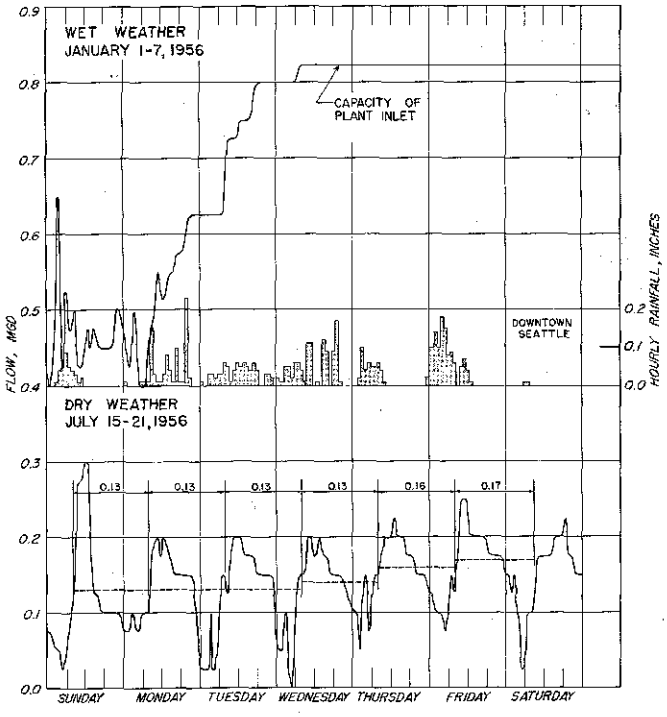


Fig. 7-12. Hourly Variation in Flow at Bellevue Sewage Treatment Plant

Wet weather flows are five or six times average dry weather flows and are apparently limited by the hydraulics of the influent sewer or inlet works. Based on plant meter records.

Bryn Mawr-Lake Ridge Sewer District

Due to a meter outage during the summer of 1956, records of dry weather flow at the Bryn Mawr plant were obtained for the period from November 24 through November 30, 1956 and are plotted in Fig. 7-13. During this period, the average flow was 0.21 mgd, or 47 gpcd. Flow rates during most of the wet weather period from January 1 to January 7, 1956 were in excess of the treatment plant meter capacity of 1.1 mgd. Winter water consumption averages 48 gpcd.

City of Kent

Dry season flows as recorded at the Kent sewage treatment plant from July 15 through July 21, 1956, averaged 0.7 mgd, or 117 gpcd (Fig. 7-14). These flows include an estimated delivery of 0.5 mgd from three cannery operations. By deducting the cannery contribution, the net flow from other sources becomes 0.2 mgd, or 50 gpcd.

City of Kirkland

At the Kirkland sewage treatment plant, dry weather flows during the period from June 30 through July 6, 1957, averaged 0.3 mgd, or 52 gpcd (Fig. 7-15). Wet weather flows, as recorded from January 1 through January 7, 1956, reached a peak of 2.3 mgd. Per capita water consumption during the winter months is estimated by the city to average 55 gpd.

Analyses were made of composite samples collected at the treatment plant for seven days during the period from July 9 through July 22, 1957 (Table 7-3). Average per capita values, as computed from the analysis results, are 0.19 pounds per day for suspended solids and 0.09 pounds per day for BOD. Although the latter is low for a strictly domestic sewage, the suspended solids value is higher than normal.

City of Renton

Records of typical wet and dry season flows at the Renton sewage treatment plant are plotted in Fig. 7-16. Dry weather flow, as metered from July 14 through July 21, 1956, averaged 0.7 mgd, representing a per capita rate of 42 gpd. Wet season flow during the period from January 1 through January 8, 1957, averaged approximately 1 mgd, with a peak rate of 3 mgd recorded on January 7. Analyses made by treatment plant personnel on samples collected during dry weather show a BOD value of 110 to 150 ppm and an average suspended solids value of 150 ppm.

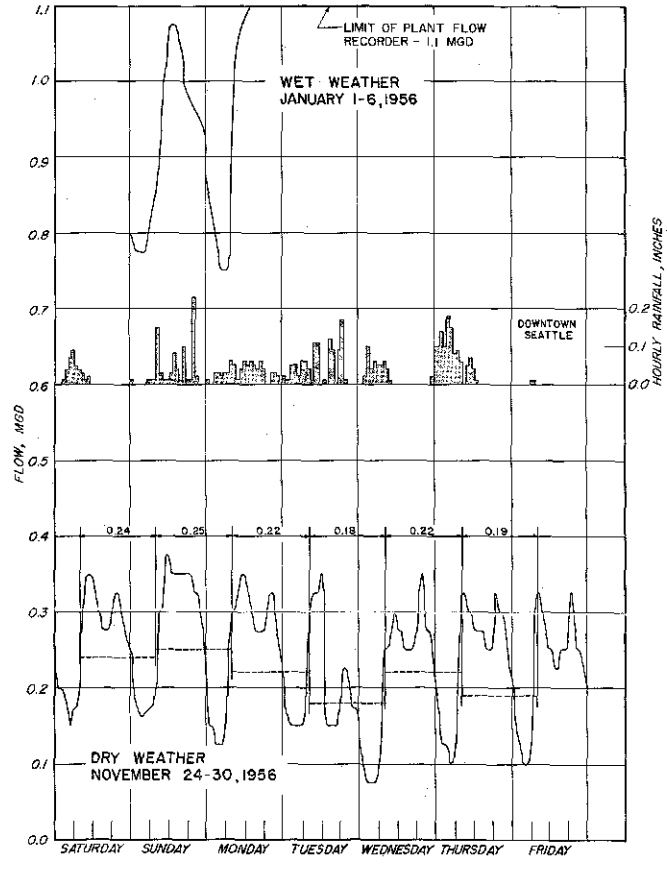


Fig. 7-13. Hourly Variation in Flow at Bryn Mawr - Lake Ridge Sewage Treatment Plant

Wet weather flow rates regularly exceed the capacity of the influent meter and bear no resemblance to the typical hourly flow variations occurring during dry weather. Based on plant meter records.

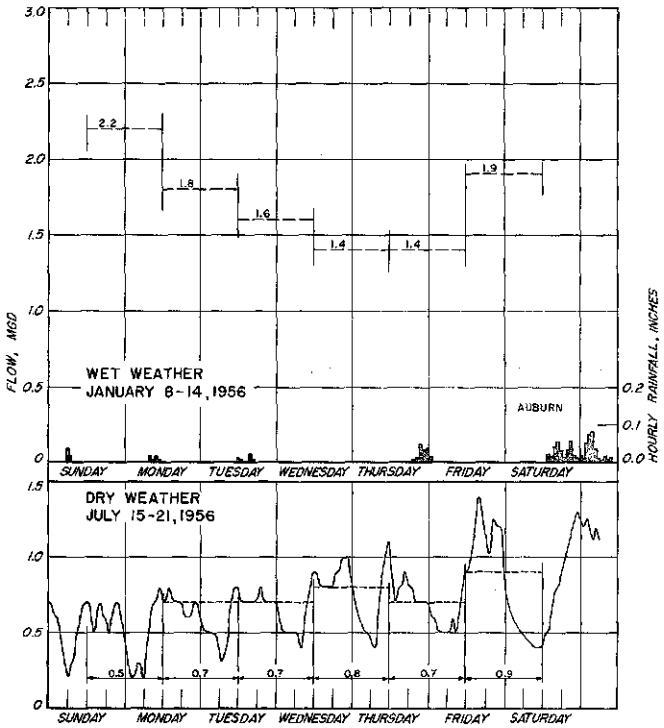


Fig. 7-14. Hourly Variation in Flow at Kent Sewage Treatment Plant

Canneries tributary to the Kent plant account for about 0.5 mgd of the average daily flow during dry weather. Wet weather flows, for which daily averages only are shown, indicate that both infiltration and storm inflow are prevalent. Flow rate indicator is inaccurate at higher flows during wet weather.

Shorewood Apartments

Wet and dry season flows recorded at the Shorewood Apartments sewage treatment plant show that this is one of the few systems in the metropolitan area which is not subject to a sizeable increase in flow during the winter months. Wet season flow (Fig. 7-17) is less than the dry season flow in the same year, apparently because of reduced occupancy during the winter. Based on complete occupancy, the dry season flow of 0.2 mgd represents a per capita rate of 87 gpd. Water consumption during the winter amounts to 70 gpcd.

As indicated in Table 7-3, sewage samples were collected at the Shorewood treatment plant from June 20 through July 1, 1957. Suspended solids and BOD contributions, as computed from the analysis results, average 0.14 and 0.16 ppcd, respectively, for an estimated connected population of 2,300 persons.

Southwest Suburban Sewer District

Construction of the Southwest Suburban sewage treatment plant and of the majority of its tributary system was completed in February of 1957. Dry weather flow, as metered from August 25 through August 31, averaged 1.3 mgd (Fig. 7-18), representing a per capita contribution of 100 gpd for the

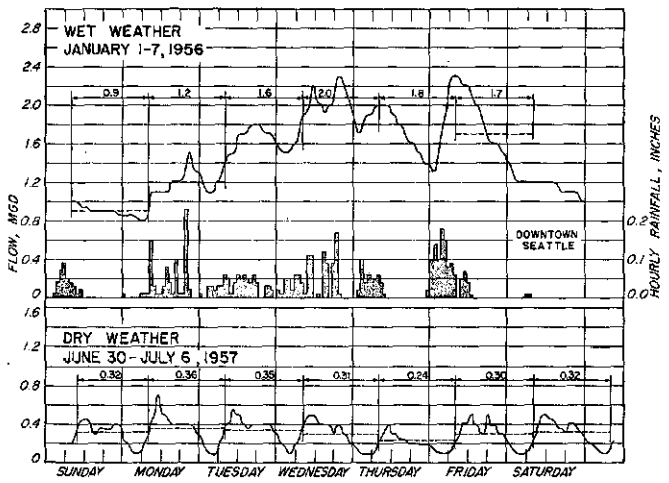


Fig. 7-15. Hourly Variation in Flow at Kirkland Sewage Treatment Plant

Peak wet weather flow of 2.3 mgd is about seven times the average dry weather flow. An even higher peak flow, 3.5 mgd, was recorded December 20, 1955. Based on plant meter records.

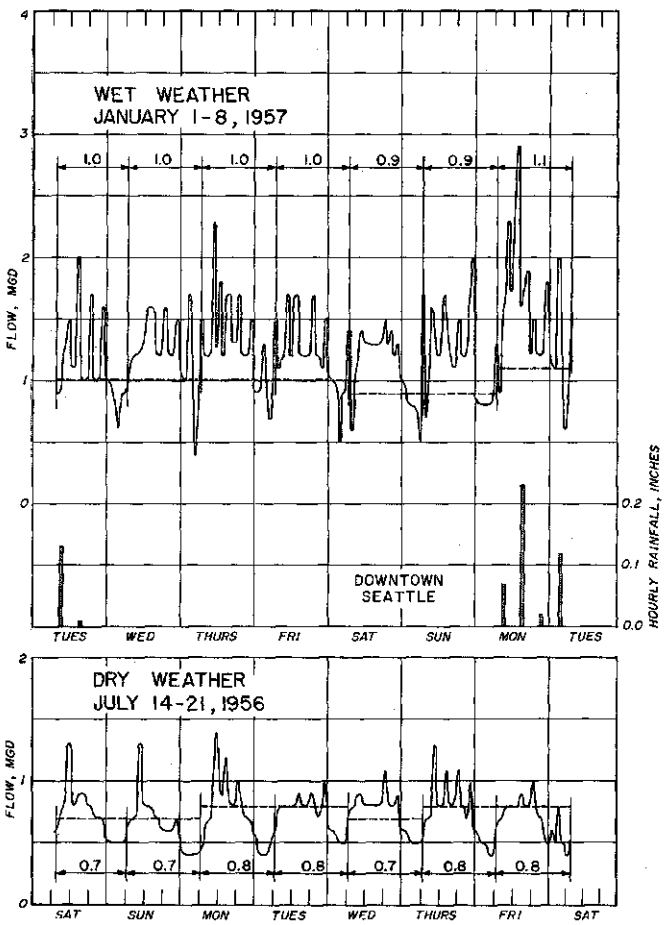


Fig. 7-16. Hourly Variation in Flow at Renton Sewage Treatment Plant

Average dry weather flow increased from 0.7 mgd to 0.9 mgd late in the summer of 1957, and is attributed to increased development resulting from expansion of the Boeing factory at Renton. Based on plant meter records.

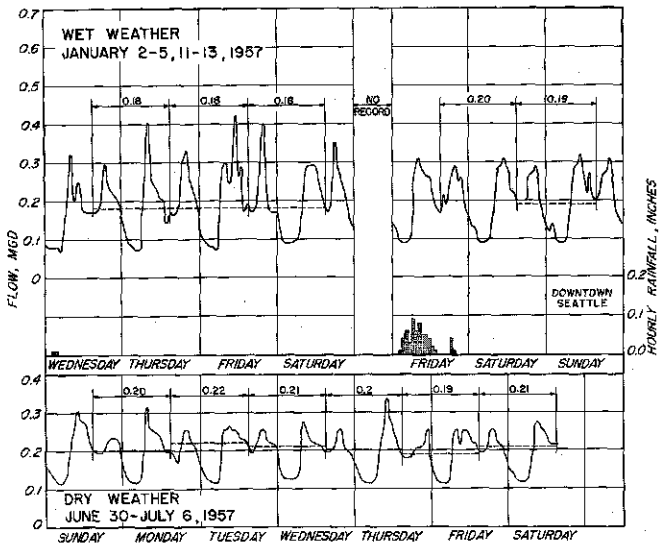


Fig. 7-17. Hourly Variation in Flow at Shorewood Apartments Sewage Treatment Plant

Average dry weather flow during the summer season exceeds wet season flow by as much as 0.3 mgd and is due, in part, to an increase in contributory population during the summer months. Based on plant meter records.

tributary population of 12,600 persons. Water consumption during winter months averaged nearly 60 gpcd.

Wet weather flows during the week beginning November 10 reached a peak rate of 3.0 mgd on November 13. On February 26, shortly after the treatment plant had been placed in service, a peak of 5.4 mgd was recorded.

Additional flow data were obtained for both wet and dry weather conditions by means of a meter installed temporarily at Pumping Station No. 1 (Fig. 6-9). Sewage reaching this station is generated in a 1,200 acre area near Lake Burien and comes also from an area immediately to the east and south. Dry weather flow from the two areas, which have an estimated total population of 6,750, averaged 0.5 mgd, or 74 gpcd (Fig. 7-19).

Sewage samples were taken at the Southwest Suburban treatment plant from August 12 through September 23, 1957. Values calculated from the analysis results (Table 7-3) reveal per capita contribution of 0.17 pounds per day for BOD and 0.23 pounds per day for suspended solids. These values are higher than those prevailing elsewhere in the metropolitan area and may be due in part to the high proportion of new homes equipped with garbage grinders.

Relative Freshness of Metropolitan Area Sewage

A characteristic common to the sewage from almost all parts of the metropolitan area is its generally fresh condition. This stems mainly from the large amount of ground water which enters the sewers. In addition to its diluting effect, ground water serves also to reduce sewage temperatures and thus to inhibit bacterial decomposition. Sewage freshness is reflected by the results of numerous dissolved oxygen tests which have been made in recent years by the engineering testing

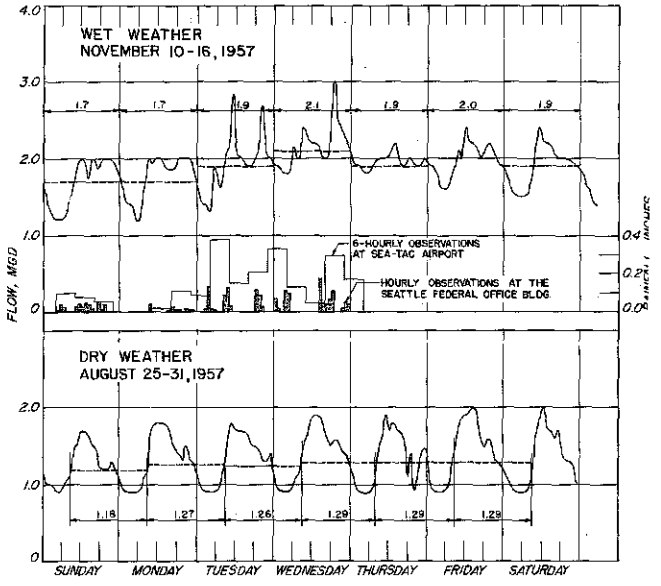


Fig. 7-18. Hourly Variation in Flow at Southwest Suburban Sewage Treatment Plant

Compared to other plants in the metropolitan area, wet weather flows show a much smaller increase in proportion to dry weather flows. This is attributed to the newness of the system as a whole and the use of approved jointing materials. Based on plant meter records.

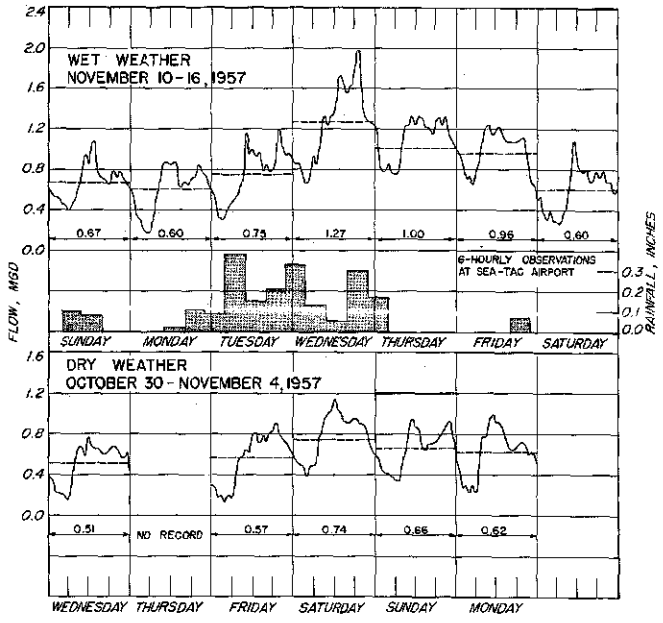
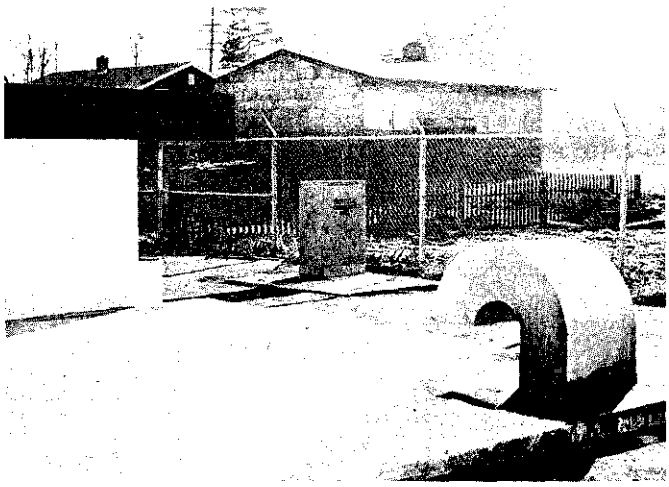


Fig. 7-19. Hourly Variation in Flow at Southwest Suburban Pumping Station No. 1

Sewers tributary to this station were constructed in 1957 and obviously have connections which allow entry of storm water.



FLOW METER installation of the bubbler type on influent sewer at Southwest Suburban Sewer District Pumping Station No. 1. Note proximity of residence beyond fence.

laboratory of the city of Seattle. These tests show concentrations ranging from 4 to 8 ppm in the winter months and of 2 ppm or more in summer months. In the summer, values below 2 ppm are found only near the downstream ends of the larger trunks.

Two benefits normally associated with weak, fresh sewage are the virtual elimination of odor conditions in and about pumping stations and treatment works, and the absence of concrete corrosion by hydrogen sulfide. In the Southwest Suburban Sewer District, however, the treatment plant influent was devoid of oxygen during summer months and had to be chlorinated for odor control. This condition appears to have been brought about by a combination of two factors, the first a low rate of infiltration, and the second a low rate of flow associated with the inception of a new system.

Occasional odor problems can be expected in other suburban areas as the sewerage systems are expanded.

Table 7-3. Sewage Characteristics at Sampling Points Outside Seattle

Date	Day	Average flow, mgd	BOD					Solids					
			Raw		Settled[a]	Total		Suspended			Settleable		
			ppm	lb/day	ppm	ppm	Volatile ppm	ppm	lb/day	Volatile ppm	ml/l	ppm	Volatile ppm
Kirkland Plant, July 1957													
9	Thur	0.32	210	561	122	683	411	564	1,505	490	12	468	379
10	Wed	0.31	225	580	120	778	463	478	1,230	382	15	378	274
11	Thur	0.31	165	424	109	558	334	254	653	231	10	196	148
12	Fri	0.30	164	409	104	686	494	224	558	192	6	196	165
16	Tue	0.30	174	428	80	666	392	478	1,175	376	11.5	400	304
18	Thur	0.33	201	560	118	644	372	330	919	240	10.5	146	106
22	Mon	0.33	295	806	200	788	482	582	1,590	425	15	424	370
Weighted average				540					1,090				
Shorewood Apartments, June and July 1957													
20	Thur	0.21	211	370	130	506	308	238	416	214	10	150	142
21	Fri	0.21	180	314	129	534	342	300	523	280	8	250	242
22	Sat	0.20	150	250	120	498	410	150	250	132	7	102	94
25	Tue	0.23	210	404	165	560	312	170	326	129	7.5	76	35
1	Mon	0.21	163	285	127	458	247	161	282	147	7.5	108	99
Weighted average				325					358				
Southwest Suburban Plant, August and September 1957													
12	Mon	1.3	171	1,850	82	656	403	428	4,640	318	11	344	296
13	Tue	1.3	206	2,230	103	652	408	334	3,610	258	13	254	240
15	Thur	1.3	230	2,490	104	544	294	232	2,520	192	10	196	182
16	Fri	1.3	204	2,210	130	678	428	254	2,750	230	10	208	188
19[b]	Thur	1.3	161	1,750	88	498	214	180	1,950	162	5	80	74
23[b]	Mon	1.3	180	1,950	138	520	338	202	2,190	166	8	96	62
Weighted average				2,080					2,950				

All analyses performed on raw sewage samples. See Fig. 7-1 for location of sampling points.

Temperature and pH during sampling period: Kirkland, 18° C, 7.3; Shorewood Apartments, 22° C; Southwest Suburban, 17° C, 7.1.

[a]*One hour settling period.*

[b]*September.*

They are not likely, however, to be serious nor are they likely to become a matter of general concern. For the most part, sulfide formation will be minimized not only by natural conditions of temperature and water quality but by taking advantage of a topography which makes it possible to maintain adequate velocities in all sewers.

Presence of Grit in Combined Sewage

One of the most difficult problems inherent in combined sewerage systems is that involved in removing grit brought in by storm flows. Street washings, by their nature, contain an appreciable quantity of gritty material. This material, combined with that introduced by the sanding of icy streets in winter months, is picked up in combined sewers and conveyed therein to points of disposal. No problems are encountered, of course, where the sewers terminate in direct discharges to open bodies of water. On the other hand, the delivery of grit laden sewage to a sewage treatment plant requires that adequate facilities be provided for its removal.

While it is desirable for design purposes to develop information concerning the quantity and the particle size of grit carried in a sewer system, it is virtually impossible to obtain such information in advance. For that reason, it is necessary to rely instead on experience at existing plants operating in the same vicinity under similar conditions.

With respect to the combined sewers in Seattle, operating records at the Diagonal Avenue treatment plant show but little grit removal during wet weather periods. Removal during dry weather averages 2 cubic feet per million gallons and reaches a maximum of 5 cubic feet. There are two reasons for the low removals during wet weather. First, velocities through the grit removal unit are such that the detention time therein is not sufficient to permit deposition and settlement. As a consequence, grit accumulates in the digester, thereby reducing its capacity and necessitating periodic removal. Second, two flow regulators at Michigan and Brandon streets upstream from the plant bypass a large portion of the storm flow to nearby receiving waters. These bypassed flows are taken from the lower portion of the sewer and thus carry the heavier particles of grit along with them.

Samples of grit were obtained from the digester at the Diagonal Avenue plant and were analyzed for gradation by the city engineering laboratories. Results of these analyses, together with those for grit found at the bottom of the North Trunk and Ballard interceptors, are presented in Fig. 7-20. This figure also includes a grading analysis of sand similar to that used for sanding city streets.

Grit from the digester consists mainly of material passing a 40-mesh screen, thus indicating that the grit removal device takes out most of the larger and heavier particles.

The high percentage of coarse material in the samples from the trunk sewers results from the classifying action of the storm flows and indicates that most of the fine material is kept in suspension and disposed of through the outfall.

In the North Trunk interceptor, 25 per cent of the deposit is 3 inches and larger in size. By comparison, all of the sand used for street sanding passes a 1/4-inch screen and has a gradation curve lying between those of the digester and sewer samples.

Grit in the bottom of the North Trunk sewer at Thorndyke Avenue and Emerson Street was found to be cemented together by an asphaltic material and have an average depth of two feet. This same cemented material was found off the submarine outfall of the North Trunk.

INFILTRATION AND STORM INFLOW

As used herein, the term "infiltration" denotes the flow or movement of water through the interstices or pores of a soil or other porous medium, and the subsequent access of such water to a sewer through cracks, breaks, and defective joints. On the other hand, the term "storm inflow" denotes the flow which

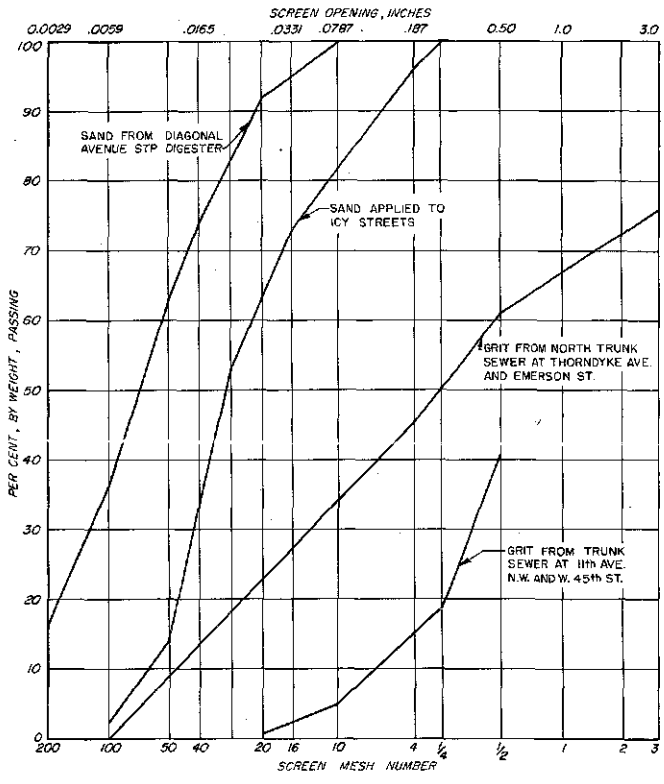


Fig. 7-20. Sieve Analyses of Grit Samples

Based on analyses by Engineering Testing Laboratory of the city of Seattle.

enters a sanitary sewer more or less directly during a rainfall and for a short period thereafter. These definitions overlap to some extent in that rainstorms, if long enough in duration, will cause a rise in ground water level and a resulting increase in the rate of inflow from saturated soils. Both storm inflow and infiltration are expressed herein as gallons per acre per day (gpad).

In general, infiltration and storm inflow are regarded as components of sewage volume. For the purpose of the present study, however, they require special consideration because of the serious problems they have created in most of the sewerage systems serving the metropolitan area. Severe infiltration is occurring, for example, in systems consisting entirely of separate sanitary sewers, most of which are designed to accommodate no more than a nominal amount of infiltration. Similarly, most of the sewage treatment plants are overtaxed with respect to hydraulic capacity and some of them, under storm conditions, either overflow or bypass raw sewage.

Combined systems, by their nature, are designed to accommodate large quantities of storm water. For that reason, the problem of ground water infiltration becomes important only as it relates to interceptor design. Interceptor sewers are designed normally to accommodate dry weather flow plus storm flow from low intensity rains, and to overflow or bypass during storms of moderate or higher intensity. Without an adequate allowance for dry weather infiltration, bypassing may be necessary even in the absence of rainfall.

Lake City System

As agreed upon in undertaking the sewerage survey, a detailed study was made of infiltration in the Lake City sewerage system. Elsewhere in the metropolitan area, studies were limited in scope and were undertaken primarily for corroborative purposes.

As a matter of background information, it should be noted that the entire Lake City system is of recent origin. Sewer construction did not begin until 1949, although it continued at a rapid rate thereafter until December of 1954. At that time, the State Pollution Control Commission issued a directive to the effect that no additions to the collection system would be approved until the sewage treatment plant, completed and placed in operation in late 1953, was enlarged to accommodate the excessive flow already being developed. In 1957, following the start of treatment plant additions, the commission ruling was withdrawn and further sewer construction was undertaken. Between 1955 and 1957, however, the sewered area remained essentially static at 2,900 acres. This system serves an estimated population of 25,000 persons and consists of 90 miles of sewers plus about 160 miles of house connections. Most of the sewers are 8 inches in diameter.

Available data on infiltration, consisting solely of flow records at the sewage treatment plant, indicate that troubles with infiltration and storm inflow have been more or less continuous ever since the plant was first placed in service. In 1954, the firm of Hill and Ingman, consulting engineers to the district, submitted a report on the problem of excessive flows at the treatment plant and the steps to take to correct them. Fig. 7-21, adapted from that report, is a plot of the hourly variation in flow for typical wet and dry weather periods in 1954, the first full year of plant operation. An analysis of the records for that year reveals that over 60 per cent of the total flow of approximately 1,300 million gallons, or 800 million gallons, was attributable to infiltration and storm inflow. On that basis, infiltration and inflow were roughly equivalent to a third of the annual rainfall received by the sewered area of 2,900 acres.

During the wet season, particularly when rain has been falling for several days at a low or moderate rate, a high intensity rainfall is manifested almost immediately by a sudden increase in metered flow at the sewage treatment plant. Flows at such times damp out any normal variation and result in a pattern similar to that of a storm hydrograph. Furthermore, they exceed the maximum rate which can be indicated by the treatment plant meter, which is 8.0 mgd. Readings on a depth gage ahead of the metering flume indicated flows as high as 11 mgd during the first year of operation.

Flows return almost immediately to near normal following the cessation of a heavy rainstorm. At such times, however, the minimum between the hours of 1 a.m. and 6 a.m. is about 1 mgd higher than it is normally. A period of two to three days usually elapses before the minimum returns to its pre-storm level.

In the five years the system was operated by the district, no action was taken toward a solution of the infiltration problem. Recently, however, specifications prepared by the city of Seattle in connection with the construction of sewers to serve a new area in Lake City call for the use of improved jointing materials. It can be expected, therefore, that infiltration in this particular area will be reduced to an acceptable level.

A contract in the amount of $760,000 was awarded on December 26, 1956 for enlargement of the sewage treatment plant. The enlarged plant has a design capacity of 10.5 mgd and a hydraulic capacity of 25 mgd.

Scope of Infiltration Study

Studies of the infiltration problem were directed toward the attainment of several objectives. These were:

1. The development of information concerning the magnitude of infiltration and storm inflow in various sections of the Lake City system. Information thus developed provides possible clues as to where remedial measures might be most effective.

2. The determination, if possible, of the actual sources of infiltration and inflow.

3. The development of measures to reduce infiltration in existing facilities as well as in facilities to be constructed in the future.

4. The development of design criteria both for areas presently sewered and areas to be sewered in the future.

As a first step in achieving the foregoing objectives, a program of continuous metering was established in trunk sewers and pumping stations tributary to the sewage treatment plant (Fig. 7-22). Continuous metering of flows was augmented by depth of flow measurements made in 50 manholes at various locations throughout the district. These measurements were obtained with the assistance of city sewer maintenance personnel and were made in May and again in August during low flow hours from 1 a.m. to 6 a.m.

Results of Infiltration Measurements

Unfortunately, the continuous metering program failed to achieve all of the desired results. Rainfall after April 14, the date when meter installations were completed, was abnormally low. In the following 30 days, rainfall amounted to 0.3 inches as compared to a normal of 2.0 inches. The only storm of sufficient intensity to be of any value to the infiltration study occurred on May 17 and May 18, 1957.

On May 16, typical dry season conditions prevailed at the two trunk sewer metering stations, as well as at the treatment plant (Fig. 7-23). In the early morning hours, 1 a.m. to 6 a.m., the plotted data show little or no variation in flow. Remembering, therefore, that the Lake City system consists entirely of sanitary sewers and that it serves an essentially residential area, it can be assumed that the flow during those hours was due almost wholly to the infiltration of ground water. In other words, the rate of 1.8 mgd then recorded represents a district-wide infiltration rate of 600 gpad.

Flows recorded during the several light rainfalls which occurred on May 17 and 18 are plotted in Fig. 7-24 and reflect the effect of a sudden influx of rain water in the early morning hours of May 18. At that time, the flow rate increased by approximately 1 mgd following a rainfall of nearly 0.20 inches in 4 hours. Based on flows at the two trunk sewer metering sta-

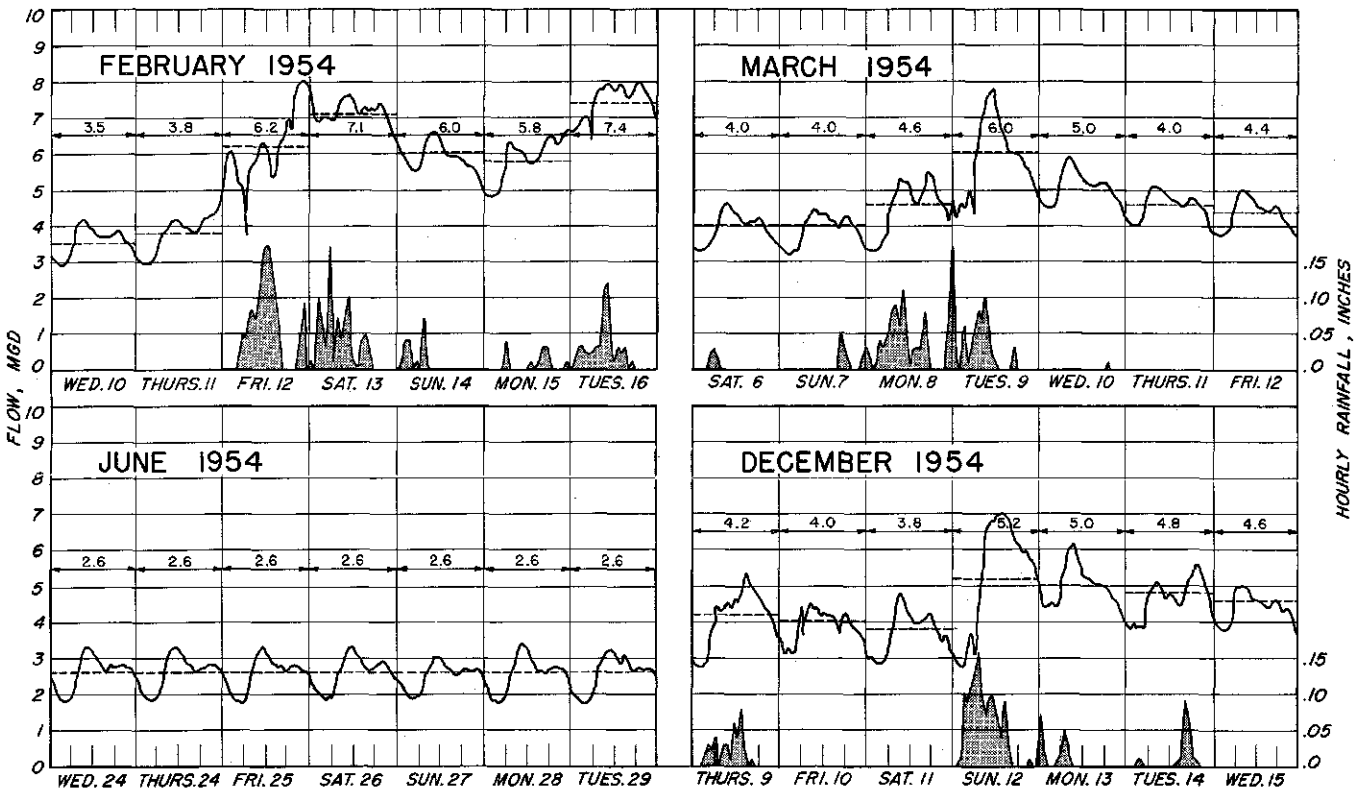


Fig. 7-21. Seasonal Variations in Hourly Flows at Lake City Sewage Treatment Plant

Average daily flows have consistently equalled or exceeded the treatment plant capacity of 2.5 mgd. Minimum rates decrease daily for several days following rainfall, thus indicating that rain water percolating downward through the soil gains access to the sewers. Based on report by Hill and Ingman, consulting engineers to Lake City Sewer District.

tions, the storm inflow contributions on an areal basis were about as follows:

Westerly trunk	0.35 mgd	1050 gpad
Northerly trunk	0.35 mgd	360 gpad
Remainder of district	0.40 mgd	390 gpad

Assuming that all of the early morning flow was infiltration plus storm inflow, the total thereof on May 18 amounted to 1,000 gpad for the entire district and to 2,200 gpad for the area served by the westerly trunk. These figures were confirmed by a rainfall of about equal intensity which occurred later the same day.

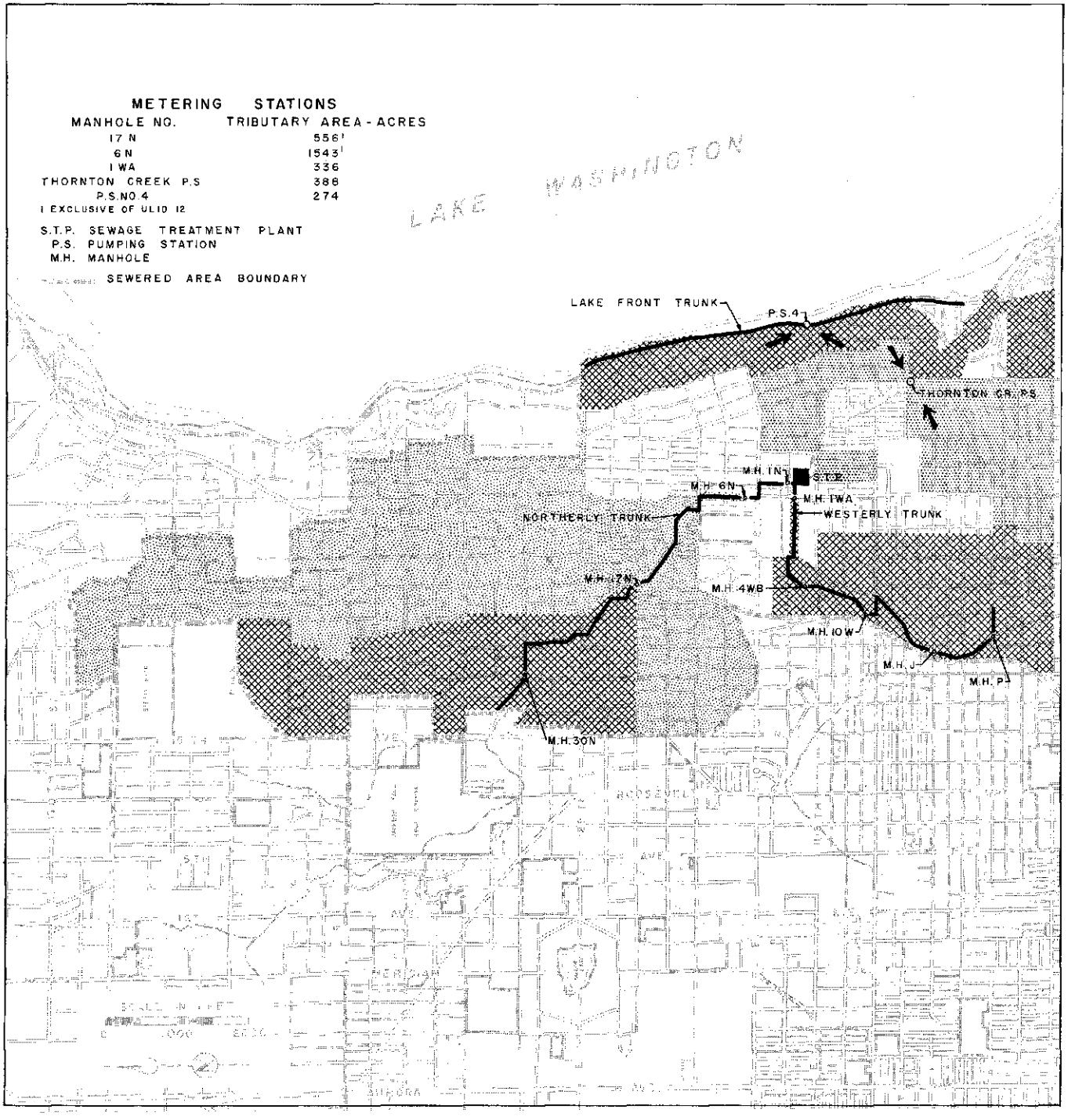


Fig. 7-22. Principal Sewers and Metering Stations in Lake City System

Continuous flow records were obtained from metering points established in manholes 1N, 6N, 17N, and 1WA, and from the two pumping stations. Additionally, manual gagings were made at selected manholes, especially on the westerly trunk, to determine the location of areas subject to excessive rates of infiltration.

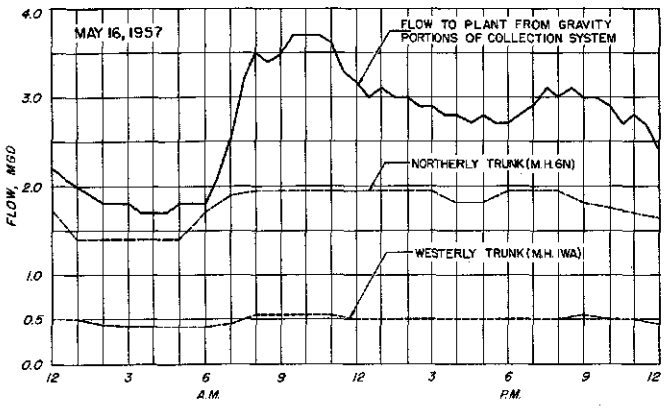


Fig. 7-23. Dry Weather Flow in Gravity Portions of Lake City System

Dry weather flows from the northerly and westerly trunks of the Lake City system account for a large proportion of the flow reaching the treatment plant by gravity.

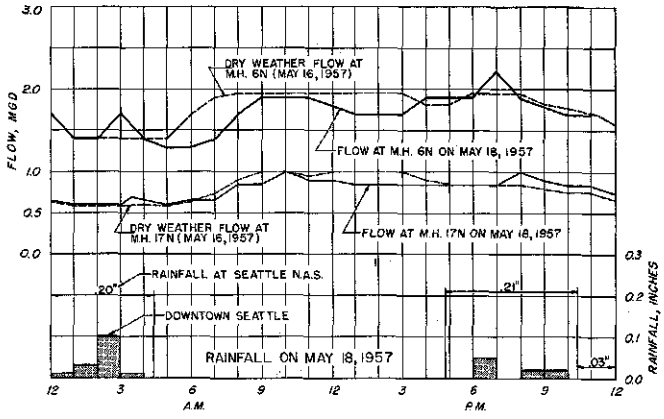


Fig. 7-25. Comparison of Dry and Wet Weather Flows in Northerly Trunk of Lake City System

A significant amount of storm inflow enters the northerly trunk between manhole 6N and manhole 17N.

A second metering station on the northerly trunk, situated upstream from the first, provided additional information on the tributary area (Fig. 7-25). Since flow from the upper station is tributary to that at the lower station, it is evident that the upswing in inflow which occurred at the latter reflects additional pick up between the two points.

As determined from flow measurements recorded at five metering points (Table 7-4), infiltration decreased from 915 gpad in May to 670 gpad by the end of August. Two conclusions are thus indicated. First, sewers in the Lake City system are subject to considerable leakage, and second, the rate of infiltration decreases with the decline in ground water level which occurs gradually during the summer season.

Seasonal low flow gagings listed in Tables 7-5 and 7-6 serve to identify the major sources of infiltration. On May 15-16, 1957, incremental flows from the upper reaches of the northerly trunk sewer showed rates of 1,700 gpad at manhole 33 and of 1,620 gpad at manhole 28AN. Both rates are relatively high, considering that the gagings in question were preceded by a dry period going back to approximately April 20. Proceeding downstream, as additional areas with little infiltration are added, the cumulated rate diminishes to 960 gpad at manhole 6N.

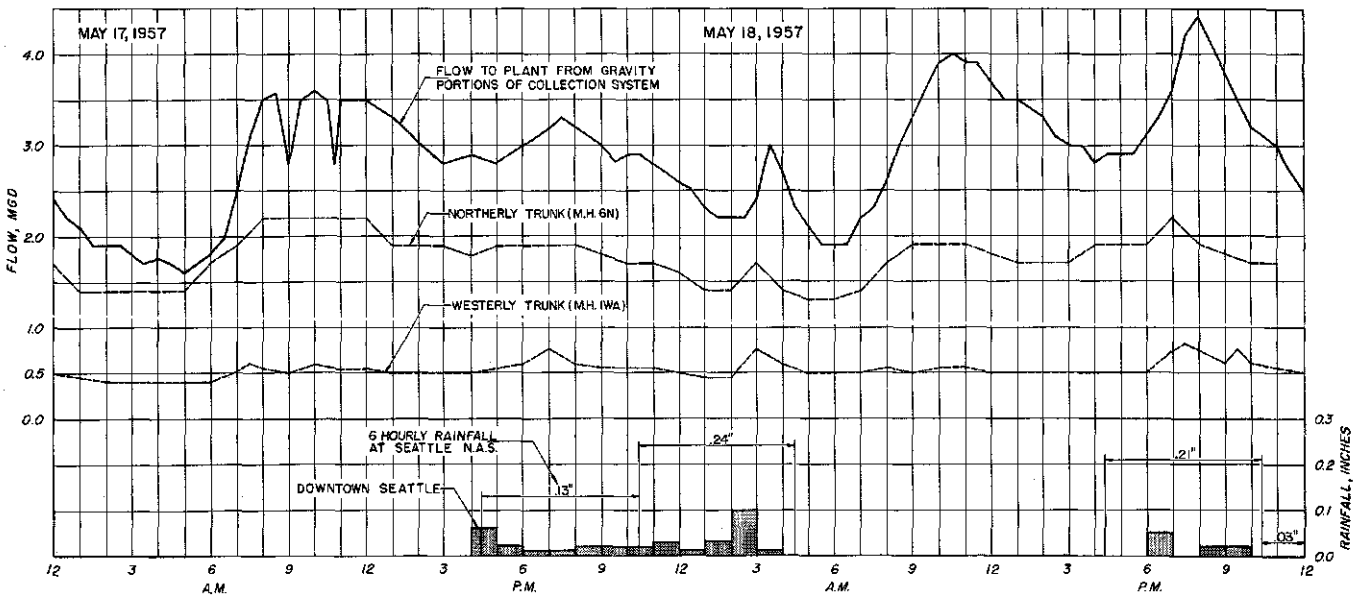


Fig. 7-24. Wet Weather Flow in Gravity Portions of Lake City System

After a slight rainfall on the evening of May 17, the increase in flow at the treatment plant was closely matched by an increased flow in the westerly trunk. Rainfall about 3 a.m. the following morning caused an increase in flow of 1.0 mgd at the plant. About 0.7 mgd of this was attributable to increased flow from the westerly and northerly trunks. Comparable increases occurred early in the evening of May 18.

Table 7-4. Summary of Infiltration Measurements, Lake City, 1957

Tributary area[a]		May 15-16 1957		Aug. 30-31 1957	
Metering point	acres	mgd	gpad	mgd	gpad
MH 6N	1,543	1.4	910	–	–
MH 1N	1,817	–	–	1.0	550
MH 1WA	336	0.42	1,250	0.45	1,340
Pumping Station No. 4	274	0.16	585	0.12	425
Thornton Creek Pumping Station	388	0.35	900	0.31	800
Summary	2,541	2.33	915	–	–
	2,815	–	–	1.88	670

[a]*See Fig. 7-23 for location.*

A comparison of the incremental rates on May 15 with those on August 30 shows a decline in all areas along the northerly trunk, with the exception of that tributary to manhole 28AN. The increase at this point apparently was caused by trench drainage from construction under way in ULID 12.

With respect to the westerly trunk, the early morning record (Table 7-5) reveals a high rate of infiltration, 2,080 gpad, in the tributary area between manholes 130 and 4WB. When these measurements were made, it was observed also that a continuous stream of ground water was entering manhole 4WB through a cracked section.

In order to determine the particular locality responsible for the high rate of infiltration, the August measurements (Table 7-6) included gagings at manhole 10W, which is situated between manholes 130 and 4WB. Readings there obtained demonstrated that most of the leakage was occurring in an 86 acre area tributary to the trunk between manholes 10W and 1WA.

Further information regarding minimum flows was obtained by timing pump operation at two pumping stations, Thornton Creek and Station No. 4. While some areas tributary to the Thornton Creek station (Fig. 7-22) show high infiltration rates (Table 7-5 and Table 7-6), their total contributions are relatively small and thus have no appreciable effect on the total flow. A major portion of the minimum flow at this station is from the area tributary to manhole 657 and from other areas which were not included in the depth gaging program.

Pumping Station No. 4 (Fig. 7-22) receives sewage from the sewered area of Lake City fronting on Lake Washington, including that picked up in a lake front interceptor. Infiltration in this area, as indicated by minimum flow measurements at the pumping sta-

Table 7-5. Minimum Flow Gagings, Lake City System, May 15-16, 1957.

Manhole number	Cumulative			Incremental		
	Tributary area, acres	Flow, gpd	Infiltration, gpad	Tributary area, acres	Flow, gpd	Infiltration, gpad
Northerly Trunk						
33	31	52,000	1,700	31	52,000	1,700
29	284	302,000	1,060	253	250,000	1,000
28AN	391	475,000	1,220	107	173,000	1,620
17N	556	600,000	1,080	165	125,000	760
6N	1,543	1,400,000	910	987	800,000	810
Westerly Trunk						
P	37	19,000	500	37	19,000	500
O	64	45,000	700	27	26,000	960
130	141	61,000	435	77	16,000	207
4WB	312	416,000	1,340	171	355,000	2,080
1WA	336	420,000	1,250	24	4,000	170
Thornton Creek Pumping Station						
499	68	25,000	368	68	25,000	368
467	125	74,000	590	57	49,000	865
571	135	88,000	650	10	14,000	1,400
593	147	121,000	820	12	33,000	2,750
657	234	196,000	840	87	75,000	860
Pumping station	388	350,000	900	154	154,000	1,000
Pumping Station No. 4						
344	6	0	0	6	0	0
390	54	19,000	350	48	19,000	390
429	68	20,000	290	14	900	61
427	94	56,000	590	26	36,000	1,400
Pumping station	274	160,000	585	180	104,000	580

Table 7-6. Minimum Flow Gagings, Lake City System, August 30-31, 1957

Manhole number	Cumulative			Incremental		
	Tributary area, acres	Flow, gpd	Infiltration, gpad	Tributary area, acres	Flow, gpd	Infiltration, gpad
Northerly Trunk						
33	31	17,000	560	31	17,000	560
29	284	238,000	840	253	221,000	875
28AN	391	605,000	1,550	107	367,000	3,400
17N	556	650,000	1,170	165	45,000	270
2-2A	852	810,000	950	296	160,000	540
867	1,298	1,070,000	825	446	259,000	580
1N	1,817	1,000,000	550	519	–	–
Westerly Trunk						
P	37	16,000	425	37	15,800	425
O	64	32,000	500	27	16,000	600
130	141	42,000	300	77	10,000	130
10W	250	117,000	470	109	75,000	690
4WB	312	345,000	1,100	62	228,000	3,700
1WA	336	450,000	1,340	24	105,000	4,400
Thornton Creek Pumping Station						
499	68	13,000	190	68	13,000	190
467	125	29,000	230	57	15,800	280
593	137	64,000	470	12	34,600	2,880
657	224	156,000	700	87	92,000	1,050
Pumping station	388	310,000	800	164	154,000	940
Pumping Station No. 4						
344	6	0	0	6	0	0
390	54	17,000	310	48	17,300	360
429	68	22,000	320	14	4,300	310
Pumping station	274	116,000	425	206	94,000	460

Table 7-7. Infiltration and Storm Inflow, Lake City Plant

Month	Flow, mgd			Storm inflow[c]		Infiltration[d]	
	Minimum[a]	Peak	Normal[b]	mgd	gpad	mgd	gpad
1956							
January	3.0	14.0	4.0	10	3,400	3.0	1,000
1954							
February	3.0	10.0	4.0	6.0	2,100	3.0	1,000
March	3.4	7.8	4.3	3.5	1,200	3.4	1,200
April	2.6	4.8	3.8	1.0	340	2.6	890
May	2.0	3.5	3.5	0	0	2.0	690
June	1.5	3.8	3.8	–	–	1.5	510
July	1.9	4.5	3.4	1.1	380	1.9	650
August	1.6	3.6	3.4	0.2	69	1.6	550
September	1.6	4.3	3.5	0.8	270	1.6	550
October	1.6	3.7	3.7	0	0	1.6	550
November	2.3	5.2	3.3	1.9	650	2.3	790
December	2.7	7.0	3.5	3.5	1,200	2.7	930

Tributary area, 2,920 acres.

[a]*Minimum flow rate for a period of relatively dry days.*

[b]*At same time as peak flow but on a previous dry day.*

[c]*Peak rate minus normal rate.*

[d]*Assuming minimum flow predominantly infiltration.*

tion and at various manholes, is consistently low. Manhole 344, which is in the area tributary to Station No. 4, was the only one of the 50 gaged in the entire Lake City district which showed no early morning flow.

Areas of the Lake City sewer system subject to summer infiltration rates in excess of 1,500 gpad are depicted in Fig. 7-26. Conditions there indicated are based on results of early morning gagings at the 50 manholes.

Because of the lack of storm flows during the metering program, information regarding wet weather conditions had to be obtained from the treatment plant meter charts (Fig. 7-27). Readings for January 6,

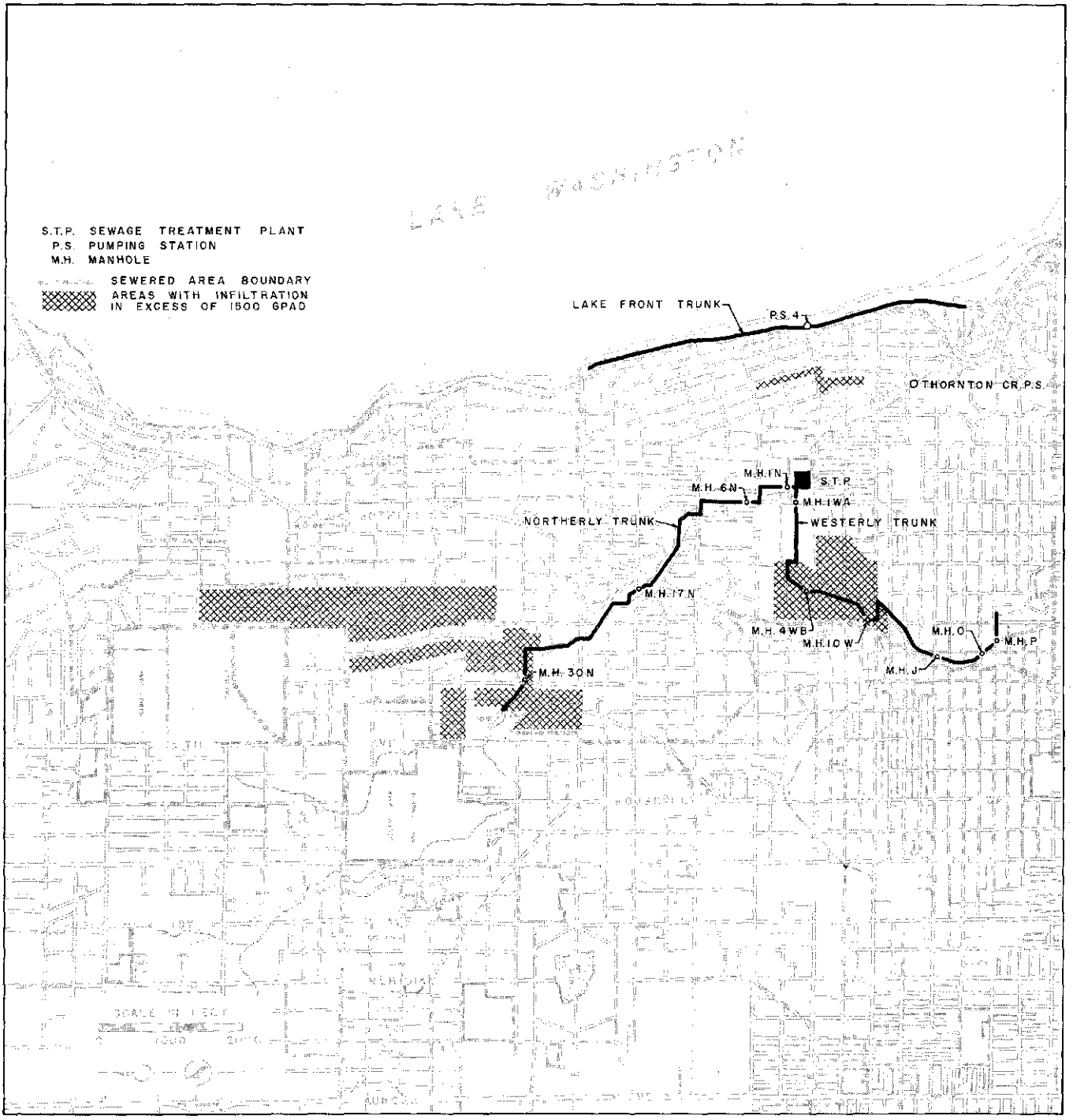


Fig. 7-26. Areas with Excessive Infiltration Rates in Lake City System

Based on results of the metering and minimum flow gaging programs. Remedial measures to reduce the quantities of storm inflow and infiltration to sanitary sewers should be undertaken first in the areas here indicated.

1956 are included because the peak rate of input on that day was the highest ever observed at the treatment plant.

Peak rates of infiltration and storm inflow, as obtained from the curves in Fig. 7-27, are listed in Table 7-7. It will be seen that the peak storm rate amounted to 3,400 gpad and that winter infiltration ranged from 1,000 to 1,200 gpad. During the summer months, storm inflow rates were generally less than 100 gpad, while infiltration ranged from 500 to 650 gpad.

Hourly records of storm flows at the Lake City treatment plant (Fig. 7-21) are indicative of two general conditions.

1. A sudden increase in flow with the advent of a storm. This is due largely to direct connections of catch basins, downspouts, and other illegal sources of storm input.

2. A gradual reduction in minimum flows at the end of a storm. This indicates a continuing but diminishing input of storm water stored in surface soils at an elevation higher than that of the normal ground water. Water thus accumulated may enter the sewer system through faulty joints in house connections or through foundation drains. The latter are installed for the purpose of protecting basement walls and may be directly connected to house sewers. No information is available, however, as to the number of them in use in the Lake City area.

A study in Erie County, New York,[1] showed an average inflow from foundation drains of 3,000 gpd

[1]*Spencer, R.C.,* Standards for Sanitary Sewers and Present Community Needs, *Sewage and Industrial Wastes, 26, September 1954.*

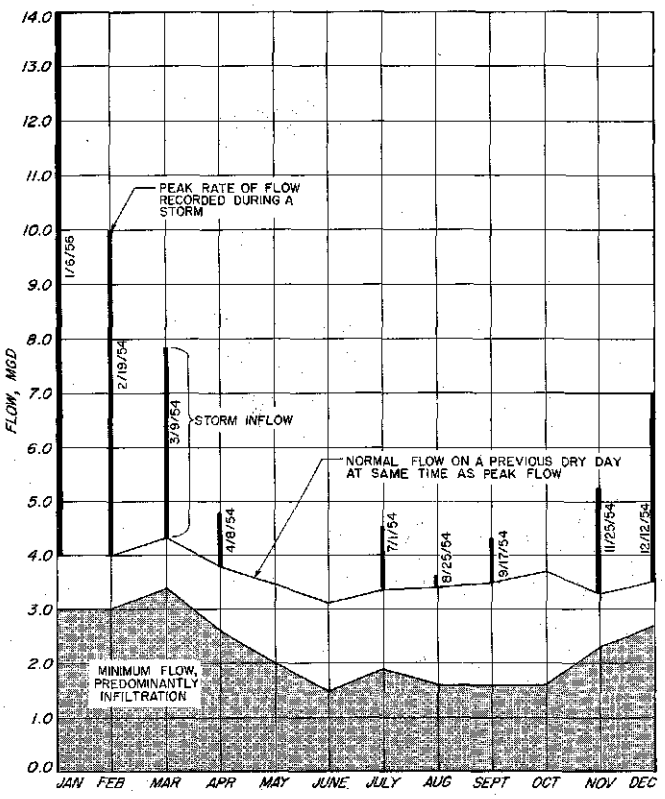


Fig. 7-27. Average Monthly Infiltration and Peak Storm Flows at Lake City Sewage Treatment Plant

The upper limit of the shaded area represents the average, by months, of minimum daily rates of flow. Storm inflow is represented by the difference between the peak rate and the normal dry weather rate at the same time of day or days preceding the storm. Storm inflow values are the maximum recorded each month. Based on plant flow records.

Table 7-8. Infiltration and Storm Inflow, Other Separate Sanitary Systems

Sewage treatment plant	Tributary area, acres	Dry weather infiltration[a]		Wet weather infiltration and storm inflow[a]	
		mgd	gpad	mgd	gpad
City of Seattle — Greenwood plant	1,320	0.4[b]	300	3.6	2,700
Auburn	1,300	0.2	150	–	–
Bellevue	700	0.02	30	0.8[c]	1,140
Bryn Mawr	510	0.08	160	1.1[c]	2,200
Kent	1,100	0.2	180		
Kirkland	1,300	0.15	115	1.9	1,500
Renton	2,100	0.4	190	1.3[d]	600
				0.5[e]	240
Shorewood Apartments	75	0.12	1,600	0.07	900
Southwest Suburban	1,200	0.21	175	1.45[f]	1,200

Hourly variations in flow for typical wet and dry weather weeks shown on Figs. 7-6 and 7-12 through 7-19.

[a]*Flow rate during early morning hours except where noted.*

[b]*Average dry weather flow less winter water consumption for area.*

[c]*Capacity of flow meter, rate shown probably exceeded.*

[d]*Storm inflow occurred during other than early morning hours.*

[e]*Infiltration only.*

[f]*Data for separate system tributary to pumping station No. 1.*

ROOF LEADERS which disappear below ground should be suspected as a possible source of storm inflow in areas served by separate sanitary sewers. By pouring a small quantity of water containing dye into the roof gutter and observing the sewage flow at a downstream manhole, it can readily be determined whether or not a leader is connected to the sewer.

per house in a suburban residential district. At four houses per acre, this flow would amount to a total of 12,000 gpad. Hence, if drains from only one house in twelve were connected to a sanitary sewer, the resulting increase in flow would be 1,000 gpad.

Based on the findings here reported, it is concluded that rates of infiltration and storm inflow vary over a wide range in the Lake City system. Certain local areas have excessive rates and should be examined further under the remedial program described later in this report.

Infiltration and Storm Inflow in Other Separate Systems

Information on infiltration and storm inflow in other metropolitan area systems served by separate sanitary sewers was obtained by examining the meter charts at a number of treatment plants. There is some doubt, however, as to the reliability of the flow rates indicated by certain of these meters under storm flow conditions. This is because most of the plants are heavily overloaded at such times and because there is a possibility also of overflow upstream from the meter.

Peak rates of infiltration and storm inflow, as determined from meter readings recorded early in 1956, are listed in Table 7-8. This table also lists the dry weather rates and is based on data presented previously in the section dealing with sewage flows.

In considering the foregoing data, it should be recognized that leakage and inflow can be expected in the future to receive increasing attention on the part of engineers concerned with sewer system design. This trend is already apparent and is manifested in some areas by the use of concrete pipe with rubber ring joints.

To determine the effect of the rubber rings, a limited metering program was undertaken in several areas served by the Southwest Suburban Sewer District. In so doing, flows from the White Center system of combined sewers, which are tributary to the sewage treatment plant, were excluded by establishing the metering station on the 24-inch influent line at Pumping Station No. 1 (Fig. 6-9). This station, situated near Lake Burien, serves an area of approximately 1,200 acres and receives sewage both by gravity flow and by pumping. Pumped flows originate in Pumping Station No. 3, which discharges into the gravity system a short distance upstream from Station No. 1.

Dry weather flows at Station No. 1, as measured from October 30 to November 4, 1957, averaged 0.60 mgd and dropped to a minimum of 0.10 mgd during the early morning hours (Fig. 7-19). In the next week, however, following the first prolonged rainfall of the season, the average flow jumped to 1.27 mgd on November 13 and the minimum to 0.80 mgd on November 14.

To ascertain the source of storm inflow, separate measurements were made of the gravity flow and of that originating at Station No. 3 (Fig. 7-28). It is evident from the plotted data that a major portion of the inflow originated in the gravity system. For the period of measurement, the flow from Station 3 re-

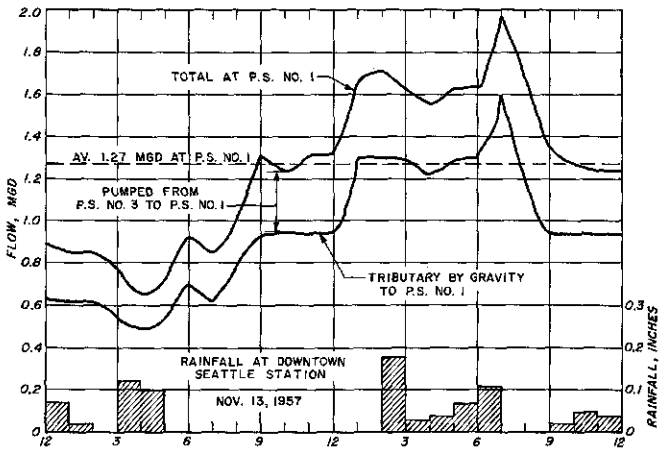


Fig. 7-28. Wet Weather Flow at Southwest Suburban Pumping Station No. 1

Flow measurements during a rain storm on November 13, 1957 reveal that most of the storm inflow originates in the area tributary by gravity rather than in the area served by Pumping Station No. 3.

Table 7-9. Infiltration and Storm Inflow, Southwest Suburban Sewer District

Tributary area		Infiltration				Storm inflow	
		Dry season		Wet season			
Designation	Acres	mgd	gpad	mgd	gpad	mgd	gpad
Pumping Station No. 1[a]	520	0.10[b]	192	0.24[c]	460	1.0[d]	1,920
Pumping Station No. 3	680	0.11[b]	162	0.11[c]	162	0.1[d]	147
Total at Pumping Station No. 1[e]	1,200	0.21[b]	175	0.35[c]	291	1.1[d]	915

See Fig. 6-9 for location of pumping stations.

[a]*For tributary area served by gravity system.*

[b]*Minimum flow - October 30, 1957.*

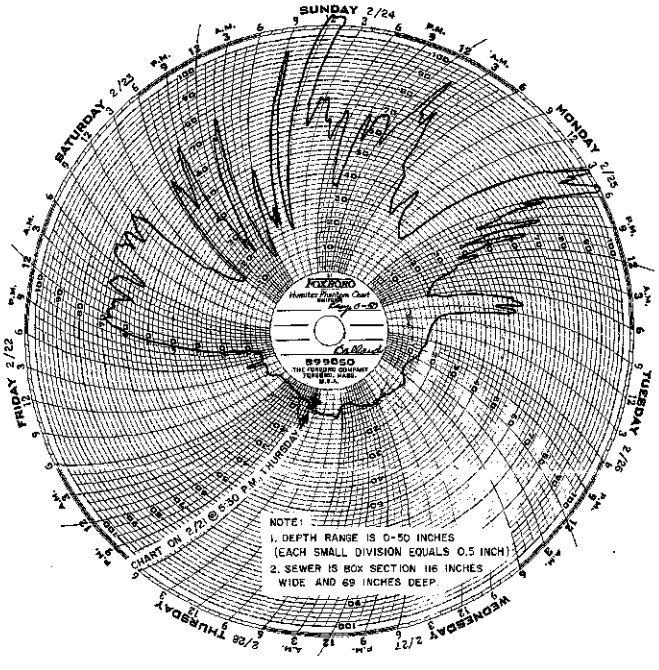
[c]*Minimum flow - December 4, 1957.*

[d]*Peak flow on November 13, 1957, less normal dry weather peak flow.*

[e]*Pumping station No. 3 discharges to system tributary to No. 1.*

mained essentially the same in wet weather despite the fact that the tributary areas are about equal in extent and that Station 3 serves a total of 1,075 connections as compared to 851 in the gravity system.

In the area tributary by gravity to Station No. 1, peak storm inflow amounted to 1,920 gpad (Table 7-9). By comparison, the peak in the area tributary to Station No. 3 was only 145 gpad. If it can be assumed that ground water conditions in the two areas are equivalent, the latter figure seems to indicate that rubber ring joints are effective in minimizing infiltration. On that basis, the high rate of inflow in the gravity area indicates that storm water is reaching the sewers from sources other than faulty joints.



DEPTH OF FLOW CHART from portable meter installed at Ballard trunk metering station shows wet and dry weather conditions during week of February 21-28, 1957. A side weir, located at this point, overflows combined sewage to the Ship Canal when the depth of flow exceeds 31 inches. During a storm shortly before noon on Sunday, February 24, the overflow outlet became clogged and sewage overflowed through the manhole onto the ground surface. This resulted in partial inundation of the immediate area.

Combined Systems

Information concerning infiltration in the combined sewers of the city of Seattle was obtained from meter charts at the Diagonal Avenue plant and from measurements at three metering stations (Fig. 7-3). These sources yielded the data presented in Table 7-10, which shows an average infiltration rate of 950 gpad during the dry season. Wet season infiltration at the Rainier and Ballard stations, neither of which is subject to upstream overflows, averaged 2,100 gpad.

In arriving at the figure for the area tributary to the Rainier and North Trunk metering stations, it was assumed that the minimum flow again was due entirely to infiltration. This assumption, however, is not entirely valid because of the light industrial and commercial operations which are being carried

Table 7-10. Infiltration in Seattle Combined Systems

Sewered area		Infiltration			
		Dry season		Wet season	
Designation	Acres	mgd	gpad	mgd	gpad
North Trunk	12,700	14	1,100	b	b
Diagonal Avenue	5,100	3.0	600	b	b
Rainier	1,100	2.0	1,800	1.7	1,550
Ballard	1,250	0.2	150	3.2[c]	2,500
Total	20,150	19.2	950	4.9	2,100

See Fig. 7-3 for locations.

[a]*In absence of rainfall.*

[b]*No metering during winter; large number of overflows upstream render data meaningless.*

[c]*Average flow less winter water consumption for area.*

on in areas tributary to those stations. Infiltration in the area tributary to the Ballard metering point was computed by deducting average winter water consumption from the average sewage flow.

CHARACTERISTICS OF INDUSTRIAL WASTE

Industrial waste disposal conditions and practices are subject to increasingly critical scrutiny by regulatory agencies and by various groups interested in protecting and preserving surface water resources. As a consequence, more rigid controls and more rigid standards with respect to disposal operations can be anticipated in the future. Liquid wastes presently discharged directly to receiving waters either will have to be treated by the responsible industry or will have to be disposed of in a publicly owned sewage treatment works. Except for very large quantities and for waste requiring special treatment, the latter method is generally the more feasible from the standpoints of economy and convenience.

For planning purposes, the two most important criteria with respect to the handling of industrial wastes are those of volume and composition. As to volume, which is the more critical of the two, sufficient capacity must be provided in trunk and intercepting sewers to avoid costly and unnecessarily paralleling at some later date. Treatment capacity, except for such structures and channels as are designed to meet ultimate requirements, may be provided as it becomes necessary. With sufficient hydraulic capacity in basic units, future increases in BOD and suspended solids loadings can be accommodated satisfactorily by the incremental addition of sedimentation tanks and sludge digestion facilities.

As a final note on the general problem of industrial waste disposal, it should be pointed out that a public sewerage authority is not necessarily obligated to receive such wastes without regard to their volume or composition. Depending on local circumstances, varying degrees of pretreatment may be required of an industry prior to acceptance of its wastes.

Sources and Volume of Industrial Waste

In Washington, every industry producing a waterborne waste, whether it be discharged independently or to a public sewer system, is required to obtain a permit from the State Pollution Control Commission. An abstract of data pertaining to all such permits and permit applications in the metropolitan area was provided by the Commission and is the principal source of information used in this report. This abstract covers a total of 150 industries, of which 58 produce waste flows of 50,000 gpd or more (Table 7-11).

Because of the seasonal nature of some of the industries, it is not likely that the maximum discharge rates occur simultaneously. As a consequence, the totals given in Table 7-11 are higher than those which are actually encountered.

For convenience in sewerage planning, industrial waste contributions from areas which have no significant seasonal operations, can be estimated on a gross acre basis by using records of winter water consumption. To that end an analysis was made of

Table 7-11. Major Industrial Waste Sources, 1957

Industry group	Seattle			Auburn	Kent	Renton	Others	Total in metropolitan area
	Puget Sound	Elliott Bay	Duwamish Waterway					
Number								
Food and beverage	5	7	8	2	3		1	26
Metals	1	2	7		1			11
Metal plating			6			2		8
Chemicals			4					4
Miscellaneous	1	1	3			2	2	9
Total	7	10	28	2	4	4	3	58
Waste quantity, 1,000 gpd[a]								
Food and beverage	806	1,759	2,000	221	970		100	5,856
Metals	285	1,380	21,261		110			23,036
Metal plating			6,769			1,686		8,455
Chemicals			2,764					2,764
Miscellaneous	4,338	84	1,119			530	402	6,473
Total	5,429	3,223	33,913	221	1,080	2,216	502	46,584

Source: "Industrial Waste Sources in the Seattle Metropolitan Area," Pollution Control Commission, State of Washington, 1957.

Data for industries having maximum waste volumes less than 50,000 gpd are not included in this table.

[a]*For maximum day; includes estimated maximums for seasonal industries.*

winter water use in a 975 acre tract situated along Duwamish Waterway from Harbor Island to Boeing Field. At present between 20 and 30 per cent of this area either is vacant or is reserved by existing industries for future expansion.

As reported in Chapter 4, winter use in the 975 acre tract averaged 3.1 mgd, or 3,200 gpad. In the three meter routes for which the data were obtained, the average ranged from 1,840 to 3,700 gpad.

At the risk of being repetitive, it should be pointed out again that higher water consumption in this particular area occurs during summer months when large quantities are used for gravel washing and cold storage operations. In both of these, however, the resulting wastes are suitable for direct discharge to adjacent waterways and thus are not significant with respect to sewerage planning.

Strength and Composition of Industrial Waste

Virtually no information is available concerning the strength and composition of specific industrial wastes in the Seattle area. Based, however, on analyses of samples taken from the influent to the Diagonal Avenue treatment plant and from the North Trunk sewer, it is evident that the industrial waste component of these flows has a strength no greater than that of the sanitary sewage. For the Diagonal Avenue plant, after deducting the BOD contribution of the resident population tributary thereto, it is estimated that the industrial wastes have a population equivalent of only 15,000 persons. The nature of the principal industries tributary to this plant and the approximate volumes of waste they produce are given in Table 7-12.

The presence in industrial waste of substances toxic to fish and other aquatic life is a matter of great concern in the Puget Sound region, particularly where disposal is to inland waters which provide little dilution. Wastes from metal plating works, of which there are a relatively large number in the Seattle area (Table 7-11), are especially hazardous. Separate pretreatment of these wastes, as well as of certain others which are more than normally hazardous, is now required by the Pollution Control Commission. In some instances, pretreatment may have to be continued unless dilution by sanitary and other flows increases to an extent sufficient to reduce the concentration of toxic materials to tolerable limits. Many of these materials cannot be removed by ordinary sewage treatment processes. Furthermore, some of them are likely to have a deleterious effect on the biological processes involved in sludge digestion and in secondary treatment.

Despite the fact that industrial wastes reaching the Diagonal Avenue plant appear to have caused little or no operational trouble, the same situation will not necessarily prevail when treatment facilities are provided for the remainder of the Seattle area. For example, the bituminous material found in the North Trunk is capable of creating operating difficulties. Other materials which should be excluded from a sewer system include those which are capable either of structural damage by direct attack or of creating an explosive atmosphere. In other words, strong acids and alkalis should be excluded, as should gasoline, kerosene and other petroleum products.

Table 7-12. Sources of Industrial Wastes Tributary to Diagonal Avenue Sewage Treatment Plant

Nature of operation	Number of industries	Waste volume,[a] 1,000 gpd
Adhesives and related chemicals	2	851
Beverage bottling	1	2.5
Cement handling and distribution	1	0.5
Compressed gasses	1	1,150
Food canning	1	170
Metal plating	2	65.7
Sawmill	1	5
Steel fabrication	1	5
Truck manufacturing	1	225
Other	3	72.5
Total	14	2,547.2

Source: "Industrial Waste Sources in the Seattle Metropolitan Area", Pollution Control Commission, State of Washington, 1957.

[a]*Maximum day.*

Experience in other cities has shown that industrial waste abuses of a sewer system, while usually unintentional, are often undetected until treatment works are constructed. Prevention of such abuses is best achieved by (1) tracing such materials to their sources, (2) maintaining an educational program for waste producing industries, and (3) enacting a comprehensive ordinance governing the use of the sewerage system. Seattle does not now have such an ordinance.

SUMMARY OF SEWAGE CHARACTERISTICS

Data with respect to sewage volume and composition, as measured in metropolitan Seattle for both residential and combined residential and industrial areas, are summarized in Table 7-13. Figures there listed, coupled with appropriate supporting information, are utilized in developing basic criteria for planning and design purposes.

Volume of Sanitary Sewage

A sanitary sewage contribution of 60 gpcd is considered suitable for design purposes and represents a moderate increase compared to the present contribution. For trunk sewers within a major sewerage area, a peak flow of 175 per cent of average is selected and

Table 7-13. Summary of Sewage Characteristics

Location	Estimated population[a]	Tributary area, acres	Sewage strength and composition				Winter water use, gpcd	Sewage volume		Infiltration	
			BOD		Suspended solids			Dry weather, mgd	Wet weather, mgd	Dry season, gpad	Wet[b] season, gpad
			lb/day	ppcd	lb/day	ppcd					
Seattle											
North Trunk	195,000	12,700	32,400	0.17	47,000	0.24		35.0		1,100	
Diagonal Avenue	30,000	5,100	4,470	0.15	7,590	0.25		4.3		600	
Lake City	25,000	2,920	3,500	0.14	5,290	0.21	53	3.0	8.0	550	4,400
Ballard	24,500	1,250	2,140	0.09	2,150	0.09	58	1.7	5.0[c]	150	2,500
Rainier	22,000	1,100						3.0	2.5[c]	1,800	1,550
Greenwood	16,000	1,320	1,330	0.08	1,710	0.11	62	1.4	3.4	300	2,700
Auburn	2,000	1,280					57	0.6		150	
Bellevue	4,100	700					57	0.14	0.8	30	1,140
Bryn Mawr	4,500	512					48	0.2	1.1	160	2,200
Kent	4,000	1,090					66	0.7	2.2	180	2,200
Kirkland	5,750	1,280	540	0.09	1,090	0.19	55	0.3	2.0	115	1,500
Renton	16,500	2,110						0.7	1.0	190	840
Shorewood	2,300	75	325	0.14	358	0.16	62	0.2	0.2	1,600	900
Southwest Suburban	12,600	1,200	2,080	0.17	2,950	0.23	60	1.3	3.8	175[e]	1,206[e]
Total	369,250	32,637	46,785		68,138			52.5	31.7		
Weighted average				0.15		0.22	56				

[a]*Based on estimates from Seattle Planning Commission and on data furnished by individual sewerage agencies.*

[b]*Wet season values include storm inflow except as noted for systems served by combined sewers.*

[c]*Combined system; values are for average flow during wet season exclusive of storm runoff.*

[d]*Based on contributory population of 311,000 included in sampling program.*

[e]*For separate sanitary system tributary to pumping station No. 1 (Table 7-9).*

is believed sufficient to allow properly for momentary peaks. For trunk sewers serving more than one major sewerage area and for sewage treatment plants, a ratio of 150 per cent of the averages is selected and is considered appropriate for the large areas which such facilities normally serve.

Volume of Industrial Waste

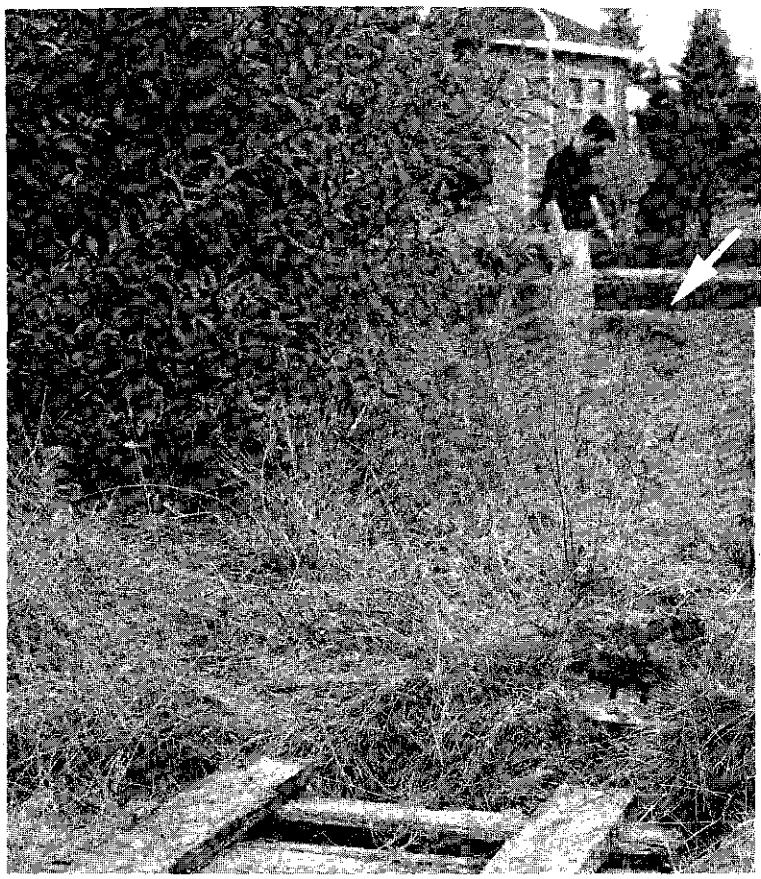
For presently developed industrial zones, an average waste contribution of 4,000 gpad and a peak of 8,000 gpad are in line with present experience. In the case, however, of new industrial zones, the tendency toward less intense land utilization is resulting in a lower unit use of water per acre and thus in a correspondingly lower unit production of liquid wastes. This tendency stems from the provision of adequate parking space for employees, of adequate room for essential transportation facilities, and of landscaping to create a pleasant working environment. For new light industrial zones and for heavy industrial zones occupying an area of 1,000 acres or more, an allowance of 2,000 gpad for average flow is considered sufficient. For heavy industrial zones smaller than 1,000 acres, the possibility that one or two fairly "wet" industries might locate therein calls for application of a higher allowance, namely, an average of 4,000 gpad. With respect to peak rates of flow, values equal to 300 per cent of average for light industrial areas and 200 per cent of average for heavy industrial areas are appropriate.

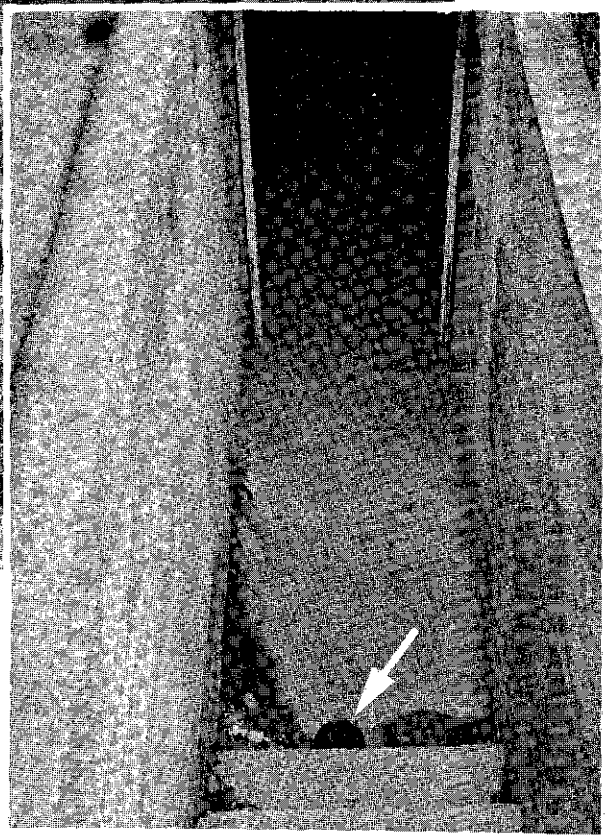
It is assumed for all new areas that cooling water and other clean waste water will be disposed of separately. In all cases, of course, infiltration and storm flow allowances must be added to the liquid waste allowances.

Infiltration and Storm Inflow

Improvements in construction practices, coupled with new developments in pipe jointing techniques, have demonstrated elsewhere that infiltration rates can be achieved in future construction which will be substantially lower than those observed during the present survey. Likewise, the provision of adequate storm drainage in areas presently lacking such facilities, supplemented by a program to eliminate obvious sources of inflow, can bring about a major reduction in direct storm inflow to sanitary systems.

Infiltration. Minimum flows measured during the metering program naturally included a small amount of sanitary sewage in addition to ground water. For that reason, and because of the reduction of infiltration which could be achieved by the procedures outlined above, the quantities selected as applicable to





ILLEGAL CONNECTIONS, of which these two are typical examples, permit storm inflow to the Lake City system. In the photograph above overflow from the wooden catch basin in foreground is conveyed to manhole in background (arrow). In the photograph at right, the drain (arrow) at the base of an exterior stairwell conveys stairwell drainage, as well as that from an adjacent patio, to a sanitary sewer.

existing construction are 1,200 gpad for the wet season and 300 gpad for the dry season. These apply to both combined and separate sewers. In the design of interceptor sewers, however, consideration will have to be given to infiltration conditions in each specific sewerage area.

For new construction involving separate sanitary sewers, a wet season allowance of 600 gpad will provide a safety factor of about 100 per cent over and above that which could reasonably be attained. This allowance implies the use of suitable jointing materials and the provision of adequate inspection during construction of sewers and house connections. Because of reduced ground water levels, a design allowance of 300 gpad will suffice for dry season conditions.

Storm Inflow to Sanitary Sewers. Storm inflow to existing sanitary sewers can be reduced materially by means of the corrective measures described in Chapter 16. An allowance of 2,000 gpad, which rate is seldom exceeded in the Lake City system, is therefore suitable for design purposes. In other words, the combined infiltration and inflow allowance amount to a total of 3,200 gpad. Records for all other separate systems indicate that the wet weather rates therein do not exceed this total.

In the case of new construction, assuming availability of adequate storm drainage facilities and appropriate regulations to prevent direct connection of catch basins, downspouts and foundation drains, no allowance need be made for summer storms. For winter storms, an allowance of 500 gpad is advisable in order to accommodate minor inflow from undetected sources.

Composition

In recent years, a general increase in the strength of sanitary sewage has been noted in many cities and

is believed to reflect the effect of several factors, notably the increased use of home garbage grinders. Design criteria must allow for a continuation of this trend. In the case, therefore, of treatment plants serving areas of primarily residential and commercial development, unit values appropriate for planning purposes are 0.20 ppcd for BOD and 0.25 ppcd for suspended solids.

For the Diagonal Avenue treatment plant, which serves a diversified industrial area, available analysis results (Table 7-13) indicate that the strength of the wastes now being received is similar on the whole to that of sanitary sewage. As far as the future is concerned, it is possible that the strength of the industrial wastes may increase substantially as a result of changes in the industrial pattern. This, however, will pose no particular problem, providing trunk sewers and treatment plant structures are designed with a capacity sufficient to accommodate the increased flow. On that basis, any increase in strength could be accommodated by constructing additional digestion and sludge handling facilities.

Assuming, therefore, that sewage and industrial waste strengths will remain equal, equivalent population values can be assigned to industrial areas on the basis of average daily waste volume. For the contributions referred to above, the equivalent populations are 33 persons per acre at 2,000 gpad, and 67 persons per acre at 4,000 gpad. As applied to the metropolitan area as a whole, this method of calculation results in a BOD and suspended solids allowance for industrial waste in an amount equal to 40 per cent of that for sanitary sewage.

With reference to the grit transported in sewage from a combined system, design should be based on removal of the large amounts carried by storm flows. As pointed out earlier, existing facilities at the Diagonal Avenue sewage treatment plant remove an average of about 5 cubic feet of grit per million gallons under dry weather conditions. This value is in line with average dry weather removals recorded at treatment plants in San Francisco which serve combined systems.

In wet weather, as much as 50 cubic feet per million gallons is removed at the San Francisco plant. At Seattle, where it is necessary to use from 600 to 1,000 yards of sand per day for sanding icy streets, much larger amounts of grit can be expected. Under storm flow conditions, therefore, an allowance of 80 cubic feet of grit per million gallons is appropriate.

Sewage from systems which are entirely separate normally carries only 0.2 to 0.4 cubic feet of grit per million gallons. Deposits which accumulate during dry weather, however, are flushed out by peak wet weather flows and may, at such times, result in a grit load as high as 5 cubic feet per million gallons.

Chapter 8
ENVIRONMENTAL AND ECONOMIC EFFECTS OF SEWERAGE AND DRAINAGE DEFICIENCIES

Environmental effects stemming from sewerage and drainage deficiencies in the metropolitan Seattle area range from minor nuisances to conditions involving a significant hazard to community health and well being. Economic effects, in addition to those involving damage to property, include losses due to impairment of fisheries resources and of areas devoted largely to recreational activity. For convenience in presentation, sewerage deficiencies will be considered under each of two categories, namely, those associated with community disposal systems and those associated with individual disposal systems. Additionally, consideration will be given briefly to the effects of these systems, both community and individual, on the waters of Lake Washington and Puget Sound. And finally, effects attributable to inadequate drainage will be outlined and some of the problems relating to conditions in suburban areas will be discussed.

COMMUNITY SEWERAGE SYSTEMS

In community sewerage systems, the deficiencies which lead to nuisance conditions and other objectionable effects consist generally of insufficient sewer capacity, leaking sewers, illicit drain connections, and either inadequate treatment or no treatment. Many of the facilities presently in use not only are incapable of meeting future needs but are already overloaded or otherwise inadequate.

Overloaded collection sewers, whether due to an inherent lack of capacity or to excessive infiltration and storm inflow are manifested by overflowing manholes, localized flooding, and backing up of sewage into basements. These conditions result, in turn, in local nuisance and inconvenience, in property damage, and in situations hazardous to public health.

Overloading of Seattle Sewers

Many of the combined sewers now serving the city of Seattle were designed and constructed around the turn of the century. In view, therefore, of their age and of the heavy burden which has been imposed on them as a result of the unprecedented population growths in recent years, it is not surprising that various shortcomings are becoming increasingly apparent. What is surprising, perhaps, is the fact that most of these old sewers are still in reasonably good condition and are deficient only to the extent that they lack the capacity to carry the combined flows of sewage and storm drainage.

On the basis of design criteria employed until recently, overloading of the combined sewers can be

DURING PERIODS OF RAINFALL, combined sewers in Seattle frequently become overloaded. In this view, combined sewage and storm water is surging from two manholes.

Fig. 8-1. Sources of Sewage Backup Complaints in Seattle, 1952 - 57

MERCER ISLAND
LAKE WASHINGTON
SUNSET HIGHWAY
LAKE WASHINGTON PONTOON BRIDGE
ELLIOTT BAY
HARBOR ISLAND
DUWAMISH WATERWAY
DUWAMISH HEAD
SMITH COVE
ALKI POINT
WILLIAMS POINT
FAUNTLEROY COVE
BRACE POINT
PUGET SOUND
BOEING FIELD
PROPOSED WATERWAY
RAINIER AVE
EMPIRE WAY
BEACON AVE
COLUMBIAN WAY
PROPOSED FREEWAY
AIRPORT WAY
CORSON ST
E MARGINAL WAY
W MARGINAL WAY
HIGHLAND PKWY
DELRIDGE WAY
W AVALON WAY
FAUNTLEROY AVE
CALIFORNIA AVE
48 TH AVE SW
BEACH DR
MARINE VIEW DR
16 TH AVE SW
35 TH AVE SW
14 TH AVE S
1 ST AVE S
4 TH AVE S
23 RD AVE
FAIRVIEW
ANNE AVE
OLYMPIC PL
ELLIOTT AVE
ALASKAN WAY
VIADUCT
C.M.ST.P.&P.R.R.
G.N.RY.
N.PRY.
S.P.&S.R.R.
S.P.&S.R.R.
PACIFIC HWY
DES MOINES WAY
GLENDALE AVE
MEYERS WAY
AQUA WAY
ROXBURY ST
106 TH ST
BARTON ST
HOLDEN ST
THISTLE ST
ADMIRAL WAY
HARBOR AVE
ALKI AVE
SPOKANE ST
DEARBORN ST
YESLER WAY
BOREN AVE
DENNY WAY
MERCER ST
MADISON ST
WASHINGTON BLVD
LAKE WAY
ORCAS ST
GRAHAM ST
RENTON AVE
RAINIER AVE

expected to occur on an average of once every two years (Chapter 6). Furthermore, most of these sewers are serving systems which have been extended far beyond the limit anticipated at the time of design. As a consequence, many of them become overloaded during periods of even light rainfall and are thus responsible for relatively frequent trouble with backed-up sewage and overflowing manholes.

In 1951, the city engineering department made an extensive study of sewer capacities. Initial results of that study, as reported by the department in 1952,[1] indicated that, even on the original basis of design, over 50 per cent of the sewers serving the city were overloaded. These studies have since been continued and have served to substantiate the initial findings.

Since 1951, the city engineering department has been keeping a record of complaints about sewage backup. This record is plotted in Fig. 8-1, which shows the number and sources of complaints for a 6-year period ending in the spring of 1957. In all, 692 complaints were received during the six years, representing an average of about 115 per year. In reviewing this record, it should be noted that the recorded complaints cover only those which were accompanied by a claim for alleged damage. Numerous complaints are received following almost every storm but are not officially recorded unless they involve a claim for damage.

Table 8-1 lists the number of claims associated with the six heaviest storms during the six-year period. As there indicated, these storms resulted in a total of 314 claims, ranging from 36 for the storm of June 4, 1956 to 68 for the storm of December 19, 1953. Since heavy storms account for less than one-half of the total number of complaints, it appears that the sewers in some areas become overloaded during storms of lesser severity.

According to sewer maintenance personnel, flooding of streets and washouts in the vicinity of overflowing manholes occur at frequent intervals. One such incident took place during the storm of February 24, 1957 at a survey metering station situated at 11th Avenue and West 45th Street. In this case, an overflow from the manhole caused a street washout 25 feet in diameter and 30 inches deep.

Although the total expense incurred by the city in the payment of claims and in the repair of damages due to sewage backups reportedly falls far short of the capital outlay which would be required for corrective purposes, no one is inclined to contend that such incidents should be condoned or allowed to continue.

[1]Planning and Progress, Seattle City-Wide Sewage Disposal Problem, *Seattle Engineering Department, 1952.*

Table 8-1. Complaints of Sewage Backups Received by the City of Seattle following the Heaviest Storms of Recent Years

Date of Storm	Complaints Received[a]
June 29, 1952	56
September 30, 1953	54
December 19, 1953	68
June 4, 1956	36
February 24 and 25, 1957	54
April 18 and 19, 1957	46
Total for six storms	314

Source: Seattle Engineering Department.

[a]*Only complaints involving alleged damage are recorded.*

Basements flooded with backed-up sewage, if not an actual hazard to health, are certainly a source of severe discomfort and annoyance. As such, they represent a dereliction in sewerage service which should be minimized or eliminated.

From an economic standpoint, heavy surcharging can lead to severe damage of sewers and their appurtenant structures and thus to a possible outlay for necessary repair and replacements. This condition is especially true in the older brick sewers.

In considering the present overload problem, it is important to bear in mind the fact that substantial portions of the areas tributary to already inadequate sewers are not as yet fully developed. Obviously, therefore, flooding and overflow conditions are bound to become increasingly serious unless measures are taken in the meanwhile to provide necessary relief.

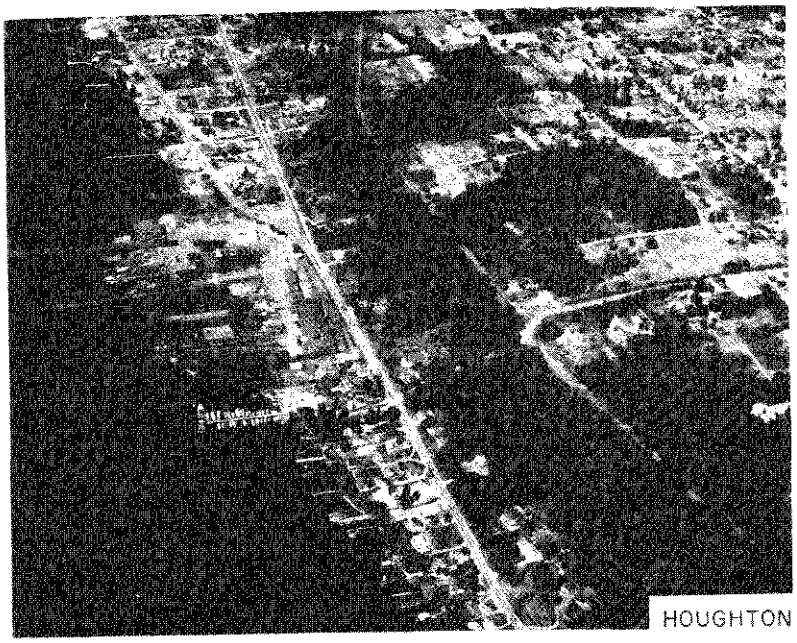
Deficiencies in Other Systems

Serious overload conditions prevail also in many public sewerage systems outside Seattle (Chapter 7). Although many of these systems have been constructed since World War II, infiltration and storm inflow quantities already overtax both sewer and treatment plant capacities. In some cases, there is little or no capacity left either for increased sanitary sewage flows which will result from future increases in population, or for the additional infiltration and storm inflow which will result from collection system extensions.

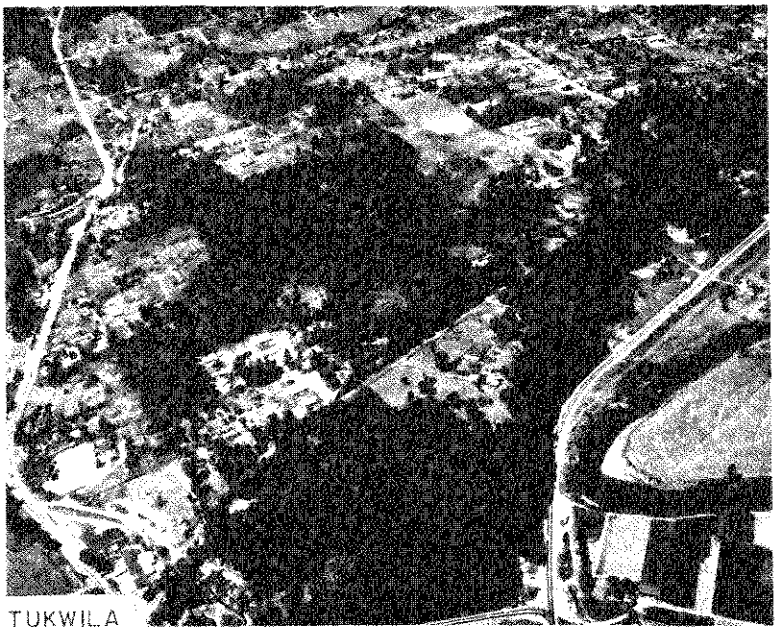
INDIVIDUAL SEWAGE DISPOSAL SYSTEMS

Approximately 30 per cent of the metropolitan area population still lacks public sewerage service. Of this group, about one-sixth lives on farms and other sparsely settled areas, and the balance lives in sub-

DEVELOPMENT IN UNSEWERED AREAS. Within the metropolitan area, a total of approximately 100 square miles is now in urgent need of public sewerage. In addition to suburban developments north and south of Seattle and east of Lake Washington, the unsewered area includes eleven incorporated cities. These aerial views illustrate the extent of development dependent on individual sewage disposal systems.

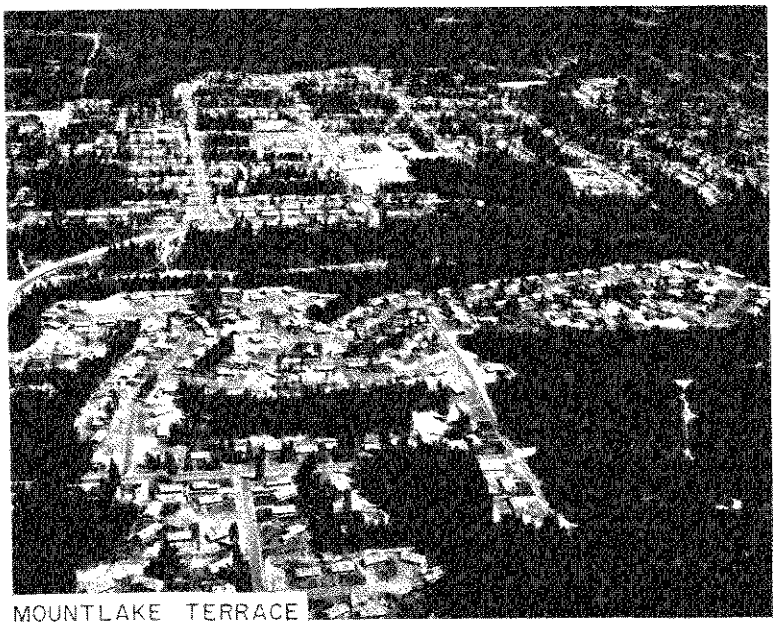


HOUGHTON



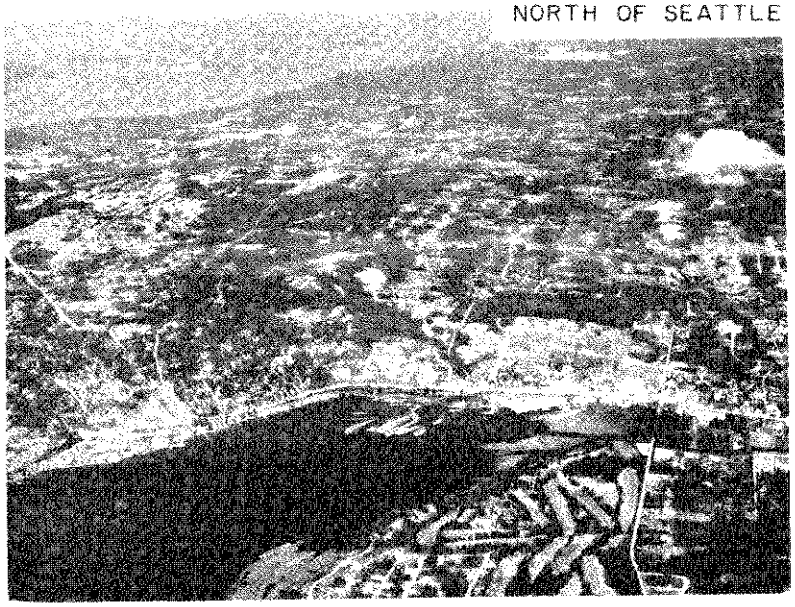
TUKWILA

BOTHELL

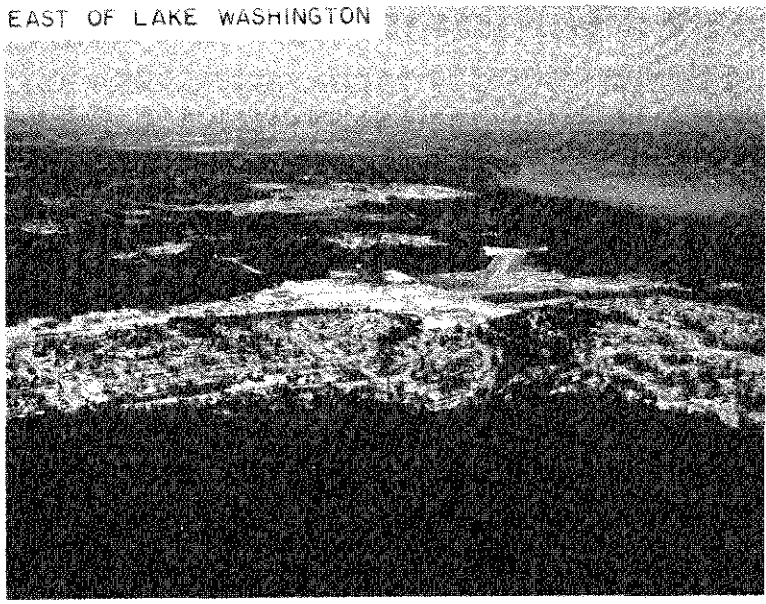


MOUNTLAKE TERRACE

NORTH OF SEATTLE



EAST OF LAKE WASHINGTON



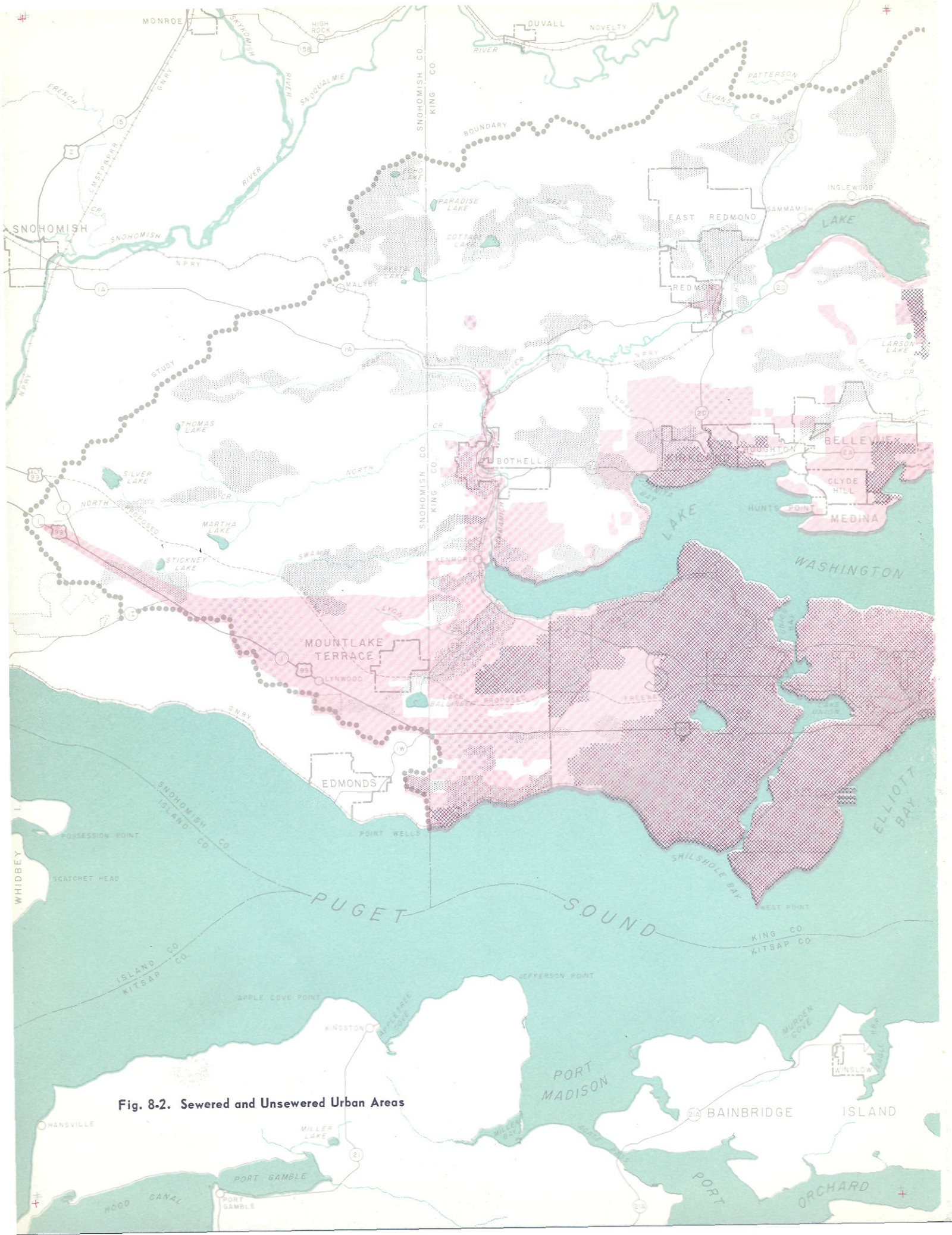
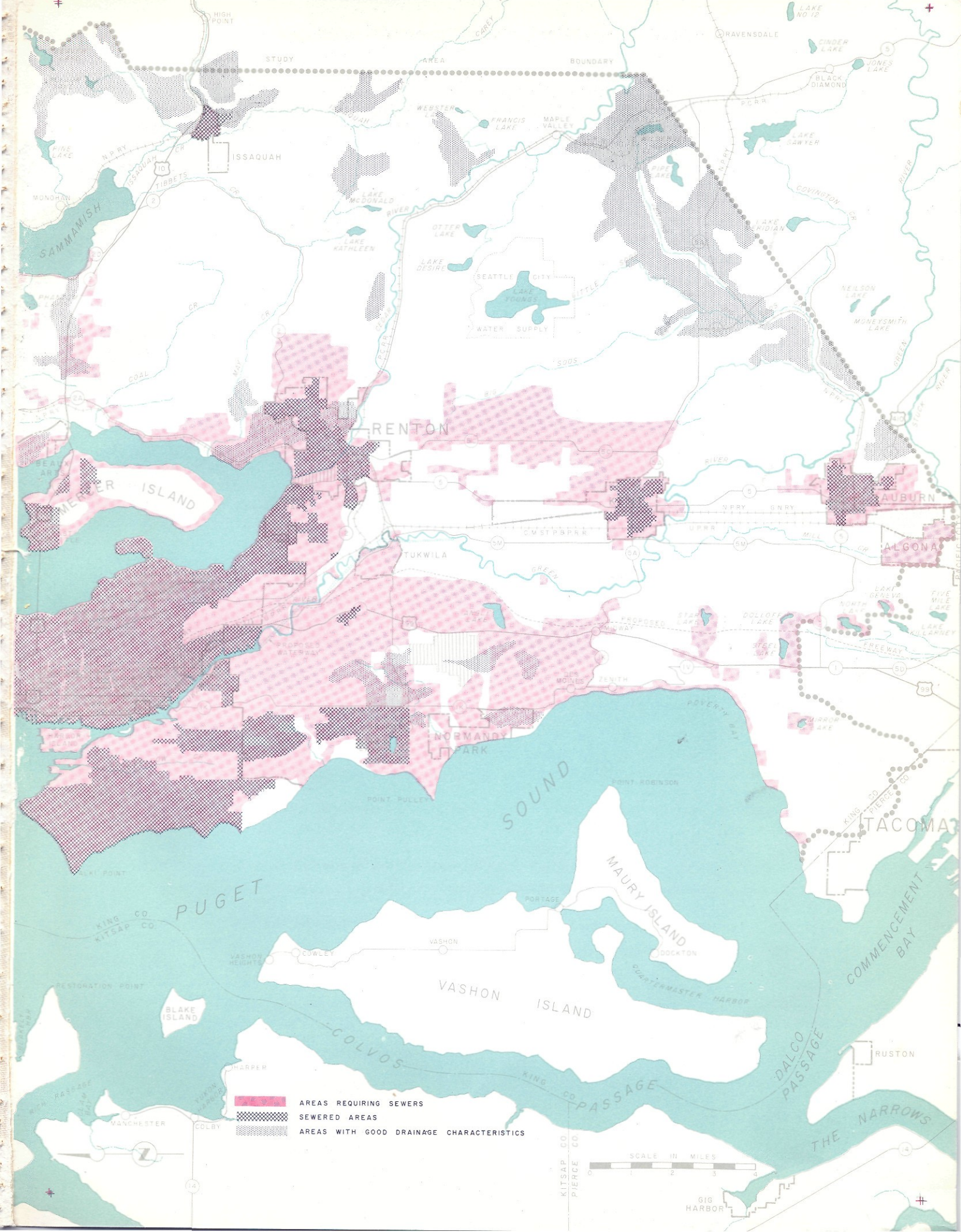


Fig. 8-2. Sewered and Unsewered Urban Areas


STUDY AREA BOUNDARY
HIGH POINT
RAVENSDALE
LAKE NO 12
CINDER LAKE
JONES LAKE
BLACK DIAMOND
WEBSTER
FRANCIS LAKE
MAPLE VALLEY
LAKE SAWYER
PIPE LAKE
ISSAQUAH
HINE LAKE
MONOHAN
SAMMAMISH
TIBBETS CR
LAKE McDONALD
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
LAKE HIDIAN
COVINGTON CR
SEATTLE CITY
LAKE YOUNGS
WATER SUPPLY
NELSON LAKE
MONEYSMITH LAKE
SOOS
COAL CR
MAY CR
RENTON
MERCER ISLAND
AUBURN
TUKWILA
ALGONA
LAKE GREEVA
FIVE MILE LAKE
LAKE KILLARNEY
DOLLOFF LAKE
PROPOSED FREEWAY
ZENITH
NORMANDY PARK
POVERTY BAY
POINT ROBINSON
POINT PULLEY
SOUND
PUGET
TACOMA
KING CO
PIERCE CO
MAURY ISLAND
PORTAGE
DOCKTON
QUARTERMASTER HARBOR
VASHON
COWLEY
VASHON HEIGHTS
VASHON ISLAND
COMMENCEMENT BAY
RESTORATION POINT
BLAKE ISLAND
COLVOS
KITSAP CO
PASSAGE
DALCO PASSAGE
RUSTON
HARPER
MANCHESTER
COLBY
THE NARROWS
GIG HARBOR
AREAS REQUIRING SEWERS
SEWERED AREAS
AREAS WITH GOOD DRAINAGE CHARACTERISTICS
SCALE IN MILES
0 1 2 3 4

urban areas and cities. In the latter case, insanitary conditions and economic problems brought on by the lack of public sewerage have been a matter of increasing concern not only to the residents directly affected but to local officials and health and pollution control authorities.

Although previous reports on sewerage problems have mentioned those of the unsewered areas, no attempt has been made either to determine the geographical extent of such areas or to assess their importance in terms of economic and other factors. As a part of the present survey, therefore, the location and extent of these areas has been determined and information has been obtained concerning their current problems.

Basis for Need of Public Sewerage

Experience in the metropolitan area indicates that it usually becomes advantageous to construct public sewers when the population density reaches about 4 persons per acre. At this stage of development, individual systems are subject to relatively frequent failure and thus are likely to be more costly than public sewerage. Failures occur when the surrounding surface soils become saturated with sewage effluent and drainage is either impeded or stopped altogether

Extent of Area in Need of Public Sewerage

In determining the extent to which public sewerage is presently required, it is necessary first to ascertain which portions of the metropolitan area fall within the density requirement. To that end, preliminary information was obtained from census tract data, from existing land use maps, and from aerial survey maps and photographs. This procedure, augmented by direct areal inspection, resulted in the delineation of about 200 square miles having a population density of 4 or more per acre (Fig. 8-2). Subtracting from this figure the 87 square-mile area which is already sewered (Table 6-1) leaves a gross unsewered area of 113 square miles. After deducting allowances for minor uninhabitable areas and for small vacant tracts which lie within developed areas and were unavoidable included in the gross total, the net area in need of sewers amounts to 100 square miles.

For the study area as a whole, the population of the portion without public sewers amounts at present to a total of 260,000. This represents the study area population (Table 5-9) less (1) the population of the sewered area (Table 6-1) and (2) an estimated 40,000 persons residing in rural portions. In terms of density, a total of 260,000 in an area of 100 square miles amounts to a little over 4 persons per acre.

Sewage Disposal in Unsewered Areas

Approximately 85,000 individual household sewage disposal systems are now in use in the unsewered area depicted in Fig. 8-2. Almost all of these installations depend on soil leaching systems for the disposal of septic tank effluent. Leaching is accomplished usually by means of tile drains laid several feet beneath the ground surface. Some private systems, however, employ dry wells and others discharge directly to roadside ditches and to streams and lakes.

Effect of Unfavorable Soil Conditions

Optimum soil drainage conditions prevail in only a small percentage of the metropolitan area (Fig. 8-2). In the remainder of the area, drainage is either poor or variable. It is poor, of course, where surface soils are impervious, and is variable either where surface formations are underlain by an impervious stratum or where the area is subject to a high ground water table or flooding (Fig. 3-5).

Under optimum conditions of soil drainage, and providing the septic tank is adequately maintained, a leaching system may function satisfactorily for many years. On the other hand, in soils with poor drainage characteristics or in areas where ground water is too near the surface, leaching failures may develop after only a short period of use. In such an event, the tank effluent pools on the ground surface or seeps into roadside ditches. In hilly areas where the surface soils are underlain by an impervious stratum, the effluent may be transported laterally for a considerable distance, ultimately seeping out of hillsides, accumulating in low areas, or finding its way into surface waters.

Leaching system failures are usually more or less isolated during the early stages of a subdivision development. As the number of septic tanks increases, however, the effluent effects tend to become cumulative and result eventually in saturation of the surface soils. When that happens, leaching line failures are frequent and conditions develop which not only are a nuisance but a serious menace to community health.

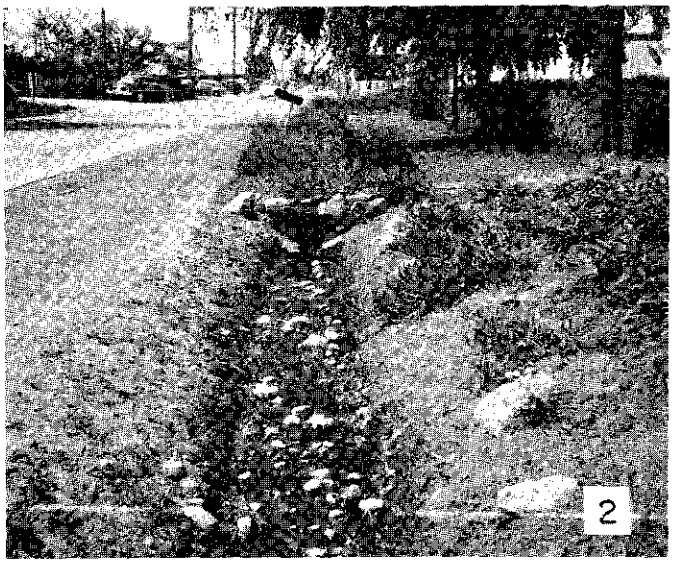
Effects of Crowded Installations

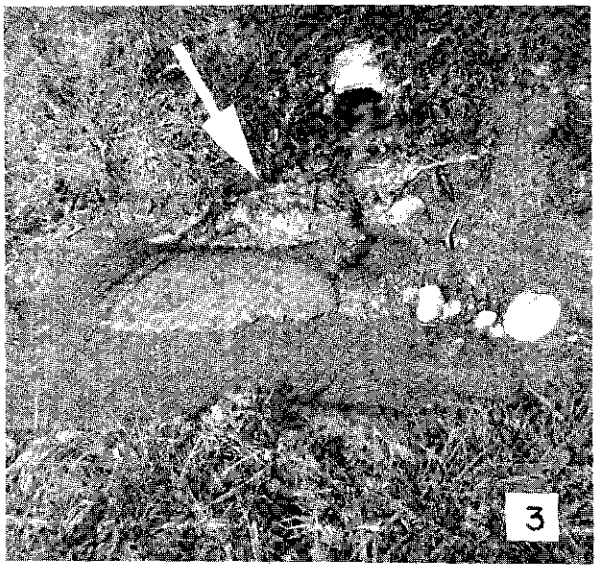
The consequences of too many private disposal systems in localities where drainage is inadequate are revealed in records of the Seattle-King County Health Department. These records show that the department is called upon during an average year to make approximately 6,000 inspections of private system failures. Many other failures are known to occur but are corrected without being referred to the department. Based on the reported cases alone, at least one out of every fourteen units fails each year. These failures are common in the heavily developed sections north and south of Seattle, in southwesterly Snohomish

(1) CLOUDY SEPTIC TANK EFFLUENT is visible in the discharge from this storm drain.

(2) THIS IS ONE OF MANY ROADSIDE DITCHES carrying a continuous flow of septic tank effluent.

(3) DIRECT DISCHARGE TO OPEN DITCH. Note sludge deposit.





County, and along the eastern shoreline of Lake Washington.

During the course of the survey, septic tank overflows were observed at numerous locations. In some parts of the study area, conditions have become so intolerable that the issuance of building permits has been greatly curtailed. In others, the permits require leaching lines of such an extent as to make the installation cost prohibitive. Faced with these prospects, subdividers and builders frequently are unable to proceed with proposed developments.

Cost of Sewage Disposal in Unsewered Areas

Health Department records for the past six years indicate that private disposal systems have been constructed at the rate of about 6,000 per year. At an estimated average cost of $375 per installation, this means that the annual investment in such systems is about $2,250,000.

Reasonable maintenance practice requires that the accumulated solids be removed from septic tanks at least once every five years. On that basis, approximately 17,000 of the 85,000 units now in use require cleaning every year. At the prevailing rate of $30

IMPERVIOUS SOIL CONDITIONS (photo 1), indicated here by local flooding, retard downward drainage of septic tank effluent.

SEEPAGE OF SEPTIC TANK EFFLUENT (photo 2, arrow) toward roadside ditch results from leaching line failure brought about by impervious soil condition.

PERIODIC RENOVATION of leaching lines (photo 3) is required in areas where downward drainage is inadequate.

per cleaning, this represents an annual expenditure of about $510,000.

Costs of repairing leaching line failures vary over a wide range. While minor repairs may suffice in some cases, replacement of part or all of the drain lines is often required. At the present rate of about $2.00 per lineal foot, the cost of total replacement may run well over $300 per system.

Based on discussions with Health Department inspectors, the average expenditure for inspection and repairs is estimated to be not less than $75 per failure. For the 6,000 reported each year, the total annual cost thus amounts to at least $450,000. Since many failures are not reported, the actual total is unquestionably much higher.

It is evident from the foregoing figures that the total annual expenditure for new construction and for the maintenance and repair of private disposal systems aggregates over $3,200,000. In comparison, an annual outlay of that magnitude would finance nearly $50 million worth of capital improvements and would provide local sewers for essentially all of the unsewered areas shown in Fig. 8-2. This comparison is based on an average cost for sewerage facilities of $800 per acre, a 5 per cent interest rate on invested capital, and a 30-year bond life. While additional expenditures for trunk sewers and treatment and disposal works would also be entailed, local sewers so provided would be capable of serving about one-half million persons, or about twice the number served by all the private systems now in use.

REMOVAL OF SEWAGE FROM INLAND AREAS

Coincident with the construction of local sewers, facilities must be provided for conveyance of the sewage to points of treatment and disposal. In some communities, particularly those situated adjacent to bodies of water capable of receiving sewage, the distance involved in conveyance to treatment and disposal points is relatively short and causes no problem from the standpoint of cost. In others, where suitable disposal sites are not close at hand, long and costly sewers are required and may impose a financial hardship of almost insurmountable magnitude.

Many inland communities within the study area have found themselves in the latter predicament. Despite their desire to construct urgently needed local sewers, and their financial capability to do so, they have been utterly unable to finance the sewers which are required to convey the sewage to treatment and disposal sites outside their immediate areas. This situation prevails in most of the densely populated inland sec-

tions of the metropolitan area and is particularly acute in the Lake Washington drainage basin.

Recent studies have indicated that nutrient substances contained in sewage effluents being discharged into Lake Washington are causing an excessive algal growth which threatens both its natural beauty and its value for recreational purposes. To prevent further degradation, both the Pollution Control Commission and the State Department of Public Health have enacted regulations designed to bring about the elimination of sewage discharges.

Compliance with these regulations will, of course, necessitate the construction of sewers wherein all sewage will be picked up and carried out of the drainage basin for treatment and disposal elsewhere. Except for a relatively small portion of the basin which is now served by the city of Seattle, no such facilities are presently available and, because of the distances involved, no way has yet been found to finance their construction.

Certain problems are developing in the absence of necessary conveyance sewers and can be expected to continue until such a time as corrective action is undertaken. These problems include:

1. A delay in the construction of vitally needed local sewerage facilities.
2. The continued installation of private sewage disposal systems, many of which are doomed to failure.
3. An exhaustion of financial resources through the construction of temporary treatment works.
4. A curtailment of desirable land developments.
5. A further degradation of Lake Washington and other inland lakes.

CONDITIONS IN ENVIRONMENTAL WATERS

It would be superfluous at this point to stress the importance of environmental waters as a major asset of the metropolitan Seattle area. Until recently, however, their value has been taken for granted and little has been done to minimize or prevent their fouling by discharges of sanitary sewage and industrial waste. With recognition fully established, the problem now is one of determining the extent to which such waters are being degraded and the steps that must be taken in the future to assure effective maintenance of acceptable conditions.

The first extensive study to ascertain the effect of sewage and waste disposal operations on the waters of the area was made by the State Pollution Control Commission in 1942 and 1943. Within the past ten years, several additional studies have been made and between them have covered all of the waters presently subject to sewage and waste discharges. In scope, these studies were concerned with such factors as bacterial contamination, nuisance conditions, chemical and biological effects, and nutrient enrichment of Lake Washington.

Bacterial Contamination

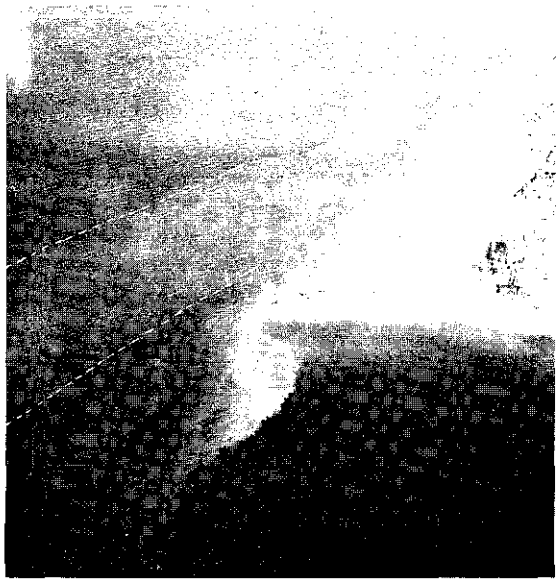
In accordance with standard public health practice, bacterial contamination of water affected by sewage disposal practices is expressed in terms of the MPN (most probable number) of organisms of the coliform group. Members of this group, though not pathogenic or disease producing, are present in immense numbers in fecal discharges of all warm blooded animals, including man. Since the group as a whole is readily cultured and identified, these organisms are a useful indicator of the possible presence of pathogenic bacteria of sewage origin. Certain members of the coliform group, however, propagate in surface soils and, as a result, are normally present to some extent in all natural surface waters. Additionally, drainage from fields fertilized with animal manures frequently is a source of high counts in surface waters.

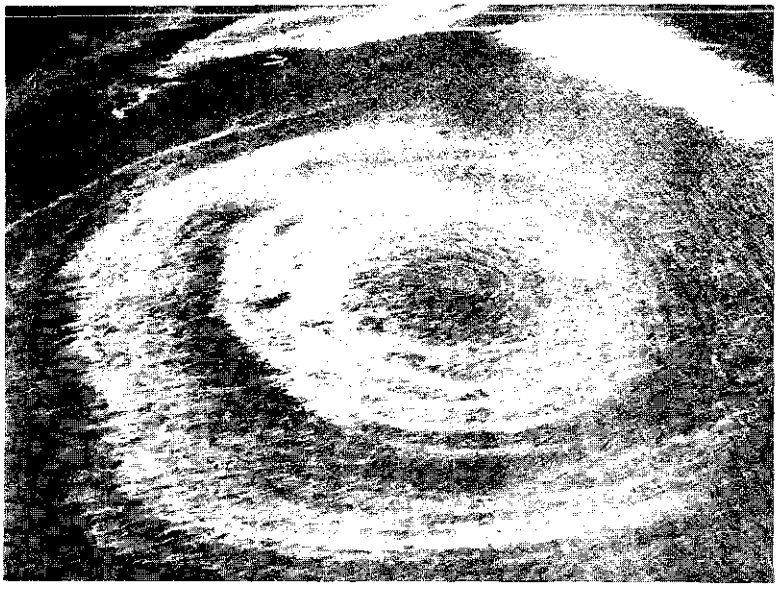
In the case of waters used for recreational purposes, very little evidence is available which relates the degree of coliform contamination to the incidence of water-borne disease. As a result, bathing water standards of governmental health agencies are far from consistent, ranging in upper limits from 240 to 2,400 MPN per 100 ml. Further, these are variously expressed as the arithmetic mean, median, or geometric mean of a series of samples or, in some cases, the maximum for a single sample.[2] It is evident, therefore, that a specific coliform limit does not necessarily define a line between safe and hazardous water but rather is intended as a desirable and, presumably, a reasonably attainable goal.

Over the past fifteen years, several standards of bacteriological quality for bathing purposes have been administratively applied by public health and pollution control agencies in the state of Washington. More recently, however, the Washington Pollution Control Commission has adopted water quality objectives promulgated by the Pollution Control Council, Pacific Northwest Area. For bathing areas, these objectives prescribe that the average MPN in a representative group of samples should not exceed 240 per 100 ml and that this number should not be exceeded in more than 20 per cent of the individual samples examined. This standard is referred to hereinafter as the Washington standard.

Because the Washington standard is believed to be somewhat stringent, the studies reported below made use also of a more lenient standard under which it is stipulated that not more than 20 per cent of a representative number of samples should have a coliform

[2] *Garber, W. F.,* Bacterial Standards for Bathing Waters, *Sewage and Industrial Wastes, 28-6-795 (June 1956).*





SEWAGE "BUBBLE" at terminus of North Trunk sewer outfall (right photo). About 40 mgd of untreated sewage is discharged at this point. Path of sewage field from North Trunk outfall during flood tide (left photo) is distinguishable from normal turbidity along shoreline of West Point. On ebb tides, sewage field was traced to Golden Gardens, two miles north of outfall.

count in excess of 1,000 per 100 ml. Actually, however, it makes little difference which standard is applied, as the coliform count in most of the samples was found to exceed the higher limit.

Summary of Bacteriological Studies, 1942-1951. A sanitary survey of Lake Washington conducted by the State Pollution Commission in 1942 and 1943 included the bacteriological examination of 536 samples taken at 36 sampling stations along the shoreline. A summary of the findings presented in the report on this survey[3] shows that conditions at 27 of the 36 sampling stations failed to meet the Washington standard. On that basis, the commission concluded that substantial sections of the shoreline were unsatisfactory for bathing purposes at the time of the survey.

As a part of his study in 1947 and 1948 of the Seattle sewerage problem,[4] Dr. Abel Wolman arranged for the collection and bacteriological examination of samples of the shore waters of Puget Sound. Under this program, which covered seven major public bathing areas on the sound within Seattle and involved six different sampling periods, the total number of samples amounted to 866, or approximately 125 from each beach. A summary of the results then obtained, as given in the report by Dr. Wolman, showed that not a single beach met the Washington standard during any of the six sampling periods.

In 1949, the State Pollution Control Commission undertook an investigation of pollution problems in Puget Sound. This work was under the direction of Professor R. O. Sylvester of the Department of Civil Engineering, University of Washington, and included an extensive study of bacterial conditions in the shore waters of Puget Sound and in the lower reaches of Green River. A total of 281 samples was taken at 24 sampling stations, 3 of which were in Green River and 21 were situated along the sound between Richmond Beach and the mouth of Salmon Creek. Here again it was found that the Washington standard was exceeded at every station and that few of the stations met the less stringent standards based on an MPN value of 1,000. The report on this work [5] recommended that essentially all of Seattle's Puget Sound beaches be closed to bathing.

In November 1950, the Seattle Engineering Department reinstituted the sampling program carried on during the Wolman survey. For a period of about 13 months, samples were collected on the average of at least once a week at 44 sampling stations situated at bathing areas in the sound and in Lake Washington. In all, 3,150 samples were analyzed, including 1,475 from Lake Washington and 1,675 from Puget Sound. A summary of these analyses, as set forth in a report issued by the city engineer in 1952, indicates that both of the two standards of bacteriological quality were exceeded at all of the Puget Sound stations. At Lake Washington, the Washington standard was exceeded at all stations and 60 per cent of the stations failed to meet the standard based on an MPN of 1,000.

[3] Sources and Extent of Lake Washington Pollution, *Bulletin No. 29, June 1943, State Pollution Commission.*

[4] City of Seattle, Report of Sewage Disposal, 1948, *by Abel Wolman.*

[5] Puget Sound Pollution, Seattle Metropolitan Area, 1949, *R. O. Sylvester and Associates.*

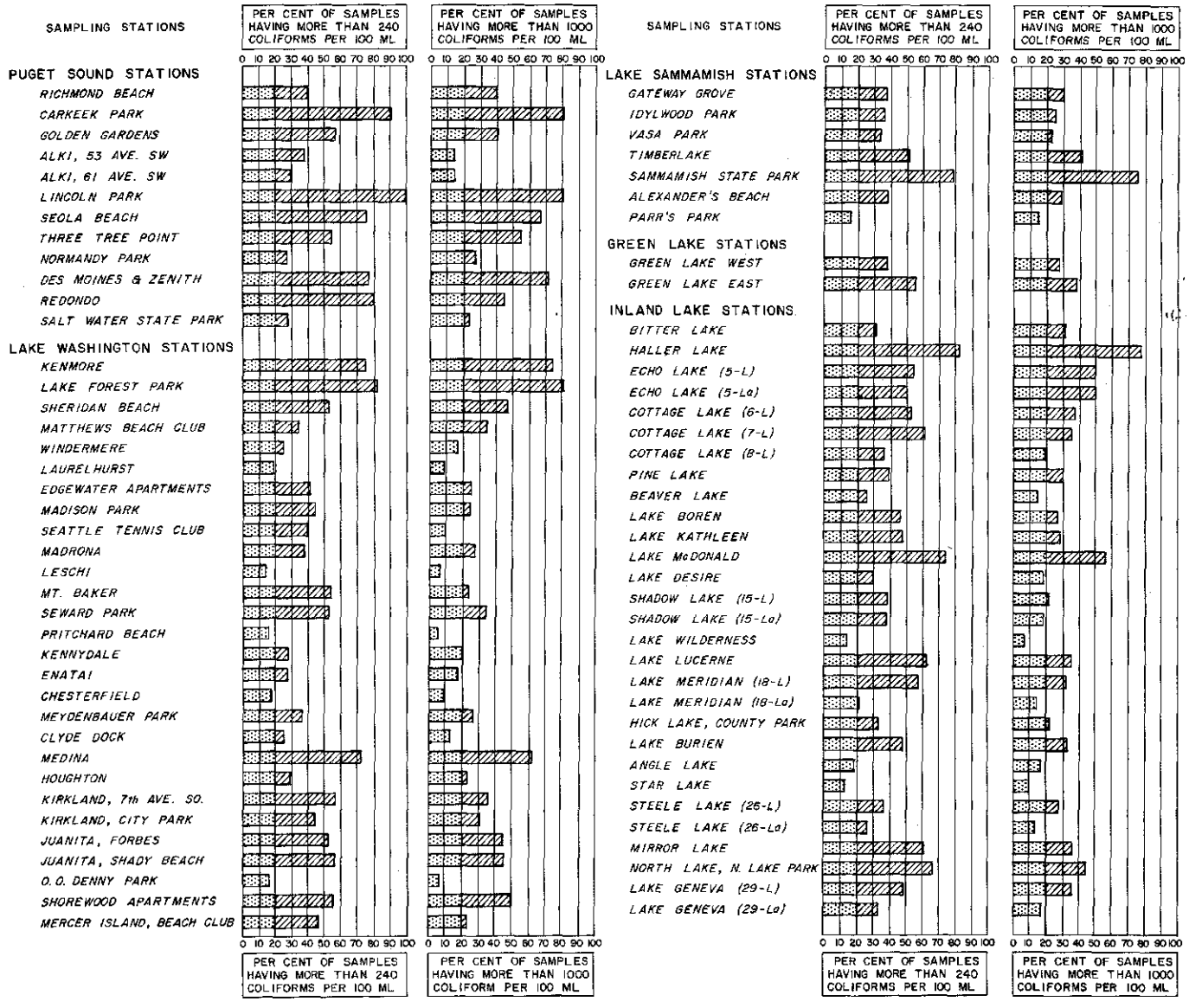


Fig. 8-3. Bacteriological Quality of Bathing Waters in King County

Based on results of sampling program conducted by Seattle-King County Health Department, 1952 to 1956 inclusive (see Table 8-2). Shaded area shows per cent in excess of the indicated standards.

Seattle-King County Health Department Bacteriological Study, 1952-56. The most recent and by far the most comprehensive study of the bacterial quality of the surface waters of the metropolitan area was conducted by the Seattle-King County Health Department during the years 1952 through 1956. This study covered essentially all of the public and semipublic bathing beaches of the area, including beaches at the smaller lakes. In all, conditions were checked at 78 beaches and 3,250 samples were collected and analyzed. From 5 to as many as 138 samples were taken at each station, the total depending generally on the extent of bathing activity.

Results of the health department survey (Table 8-2 and Figure 8-3) show that 69 of the 78 beaches failed to meet the Washington standard and that 55 failed also to meet the standard based on an MPN value of 1,000. None of the 12 Puget Sound stations and only 25 of the Lake Washington stations met the Washington standard, while only 2 stations on the sound and 10 on Lake Washington met the less stringent standard. These findings substantiate those of the previous studies.

Lake Washington beaches adjacent to unsewered residential areas, such as Kenmore, Lake Forest Park and Sheridan Beach, are among the most heavily contaminated. Two factors probably account for this condition. First, because of the poor drainage char-

Table 8-2. Bacteriological Quality of Bathing Waters in King County

Sampling station designation	Location of sampling station[a]	Number of samples analyzed	Years sampling conducted	Coliform organisms, MPN[b] per 100 ml		Per cent of samples	
				Lowest value found	Highest value found	MPN[b] greater than 240	MPN[b] greater than 1,000
1	Richmond Beach	5	53,55	23	24,000	40	40
2	Carkeek Park	22	54,55,56	62	240,000	91	82
3	Golden Gardens	27	55,56	23	7,000	57	41
4	Alki, 53 Avenue SW	21	55,56	23	2,400	38	14
5	Alki, 61 Avenue SW	21	55,56	62	7,000	29	14
6	Lincoln Park	31	55,56	700	24,000	100	81
7	Seola Beach	24	52,53,55,56	240	24,000	75	67
8	Three Tree Point	45	52,53,55,56	0	240,000	55	55
9	Normandy Park	30	52,53,55	0	24,000	27	27
10	Des Moines and Zenith	53	52,53,55,56	15	240,000	77	72
11	Redondo	40	52,53,55,56	23	24,000	80	45
12	Salt Water State Park	39	52,53,55,56	0	240,000	28	23
W-1	Kenmore	24	53,54,55	0	240,000	75	75
W-2	Lake Forest Park	122	53,54,55,56	38	240,000	83	82
W-3	Sheridan Beach	112	53,54,55,56	15	240,000	53	47
W-4	Matthews Beach Club	18	53,55,56	38	24,000	34	34
W-5	Windermere	36	55,56	0	24,000	25	17
W-6	Laurelhurst	32	55,56	0	7,000	19	9
W-6a	Edgewater Apartments	12	56	46	24,000	42	25
W-7	Madison Park	34	55,56	23	7,000	44	24
W-8	Seattle Tennis Club	10	55,56	23	24,000	40	10
W-9	Madrona	37	55,56	46	24,000	38	27
W-10	Leschi	14	55,56	0	24,000	14	7
W-11	Mt. Baker	26	55,56	13	7,000	54	23
W-12	Seward Park	38	55,56	0	24,000	53	34
W-14	Pritchard Beach	37	55,56	6	24,000	16	5
W-15	Kennydale	85	52,53,54,55,56	13	240,000	28	20
W-16	Enatai	104	53,54,55,56	13	240,000	28	18
W-17	Chesterfield	22	53,55,56	15	24,000	18	9
W-18	Meydenbauer Park	48	54,55,56	22	24,000	37	27
W-19	Clyde Dock	16	54,55,56	23	2,400	25	13
W-20	Medina	57	52,53,54,55	0	24,000	72	63
W-21	Houghton	62	53,54,55,56	0	24,000	29	23
W-22	Kirkland, 7th Avenue So.	33	53,54,55,56	38	2,400	57	36
W-23	Kirkland, City Park	54	53,54,55,56	6	24,000	44	31
W-24	Juanita, Forbes	91	52,53,54,55,56	0	240,000	53	45
W-25	Juanita, Shady Beach	68	52,53,54,55,56	23	24,000	57	46
W-26	O. O. Denny Park	42	52,53,54,55,56	46	24,000	17	7
W-28	Shorewood Apartments	138	52,53,54,55,56	0	240,000	56	50
W-32	Mercer Island, Beach Club	30	53,54,55,56	6	7,000	47	23
S-1	Gateway Grove	37	52,53,54,55,56	46	24,000	38	30
S-2	Idylwood Park	41	52,53,54,55,56	0	24,000	37	25
S-4	Vasa Park	44	52,53,54,55,56	22	240,000	34	23
S-5	Timberlake	31	53,54,55,56	0	240,000	52	42
S-6	Sammamish State Park	113	52,53,54,55,56	15	240,000	79	75
S-7	Alexander's Beach	24	52,53,55,56	88	2,400	38	29
S-8	Parr's Park	20	52,53,55,56	0	2,400	15	15
1-L	Green Lake West	39	55,56	0	24,000	38	28
2-L	Green Lake East	42	55,56	6	24,000	55	38
3-L	Bitter Lake	45	52,53,54	12	2,400	31	31
4-L	Haller Lake	113	52,53,54,55,56	5	240,000	82	77
5-L	Echo Lake	52	52,53,55,56	21	240,000	54	50
5-La	Echo Lake	32	52,56	21	24,000	50	50

Continued on next page

Table 8-2. Continued

Sampling station designation	Location of sampling station[a]	Number of samples analyzed	Years sampling conducted	Coliform organisms, MPN[b] per 100 ml		Per cent of samples	
				Lowest value found	Highest value found	MPN[b] greater than 240	MPN[b] greater than 1,000
6-L	Cottage Lake	30	52,53,54,55,56	0	24,000	53	37
7-L	Cottage Lake	23	52,53,54,55,56	0	2,400	61	35
8-L	Cottage Lake	25	52,53,55,56	38	7,000	36	20
9-L	Pine Lake	23	52,53,54,55,56	21	70,000	39	30
10-L	Beaver Lake	27	52,53,54,55,56	0	24,000	26	15
11-L	Lake Boren	11	55,56	23	24,000	46	27
12-L	Lake Kathleen	32	55,56	13	7,000	47	28
13-L	Lake McDonald	23	55,56	23	24,000	74	57
14-L	Lake Desire	27	55,56	13	24,000	30	19
15-L	Shadow Lake	58	53,54,55,56	6	7,000	38	21
15-La	Shadow Lake	29	53,56	46	24,000	38	19
16-L	Lake Wilderness	58	53,54,55,56	46	2,400	14	7
17-L	Lake Lucerne	66	55,56	21	24,000	62	35
18-L	Lake Meridian	64	53,54,55,56	6	24,000	58	33
18-La	Lake Meridian	14	53,56	21	2,400	21	14
22-L	Hick Lake, County Park	76	52,53,54,55,56	0	24,000	33	22
23-L	Lake Burien	36	53,54,55,56	23	24,000	47	33
24-L	Angle Lake	48	52,53,54,55,56	0	240,000	19	17
25-L	Star Lake	39	52,55,56	22	7,000	13	10
26-L	Steele Lake	25	52,53,55,56	21	24,000	36	28
26-La	Steele Lake	23	52,53,54,55,56	15	2,400	26	13
27-L	Mirror Lake	31	52,55,56	23	7,000	61	35
28-L	North Lake, North Lake Park	18	53,55,56	62	24,000	67	44
29-L	Lake Geneva	29	53,55,56	6	24,000	48	35
29-La	Lake Geneva	18	53,54,55,56	0	24,000	33	17

Data from bacteriological surveys conducted by the Seattle - King County Department of Public Health during the years 1952 to 1956 inclusive.

[a]*See Fig. 8-4.*

[b]*MPN - Most probable number.*

acteristics of the soils in these localities, septic tank effluents are entering the lake. And second, heavily contaminated runoff is entering the lake in streams which meander through densely populated unsewered areas.

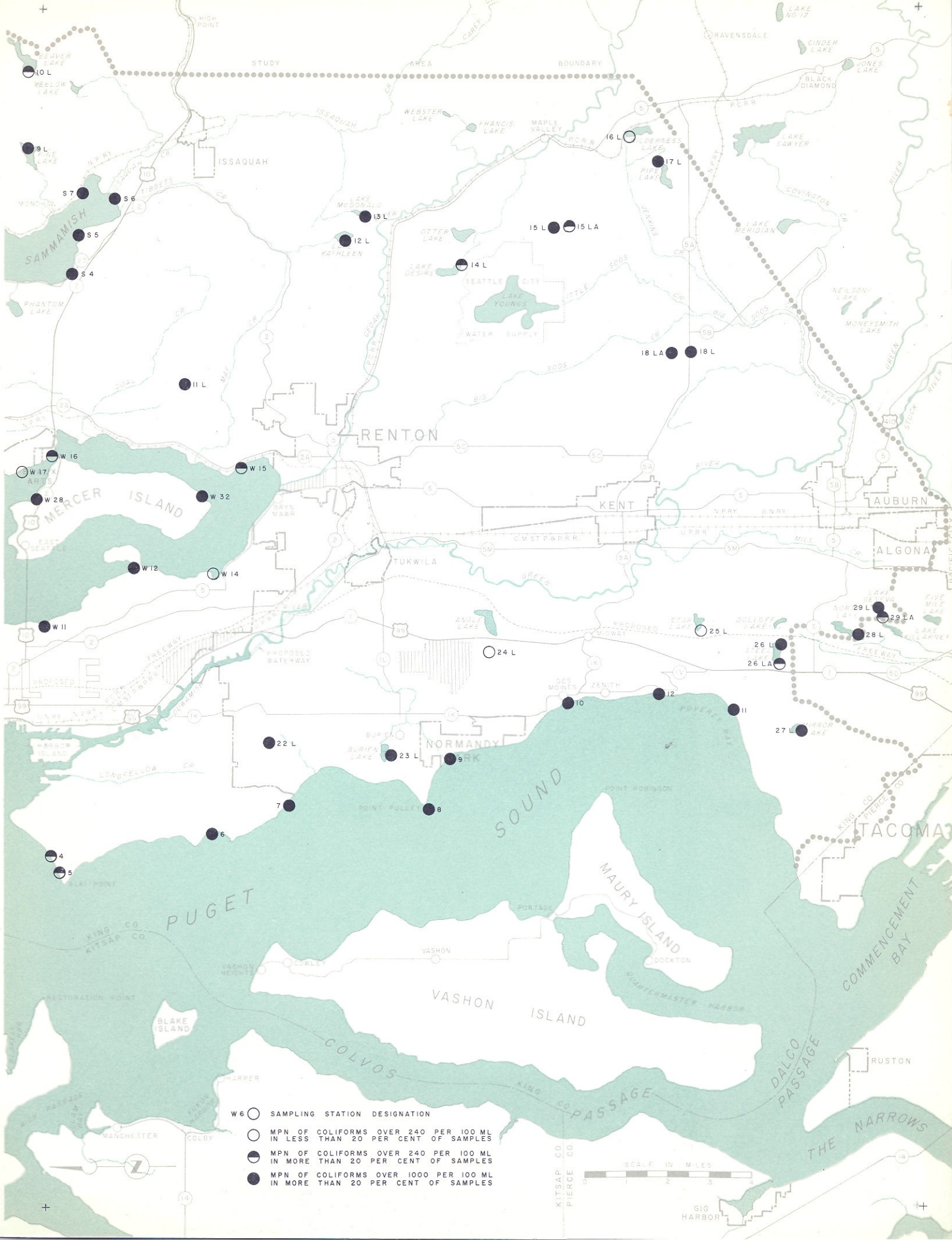
A similar situation apparently prevails at smaller lakes where residential developments along shorelines rely on private disposal systems. Only 3 of a total of 36 sampling stations at these lakes were found to comply with the Washington standard and only 11 complied with the MPN limit of 1,000 per 100 ml.

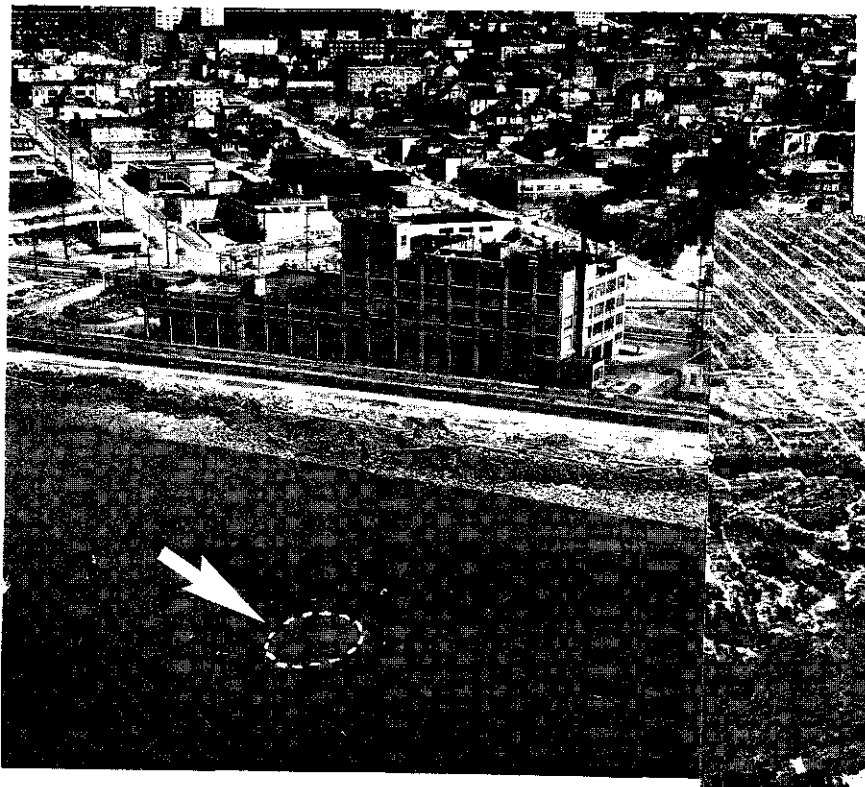
Except in several minor instances, no improvements have been made since the foregoing study was completed which would alter measurably the conditions existing at the time the samples were taken. Locations of the sampling stations, together with conditions in terms of compliance with the standards of quality, are shown in Fig. 8-4.

Surface drainage accounts, of course, for a certain portion of the bacterial densities found at all stations. At most stations, however, the maximum densities were far too high and violations of the standards were far too frequent to attribute the results to natural causes alone. In this regard, the 1949 report of the Pollution Control Commission states: "The natural coliform median for the salt water area, as determined from sampling data over a number of years, is 23 (or less) MPN per 100 ml. This is the value which can be said is due to normal land runoff and waterfowl. ...Any count in excess of this may be attributed to the presence of sewage. This is particularly true in areas of known sewage discharge in which cases there can be little doubt as to the origin of the coliform organisms."

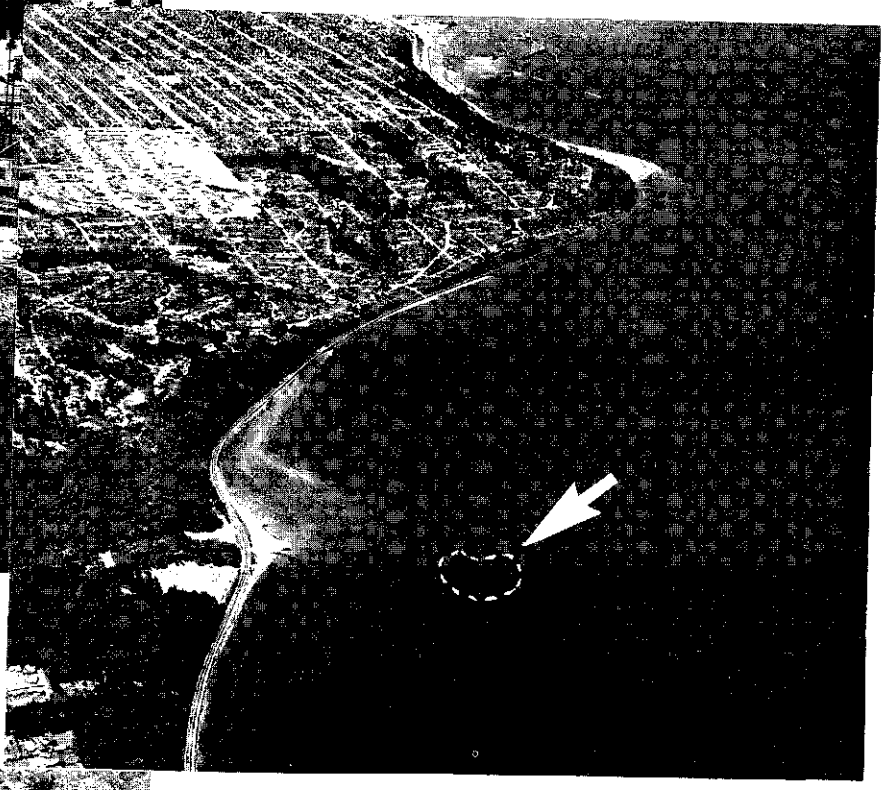
While it has been argued that there is little epidemiological evidence linking use of contaminated beaches with an actual outbreak of disease, conditions at these beaches are at least a potential hazard and have long been a source of concern to public health authorities. In any case, since coliform organisms are a known constituent of sewage, they should be held to a minimum in recreational waters for esthetic reasons if for no other.

Fig. 8-4. Location of and Conditions at Bacteriological Sampling Stations

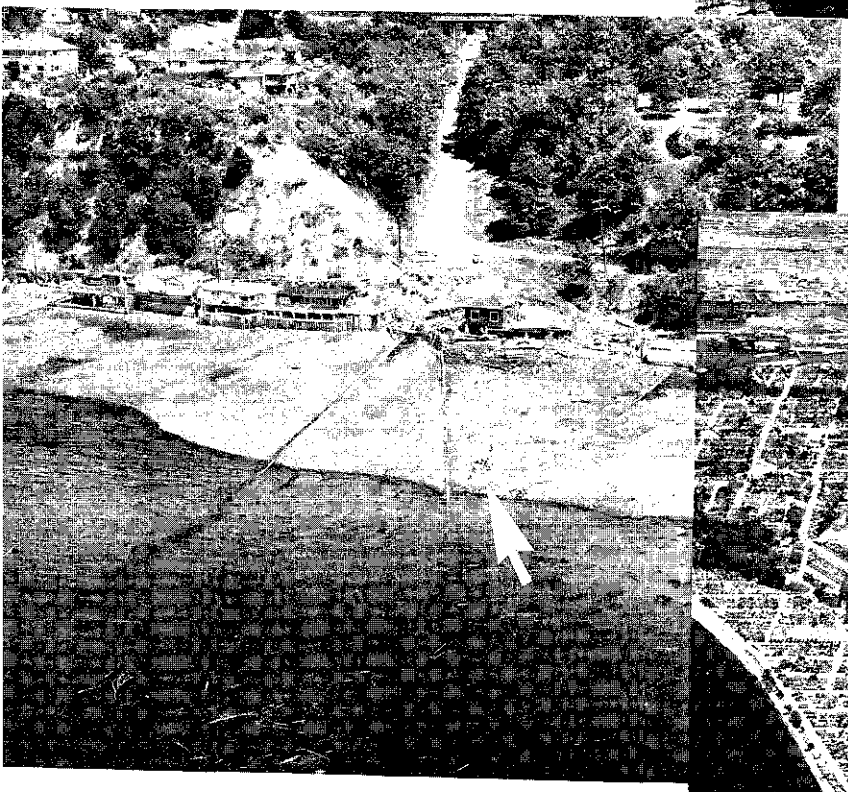

STUDY AREA BOUNDARY
ISSAQUAH
SAMMAMISH
MERCER ISLAND
RENTON
KENT
AUBURN
ALGONA
TUKWILA
NORMANDY PARK
TACOMA
PUGET SOUND
MAURY ISLAND
VASHON ISLAND
COMMENCEMENT BAY
DALCO PASSAGE
COLVOS PASSAGE
THE NARROWS
RUSTON
GIG HARBOR
W 6 SAMPLING STATION DESIGNATION
MPN OF COLIFORMS OVER 240 PER 100 ML IN LESS THAN 20 PER CENT OF SAMPLES
MPN OF COLIFORMS OVER 240 PER 100 ML IN MORE THAN 20 PER CENT OF SAMPLES
MPN OF COLIFORMS OVER 1000 PER 100 ML IN MORE THAN 20 PER CENT OF SAMPLES
SCALE IN MILES



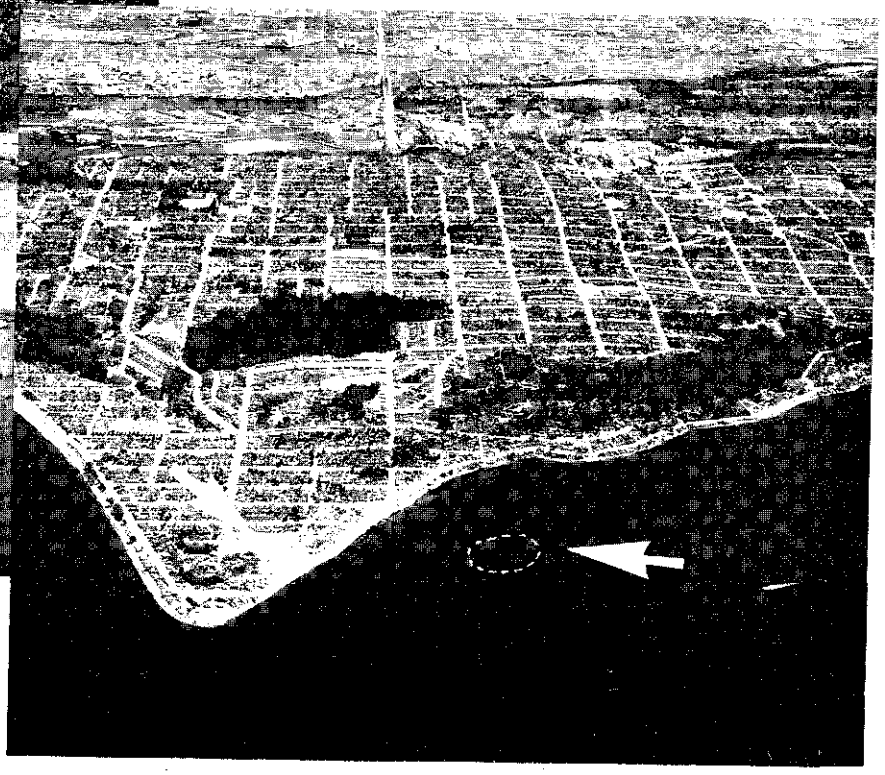
RAW SEWAGE OUTFALL near foot of Denny Way is one of 22 such discharges along the Elliott Bay waterfront.



SEWAGE IS DISCHARGED offshore from Carkeek Park public bathing beach at mouth of Piper Creek.



TWIN RAW SEWAGE OUTFALLS at foot of 32nd Avenue West. Note floating debris and children at play at water's edge (arrow).



RAW SEWAGE "BUBBLE" off Alki Point (right arrow). This ischarge will be eliminated when the new treatment plant, now nder construction, (left arrow) is completed.

As a final note, it should be pointed out (1) that many of the salt water bathing areas are at beaches which are utilized also for clamming, and (2) that Lake Washington is utilized at some locations as a source of domestic water supply. Both of these uses call for far more stringent standards than those applicable to bathing waters.

Nuisance Conditions

Raw sewage, in addition to being heavily infested with bacteria, contains both coarse and finely divided suspended and floatable substances, a large proportion of which is putrescible. When discharged into a body of water, the heavy components of sewage usually settle at the bottom, whereas the lighter components either are dispersed through the water or float at the surface. Depending on wind conditions and on current and other water movements, these substances may be transported for considerable distances, ending eventually as shore deposits, as accumulations in quiet areas, or as attachments to physical obstructions such as piers, boathouses and sea walls. Wherever found, they are unsightly and are productive usually of disagreeable odors, bottom deposits, and slime and fungal growths.

Nuisance conditions of the type just described prevail in and adjacent to many local water areas and are inevitable in view of the large number of raw sewage discharges and combined sewer overflows. Previous reports refer to them frequently. In 1948, Dr. Wolman pointed out that physical conditions were poor in inner Elliott Bay, in Lake Union, in parts of the West Seattle shoreline, and in the vicinity of the North Trunk outfall. In 1949, the Pollution Control Commission report on pollution in Puget Sound made repeated reference to the presence of sewage solids on beaches and to grease slicks of sewage origin in the offshore waters.

Various studies conducted as part of the present survey, such as the current studies reported in Chapter 11, afforded ample opportunity for direct observation of nuisance conditions in receiving water areas. Such conditions were most noticeable in the general vicinity of raw sewage outfalls and, to a lesser extent, in the vicinity of combined sewer overflows. At times, however, unsightly conditions and disagreeable odors were found to prevail over relatively large areas.

Sewage debris of every description was found regularly on many of the beaches opposite points of outfall. Debris was frequently found also in the vicinity of points of overflow along the west Lake Washington shoreline. At times, unsightly sewage solids, grease slicks and clouds of turbidity were traced for considerable distances offshore and were observed at other times to travel directly onshore. Odor conditions, which invariably were present near many of the points of raw sewage discharge, at times were noticeable several miles inland from the North Trunk outfall and over considerable areas along Elliott Bay and Duwamish River.

Nuisance effects at any given point can be expected to vary considerably, depending on current, tide and wind conditions. Nevertheless, it can be concluded (1) that nuisances in one form or another prevail continuously in some areas; (2) that there is probably no section of shoreline along Puget Sound, Elliott Bay or Duwamish River which is not at times affected to some degree; and (3) that local nuisances frequently prevail along the Lake Washington shoreline. Visible evidence of sewage from raw sewage outfalls, from combined sewer overflows, and from individual houseboat and pleasure craft discharges was seen more or less continuously in Lake Washington and the Ship Canal. Bypassing and overloading at some of the sewage treatment plants, which occur at rather frequent intervals during the wet season, produce noticeable though less pronounced effects in Puget Sound, Green River and Lake Washington.

Chemical and Biological Effects

Fortunately, Puget Sound has a tremendous capacity to dilute, disperse and assimilate the large quantities of oxygen consuming materials contained in the sewage and industrial waste discharges which it presently receives. Results of previous studies, coupled with those obtained from a limited number of analyses made during the survey, indicate that such discharges, except in the immediate vicinity of the largest outfalls, have essentially no effect on either the dissolved oxygen content or the chemical quality of the waters of the sound.

An examination of a series of 40 bottom samples taken in the vicinity of the North Trunk outfall indicated that such deposits are generally concentrated within a radius of 1,000 feet of the point of discharge. It indicated also that their effect on biological conditions, even within the area of concentration, is limited and that they produce no measurable effects beyond 1,000 feet from the point of discharge (Appendix C). Minor sludge deposits undoubtedly exist in the vicinity of all raw sewage outfalls but their effects are probably even more localized than those observed at the North Trunk outfall.

Recent studies (Chapter 12) have shown that dissolved oxygen concentrations in the lower reaches of Duwamish River have approached and, at times, have been reduced below the generally recognized limit of 5.0 ppm which is required to maintain favorable conditions for fish and other forms of aquatic life. Concentrations as low as 4.8 ppm have been found rather consistently in the vicinity of the Spokane Street bridge during critical periods of the year. In this connection

it should be noted that there are times when the oxygen content of the salt water entering the river with the tide approaches the critical limit. It is apparent, nonetheless, that the low oxygen condition in the river is due to the discharge therein of large quantities both of raw and primary treated sewage and of industrial waste. This condition, together with what it means in relation to the valuable fisheries resources of the river, is discussed in detail in Chapter 12.

In terms of general public interest, the one problem which has caused the greatest concern in recent years is the degradation of Lake Washington due to nutrient enrichment brought about by the discharge of sewage effluents. Numerous reports indicate that algal "blooms", with related odors, bottom sediment accumulations, and oxygen depletion, have been greatly accelerated in recent years. At the same time, there have been repeated warnings that the lake is approaching a state of eutrophication, meaning permanent impairment. Because of the importance of this problem, all of Chapter 10 is devoted to a discussion of the biology of Lake Washington and to a detailed analysis of the various factors which contribute to its degradation.

EFFECTS OF INADEQUATE DRAINAGE

Effects of inadequate drainage within Seattle were discussed earlier in this chapter in the section dealing with the overloading of combined sewers. Outside Seattle, drainage problems result largely from the lack of suitable facilities. As noted in Table 6-20, Lake Hills is the only community which has a properly planned system of storm drains.

Very little information is available or could be obtained in regard to drainage conditions in areas which either have no storm drains or are served by drains of inadequate capacity. As a consequence, it has not been possible either to determine the extent to which serious flooding has occurred in these areas or to assess the resulting damage in terms of its economic impact. In the course of the survey, however, it was observed that localized flooding occurs in suburban areas during periods of moderate to heavy rainfall.

From the standpoint of environmental effects, periodic flooding of suburban areas creates a multitude of hazards. In addition to the usual discomfort and inconvenience, public health and safety are endangered, especially in the case of children and their frequent urge to explore storm water accumulations. Likewise, traffic movement is impeded, water supply wells may become contaminated, and normal activity is curtailed.

Economic effects attributable to poor drainage are manifested most obviously by property damage which, depending on rainfall and local conditions, may range from minor to severe. Less apparent is the effect brought about by the practice in many areas of connecting roof, foundation, yard and even street drains to the sanitary sewers. This practice necessitates the oversizing of intercepting sewers and provision of greater pumping and hydraulic capacity at treatment plants than would otherwise be required. As stated earlier in Chapter 7, wet weather flows up to six times the average dry weather flow are being experienced at some locations.

Problems associated with inadequate drainage can be expected to become increasingly severe. Development of suburban communities, with a consequent increase in the areal extent of pavement, roofs, and other impervious surfaces, leads to an increase in storm water runoff and thus to a furtherance of the flood hazard where drainage facilities are inadequate. This situation will continue until such a time as a remedial program is initiated and construction of adequate facilities is undertaken on a systematic and properly integrated basis.

It is not intended to imply that the need for drainage improvements has been ignored. On the contrary, comprehensive studies have been made in some areas and plans for relief have been developed. Apparently, however, no way has been found as yet to finance construction of the recommended facilities.

Drainage problems in suburban areas have received considerable attention on the part of E. L. Evans, county road engineer of King County. As long ago as 1950, Mr. Evans advocated the enactment of legislation under which funds would be made available for the control of flood conditions. In a letter dated December 28, 1950, which was addressed to Charles O. Carroll, prosecuting attorney of King County, Mr. Evans suggested "that an effort be made to have legislation passed which would provide means by which the county could take care of drainage of many depressed areas or pot holes which cannot be drained naturally but are subject to flooding during the winter months, similar to the conditions encountered in the Oak Lake District which we took care of last year.

"We are constantly encountering flood conditions in low areas which have been developed in permitting faster runs than when the territory was in its natural state. We felt that the people living in these areas were entitled to relief the same as those living along a river bank.

"The Board of County Commissioners some time ago requested me to confer with your office, Planning Commission, Health Department, and other interested agencies in an effort to have a bill drawn revising the present flood control laws to take care of these conditions mentioned."

Approximately five years later, Mr. Evans wrote to the Board of County Commissioners, again stressing the flood problem and requesting legislative action.

The following comments from his letter of January 9, 1956 are of particular interest:

"The recent period of heavy rainfall has again emphasized the serious need for proper and adequate drainage disposal facilities for the densely populated suburban areas.

"This office has been literally besieged by demands for relief from this so-called 'outlawed water'. Our expression of sympathy and explanation that the county has no legal means of helping is of little consolation to those people whose homes are being flooded. Many contend the county does have responsibility since they control development of the area through platting and building regulations. Comprehensive studies have been made and feasible plans developed which would, if permitted, control these flash floods. The problem, however, is to obtain funds and legal authority to do the work. County roads, too, would be much easier to construct and maintain without this excessive moisture.

"A number of years ago we recognized the problem and by using improvement funds, supplemented by an appropriation from the state, corrected a flooding situation in the Oak Lake area. Subsequently an opinion obtained from the Prosecuting Attorney ended our program. River funds could not be used for this purpose. Flood damage from rivers could be alleviated but similar damage from drainage could not.

"Our attempt to amend the state law in 1951 was unsuccessful and since then the problem has become increasingly acute. I believe laws are needed now to permit the handling of storm drainage on a metropolitan basis."

It is apparent from the foregoing comments that county authorities are fully familiar with the drainage problems of suburban areas. It is apparent also that local flooding and resultant damage will become increasingly severe until a means is found both to attack the problem on an area-wide basis and to finance construction of the necessary facilities.

PART IV

BASIS FOR SEWERAGE AND DRAINAGE PLANNING

Chapter 9. Principles and Functions of Sewerage and Drainage

Chapter 10. Sewage Disposal in Lake Washington

Chapter 11. Sewage Disposal in Puget Sound

Chapter 12. Sewage Disposal in the Green–Duwamish River

Chapter 13. Design Criteria and Basis of Cost Estimates

Chapter 14. Sewerage and Drainage Subareas

Chapter 9

PRINCIPLES AND FUNCTIONS OF SEWERAGE AND DRAINAGE

Sewerage, along with waterworks, ranks among the oldest of public works practices. Its recorded history dates back nearly 3,000 years to Babylonian times in the seventh century before Christ. Remnants of ancient sewers have been found in Jerusalem and in Grecian cities, and the *cloaca maxima* a large sewer constructed to drain the Roman Forum, is still in use today. For nearly thirty centuries, sewerage has been allied with the growth of urban centers. In all modern cities it represents one of the most vital of public services and accounts for one of the major expenditures of public funds.

That a practice of such ancient origin and of such continuous and universal use should be one of the least understood of all public works practices is one of the oddities of modern society. Despite the integral part sewerage plays in present day living, the average citizen has little interest and, hence, little knowledge regarding the basic concepts and processes involved. Since sewers are out of sight, he is inclined to take them for granted, except during periods when they fail to function or until his particular recreational or other interest is affected by pollution or nuisance. Modern education and public information programs, somehow, have failed to break through this armor of indifference.

To provide a basis for understanding the problems discussed in this report as well as the solutions recommended, this chapter is designed to acquaint the reader with the purposes of sewerage systems, the functions performed by such systems, and the nature of the agencies engaged in providing public sewerage service.

PURPOSES OF SEWERAGE

In discussing the subject of why sewerage is undertaken, two general aspects are of importance, namely, those of public health and water pollution. Although early sewerage systems accomplished the purpose of removing sewage from the areas in which it was produced, disposal practices brought about problems of water pollution and its control.

Public Health Aspect of Sewerage

Disastrous human experience has amply demonstrated that the excreta of man constitutes one of the original vehicles of infection. It follows, therefore, that the proper collection and the safe disposal of this waste are essential to the protection of public health. In modern communities collection is accomplished by means of underground conduits in which water is utilized as the means of conveyance. These conduits and their appurtenances, together with the facilities for treatment and disposal, constitute a sewerage system.

Sewage may be defined broadly as the water-borne by-product of man and his environment. As produced in a sewerage system serving a large metropolitan area, sewage contains not only human excreta but an almost infinite variety of waste materials, ranging in quality from inert and harmless to toxic, noxious and infectious. Fresh sewage is relatively free of odor and is light gray in color. Stale sewage, or sewage in which dissolved oxygen has been depleted, is malodorous and noisome and ranges in color from dark gray to black.

All sewage, regardless of its origin and whether it be fresh or stale, is a potential hazard to public health and to individual as well as community comfort and well being. As such, it must be removed promptly from all premises in which it originates and must be disposed of in a manner which is both safe and completely innocuous.

Water Pollution Aspects of Sewerage

Early sewers were utilized almost exclusively for the removal of surface drainage. Direct connections from houses were in most instances forbidden in an effort to prevent fouling of the sewers. Putrescible material which gained entrance through street washings or through the illicit dumping of household wastes was flushed out periodically to reduce odor nuisance. The deplorable sanitary conditions associated with the disposal of household wastes in urban areas, however, prompted a continual search for better methods of waste disposal and led finally to development of the modern water carriage system of sewerage. Paradoxically, this solution to the problem of removal of waste from premises led to a new and, in many ways, an even more complex problem, namely, water pollution.

Development of Water Pollution Problems. Construction of the first authentic system of modern sewerage was undertaken in 1843 in connection with the rehabilitation of that portion of Hamburg, Germany, which had been destroyed in the conflagration of 1842. Twenty-

five years later, a committee of experts found the Hamburg system to be in excellent physical condition and almost without odor. Thus, modern sewerage became firmly established as an accepted practice.

Since sewage treatment was virtually non-existent in the early years of the 19th century, and since the dangers of water pollution were then unrecognized, it was only natural that sewers were almost universally designed to carry both domestic sewage and storm runoff and to discharge into the nearest convenient waters. It soon became apparent that while the water carriage system of sewerage had effectively accomplished the removal of wastes from premises, the nuisances and menaces to health which it sought to correct had, in effect, only been transferred from land to water. Moreover, the heavy concentration of sewage at the various points of discharge often resulted in fouling of the receiving waters to such an extent as to cause interference with activities involving the use of these waters. As the population connected to the sewers increased, these pollutional effects became more widespread and it was not long before water pollution became a serious community problem.

Significance of Water Pollution. Although the abatement of disease potential is manifestly the most important single benefit attributable to good sewerage practice, the protection of the quality of water resources through proper waste disposal operations has come to be recognized as an economic necessity. Obviously, this is because many segments of our economy and nearly every aspect of modern living are directly dependent on water use. It is obvious also that no condition can be tolerated which degrades or impairs the quality of a receiving water to such an extent that the other beneficial uses of the water become adversely affected.

Because of the undesirable constituents contained in untreated sewage and in many industrial wastes, the discharge of these wastes is by nature inimical to other water uses within the zone of influence of the discharge. On the other hand, sewage disposal is in itself an important water use, since streams and water bodies often provide the only financially feasible means for the disposal of community wastes. Discharged indiscriminately, however, these wastes can so alter the natural bacteriological, chemical, biological and physical characteristics of receiving waters as to render them partially or wholly unfit for other uses of a higher order. Many of the nation's rivers, lakes and coastal waters, including a substantial part of the environmental waters of the metropolitan Seattle area (Chapter 8), have been thus degraded. It is essential, therefore, that the conflict of interests which is created by the necessity for both economical waste disposal and the maximum utilization of water for other purposes be minimized to the fullest possible extent.

Control of Water Pollution in Washington

In the past ten to fifteen years, the nation as a whole has become increasingly aware of its water resources and of the damage thereto being caused by discharges of sewage and industrial wastes. As a consequence, federal, state and local laws relating to the control of water pollution have been enacted. In general, the federal government has final jurisdiction with regard to the pollution of interstate waters, while the states exercise control within their respective boundaries. Requirements established under state and other legislation are designed to protect the public health, to maintain receiving water quality consistent with beneficial water uses, and to prevent nuisance in the vicinity of points of waste disposal.

Responsibility for the control of pollution in all waters of the state of Washington, both inland and coastal, is presently vested in the Pollution Control Commission. Established in March 1945, this commission is empowered to prohibit the pollution of state waters and to prescribe and enforce rules and regulations governing their use for waste disposal.

Members of the Pollution Control Commission include the directors of the following state departments: Conservation and Development, Fisheries, Game, Health, and Agriculture. In addition, the Chief, Division of Engineering and Sanitation, State Department of Health, serves as technical secretary of the commission. All operations of the commission are under the direct charge and supervision of a director who is appointed by the governor.

Approval and Permit Procedures. As provided under state law, all plans and specifications for the construction of new sewerage systems, or for improvements or extensions to existing sewerage systems or sewage treatment or disposal plants, must be submitted to and be approved by the commission. In addition, the state requires, as a preliminary to design, the submission and approval of a report setting forth "...a sound and economical plan for a sewage works project..."

In the case of industrial wastes, a permit is required for each discharge into state waters. An application for such a permit must be filed with the commission and must provide all pertinent information.

There are no governmental requirements for disposal of storm drainage because such discharges are normally not incompatible with other receiving water uses. It should be noted, however, that no artificial change can be made in a stream bed without a permit from the State Department of Fisheries. Further-

more, surface drainage cannot be transferred from one watershed to another without the possibility of incurring liability under common law for flood damage.

Determination of Water Uses. As a preliminary to the establishment of waste discharge requirements, it is necessary to determine the beneficial water uses to be protected. In a large area such as metropolitan Seattle, these uses must be specifically determined for each body of water.

In response to a request made as a part of this survey, the Pollution Control Commission, in 1957, requested the various agencies concerned with the use of the waters of the study area to submit a statement relative to their particular interests, both present and proposed. The following agencies were included: State Department of Game; State Department of Fisheries; State Department of Public Health; State Parks and Recreation Commission; Seattle Park Board; Seattle Planning Commission; Seattle-King County Health Department; King County Planning Commission; Port of Seattle; Association of Washington Industries; and Snohomish County Planning Commission.

The response of each agency, together with a summary by the commission, was furnished to the survey as being representative of "the total interests of state and local agencies in the water uses of the area."[1] Insofar as they pertain to possible waste disposal sites, the uses thus set forth are described in detail in Chapter 10 (Lake Washington and tributary waters), Chapter 11 (Puget Sound), and Chapter 12 (Green-Duwamish).

Quality Requirements. It is the policy of the Pollution Control Commission to consider each waste discharge problem individually, taking into account the character and quantity of the proposed discharge and its probable effect on beneficial uses of the receiving water. This approach leads to the rational utilization of public waters, since it insures the protection of all beneficial uses, and permits at the same time the legitimate and planned utilization of their capacity to assimilate discharges of sewage and industrial waste.

Water quality objectives and minimum treatment requirements prepared by the Pollution Control Council, Pacific Northwest Area, are utilized by the commission as a guide in determining requirements for waste discharges. As applied by the commission, these objectives can be summarized as follows:

1. The minimum treatment requirement for domestic sewage is primary treatment.

2. No sewage or industrial waste shall be discharged into any of the waters of the state that will cause:

a. Reduction of the dissolved oxygen content to less than five parts per million (5 ppm).

b. Hydrogen-ion concentration (pH) to be outside of the range of 6.5 to 8.5.

c. Liberation of dissolved gases, such as carbon dioxide and hydrogen sulphide or any other gases, in sufficient quantities to be deleterious to fishes or related forms.

d. Development of fungi or other growth detrimental to stream bottoms, fishes and related forms, or to health, recreation or industry.

e. Toxic conditions that are deleterious to fishes and related forms or affect the potability of drinking water.

f. Formation of organic or inorganic deposits detrimental to fishes and related forms, or to health, recreation or industry.

g. Discoloration, turbidity, scum, oily sleek, floating solids, or the coating of aquatic life with oily films.

h. Temperature to be raised above the limit of tolerance of fishes and related forms.

3. In those waters which are used or are reasonably suitable for use as drinking water supplies, shellfish culture, recreation involving bodily contact with water, or in other instances where water use presents a definite public health hazard by presence or potential presence of disease-producing organisms, the bacteriological content of a representative number of samples shall not show the presence of coliform organisms in excess of the following:

Domestic water supply (without treatment other than disinfection and removal of naturally present impurities) . . . . . 50 per 100 ml

Domestic water supply (with treatment equal to coagulation, sedimentation, filtration, disinfection and any necessary additional treatment) . 2,000 per 100 ml

Shellfish culture (median value) . . 70 per 100 ml

Recreation involving bodily contact with water . . . . . 240 per 100 ml

SEWERAGE FUNCTIONS AND METHODS

Sewerage and drainage systems consist of (1) collection works, and (2) disposal works. Collection works include conduits and pumping stations designed to remove waste water from the community either by gravity flow or by pumping. In gravity systems flow is "free", that is, it moves continuously downhill with the surface of the liquid exposed to atmospheric pressure in a manner similar to stream flow. Where the depth of the conduit or sewer becomes excessive, pumping stations and short lengths of force mains are required to lift the sew-

[1] *A report on the Water Uses in the Seattle Metropolitan Area, Washington Pollution Control Commission, October 1957.*

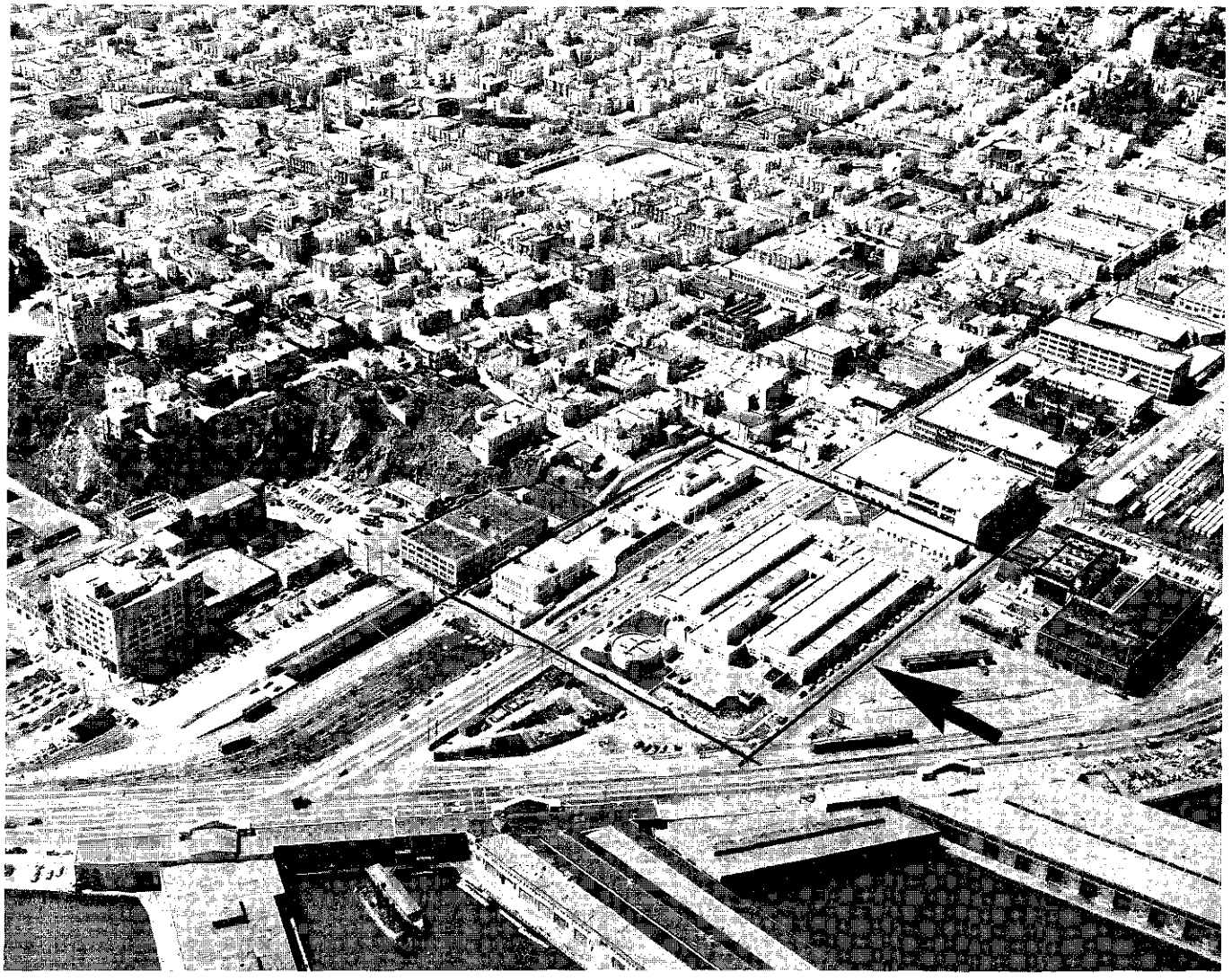
age from the deep sewer to one nearer the surface of the ground.

It is desirable at this point to direct attention again to the difference between separate and combined collection systems. Collection works designed to transport liquid wastes from households (domestic sewage) and those from manufacturing or processing plants (industrial wastes) are termed separate systems and the flow therein is called sanitary sewage. A combined system picks up storm water runoff in addition to the sanitary sewage and the flow is called combined sewage. Storm drainage systems receive only storm water runoff from streets, yards, roofs and paved areas and commonly discharge to the nearest convenient watercourse or water body.

Disposal works include the structures necessary for the final discharge of the liquid wastes either into receiving waters or onto land. In most systems, treatment of the sewage is usually required prior to final disposal. As indicated earlier, the degree of treatment is dependent upon the preservation of public health, the prevention of nuisance, and the conservation of water and land resources.

Separate versus Combined Collection Systems

It was not until the third quarter of the last century that the advantages of the separate system began to be recognized. One of the earliest installations of separate sewers in the United States was at Memphis, Tennessee, and was designed by Col. George E. Waring, Jr. About the same time, Benezette Williams designed a separate system for Pullman, Illinois. Although both systems subsequently proved to have inadequate capacities, the principles of separation proved to have merit. Primarily as a result of Waring's activities in advocating small pipe sewers in various communities, the National Board of Health engaged Rudolph Hering in 1880 to make an inspection



PRIMARY TYPE sewage treatment plant serving highly developed residential, commercial, and industrial area (North Point plant, San Francisco, California).

of current European practice. Hering was an influential leader of his generation in sanitary engineering and his views greatly affected the practice of his contemporaries. In a comprehensive report prepared after his return from Europe, Hering outlined the principles of separate versus combined sewers in a manner which remains today a thorough summing up of good practice. In 1886, shortly after Hering's report, a treatise entitled "The Separate System of Sewerage" was published by Staley and Pierson. Since that time, the construction of combined sewers in the United States has steadily declined.

Factors responsible for this decline stem from higher standards of sanitation than were considered necessary in 1886. The high cost of providing treatment capacity for combined sewage and storm flows and of constructing interceptors capable of carrying such flows without indescriminate bypassing to waterways has made the construction of combined systems generally uneconomical. In most states, furthermore, new combined systems are not permitted and, in many, extensions to existing combined systems must be by separate sewers and storm drains. As a consequence, combined systems originally constructed in many communities have subsequently been converted to separate systems. In the larger and older cities, combined systems persist because of the high cost of constructing new large and extensive sewers in congested areas. In addition, because such cities usually are situated on large rivers, lakes or tidal waters which enable considerable dilution of overflows, bypassing of combined flows of sewage and storm water can be tolerated as an economic necessity.

To ascertain what other metropolitan areas both in the United States and in Canada are doing with respect to separation of combined systems, a survey was made early in 1957 for the purposes of this report. Twenty questionnaires were sent out and 16 replies were received (Table 9-1).* Of the 16 cities and areas represented, three have sewers and storm drains which are principally separate, five are principally combined, and eight have a combination of separate and combined systems.

It will be seen that the tabulated evidence indicates a trend toward separation as sanitation standards improve. Also indicated is the conclusion that separation depends on local factors and that each case must be worked out in the light of controlling conditions.

Further facts disclosed by the separation survey are as follows:

1. Roof drainage is allowed to run over the ground surface in half the cities reporting. In many cities, however, roof drainage is discharged into the street gutter by means of a surface drain or pipe from the house.

2. Sidewalk icing caused by surface runoff from roofs during cold weather was reported to be of little consequence by half of the replies from areas where freezing occurs. Another one-third prohibit discharge across sidewalks and hence have no icing problems. The remainder report that corrective action is taken as required.

3. Foundation drainage is discharged to the sanitary sewers in seven of the places reporting, except in a few cases where storm drains are deep enough to receive it. In Baltimore, Maryland, and Vancouver, B.C., storm sewers, where provided, are designed to receive foundation drainage. Seven of ten cities stated that present residential drainage practices must be modified to conform to programs of separation under way. The other three cities, Boston, Buffalo and St. Paul, indicated that cost and other obstacles made it infeasible to require alteration of connections to effect complete separation.

Although the trend is toward construction of separate systems in most recently developed areas of these cities, this does not necessarily mean that combined systems already provided in older areas should be rebuilt. All factors must be evaluated for each specific case and decisions made accordingly. In general, the final answer will depend on (1) the capacity of existing sewers, (2) the frequency and intensity of rainfall, (3) the importance and uses of the water into which is discharged the overflow of diluted sewage and storm water, and (4) the cost of construction and maintenance.

Methods of Sewage Treatment and Disposal

A ton of sewage contains less than two pounds of suspended and dissolved organic material. Yet uncontrolled decomposition of this small quantity of matter renders the sewage offensive to sight and smell and produces esthetically objectionable effects. Sewage treatment is required in order to minimize or eliminate these effects, as well as to remove and destroy disease-causing organisms.

In essence, sewage treatment is the removal of relatively small amounts of mineral and organic material from the transporting water. Unfortunately, this operation is not simple. Some of the waste material is fairly coarse and can be removed by settling and flotation. Other portions are finely divided or dissolved and are not easily separated. Sewage treatment is accomplished in more or less separate operations or steps as follows:

1. Screening -- removal of coarse suspended and floating material.

2. Grit separation -- removal of sandy or gritty inorganic material.

3. Sedimentation -- removal of finely divided and suspended material.

Table 9-1. Summary of Drainage and Sewer Separation Survey, April 1957

Sewerage agency	Type of sewer system	Disposition of residential drainage (1) roof; (2) basement; (3) foundation	
		In existing separate systems	In separation of combined systems
City of Baltimore, Maryland	Separate	(1) As surface runoff or to gutter (3) To storm drain via side sewer	(1) As surface runoff or to gutter (3) To storm drain via side sewer
City of Boston, Massachusetts	Separate and combined	(1) Side sewer to storm drain	Usually remain in sanitary sewer
Buffalo Sewer Authority Buffalo, New York	See remarks	(1) To storm drain (2) To sanitary sewer (3) To sanitary sewer if storm drain is not deep enough	Former disposition remains unchanged
City of Chicago, Illinois	Combined	–	–
City of Detroit, Michigan	Combined	–	–
Metropolitan St. Louis Sewer District St. Louis, Missouri	Principally combined	(1) As surface runoff (2) To sanitary sewer (3) Discouraged. To sanitary sewer if used	No experience
City of Minneapolis, Minnesota	Separate and combined	(1) As surface runoff (2) To sanitary sewer	If economical: (1) As surface runoff (2) To sanitary sewer
City of New York, New York	Separate and combined	(1) To storm drain	(1) To storm drain
City of Oakland, California	Principally separate	(1) Leader to storm drain or gutter (3) May be pumped to sanitary sewer by owner	(1) Leader to storm drain or gutter (3) May be pumped to sanitary sewer. City will usually make change-over for (1)
City of Portland, Oregon	Separate and combined	(1) Usually as surface runoff (3) No special provision	(1) Usually as surface runoff (3) No special provision
City of Spokane, Washington	Principally combined	(1) Side sewer to storm drain or surface runoff (2) To sanitary sewer (3) To sanitary sewer	No experience
City of St. Paul, Minnesota	Separate and combined	(1) As surface runoff	No changes required
City of Toledo, Ohio	Principally separate	(1) Side sewer to storm drain, or to gutter, or french drain (2) To sanitary sewer (3) To sanitary sewer	(1) Side sewer to storm drain, or to gutter, or french drain (2) To sanitary sewer (3) To sanitary sewer
City of Vancouver, British Columbia	Separate and combined	(1) To storm drain (2) To sanitary sewer (3) To storm drain	(1) To storm drain (2) To sanitary sewer (3) To storm drain
Washington, D. C.	Separate and combined	(1) Surface runoff or line to gutter (2) Not allowed (3) To sanitary sewer	(1) Surface runoff or line to gutter (2) Not allowed (3) To sanitary sewer
City of Winnipeg, Manitoba	Combined	(1) As surface runoff (3) To sanitary sewer	–

Table 9-1. Continued

Method of financing local separation projects	Major problems in separation	Remarks
General fund	Problem of separating residential drainage at owners' expense.	System is nearly all separate. Major problem is controlling illicit connections.
General fund	Difficulty of effecting complete separation. High financing costs without assessing property owners.	Find it almost impossible to achieve on-site separation. Estimate separation projects are about 60 per cent effective.
General fund	No extraordinary problems.	City of Buffalo is combined. Statements regarding separate systems apply to neighboring municipalities.
–	–	System completely combined. No separation proposed.
–	–	Combined system. No attempt is being made to separate. Have interceptor system with overflow regulators.
Would be done on watershed basis	–	City of St. Louis has done no separating; considers it too costly for the city system.
General fund	–	In developed areas storm drains are constructed on streets perpendicular to sanitary sewers.
Not established	Financing.	Do not anticipate any extensive separation projects.
City Bonds or "Public Betterment Funds"	Financial and public education, particularly where side sewer changeovers are required.	When separating, a team checks all roof and yard drains for illegal connections.
General Obligation Bonds and general fund from sewer service charge	Financing is most difficult.	Storm drain system consists of short, shallow lines connecting local troublesome areas to nearest natural watercourse.
No experience	Financial problems. City now planning sewer service charge.	Separate storm sewers not deep enough to drain basement.
No experience	Financial problem of assessing costs. Difficult substructures problem in built-up areas.	
Local sewer district	Major problem is financial.	Planning to separate some areas using existing combined line for sanitary sewer.
No experience	–	Sanitary and storm line installed in same trench, both low enough to drain basements.
General fund	New lines must generally be lower than old ones – creates trunk problem. Substructures problem.	Have separated 1,400 acres. Separation in "Urban Renewal" areas will cost about $10,000 per acre.
–	Financing and physical problems of disruptions and substructures.	Winnipeg has not yet done any separating. Foregoing statements represent tentative thinking.

4. Biologic oxidation -- removal of dissolved and colloidal material.

5. Sludge reduction -- treatment and disposal of mineral and organic material removed in Step 3.

6. Disinfection -- destruction of disease-producing organisms.

Screening (step 1) is sometimes called "preliminary treatment" or "pretreatment"; sedimentation (step 3) and sludge reduction (step 5) constitute "primary treatment"; biologic oxidation (step 4) is "secondary treatment"; and the combination of primary and secondary processes is referred to generally as "complete treatment". It should be emphasized that the designation "complete treatment" applies only to the grouping of treatment units and not to the treated sewage itself. Complete treatment, meaning the complete removal of all waste matter contained in sewage, can never be attained. It can only be approached.

The relative proportion of waste material removed from sewage is known as the "degree" of treatment. For any specific case, the required degree of treatment depends both on the relative volume of receiving water and on the degree of degradation which can be permitted. It follows, therefore, that treatment plants do not necessarily contain all of the listed steps or operations. Instead, each plant embodies only those steps which are necessary in order to meet local requirements. For example, disinfection (step 6) is employed when it is necessary to protect public water supplies, bathing places, or shellfish layings against contamination from disease-producing organisms. The degree of treatment as related to receiving water capacity is illustrated in Fig. 9-1.

Preliminary Treatment. Removal of coarse suspended and floating material is accomplished by bar

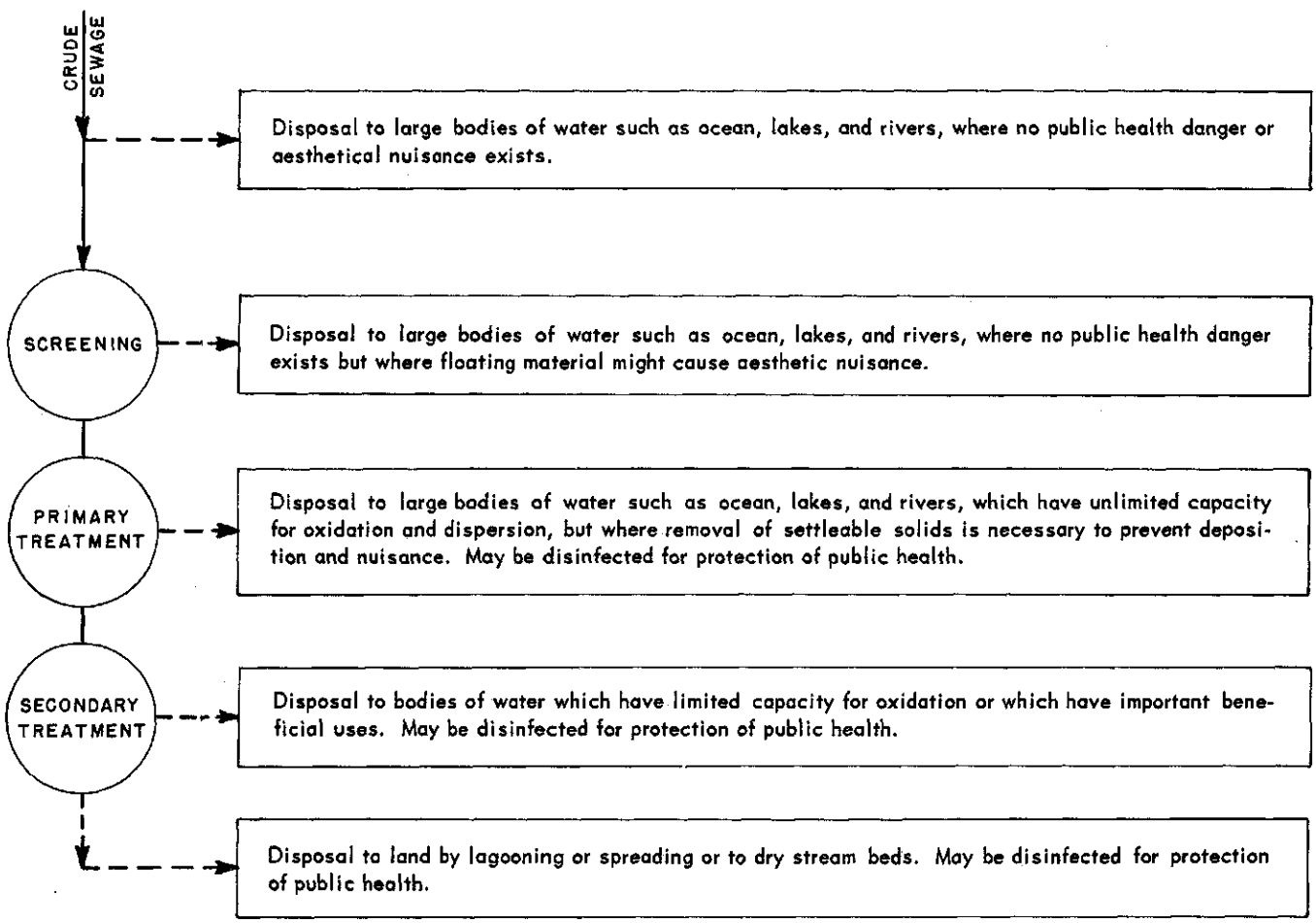


Fig. 9-1. Sewage Treatment Processes

Classification of sewage treatment processes is made on the basis of the degree of treatment. Minimum treatment is provided in primary plants, while increasing degrees of treatment are afforded in secondary plants. Primary treatment is used prior to secondary treatment or where disposal of the effluent is to be to receiving waters of capacity sufficient to ensure no danger to beneficial uses of the waters. Secondary treatment is used where disposal of the effluent is to be to receiving waters of limited capacity or onto land.

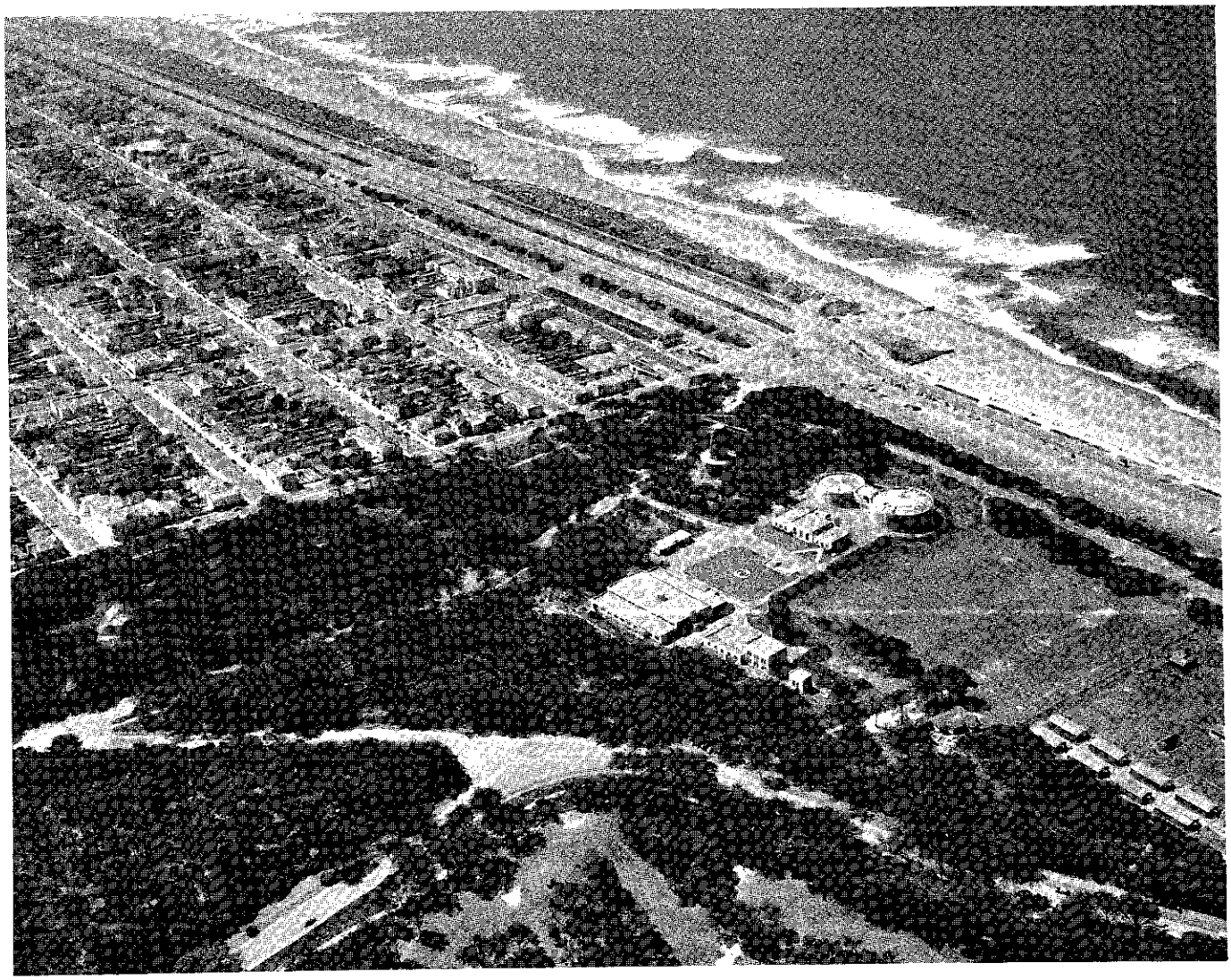
racks or bar screens and by comminuters or cutting screens. Bar racks may be hand cleaned or provided with automatic raking mechanisms. If hand cleaned, the open space between bars may be as small as 1-1/2 inches or as large as 4 inches. Clear spacing for automatically cleaned racks is usually 3/4 or 1 inch. Screenings removed from the sewage may be disposed of by burial, incineration, digestion, composting, or by grinding and returning to the plant influent.

Preliminary treatment may include the addition of chlorine to control odors and to increase the efficiency of primary treatment. Chlorine thus applied oxidizes hydrogen sulfide and other odor-producing substances and assists in reducing grease accumulations and slime growths throughout the treatment works. Dosage varies from 3 to 10 parts per million and may be controlled by the degree of septicity of the raw sewage entering the treatment plant.

Grit Separation. Grit is carried into sewers by water running off from roofs, yards, streets and other ground surfaces. Even in separate systems, some of this material gains access to the sewers. It is common practice, therefore, to provide for grit separation in all treatment plants, regardless of whether the tributary system is separate or combined. If not removed prior to sedimentation, grit settles with raw sludge in the sedimentation tanks and accumulates in the sludge digestion tanks. In addition, it causes excessive wear of mechanical equipment.

Separation of grit is accomplished by permitting the sewage to flow in a channel or tank at such a velocity that the heavy particles settle out while the lighter organic material is carried on. Grit thus deposited may be removed by mechanical scrapers or hydraulically by water ejectors. It may be dredged from the bottom of the tank by a clamshell crane.

After treatment by washing, gritty material removed from sewage is nonputrescible and generally inoffen-



SEWAGE TREATMENT PLANT in a famous park (Richmond-Sunset plant in Golden Gate Park, San Francisco, California).

sive. It may be disposed of as fill or by burial. As related to total sewage flow, grit loadings may vary from 0.2 to 0.4 cubic feet per million gallons for systems composed of sanitary sewers exclusively to as much as 80 to 100 cubic feet per million gallons for combined systems.

Sedimentation. Settleable and floatable organic solids are removed by sedimentation and skimming in primary sedimentation tanks. Material so removed is raw sludge and the liquid discharged from the tanks is primary effluent.

Primary sedimentation tanks may be rectangular or circular. Both types are normally provided with mechanical sludge and scum collecting mechanisms. In either case, it is common practice to provide for a detention period of 2 hours at average dry weather sewage flow.

Septic tanks actually perform a combination of two functions, namely, sedimentation and digestion. As sewage flows through such tanks, suspended material settles to the bottom where it gradually undergoes anaerobic digestion and partial liquefaction. Periodic cleaning of the tank is required to remove the accumulated deposit of relatively inert material. As commonly utilized for an individual household disposal system, septic tanks are sized to provide a capacity roughly equivalent to the daily sewage flow.

Biologic Oxidation. Effluent from primary sedimentation tanks contains approximately three-fourths the original quantity of organic material present in the sewage. Under controlled conditions of aeration and contact surfaces, large numbers of organisms develop which flocculate colloidal particles and absorb and feed on dissolved organic matter to the extent that the carrying water is largely freed of its organic load. Any process using such natural forces is called "biologic oxidation" or secondary treatment.

Historically, the development of biologic oxidation processes has proceeded from broad irrigation through intermittent sand filters and contact beds to trickling filters, activated sludge units, and oxidation ponds. All of these are based on the ability of microorganisms to flocculate and remove colloidal and dissolved organic matter.

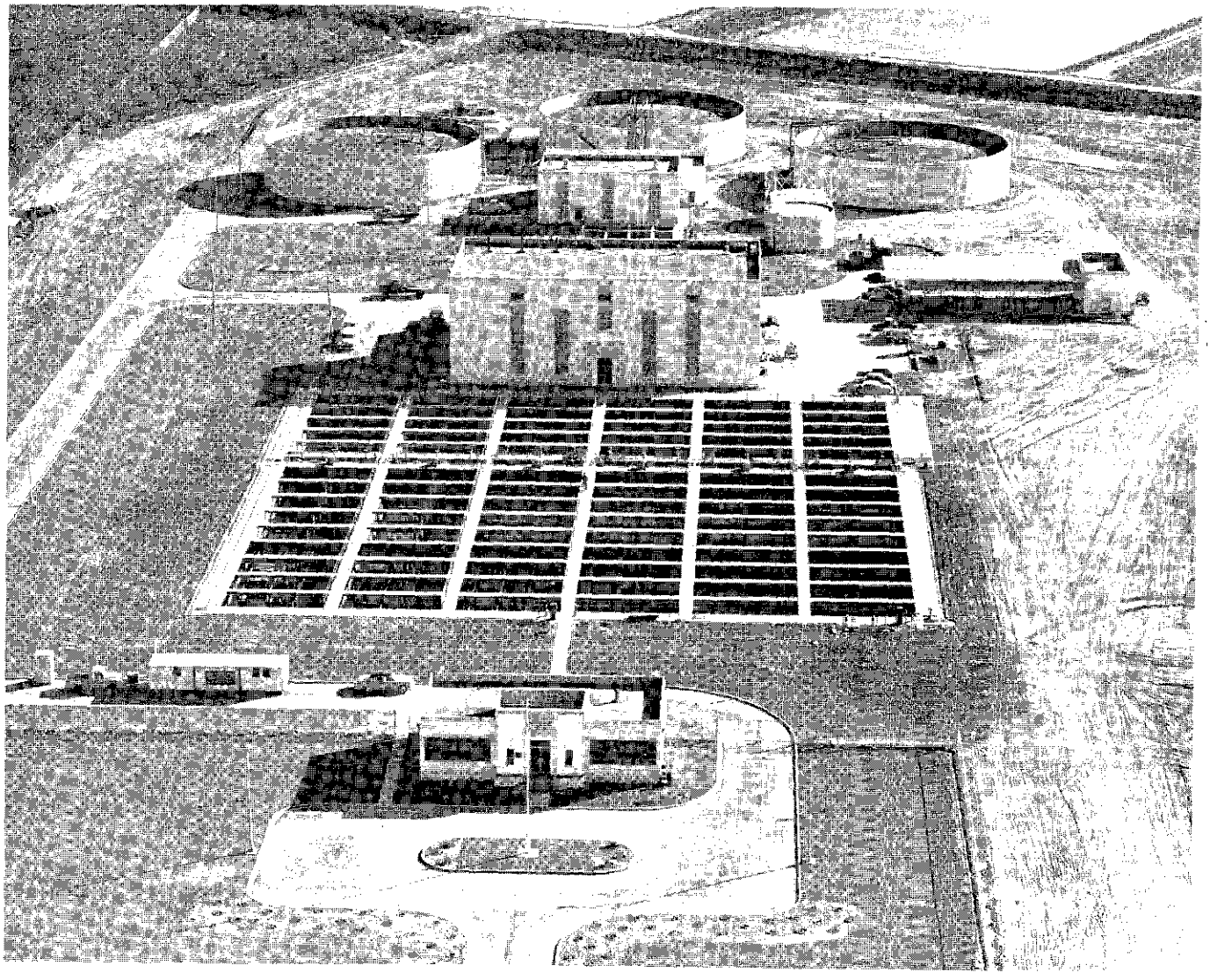
Although a number of biologic oxidation processes have been developed during the last 50 years, only two, high rate trickling filtration and the activated sludge process, are significant in their application to the Seattle area. A third process, oxidation ponds, is worthy of mention since it may be utilized in the Seattle area as a temporary measure in some localities.

A trickling filter consists of an open-bottom tank 4 to 10 feet deep filled with rock varying from 2 to 4 inches in diameter. Sewage is distributed over the rock bed from a rotary distributor and allowed to trickle downward while currents of air induced by the flow of sewage carry oxygen to organic films on the rock surfaces. Flocculated organic material in the form of bacterial slime continually sloughs or breaks away from the filter rock surfaces and, depending on local conditions, is either discharged with the plant effluent or removed by secondary sedimentation. Loadings on trickling filters, expressed in terms of weight of BOD, vary from 20 to 130 pounds per day per 1,000 cubic feet of rock, depending on degree of treatment required.

In the activated sludge process, the sewage is stirred in aeration tanks in the presence of sludge which contains the organisms required for purification. Oxygen is supplied either by air blown through tubes located along the bottom of the tank or by mechanical means. Activated sludge is maintained in sufficient concentration in the aeration tank by settling formed floc from the effluent sewage and returning it to the influent sewage. Excess activated sludge is returned to the raw sewage or concentrated and pumped to digestion tanks. Loadings on activated sludge units also depend on the degree of treatment required, and vary from 30 to 110 pounds of BOD per day per 1,000 cubic feet of aeration tank capacity.

Oxidation ponds, while probably the least expensive form of secondary treatment, are subject to limitations of available land, temperature and sunlight. Under this process, sewage first receives primary treatment and is then introduced into large open ponds which provide a detention or holding time of 20 to 30 days. Given favorable conditions of temperature and sunlight, a symbiotic association of saprophytic bacteria and algae develops. As a result, large quantities of oxygen are produced, putrescible organic matter is oxidized and stabilized, and coliform bacteria densities are greatly reduced. Oxidation ponds are normally constructed with an effective depth of about 4 feet. On that basis, they are commonly designed for a BOD loading of 50 to 75 pounds per day per acre.

Sludge Digestion. Floatable and settleable solids removed by sedimentation require further treatment prior to final disposal. This is obtained by digestion in separate tanks from which air is excluded. In these, the process of digestion is hastened considerably by heating the contents to about 95° F. During the detention period, which ranges from 20 to 60 days, the putrescible organic matter, which is roughly 50 per cent of the total organic content, is converted by biologic action to a combustible gas containing about 70 per cent methane and to a stable, nonputrescible residue termed digested sludge. As described later in this chapter, the gas may be utilized as a source



PRIMARY TYPE sewage treatment plant, located in San Jose, California.

of fuel and the digested sludge may be used as a soil conditioner. Loadings on digestion tanks, expressed in terms of total solids, vary between 0.05 and 0.25 pounds per day per cubic foot of tank capacity.

Disposal of Digested Sludge. Upon withdrawal from the digesters, sludge normally contains about 94 per cent water and may be disposed of without further treatment by: (1) lagooning in ponds which, after a number of years of filling, are covered over with earth; (2) discharging into a suitable body of water; or (3) hauling to a location where it can be utilized directly as a soil conditioner. Lagooning is practicable where sufficient land is available and where the site is reasonably isolated from other developments. Discharge to a body of water is practicable only when it may be accomplished without adversely affecting other beneficial uses. This limitation, as it applies to conditions in the metropolitan Seattle area, is considered in detail in Chapter 11. In most cases, direct utilization of wet sludge as a soil conditioner is not practicable because of hauling costs and because the demand for such material is intermittent or seasonal.

In situations where digested sludge cannot be disposed of in its liquid state, it becomes necessary to provide facilities for drying or incineration. Drying may be accomplished either by mechanical means, using vacuum filtration followed by heat, or by spreading on underdrained or permeable open-air beds where part of the liquid drains away and the remainder evaporates. Mechanical dewatering does not require as much space as open-air beds and is not subject to the climatological influences which affect drying by evaporation. In the Seattle area, for example, it would be necessary to enclose open air beds or to provide a storage capacity such that a one-year output could be dried and removed from the beds during the sum-

mer months. Incineration of digested sludge produces a completely inert material suitable for use as fill.

Disinfection. Disinfection of treated effluent is employed to reduce the concentration of pathogenic bacteria contained in sewage. Effects thereof are measured in terms of coliform organisms. The degree of disinfection which must be achieved is dependent not only on the beneficial uses of the receiving waters but on the magnitude of dilution, diffusion and other factors which tend to reduce the concentration of the sewage in general and of the pathogens in particular. While disinfection is normally accomplished by chlorination, it should be noted that secondary treatment in oxidation ponds can produce an effluent of almost comparable bacteriological quality.

Reaction or contact time for purposes of chlorination depends on the degree of disinfection required and ranges generally from 20 to 30 minutes. This may be provided in tanks constructed for such a purpose or may be obtained in the course of flow through an outfall from the treatment works.

Effluent Disposal. Depending on the particular location, sewage effluent may be discharged to bodies of water or onto land. In either case, the feasibility of disposal must be evaluated in terms of its effect on physical, chemical, and biologic conditions which, in turn, determine the extent to which beneficial uses may be endangered. Primary interest in the metropolitan Seattle area, quite naturally, is centered in disposal to bodies of water.

By-Product Recovery and Utilization

So far as presently known, substances recoverable from sewage treatment processes never have had a sales value commensurate with the costs of their recovery. Nevertheless, some of the cost of treatment may be defrayed by utilization or sale of (1) water for industrial, agricultural, or other use; (2) combustible gas as a source of fuel; and (3) digested sludge as a soil conditioner.

Water. Water reclaimed from sewage can be utilized economically only in those cases where its cost is less than that of obtaining water from additional natural sources. Reclamation costs depend upon the additional treatment required to produce a usable supply from a sewage effluent which is otherwise suitable for disposal to the local receiving waters. This additional cost is not, of course, chargeable to the sewage disposal function. In any case, the relative abundance of rainfall and of natural water supplies in the metropolitan Seattle area is such that sewage reclamation is not likely to be an economically attractive project.

Gas. Gas produced in the process of sludge digestion can be utilized economically as a fuel for (1) internal combustion engines, either driving sewage pumps or generating power for plant purposes, (2) digester heating, or (3) space heating. Sludge gas is produced in a quantity ranging from 0.5 to 1.3 cubic feet per capita per day and has a heat content in the order of 600 btu per cubic foot. It is recognized, however, that production may decrease during the winter at plants served by combined sewers. For that reason, gas storage facilities should be provided at all such plants.

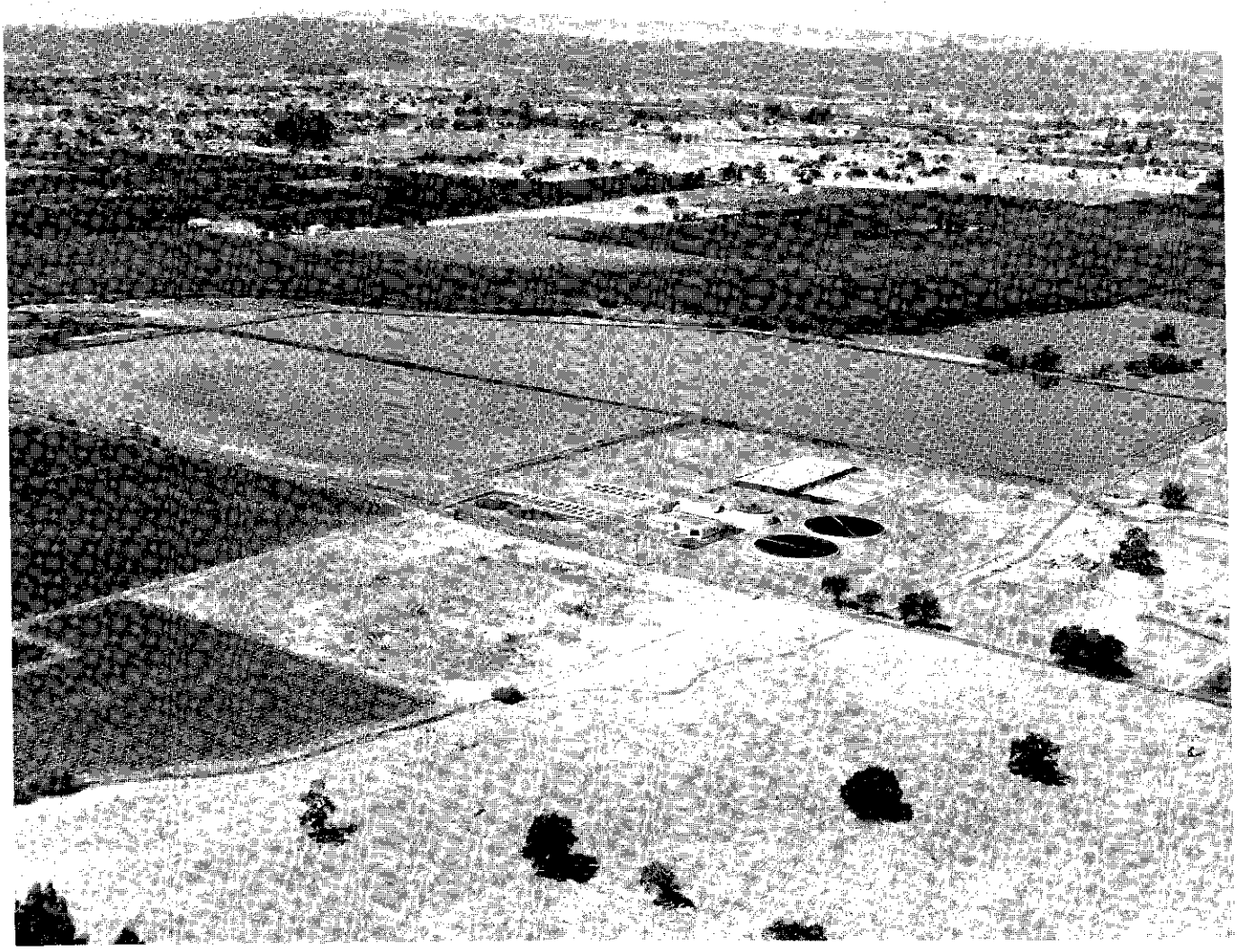
Digested Sludge. Dry sludge contains about 50 per cent of organic matter, of which from 1 to 2 per cent is organic nitrogen. It also contains very small percentages of potassium and phosphate. In all, the characteristics of digested sludge are such that it is an excellent soil builder or soil conditioner. Various chemicals must be added in appropriate amounts, however, to bring the concentration of nitrogen and other constituents up to those available in commercial fertilizers.

The cost of preparing digested sludge for further use depends, of course, on the nature of the least expensive satisfactory method of disposal. Any cost in excess of that involved in the latter should be considered as the cost of digested sludge reclamation. Since relatively inexpensive methods of disposal are available in the metropolitan Seattle area, the cost attributable to sludge reclamation would be relatively high.

AGENCIES FOR PROVIDING SEWERAGE SERVICE

In an area comprising a number of scattered and widely separated cities, each such city normally is independently responsible for the provision and operation of its own sewerage system. With that type of development, there is little or no reason to consider the establishment of any single agency to take over the sewerage function and to provide sewerage service to the area as a whole. On the other hand, in the case of a metropolitan area with one or more large cities surrounded by a number of other incorporated cities and unincorporated communities, experience has demonstrated not only the desirability but the necessity of establishing a metropolitan sewerage agency.

Perhaps the greatest single benefit which can be achieved through the establishment of a metropolitan agency is the elimination of a multiplicity of adjacent but nevertheless independent treatment and disposal operations. This can be achieved through construction of intercepting sewers for conveyance of sewage to a central treatment plant. Experience indicates that a central plant can be operated more economically



COMPLETE TYPE sewage treatment plant at Santa Rosa, California, showing oxidation ponds.

and with a higher degree of reliability and performance than can a group of independent plants. It is frequently possible, furthermore, to construct centralized treatment works at an isolated site of lesser actual or potential value than is the case with smaller local plants which are perforce located in areas of denser population. In addition, effluent from a centralized plant can often be conveyed economically to a more suitable point for disposal than can effluents from numerous independent plants.

Throughout the conduct of this survey it has become increasingly apparent that the multitude of problems relating to sewage collection, treatment and disposal in the metropolitan Seattle area can be solved only through the formation and implementation of a metropolitan sewerage agency. This conclusion, of course, is by no means new or original. As far back as 1890, Benezette Williams saw the need for a centralized sewerage authority in the Seattle area. In 1927, the Municipal League of Seattle and King County published in the Seattle Municipal News of March 19 an article concerning "A Sewerage Commission for the City of Seattle with Statutes and Regulations Bearing Upon Sewers and Sewage." At that time, the formation of a metropolitan sewerage district to comprise some or all of King, Pierce, and Snohomish counties was seriously advocated. Subsequently, a metropolitan Sewerage Commission was created by the Commissioners of King County in 1933. Both Mr. E. French Chase, its chief engineer, and Dr. Abel Wolman, in his influential report of 1948, recommended a centralized sewerage authority.

Because of the general interest which has developed in the Seattle area with reference to metropolitan sewerage agencies, it is appropriate to consider briefly (1) some of the similar organizations in this country and abroad and (2) the types of sewerage agencies which can be formed under laws of the state of Washington.

Metropolitan Sewerage Agencies

Although development of a complete and detailed inventory of all metropolitan sewerage agencies is beyond the scope of this report, the data presented in Table 9-2 indicate that the practice of dealing with sewerage on a metropolitan area-wide basis has been adopted throughout a large part of the world. Historically, the development of metropolitan area sewerage began in London, England, in 1848 with establishment of the Metropolitan Sewerage Commission. Intolerable conditions in the Thames River were subsequently ameliorated by the construction of intercepting sewers and sewage treatment plants under direction of this over-all authority.

In the United States, the first agencies of a metropolitan nature were established in Chicago and in Boston around 1890. Since then, agencies have been organized in most of the other metropolitan areas of the country. On the Pacific Coast, the two best known examples are the Los Angeles County Sanitation District and the East Bay Municipal Utility District in the San Francisco Bay area.

In the Los Angeles County Sanitation Districts, the sewage of 2.5 million people is conveyed to a single treatment plant in the southwestern part of the county, with effluent disposal to the Pacific Ocean in deep water 5,000 to 8,000 feet offshore. In the 19 sanitation districts associated in this enterprise are 41 incorporated cities, 7 of which each have populations of 50,000 or more. Per capita costs under this inclusive scheme are but a fraction of those which would prevail if treatment and disposal were obtained instead

Table 9-2. Some Metropolitan Sewerage Schemes

Date formed	Agency	Location	Municipalities included	Population served	Treatment works	
					Name	Capacity, mgd
1848	London County Council	London, England		4,500,000	Northern Outfall Southern Outfall	120 220
1889	Sanitary District of Chicago	Chicago, Illinois	60	4,400,000	Calumet North Side Southwest West Side	136 250 400 472
1889	Metropolitan District Commission	Boston, Massachusetts	34	2,000,000	Deer Island[a] Nut Island	145 112
1902	Passaic Valley Sewerage Commission	Passaic Valley, New Jersey	28	1,100,000	New York Bay	145
1913	Ruhrverband	Ruhr Valley, Germany	22	1,500,000		
1914	Greater Vancouver Sewerage and Drainage Board[b]	Vancouver, British Columbia	4	480,000	Iona Island[c]	40
1923	Los Angeles County Sanitation Districts	Los Angeles, California	41	2,500,000	Joint Disposal Plant[d]	200
1935	West Middlesex Drainage Board	Middlesex County, England	28	1,400,000	Mogden	300
1937	Colne Valley Sewerage Board	Hertfordshire, England	13	430,000	Maple Lodge	60
1941	East Bay Municipal Utility District[e]	Oakland, California	6	625,000	Oakland	128
1944	Auckland Metropolitan Drainage Board	Auckland, New Zealand	17	300,000	Manukau[a]	66
1946	Allegheny County Sanitary Authority	Pittsburgh, Penn.	129	1,600,000	Pittsburgh[f]	150
1948	City of Cincinnati	Cincinnati, Ohio	24	700,000	Little Miami Mill Creek	29 120

[a]*Under construction.*

[b]*Formed as Vancouver and Districts Joint Sewerage and Drainage Board and reorganized in 1956.*

[c]*Under design.*

[d]*Districts also have five small plants.*

[e]*Special Assessment District No. 1 formed for sewerage purposes.*

[f]*Plant serves population of 1,250,000 in 62 municipalities.*

at a considerable number of plants at widely scattered locations.

In the Los Angeles metropolitan area, the experience of Pasadena is typical of many in that area and exemplifies the savings which can be achieved through centralized sewer planning. In 1945, Pasadena and three smaller cities were faced with the alternatives either of enlarging their existing treatment plant or of abandoning that plant and constructing a trunk sewer to the sanitation district interceptor. Adoption of the latter alternative, which involved the construction of a 25-mile trunk and the purchase of rights to use the district facilities, enabled these four cities with a population of about 220,000 to effect an average annual saving of over one million dollars.

Facilities operated by the East Bay Municipal Utility District serve six cities on the east shore of San Francisco Bay. These cities presently have an estimated total population of about 625,000. Sewage is collected by two main interceptors and conveyed therein to a single plant for treatment and disposal. Effluent, as well as digested sludge, is discharged through multiple outlets into the adjoining waters of San Francisco Bay. Elaborate studies conducted in 1940-41 by a survey board clearly indicated the economic as well as other important advantages of such an enterprise. Its actual consummation has served to demonstrate the validity of those findings.

Sewerage Agencies in Washington

Washington state laws determine the legal means whereby sewerage agencies can be formed and sewerage projects can be initiated, financed, constructed, maintained and operated. Until 1957, when legislation was enacted enabling the formation of metropolitan municipal corporations, performance of sewerage functions was vested in cities and towns, various types of sewer districts, and some drainage districts. These agencies are limited to local sewerage activities and are not legally capable of undertaking area-wide projects of the magnitude and scope required in many parts of the metropolitan Seattle area. The following discussion, therefore, is restricted to the metropolitan municipal corporation.

Chapter 213 of the laws of 1957 permits an area containing two or more cities situated in one or more adjoining counties to organize a metropolitan municipal corporation for the purpose of providing one or more of the following services: (1) sewage disposal, (2) water supply, (3) public transportation, (4) garbage disposal, (5) parks and parkways, and (6) comprehensive planning. Within the same legal limitations applicable to other municipal agencies in Washington, the metropolitan corporation is given the power to issue general obligation bonds, tax anticipation warrants, revenue bonds, and special assessment bonds, and to fix rates and charges for service.

If authorized to perform the sewage disposal function, the corporation is given the following additional powers: (1) to prepare a comprehensive plan for sewage disposal and storm water drainage for the metropolitan area; (2) to design, build, maintain, and operate sewage disposal and storm water drainage facilities within and without the metropolitan area; (3) to acquire the sewerage facilities of municipalities or districts, but only with the consent of the legislative body of such municipality or district; (4) to require the discharge of sewage into its sewers; (5) to fix rates; (6) to establish standards for sewer construction; (7) to acquire, construct, maintain, and operate facilities for local collection of sewage or storm water in portions of the metropolitan area not within any city or sewer district; and (8) to perform local sewerage functions within a city or district upon consent of the legislative body of the city or district.

Formation of a metropolitan corporation can be initiated by resolution or by petition. The resolution, or concurring resolutions, calling for an election may be adopted by the city council of a central city, by the city councils of two or more component cities other than a central city, or by the board of commissioners of a central county. If initiated by a petition calling for an election on the matter, the petition must be signed by at least four per cent of the qualified voters residing within the metropolitan area and must be filed with the auditor of the central county. The resolution or petition must describe both the boundaries of the proposed area and the functions to be performed.

After a petition is filed, the next step is a public hearing by the county commissioners. If sewage disposal is a proposed function, the act expressly prohibits deletion of any portion of the area which is or can be expected to contribute to the pollution of any watercourse or body of water within the boundaries described in the petition. This provision is necessary to ensure protection of the receiving waters.

A special election is called not less than 60 days nor more than 120 days following the adoption of a resolution by the county commissioners at this preliminary hearing. As stated in the act, "If a majority of the persons voting on the proposition residing within the central city shall vote in favor thereof and a majority of the persons voting on the proposition residing in the metropolitan area outside of the central city shall vote in favor thereof, the metropolitan municipal corporation shall thereupon be established. . ."

Activities of a metropolitan municipal corporation are administered by a metropolitan council which is

composed, as detailed in the act, of elected public officials from each component city and county. Furthermore, a corporation having sewage disposal as a function is required to have an advisory council with one official from each city or district which operates sewerage facilities within its area.

Powers and limitations of a metropolitan municipal corporation with respect to the financing of sewerage and storm drainage projects are discussed in detail in Chapter 19.

Chapter 10

SEWAGE DISPOSAL IN LAKE WASHINGTON

Inland lakes and streams of the metropolitan Seattle area constitute one of its foremost attractions. Because of their natural beauty, extensive residential and recreational developments are taking place in adjacent areas. Furthermore, there is every indication that future settlement therein, as well as future use of public and private facilities on or near these waters, will increase at a rate which will at least equal the predicted rate of population growth for the metropolitan area as a whole.

Continued rapid urbanization, particularly in the vicinity of Lake Washington, has served to aggravate the need for adequate protection of these waters. Although public attention has been focused mainly on the deleterious effects of sewage disposal in Lake Washington, the influence of inland metropolitan sewage disposal practices will eventually extend to all the lakes and streams within the Lake Washington drainage basin. Complete protection of these waters is imperative to the successful carrying out of any plan for sewerage of the area.

Lying in the center of the metropolitan area, Lake Washington is the most notable of the inland waters. Because of its great scenic beauty and charm, urban developments have tended to concentrate along its shoreline (Fig. 10-1). Included are nine incorporated cities (Seattle, Kirkland, Houghton, Bellevue, Beaux Arts, Medina, Hunts Point, Clyde Hill and Renton) and numerous unincorporated residential communities. At present, approximately 20 per cent of the population of the metropolitan area resides in the immediate vicinity of the lake. As to the future, it is expected that over 50 per cent of the ultimate metropolitan population will live within the area encompassed by the Lake Washington drainage basin.

Outflow from Lake Washington is discharged to Shilshole Bay through a series of ship canals, Lake Union and the Government Locks. The lake itself is the natural drainage basin for an area of approximately 182 square miles. In addition, it receives runoff from 402 square miles of adjacent drainage basins tributary to Lake Sammamish and the Sammamish and Cedar rivers. Of the combined total of 584 square miles, 350 are situated within the study area.

WATER USES AND WATER QUALITY REQUIREMENTS

Water quality criteria to be applied to any given water must obviously be determined on the basis of the intended uses of that water. In order, therefore, to evaluate on a reasonable basis the possible effects of waste disposal, it is necessary first to establish these uses, both present and future.

Water Uses

Water uses in the Lake Washington drainage basin have been determined by the State Pollution Control Commission as part of a recent survey of water uses in the metropolitan Seattle area.[1] In all cases, of course, the principal aim of any program of sewage disposal must be the effective protection and preservation of those uses.

Lake Washington and Tributary Waters. Primary beneficial uses of Lake Washington, as described by the Pollution Control Commission, are recreational, including swimming, boating, and fishing; fish propagation; and water supply, both public and private. Within limits, these uses and functions would apply also to other lakes and to the principal tributary waters in the drainage basin.

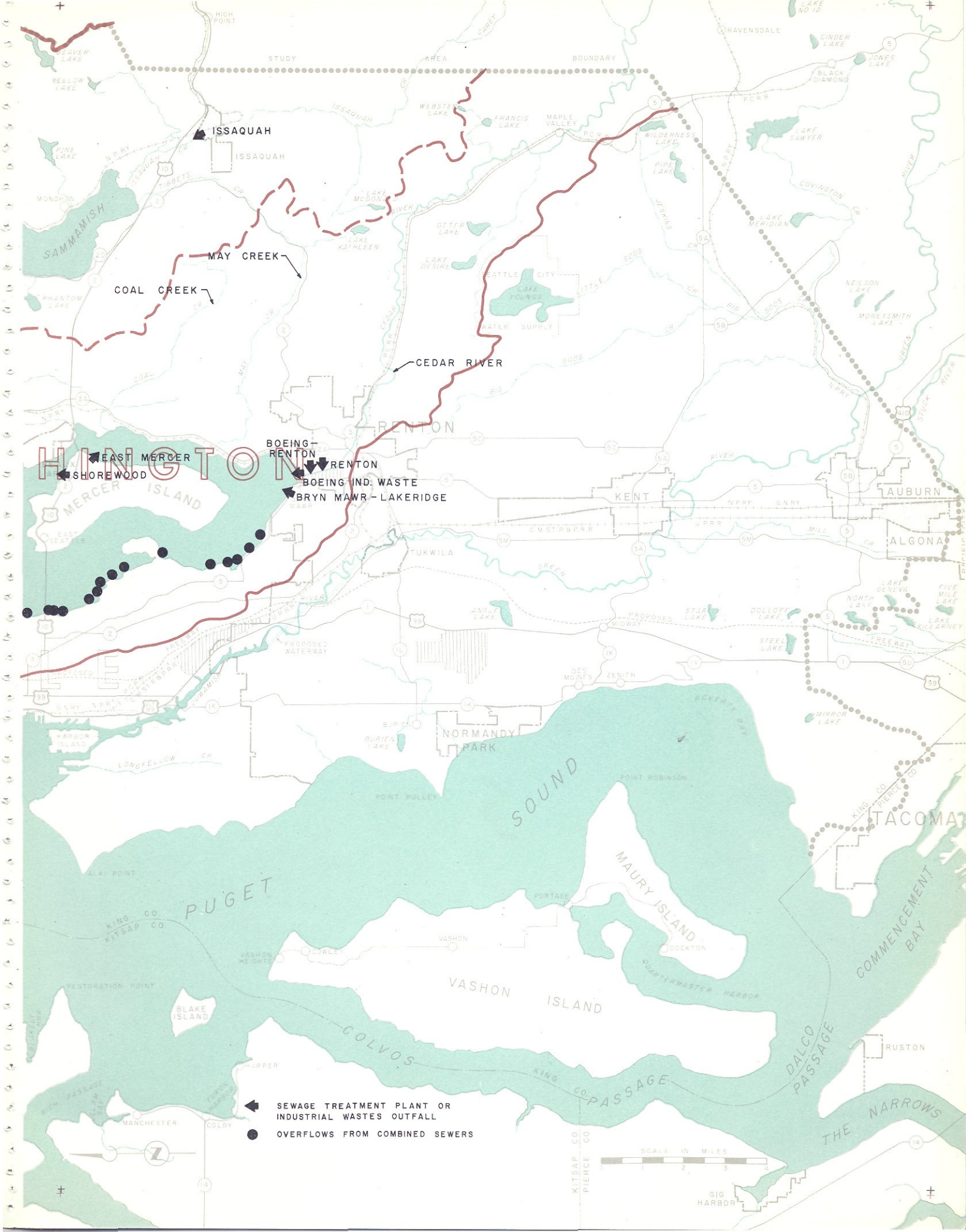
Lake Washington has developed into an outdoor recreational center for the entire metropolitan area. The city of Seattle alone maintains six public bathing beaches and parks along the lake and a number of other communities have developed similar facilities. Thousands of residents and visitors derive recreation and pleasure from the lake each year. Its economic value and esthetic importance to the metropolitan area defy calculation.

A number of communities and many individuals derive their drinking water from lakes in the basin. It is expected, however, that this practice will decrease in the future following further development of the lakes and lake fronts for residential and recreational purposes. In any event, domestic use of untreated waters in which extensive recreational activity, including swimming, is encouraged, is an insanitary practice and should not be condoned.

Lake Washington now receives discharges of sewage, both treated and untreated, from surrounding communities and lake shore residents (Fig. 10-1). Continued disposal of sewage in waters of the Lake Washington drainage basin obviously should not be in conflict with any other beneficial use of those waters.

[1] *State of Washington, Pollution Control Commission,* A Report on the Water Uses in the Metropolitan Seattle Area, *October 1957.*

Fig. 10-1. Lake Washington Drainage Basin


STUDY AREA BOUNDARY
ISSAQUAH
MAY CREEK
COAL CREEK
CEDAR RIVER
BOEING-RENTON
EAST MERCER
RENTON
SHOREWOOD
BOEING IND. WASTE
BRYN MAWR-LAKERIDGE
HINGTON
RENTON
KENT
AUBURN
ALGONA
TACOMA
MERCER ISLAND
TUKWILA
NORMANDY PARK
PUGET SOUND
MAURY ISLAND
VASHON ISLAND
COMMENCEMENT BAY
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
BLAKE ISLAND
RUSTON
GIG HARBOR
SEWAGE TREATMENT PLANT OR INDUSTRIAL WASTES OUTFALL
OVERFLOWS FROM COMBINED SEWERS
SCALE IN MILES

In discussing Lake Washington, the Pollution Control Commission has said, "For all the uses mentioned, clean and clear water, free from bacteriological contamination and objectionable algal growths, is of paramount importance."

Lake Union – Ship Canal. Waters of Lake Union and the Ship Canal are used primarily for shipping, boating and navigation. Fish migration from Puget Sound to Lake Washington also takes place through these waters. In addition, Lake Union serves as a catchment basin or sump for salt water which enters through the Government Locks and overflows the Salmon Bay sump.

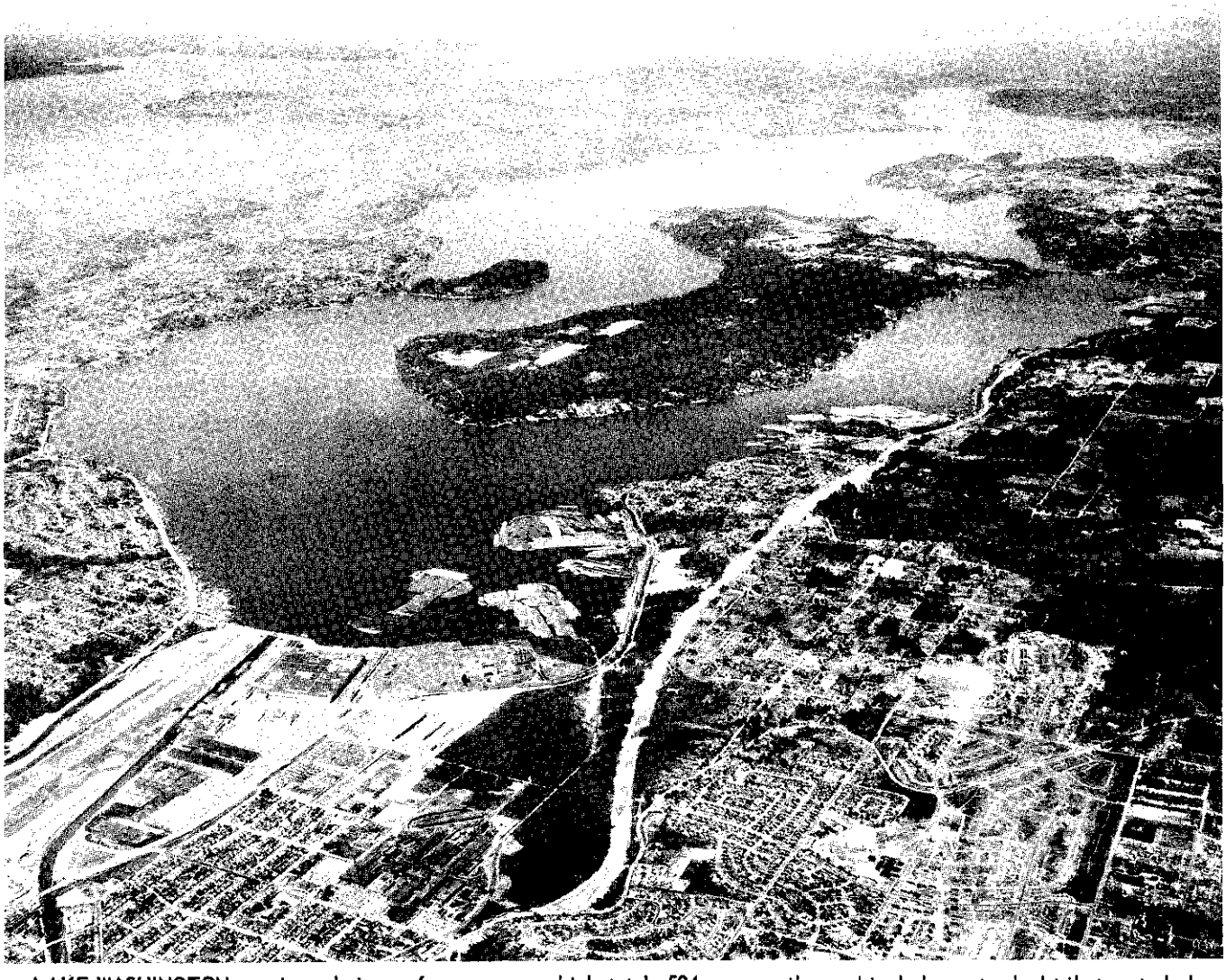
Water Quality Requirements

As related to sewage disposal, water quality requirements for waters in the Lake Washington drainage basin must take into account all the beneficial uses previously discussed. In general, these uses must be protected and the requirements for sewage disposal must be formulated in such a manner that the following effects are prevented:

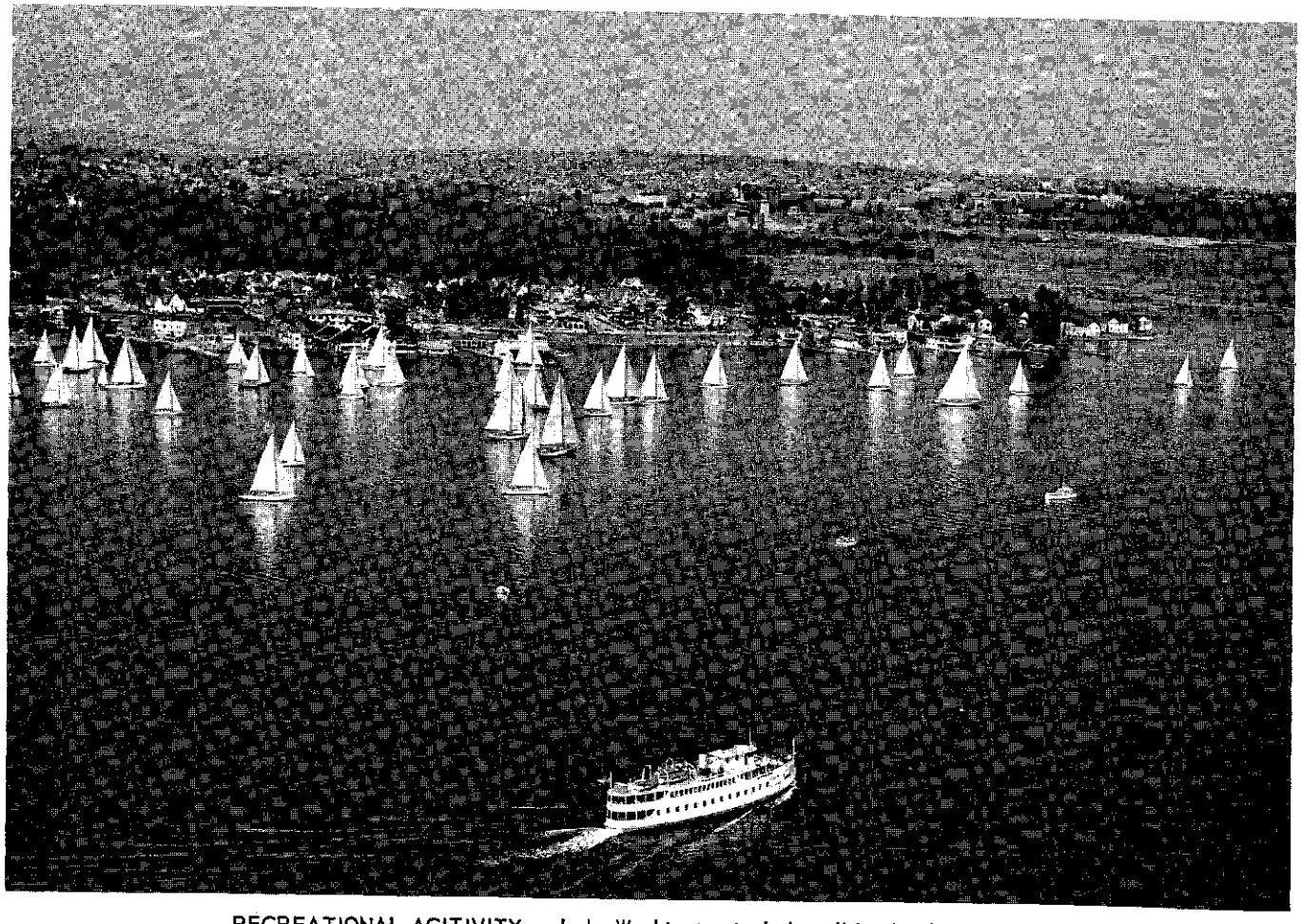
1. Contamination of either recreational or drinking waters with bacteriological organisms of human origin.
2. Physical impairment of the waters by sewage components, including solids, grease and oils.
3. Destruction or inhibition of desirable fish life.
4. Creation of nuisances as evidenced by algal blooms due to excessive mineral enrichment or fertilization of the waters.

In the case of waters of the ship canal system, including Lake Union, esthetic considerations are the controlling criteria.

Bacteriological Requirements. In its 1952-1953 investigation of Lake Washington, the Pollution Control



LAKE WASHINGTON receives drainage from an area which totals 584 square miles and includes watershed tributary to Lake Sammamish and the Sammamish and Cedar rivers. Communities bordering the lake include Seattle (upper left), Renton (foreground) and seven cities along the east shore (right).



RECREATIONAL ACITIVITY on Lake Washington includes all kinds of sport boating.

Commission employed the following bacteriological criteria:

1. Waters used for domestic purposes, untreated except for disinfection by chlorine or its equivalent--the average MPN of coliform organisms in a representative number of samples shall not exceed 50 per 100 ml or exceed this number in more than 20 per cent of the samples.

2. Waters used for domestic purposes, requiring complete treatment including coagulation, filtration and disinfection--the average MPN of coliform organisms in a representative number of samples shall not exceed 2,000 per 100 ml, or exceed this number in more than 20 per cent of the samples.

3. Waters used for bathing and swimming--the average MPN of coliform organisms in a representative number of samples shall not exceed 240 organisms per 100 ml or exceed this number in more than 20 per cent of the samples.

As stated earlier, it does not appear that continued use of untreated waters in the Lake Washington drainage basin for domestic water supply purposes is compatible with other uses. The requirement, therefore, that the bacteriological quality of the lake water be maintained at a level whereby it is suitable, in an untreated state, for domestic water supplies seems completely unrealistic. For example, coliform counts in waters used extensively for swimming and other forms of recreation frequently exceed those given for untreated drinking waters, even in the absence of sewage discharges.

The significance or meaning of bacteriological standards for the ship canal system is problematical. Obviously, these standards have no real meaning with respect to such water uses as navigation, shipping and boating. In terms of fish migration, the other principal water use for the canal system, the relationship or significance of coliform standards has yet to be established. No evidence or scientific findings of fact have been reported by fisheries biologists which would indicate that bacteriological contamination has in any way inhibited fish migration into the Lake Washington drainage basin.

In any body of water frequented by a large number of people, a certain amount of chance exposure might be anticipated. There is no rational way of interpreting the significance of inadvertent or accidental contact. In bodies of water such as the canal system

Lake Union, which are not amenable to or intended for bathing and swimming, regulatory authorities should take such action as may be necessary to keep the public informed that such use is prohibited. This situation appears to have been recognized both by the State Health Department and by the Pollution Control Commission in their investigations of the drainage basin.

Esthetic Requirements. The extent and nature of the esthetic impairment resulting from waste disposal by dilution is, in general, a function of the waste quantities involved, the relative diluting capacity of the receiving water, and the water use within the zone of influence. Waste disposal in the Lake Washington basin takes place mostly along shoreline areas and to a lesser degree in tributary streams. Consequently, the zone of immediate influence is confined to beach areas and nearshore regions. Esthetic factors of concern in these waters are:

1. Presence of undesirable tastes and odors in the water.
2. Formation of sludge banks and organic deposits in bathing beach areas.
3. Appearance of discoloration, turbidity, scum, oily sleek and floating solids.
4. Visible evidence of sewage pollution.

Protection of Fish Life. Since one of the primary water uses of the drainage basin is the propagation of game fish, certain additional water quality considerations become important. The Pollution Control Commission has promulgated water quality standards aimed at maintaining biological, chemical and physical conditions suitable for the spawning, growth and migration of desirable fish life. These standards are described in Chapter 9 and deal with the toxicity of chemical waste components, the pH (hydrogen-ion concentration), the formation of organic and inorganic deposits, and the development of physiologically intolerable conditions such as low dissolved oxygen content.

Prevention of Algal Growths. During recent years, a new aspect of sewage pollution in the Lake Washington drainage basin has come to light and has been manifested in the lake by a large increase in biological productivity. If allowed to continue, this condition is likely to become a serious threat to present uses and values of the lake.

Increased biological productivity is evidenced by excessive algal growths and is directly related to the concentration of inorganic or mineral nutrients, chiefly nitrogen and phosphorus, in the water. Sewage, whether treated or untreated, is rich in both nitrogen and phosphorus and is undoubtedly a contributing cause of excessive growths of aquatic plant life. The conversion of oligotrophic (low biological activity) lakes to eutrophic (high biological activity) lakes by the addition of nutrient-rich domestic sewage has been observed in several locations. It is apparent, therefore, that disposal of sewage in the Lake Washington drainage basin should be regulated to the extent that nuisance conditions resulting from excessive biological productivity are effectively controlled.

Pollution Control Commission Policy Relating to Sewage Discharges to Lake Washington

Early in 1956, the Pollution Control Commission, recognizing the fact that sewerage planning for the metropolitan area would be affected by the requirements for disposal of the sewage generated within the Lake Washington drainage basin, adopted the following statement of policy:

STATEMENT OF POLICY WITH RESPECT TO SEWAGE DISCHARGE INTO LAKE WASHINGTON

WHEREAS competent investigation has demonstrated that the waters of Lake Washington have reached a stage of degradation such that there is impending danger of profuse algal growths; and,

WHEREAS the conditions found in Lake Washington follow a pattern of deterioration experienced in comparable lakes elsewhere, and for which investigative records are available; and,

WHEREAS it has been determined that the major causative agent of deterioration has been sewage and treatment plant effluents discharged to these lakes by communities and individuals; and,

WHEREAS the expected population growth and industrial development will significantly increase the sources of waste in the drainage basin; and,

WHEREAS considerable numbers of people are now, and will be for some time in the future, dependent upon Lake Washington for domestic water supply; and,

WHEREAS the degradation of the waters of Lake Washington threatens the recreational uses of the Lake, particularly as to contamination of the public and private swimming beaches on its shores; and,

WHEREAS it is within the scope of responsibility of the Pollution Control Commission to foresee and determine such conditions as may be hazardous to the beneficial uses of the waters of the State, and to take such action as may be necessary and possible to attain correction of such conditions; and,

WHEREAS it is both possible and feasible to intercept sewage from the several communities in the drainage basin of Lake Washington and transport this sewage by various means and devices to the waters of Puget Sound; and,

WHEREAS it is the continuing policy of the Commission to require the treatment of sewage for discharge into Puget Sound; and,

WHEREAS it is both possible and feasible to discharge sewage thus collected and treated into the waters of Puget Sound in a manner consistent with good engineering practice;

IT SHALL, THEREFORE, be the policy of the Pollution Control Commission to adhere to the following principles in considering for approval plans for sewage treatment plants in Lake Washington drainage basin. In applying this policy, the drainage basin of Lake Sammamish is considered as and accepted to be a part of the drainage basin of Lake Washington.

1. All sewage shall be treated and all treatment plant effluents must eventually be diverted from Lake Washington and Lake Sammamish to some point or points on Puget Sound.
2. That all future expansion of existing sewage treatment plants must be designed on the basis of eventual diversion to Puget Sound.
3. That in the design of future sewer systems and sewage treatment plants where there may be two or more alternate points of discharge available, the one which most closely approaches the ultimate scheme of diversion to Puget Sound shall be the only acceptable one of the alternates.
4. That if it appears impractical or financially not feasible to select the solution in accordance with (3) above, consideration will be given to the next available alternate as a temporary solution only, and conformance to the ultimate scheme of diversion to Puget Sound will be required.
5. That all properties within reach of existing or proposed collection and treatment facilities designed in conformance with the principles set forth above, shall connect to such facilities.
6. Such facilities shall be planned to provide capacity for adjacent areas.

ADOPTED AND BECOMES EFFECTIVE this *eighth* day of *February, 1956.*

POLLUTION CONTROL COMMISSION

Signed: W. A. Galbraith, Chairman
John A. Biggs
Bernard Bucove, M.D.
Robert J. Schoettler
Sverre N. Omdahl

CAPACITY OF LAKE WASHINGTON AND TRIBUTARY WATERS TO RECEIVE SEWAGE

Disposal of untreated sewage and industrial wastes by dilution may have serious degrading effects on the quality of the receiving water. The extent and nature of these effects are obviously dependent upon both the characteristics of the waste and the dilution and self-purification capacity available at each point of discharge. Through adequate waste treatment it is possible to prevent direct physical and bacteriological impairment of water quality. On the other hand, it is not possible, by any economical sewage treatment process, to prevent chemical impairment through mineral enrichment.

Sewage treatment as practiced today consists essentially of mechanical and oxidizing processes whereby settleable and floating solids are removed and, if required, organic compounds are oxidized. Dissolved inorganic or mineral constituents are removed only incidentally during sedimentation and oxidation. Further, organic compounds are broken down during treatment to liberate inorganic nitrogen and phosphorus and thus make them more available as fertilizing agents. Consequently, even though treatment effects somewhat of a reduction in the total amounts of nitrogen and phosphorus, plant effluents actually contain more readily available nutritional material than raw sewage effluents.

Every receiving body of water has a unique natural capacity to assimilate a definite pollutional load. Maximum beneficial development of water resources for waste disposal is predicated upon a knowledge of the pollution-bearing capacity of the waters in question. In the present situation, the capacity of the drainage basin to receive sewage appears to be determined by the ability of Lake Washington to tolerate the inflow of fertilizing substances. In order to evaluate this capacity, it is necessary first to establish the nutrient balance for the drainage basin, and second to determine the biological response of the lake to various levels of nutrition.

Nutrient Balance

The nutrient balance of a lake may be defined as the net amount of nutrient materials, principally nitrogen and phosphorus, available for the purpose of promoting aquatic plant growth. To establish this balance, sources and amounts of nitrogen and phosphorus supplied to the lake must be determined, as must the amounts of each which are lost due to outflow. In addition, the quantities of nutrients stored on the lake bottom and subsequently released for reuse must be evaluated.

Previous Studies. Concern over the eutrophication of Lake Washington has resulted in many studies of lake conditions during recent years. In the first such study, which was conducted in 1933 by Scheffer and Robinson,[2] the nitrogen and phosphorus content of the lake was measured throughout the year. Additional studies have been made during subsequent years, particularly by Dr. W. T. Edmondson and Dr. J. Shapiro of the Zoology Department of the University of Washington. With the aid of a recent grant from the U.S. Public Health Service, Edmondson and Shapiro have expanded their investigations during the past year. Included in the work presently being performed are analyses of samples collected monthly at various depths from several sampling stations in the lake. Determination of nitrogen, phosphorus, dissolved oxygen, pH, chlorophyll and plankton quantities are being made on all samples thus collected.

Limnological studies of Lake Washington were made by the Pollution Control Commission from June 1952

[2] *Scheffer, V. B., and Robinson, R. J.,* A Limnological Study of Lake Washington, *Ecological Monographs, 9 (January 1939).*

to July 1953. Temperatures were determined and nitrate, phosphate and chlorophyll concentrations were measured in samples collected from the surface at 26 sampling stations. In 1955, the engineering department of the city of Seattle began an investigation of nutrient additions to the lake, primarily from the Seattle area.

Field Studies. To augment the data obtained from earlier studies and also to obtain data on both the total inflow and outflow of nutrients to the lake, sampling stations were set up during the survey at the mouth of the major tributary streams and at the outlet of the lake. This work was undertaken in cooperation with the engineering department of the city of Seattle.

Weekly samples were collected and analyzed for nitrogen and phosphorus content. In addition, samples, both composite and grab, were taken at the Lake City, Shorewood, Renton and Kirkland sewage treatment plants and were analyzed for nitrogen and phosphorus.

Nutrient Inflow. The principal sources of nitrogen and phosphorus entering Lake Washington are the major tributary streams and sewage discharges. Full evaluation of each, as well as of other minor sources, is required to determine the total nutritional load on the lake.

According to hydrologic survey data of the U.S. Geological Survey, more than 98 per cent of the natural surface runoff into the lake occurs from 9 principal tributary streams. These are Kenmore Creek, Sammamish River, McAleer Creek, Lyon Creek, Thornton Creek, Cedar River, Coal Creek, May Creek, and Mercer Slough (Fig. 10-1).

To determine the contribution of fertilizing substances from the 9 streams, analyses were made of samples collected at their outlets. Total monthly and average daily contributions of phosphorus and nitrogen during the first six months of 1957 are listed in Tables 10-1 and 10-2.

As shown in Table 10-1, the monthly contribution of orthophosphate, which is phosphorus in its most readily available form, varied from a minimum of 3,720 pounds in June to a maximum of 12,600 pounds in March. Similarly, the total dissolved phosphate content varied from a minimum of 7,290 pounds in

Table 10-1. Phosphorus Content of Streams Tributary to Lake Washington

Tributary	Drainage basin, sq mi	January 1957		February 1957		March 1957	
		Ortho-phosphate	Total dissolved phosphate	Ortho-phosphate	Total dissolved phosphate	Ortho-phosphate	Total dissolved phosphate
Cedar River	197	105	140	105	200	105	270
Coal Creek	6.7	0.4	0.6	0.9	2	3	8
Lyon Creek	3.6	0.6	1	1	2	4	9
May Creek	12.5	3	9	6	18	4	18
Mercer Slough	12.0	18	26	40	59	53	70
McAleer Creek	6.9	2	3	3	6	12	19
Sammamish River	209	127	220	254	425	197	507
Thornton Creek	12.1	5	8	10	14	29	116
Total daily		261	408	420	726	407	1,017
Total month		8,090	12,650	11,760	20,330	12,600	31,530

Tributary	April 1957		May 1957		June 1957	
	Ortho-phosphate	Total dissolved phosphate	Ortho-phosphate	Total dissolved phosphate	Ortho-phosphate	Total dissolved phosphate
Cedar River	50	160	60	260	50	120
Coal Creek	1	1	0.4	1	0.5	0.5
Lyon Creek	2	3	1	2	0.7	1
May Creek	2	5	2	4	0.6	2
Mercer Slough	24	32	5	7	5	6
McAleer Creek	6	8	2	6	2	2
Sammamish River	115	260	60	175	60	105
Thornton Creek	14	25	6	8	5	7
Total daily	214	494	136	463	124	243
Total month	6,420	14,820	4,220	14,350	3,720	7,290

Except for total monthly values, phosphorus values are average rates expressed as pounds of phosphorus (P) per day.
Values greater than 1 rounded off to nearest unit.

Table 10-2. Nitrogen Content of Streams Tributary to Lake Washington

Tributary	Drainage basin, sq mi	January 1957		February 1957		March 1957	
		Nitrate nitrogen	Total dissolved nitrogen	Nitrate nitrogen	Total dissolved nitrogen	Nitrate nitrogen	Total dissolved nitrogen
Cedar River	197	134	7,854	313	10,913	318	9,948
Coal Creek	6.7	2	202	4	394	5	405
Lyon Creek	3.6	3	36	8	158	6	126
May Creek	12.5	18	344	36	676	33	703
Mercer Slough	12.0	6	471	14	909	25	800
McAleer Creek	6.9	2	62	55	375	6	226
Sammamish River	209	261	4,331	865	8,355	540	10,110
Thornton Creek	12.1	11	264	22	567	225	685
Total daily		437	13,564	1,317	22,347	1,158	23,003
Total month		13,550	420,480	36,880	625,720	35,900	713,090

Tributary	April 1957		May 1957		June 1957	
	Nitrate nitrogen	Total dissolved nitrogen	Nitrate nitrogen	Total dissolved nitrogen	Nitrate nitrogen	Total dissolved nitrogen
Cedar River	110	19,555	48	17,328	15	11,885
Coal Creek	2	142	0.2	90	0.1	36
Lyon Creek	2	97	0.5	16	0.2	–
May Creek	11	381	3	173	1	136
Mercer Slough	8	423	3	173	3	148
McAleer Creek	3	193	1	61	0.5	20
Sammamish River	190	7,560	72	6,192	36	3,606
Thornton Creek	9	369	4	149	2	102
Total daily	335	28,720	132	24,182	58	15,933
Total month	10,050	861,600	4,090	749,640	1,740	477,990

Except for total monthly values, nitrogen values are average rates expressed as pounds of nitrogen (N) per day.

Values greater than 1 rounded off to nearest unit.

June to a maximum of 31,530 pounds in March. During the period of measurement, the orthophosphate concentration varied from 29 to 64 per cent of the total phosphate. Of the total orthophosphate in the tributary streams, a minimum of 44 per cent was contained in the waters of Sammamish River and of 23 per cent in the Cedar River.

Table 10-2, which gives the nitrogen contributions of streams tributary to Lake Washington, shows that monthly contributions of nitrate nitrogen, which is nitrogen in its most readily available form, varied from a minimum of 1,740 pounds in June to a maximum of 36,880 pounds in February. Similarly, total dissolved nitrogen ranged from a minimum of 420,480 pounds in January to a maximum of 861,600 pounds in April. At the same time, nitrate nitrogen varied from 1.2 to 5.9 per cent of the total nitrogen. As with phosphorus, the Cedar and Sammamish rivers were the heaviest contributors of nitrate nitrogen, with minimum values of 24 and 47 per cent of the total respectively.

Surface runoff or stream flow follows a definite annual pattern. Stream flow is measured continuously by the U.S. Geological Survey on a number of streams tributary to Lake Washington, including Cedar River, Sammamish River, May Creek and Mercer Slough. In addition, periodic measurements are made on other tributary streams during dry weather flow periods. Extreme flow hydrographs have been developed by the Geological Survey for the Cedar River at Renton and the Sammamish River at Bothell (Fig. 10-2). As therein indicated, maximum flows in both rivers occur during the winter months, December to March, while minimum flows occur during the summer months, July to September. Comparison of this figure with Tables 10-1 and 10-2, indicates that, as might be expected, the nutrient contribution to Lake Washington of the natural streams varies directly with the stream flow.

Based on available U.S. Geological Survey hydrological data for 1957, on extreme flow hydrographs for the entire basin, and on the analysis results for phosphorus and nitrogen, probable maximum, minimum and average annual nitrogen and phosphorus additions to Lake Washington from natural sources have been computed. These values, together with

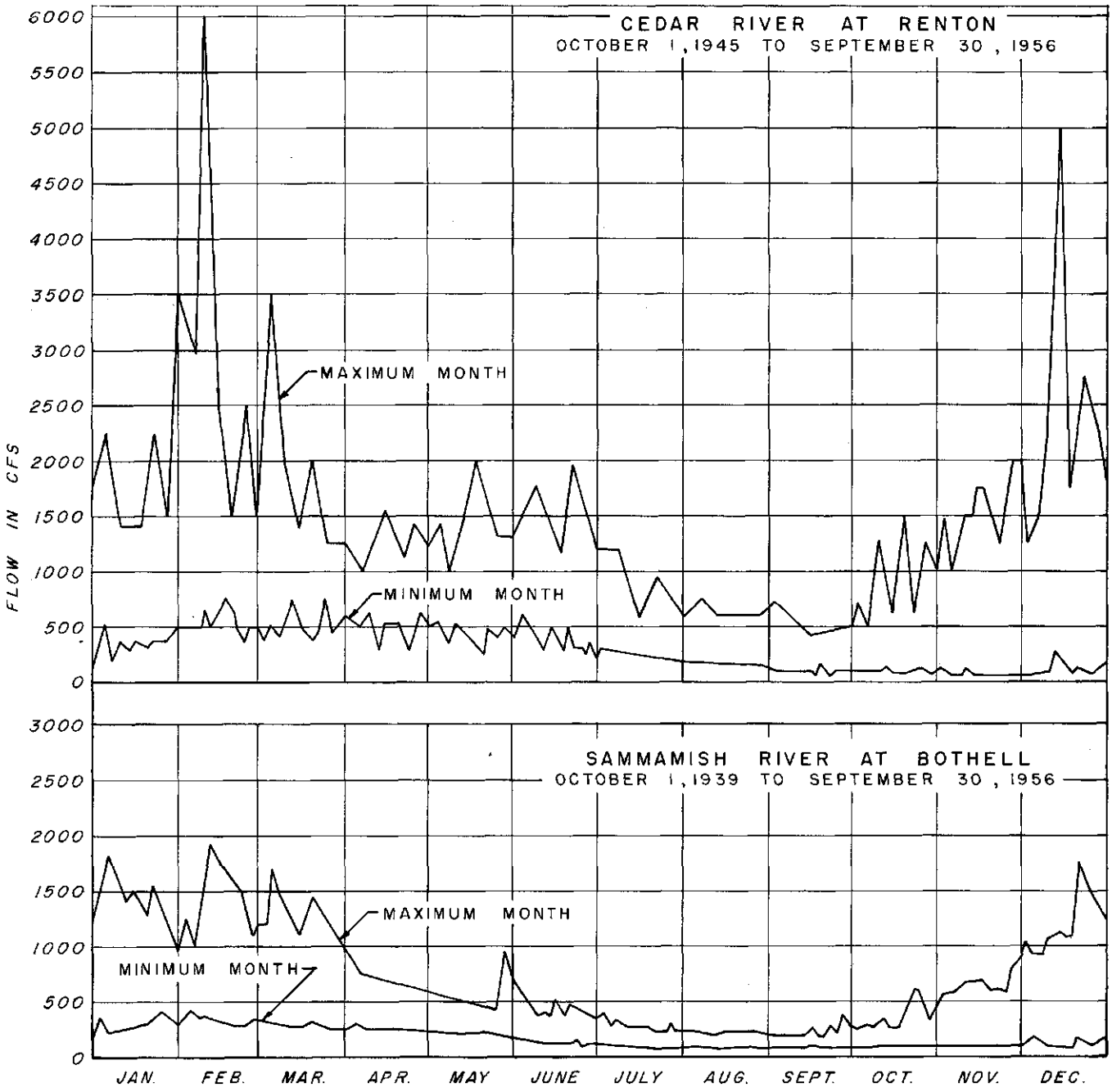


Fig. 10-2. Maximum and Minimum Monthly Flows, Cedar and Sammamish Rivers

Source: U. S. Geological Survey, "Report on Surface Water Investigations in Lake Washington Basin and Adjacent Basins", Sept. 1956.

the calculated 1957 contributions, are given in Table 10-3. As therein indicated, the annual contribution of nitrate nitrogen may be expected to range from a minimum of 63,000 pounds to a maximum of 296,000 pounds. Similarly, the annual contributions of orthophosphate are expected to range from 31,000 to 145,000 pounds.

Sewage discharges to Lake Washington emanate from many sources. Ten sewage treatment plants currently discharge directly to the lake and septic tank seepage enters the lake from areas without public sewerage facilities. As shown in Table 10-4, the estimated population contributory to the treatment plants is 64,300. Of the 89,000 persons residing in unsewered areas in the Lake Washington drainage basin, approximately 15 per cent live in areas with soils classified as unsuitable for septic tanks. Based on a 90 per cent outflow of septic tank effluents from these areas, the population contributory to the lake from unsewered areas is equivalent to 12,000 persons. In other words, the total equivalent population presently discharging sewage to the lake amounts to 76,300.

In addition to the discharges given above, sewage also reaches Lake Washington from a number of storm

Table 10-3. Estimated Annual Contributions of Nitrogen and Phosphorus to Lake Washington from Natural Sources

	Nitrogen 1,000 lbs N per year		Phosphorus 1,000 lbs P per year	
	Nitrate	Total dissolved	Ortho-phosphate	Total dissolved
10-year maximum	296	12,300	145	317
10-year average	186	7,720	91	199
10-year minimum	63	2,610	31	68
1957	141	5,850	69	151

Based on calendar year, January 1 - December 31.

water overflows on combined sewers in the city of Seattle and from untreated industrial wastes which are discharged by the Boeing aircraft plant at Renton. All combined sewers in Seattle in the Lake Washington drainage basin are intercepted by lake front interceptors having a capacity of about two and one-half times the average dry weather flow. Based on this capacity, the quantity of domestic sewage which will enter the lake during periods of storm water overflow amounts to approximately 3.5 per cent of the total annual sewage generated in the area (Fig. 13-4). For a tributary area of 9,200 acres and a population of 130,000, the overflows are equivalent to an average sewage flow rate of 0.48 mgd.

To determine the nutrient contribution from the Boeing plant, a composite sample of the industrial waste discharge was obtained during a 12-day period in April, 1957. Analysis results for this sample indicate that the nitrogen content amounts to 10 ppm as total dissolved nitrogen, while the phosphorus content is 7.3 ppm as total dissolved phosphate. Nutrient contributions were calculated on the basis of these analyses and water consumption data.

Table 10-4. Estimated Population Contributory to Lake Washington

Source	Population
Sewage treatment plants	
Lake City	26,000
Sand Point NAS	900[a]
Bryn Mawr	4,500
Renton	14,800
Boeing-Renton	6,000[a]
Bellevue	4,100
Kirkland	4,000
Shorewood Apartments	2,800
East Mercer	500
Sand Point Homes	700
Total, treatment plants	64,300
Unsewered areas[b]	12,000
Total	76,300

[a]*Equivalent residential population.*

[b]*Based on 90 per cent outflow of septic tank drainage from population residing in areas having soils classified as unsuitable for septic tanks. Total population in unsewered areas is 89,000, of which 15 per cent live in areas with unsuitable soils.*

Total monthly and average daily contributions of nitrogen and phosphorus from all sewage and industrial wastes sources for the first six months of 1957 are presented in Table 10-5. As indicated in this table, the rate of contribution of dissolved phosphates varied from a minimum of 8,340 pounds in February to a maximum of 11,550 pounds in June. Nitrogen contributions varied from a minimum of 30,600 pounds in April to a maximum of 37,200 pounds in May. Effluents from sewage treatment plants accounted for a minimum of 73 per cent of the total dissolved nitrogen.

Sewage nutrients appear to follow seasonal fluctuations or trends, with maximum values obtaining during the summer months and minimum values during the winter months. Comparison of Table 10-5 with Tables 10-1 and 10-2 indicates that this fluctuation is the reverse of the seasonal fluctuation in the contribution from natural sources. Thus, during the winter months when runoff in the tributary streams is at a maximum, natural sources account for the major amount of nutrient materials discharged to Lake Washington. Conversely, sewage sources gain in relative importance during the summer months.

Past contributions of nutrients to Lake Washington from sewage sources are difficult to estimate. The amount of sewage, both treated and untreated, entering the lake has varied considerably over the years. Prior to the construction in 1941 of intercepting sewers along the lake front in the city of Seattle, raw sewage from an estimated 40,000 to 50,000 people was discharged to the lake from the city alone. Upon completion of the interceptor, continuous discharge of raw sewage was eliminated and the only discharge at present is that which occurs periodically from the storm water overflows. As indicated previously, this amounts to about 3.5 per cent of the total sewage generated in the area.

Exclusive of septic tank drainage, it is estimated that the sewage from 10,000 people was discharged directly to the lake in 1941 and that this number increased to 64,300 in 1957 (Table 10-4). Additionally, the inorganic phosphorus content of ordinary domestic sewage has increased during the past 10 years. Most of this increase has been attributed to the use of phosphorus-rich synthetic detergents in place of soap.[3] No change appears to have occurred in the nitrogen content.

[3]*Sawyer, C. N.,* Some New Aspects of Phosphates in Relation to Lake Fertilization, *Sewage and Industrial Wastes, 24, 768 (1952).*

Conventional secondary sewage treatment removes some of the nitrogen and phosphorus but, as stated earlier, converts them into a more readily available form. Table 10-6, which gives the results of analyses made on raw sewage and plant effluent samples collected at the Kirkland and Renton sewage treatment plants, indicates that secondary treatment removes about 15 to 25 per cent of the total dissolved phosphorus and about 33 per cent of the total dissolved nitrogen. It is interesting to note that the amount of orthophosphate and nitrate was not reduced by treatment. In fact, there was an increase in both.

To determine the approximate per capita contribution of nitrogen and phosphorus, weekly composite samples of raw sewage were collected at the Kirkland, Lake City, Shorewood Apartments, and Renton sewage treatment plants during the period March to June, 1957. As given in Table 10-7, the total dissolved phosphorus and total dissolved nitrogen content of the raw sewage at these plants is equivalent to 2.9 and 6.1 pounds per capita per year, respec-

Table 10-5. Nitrogen and Phosphorus Content of Sewage and Industrial Waste Discharges to Lake Washington

Source	January 1957			February 1957		
	Total dissolved phosphate[a]	Nitrogen[a]		Total dissolved phosphate[a]	Nitrogen[a]	
		Nitrate	Total dissolved		Nitrate	Total dissolved
Sewage treatment plants[b]						
Lake City	61	4	285	73	12	418
Sand Point NAS	3	7	11	3	7	10
Bryn Mawr	10	0.5	62	10	3	67
Renton	41	3	130	39	4	146
Boeing-Renton	3	3	22	4	2	22
Bellevue	8(est)	0.5	26	9	0.5	30
Kirkland	17(est)	2	202	18	2	232
Shorewood Apartments	6	0.5	70	6	0.5	82
East Mercer	1	0.1	12	1	0.1	14
Sand Point Homes	2	0.1	17	2	0.1	20
Total, treatment plants	152	21	837	165	31	1,041
Unsewered areas[c]	47	–	97	47	–	97
Combined sewage overflows[d]	39	–	112	39	–	112
Boeing-Renton industrial waste[e]	47	–	64	47	–	64
Total daily	285	–	1,110	298	–	1,314
Total month	8,840	–	34,410	8,340	–	36,790

Source	March 1957			April 1957		
	Total dissolved phosphate[a]	Nitrogen[a]		Total dissolved phosphate[a]	Nitrogen[a]	
		Nitrate	Total dissolved		Nitrate	Total dissolved
Sewage treatment plants[b]						
Lake City	67	14	349	71	6	326
Sand Point NAS	3	8	11	3	8	12
Bryn Mawr	12	1	73	18	0.2	87
Renton	43	6	180	16	12	186
Boeing-Renton	4	2	26	6	2	36
Bellevue	6	0.3	19	5	0.9	13
Kirkland	15	2	58	19	7	58
Shorewood Apartments	5	0.6	21	6	3	21
East Mercer	1	0.1	3	1	0.4	3
Sand Point Homes	1	0.1	5	2	0.6	5
Total, treatment plants	157	34	745	147	40	747
Unsewered areas[c]	47	–	97	47	–	97
Combined sewage overflows[d]	39	–	112	39	–	112
Boeing-Renton industrial waste[e]	47	–	64	47	–	64
Total daily	290	–	1,018	280	–	1,020
Total month	8,990	–	31,560	8,400	–	30,600

Continued on next page

Table 10-5. Continued

Source	May 1957			June 1957		
	Total dissolved phosphate[a]	Nitrogen[a]		Total dissolved phosphate[a]	Nitrogen[a]	
		Nitrate	Total dissolved		Nitrate	Total dissolved
Sewage treatment plants[b]						
Lake City	96	2	426	80	0.9	346
Sand Point NAS	3	8	14	5	10	20
Bryn Mawr	10	–	51	7	–	33
Renton	64	9	239	64	11	241
Boeing-Renton	6	5	40	5	3	29
Bellevue	14	0.5	54	15	–	38
Kirkland	20	7	65	42	24	107
Shorewood Apartments	7	3	27	24	8	37
East Mercer	1	0.4	4	4	1	6
Sand Point Homes	2	0.6	7	6	2	9
Total, treatment plants	223	36	927	252	60	866
Unsewered areas[c]	47	–	97	47	–	97
Combined sewage overflows[d]	39	–	112	39	–	112
Boeing-Renton industrial waste[e]	47	–	64	47	–	64
Total daily	356	–	1,200	385	–	1,139
Total month	11,040	–	37,200	11,550	–	34,170

[a]*Except for total monthly values, phosphorus and nitrogen values are average rates expressed as pounds of element per day.*

[b]*Based on data obtained from analyses of grab samples as reported by the city engineer's office, city of Seattle.*

[c]*Based on contributory population of 12,000 (13.5% of 89,000) and annual per capita contributions of 1.4 pounds of phosphorus and 3.0 pounds of nitrogen.*

[d]*Based on contributory population of 4,500 (3.5% of 130,000) and annual per capita contributions of 3.0 pounds of phosphorus and 8.5 pounds of nitrogen.*

[e]*Based on data obtained from analysis of composite sample collected April, 1957.*

Table 10-6. Removal of Nitrogen and Phosphorus by Secondary Treatment

	Treatment plant	
	Kirkland	Renton
Raw sewage[a]		
Nitrogen, pounds per day		
Nitrate	0	0
Total dissolved	101	–
Phosphorus, pounds per day		
Ortho	21	57
Total dissolved	28	81
Plant effluent[b]		
Nitrogen, pounds per day		
Nitrate	7	9
Total dissolved	68	239
Phosphorus, pounds per day		
Ortho	20	63
Total dissolved	23	66
Per cent removed by treatment		
Total dissolved nitrogen	33	–
Total dissolved phosphorus	18	22

[a]*Based on analyses of composite samples collected during March - June, 1957.*

[b]*Based on monthly average values during March - June, 1957, as reported by city of Seattle engineer's office.*

tively. Based on reductions by treatment of 20 per cent in total phosphorus and of 33 per cent in total nitrogen, the approximate concentrations in effluents from secondary treatment plants in the metropolitan Seattle area may be expected to be equivalent to 2.3 pounds of phosphorus and 4.0 pounds of nitrogen per capita per year. Sawyer[4] reported dissolved phosphorus concentrations in effluents from secondary treatment plants equivalent to 1.2 pounds per capita per year and dissolved nitrogen concentrations equivalent to 6.0 pounds per capita per year. As a matter of comparison, raw Seattle sewage contains 3.0 pounds of dissolved phosphorus and 8.5 pounds of dissolved nitrogen per capita per year.

Because of the probable error in estimating the total contributory population, as well as the uncertainties in evaluating other factors such as per capita contributions of nutrients and the significance of septic tank leaching, it is difficult to determine the exact rate at which nutrients from sewage sources were entering the lake prior to the present survey.

[4]*Sawyer, C. N.,* Fertilization of Lakes by Agricultural and Urban Drainage, *Journal of the New England Water Works Association, LXI, 109, 1947.*

Table 10-7. Per Capita Nitrogen and Phosphorus Content of Raw Sewage

Treatment plant	Contributory population	Total dissolved phosphorus			Total dissolved nitrogen		
		Average, pounds per day	Total, pounds per year	Pounds per capita per year	Average, pounds per day	Total, pounds per year	Pounds per capita per year
Kirkland	4,000	28	10,200	2.55	75	27,400	6.85
Lake City	26,000	249	90,900	3.49	–	–	–
Shorewood Apartments	2,800	24	8,800	3.14	–	–	–
Renton	14,800	81	29,600	2.00	239	87,200	5.89
Total	47,600	–	139,500	2.93	–	114,600	6.10

Based on analyses of composite samples collected during March - June, 1957.

Based on the best available information, however, the probable rate of mineral enrichment of Lake Washington by sewage was calculated for several periods since 1916 (Table 10-8). Values thus obtained indicate that the amount of nitrogen entering the lake has more than doubled and that the amount of phosphorus has increased by almost 300 per cent since the 1916–1930 period.

Nutrient materials are received also from a number of minor sources, including storm water runoff from urban drainage and leaching of such areas as the Union Bay refuse disposal site. As stated earlier, however, the U. S. Geological Survey estimates that more than 98 per cent of the total surface runoff to Lake Washington occurs in 9 principal tributary streams. The remainder comes from creeks and ravines and from urban storm water drainage systems. Obviously, therefore, the amount of nutrients that could be brought into the lake from the minor runoff sources is so small that it can be neglected.

With regard to leaching from areas such as refuse disposal sites, a field investigation conducted in April 1957 by Shapiro indicated that the influence of the Union Bay refuse site was confined to the immediate area. It thus appears that the contribution of nutrients from such sources would be negligible.

Nutrient Outflow. Not all the nutritional elements reaching Lake Washington are used or stored in the lake. Some of these substances pass through the lake and flow out through the ship canal system and the Government Locks to Shilshole Bay. Runoff from the drainage basin, together with flow regulation at the locks, will obviously influence the flushing characteristics of the lake.

Water is required to operate the Government Locks and fish ladders, and to flush out salt water which enters through the locks. This water is supplied by the streams flowing into Lake Washington. For 8 to 10 months of the year, runoff from the basin is normally in excess of lockage requirements, but during low flow summer months additional water is needed. To meet this demand, approximately 43,000 acre-feet of water are held in storage during periods of high runoff by regulating the level of Lake Washington between elevations of 7.0 and

Table 10-8. Estimated Annual Contributions of Nitrogen and Phosphorus to Lake Washington from Sewage Sources

Period	Estimated average contributory population			Nitrogen,[a] 1,000 lb N per year	Phosphorus,[b] 1,000 lb P per year
	Treated sewage	Raw sewage	Equivalent septic tank		
1916 - 1930	–	25,000	3,200	181[c]	42[c]
1931 - 1940	–	33,000	4,300	240[c]	54[c]
1941 - 1950	12,000	17,000	5,900	226[d]	74[d]
1957	64,000	4,500	12,000	409[e]	114[e]

[a]*Total dissolved nitrogen.*

[b]*Total dissolved phosphorus.*

[c]*Based on yearly per capita contributions of nitrogen of 6.5 pounds for raw sewage and 6 pounds for treated sewage, including septic tanks, and of phosphorus of 1.5 pounds for raw sewage and 1.2 pounds for treated sewage, including septic tanks.*

[d]*Based on yearly per capita contributions of nitrogen of 7.5 pounds for raw sewage and 5.5 pounds for treated sewage, including septic tanks, and of phosphorus of 2.25 pounds for raw sewage and 2.0 pounds for treated sewage, including septic tanks.*

[e]*Two times 6-month total from Table 10-5.*

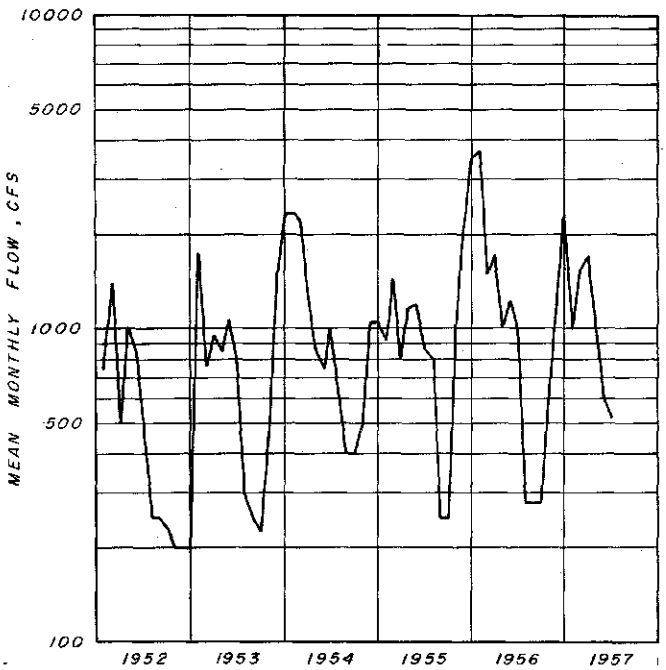


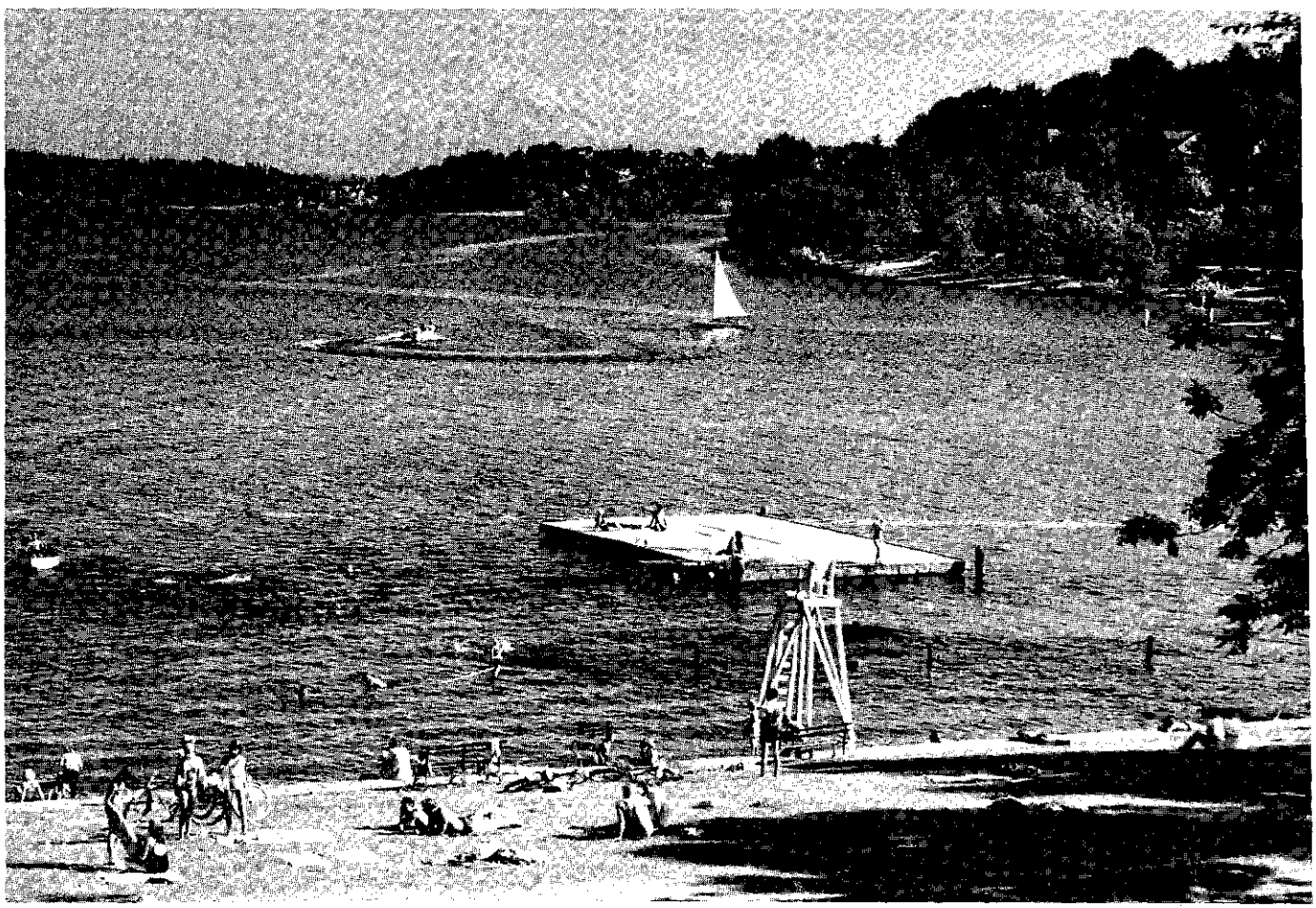
Fig. 10-3. Average Monthly Flows, Lake Washington Ship Canal at Government Locks

Based on data obtained from the U. S. Army Corps of Engineers.

8.8 feet, Seattle datum. Thus, the rate of outflow from the lake is controlled by lockage operations during two to four months of each year and by run-off from tributary streams during the balance of the year.

Average monthly flows at the Government Locks, as computed from data obtained from the U.S. Corps of Engineers for the period January, 1952 to July, 1957 are shown in Fig. 10-3. It does not appear that lockage operations have modified the total outflow from the lake to any appreciable extent. Consideration should be given, however, to the significance of future changes in rates of outflow due to changing operation of the locks.

As indicated by the minimum month flows (Fig. 10-3), lockage water consumption has increased steadily over the years. Water requirements have reached the point where insufficient water is available during dry weather years to prevent salt water from intruding into Lake Washington. Rattray and Seckel of the Oceanography Department of the University of Washington have reported on an excessive salt water inflow that occurred during the summer of 1952[5]. It was concluded that the amount



MOUNT BAKER BEACH on Lake Washington is one of six such facilities maintained by the city of Seattle.

Table 10-9. Average Outflow of Nitrogen and Phosphorus from Lake Washington

Month, 1957	Phosphorus			Nitrogen		
	Ortho, pounds per day	Total dissolved, pounds per day	Total dissolved, 1,000 pounds per month	Nitrate, pounds per day	Total dissolved, pounds per day	Total dissolved, 1,000 pounds per month
January	10.5	144	4.46	125	6,275	195
February	96	350	9.80	240	14,440	404
March	11.4	410	12.70	300	12,500	388
April	47	315	9.45	147	15,447	463
May	7.5	271	8.40	28	11,828	367
June	10.5	81	2.43	12	5,712	171

Phosphorus and nitrogen values are average rates expressed in pounds of element.

of salt water which entered at that time was very near the critical quantity required to permanently stratify the lake. It can be expected that similar conditions will occur during any extremely dry period.

Concern over future demands for lockage water may result in a modification of water use and flow regulation at the locks and dam. At present, it does not appear that such flow regulation will significantly affect the total outflow from Lake Washington. On the other hand, failure to control salt water intrusion could have serious consequences. Continued salt water intrusion will lead eventually to stratification of the lake waters, which when once established will become a permanent condition. In such an event, water below approximately 50 feet in depth will become permanently depleted of oxygen, resulting in the development of obnoxious odors and the elimination of desirable fish life. It is apparent, therefore, that these problems must be recognized in any plans for future development of the water resources of the Lake Washington basin which would reduce the amount of water available for operation of the locks during summer months.

Analysis results for samples collected at the Montlake Bridge, together with the mean monthly flow measurements at the Government Locks, were used to calculate the amount of nitrogen and phosphorus leaving the lake each month. Values so obtained (Table 10-9) show that the outflow of total dissolved phosphorus ranged from a minimum of 2,430 pounds per month in June to a maximum of 12,700 pounds in March. Similar values for total dissolved nitrogen amounted to 171,000 pounds in June and 463,000 pounds in April.

Table 10-10 gives the relationship between total inflow and outflow of nutrients during the first six months of 1957. It will be seen that the outflow of phosphorus varied between 12.7 and 40.5 per cent and averaged 29.9 per cent of the total inflow for the 6-month period. Nitrogen outflow ranged between 33.4 and 60.9 per cent of the inflow and averaged 49.0 per cent. It is interesting to note that the maximum outflow of nutrients, both total and expressed as a percentage of the total inflow, occurred during the months of maximum nutrient inflow when the contribution from natural sources was greatest. The minimum outflow, both total and as a percentage of the total inflow, occurred when the contribution of nutrients from sewage sources was relatively the greatest. This would seem to indicate that nutrients discharged to the lake by natural sources are flushed out to a greater extent than are those contributed by sewage discharges. Although it is difficult to estimate the inflow - outflow relationship for the remainder of the year, it is probable that the per cent of nutrient outflow during this period would approach the minimum values since the monthly flow at the Government Locks is generally at a minimum from July to October (Fig. 10-3).

Nutrient Storage and Reuse. The ability of a lake to store and to reuse nutrient materials is related, in part, to the mixing and flushing actions which normally take place throughout the year. In deep lakes, such as Lake Washington, stratification usually takes place during part of the year due to variation in water density brought about by temperature changes in the water mass. This phenomenon, referred to as thermal stratification, results in a surface layer of warm light water (the epilimnion) floating on top of cool heavier bottom water (the hypolimnion), with the two layers being separated by a narrow band or zone of water known as the thermocline. In temperate climates, the epilimnic water begins to cool during the fall and winter months until its density is equal to or greater than the water in the hypolimnion. When such condi-

[5] *Department of Oceanography, University of Washington,* Studies on Lake Washington Ship Canal, *Technical Report No. 15 (1953).*

Table 10-10. Relationship between Inflow and Outflow of Nutrients in Lake Washington

Month, 1957	Total dissolved phosphorus, 1,000 lb P per month					Total dissolved nitrogen, 1,000 lb N per month				
	Inflow			Outflow			Inflow		Outflow	
	Natural[a]	Sewage[b]	Total	Total[c]	Per cent of inflow	Natural[d]	Sewage[b]	Total	Total[c]	Per cent of inflow
January	12.7	8.8	21.5	4.5	20.9	420	34	454	195	43.0
February	20.3	8.3	28.6	9.8	34.3	626	37	663	404	60.9
March	31.5	9.0	40.5	12.7	31.4	713	32	745	388	52.1
April	14.8	8.4	23.2	9.4	40.5	862	31	893	463	51.8
May	14.4	11.0	25.4	8.4	33.1	750	37	787	367	46.6
June	7.3	11.6	18.9	2.4	12.7	478	34	512	171	33.4
Total, 6 months	101.0	57.1	158.1	47.2	29.9	3,849	205	4,054	1,988	49.0

[a]*From Table 10-1.*
[b]*From Table 10-5.*
[c]*From Table 10-9.*
[d]*From Table 10-2.*

tions prevail, an overturn takes place and the lake is easily mixed from top to bottom by wind action. Winter stratification may take place under freezing conditions. Warming of the surface waters in the spring may result in another condition of instability and subsequent mixing.

Lake Washington is usually homothermal, that is, of equal temperature throughout its depth, from some time in December through March. Water temperatures during this period may range from 4.5° to 8.0° C. Temperatures begin to rise in March, soon after which stratification begins and the lake is usually completely stratified by June. Fall mixing usually starts near the end of September and the lake is generally subject to thorough vertical mixing during the period of October to March. Because this period of vertical mixing coincides with the time of maximum surface water runoff and minimum biological activity, the lake is literally purged of a great deal of hypolimnic nutrient material before it can be utilized.

Rattray and Seckel,[5] based on their investigation of the Ship Canal, concluded that approximately 25 per cent of the salt in Lake Washington is flushed out annually by vertical mixing during the winter months. Thus, approximately 75 per cent of the salt contained in the lake at the beginning of the year should be left 12 months later. If there has been any inflow of salt during the period in question, the total amount present may actually increase. It appears reasonable, therefore, to assume that something similar must hold true for the dissolved nitrogen and phosphorus present in the lake.

Table 10-11. Mean Nitrogen and Phosphorus Content of Lake Washington

Year	Nitrogen, 1,000 lbs N		Phosphorus, 1,000 lbs P	
	Inorganic	Total dissolved	Ortho	Total dissolved
1933	1,270	1,970	72.8	114.4
1957	1,724	4,180	19.7	184

Content of nutrients is for period of complete mixing in the lake, January - March.

Some of the inorganic nutrients converted to plankton forms ultimately reach the lake bottom. Organic matter so deposited is subject to bacterial attack, which in turn results in the release of these nutrients for reuse. Although deposits of earlier years will continue to release fertilizing substances, much of the bottom material is sealed off by more recent deposits.

Because of incomplete data prior to 1957, yearly trends in the quantity and effects of stored nutrients in Lake Washington cannot be determined. It is possible, however, to determine the over-all change which has taken place in the 24-year period since 1933. This can be done by comparing data obtained during the 1933 studies by Scheffer and Robinson[2] with data from current studies being conducted by Edmundson and Shapiro (Table 10-11). As shown in the table, the total dissolved nitrogen content of the lake water has more than doubled since 1933, while the total dissolved phosphorus content has increased by over 1.5 times.

Based on present data (Table 10-12), the nutrients available from stored sources approximated 47 per cent and 37 per cent respectively of the total annual inflow of dissolved nitrogen and phosphorus to Lake Washington during 1957. Similar values for 1933 were 4 and 13 per cent, indicating that the amounts of nutrients available from stored sources increased by three or more times during the past 24 years. If this trend should continue, it is obvious that the quantity which may be tolerated from other sources will decrease.

Table 10-12. Estimated Nutrient Contributions to Lake Washington from Stored Sources

	1933		1957	
	Nitrogen	Phosphorus	Nitrogen	Phosphorus
Total inflow during period,[a] 1,000 pounds	3,580[b]	114[b]	2,720	122
Total outflow during period,[c] 1,000 pounds	1,970	34	1,500	37
Net inflow, 1,000 pounds	1,610	80	1,220	85
Total content of lake,[d] 1,000 pounds	1,970	114	4,180	184
Net from stored sources, 1,000 pounds	360	34	2,960	99
Total annual inflow to lake, 1,000 pounds	7,960[b]	253[b]	6,259[e]	265[e]
Per cent of total annual inflow from stored sources	4.5	13.4	47.3	37.3

[a]*Assuming that 45 per cent of yearly total enters lake during period of complete mixing, November - March.*

[b]*Based on 10-year average for natural sources from Table 10-3 and on 1931 - 1940 period for sewage sources from Table 10-8.*

[c]*Assuming that outflow equals 55 per cent of total nitrogen inflow and 30 per cent of total phosphorus inflow during period of complete mixing, November - March.*

[d]*From Table 10-11.*

[e]*From Tables 10-3 and 10-8.*

Future Nutritional Loads. Through the years, sewage has served as a steadily increasing source of nutrients. As urban development in the Lake Washington basin continues, increasing quantities of sewage will naturally become available for fertilization of the lake. On the other hand, assuming no major changes in water use, the amount contributed by natural sources will remain almost constant.

Based on information developed in Chapter 5, present and future populations in the Lake Washington drainage basin, exclusive of the portion in Seattle served by lake front interceptors, are estimated as follows:

1954	153,000
1980	437,000
2000	737,000
2030	1,048,000

As indicated by the above figures, the number of persons residing in the basin is expected to almost triple by 1980 and to be over six times the present total by 2030. These increases will result, of course, in similar increases in sewage contributions of nutritional matter.

Assuming that the entire future population will be served by separate sanitary sewers and that all sewage will be subject to complete biological treatment before discharge to the waters of the basin, the total nutritional load which may be expected in the future has been calculated and is given in Table 10-13. As indicated in this table, which also gives the nutritional contributions from natural sources, the contribution of phosphorus from sewage sources is expected to increase from the present level of 43 per cent of the total phosphorus inflow to the lake to 83.5 per cent by 1980 and to 92 per cent ultimately. Similarly, sewage contributions of nitrogen are expected to increase from the present level of 6.5 per cent to an ultimate level of 35 per cent of the total inflow.

Table 10-13. Estimated Future Annual Nutritional Contributions to Lake Washington

Year	Total dissolved phosphorus, 1,000 lb P per year					Total dissolved nitrogen, 1,000 lb N per year				
	Total	Natural sources		Sewage sources		Total	Natural sources		Sewage sources	
		Total	Per cent of total	Total	Per cent of total		Total	Per cent of total	Total	Per cent of total
1957	265	151[a]	57.0	114[b]	43.0	6,259	5,850[a]	93.5	409[b]	6.5
1980	1,204	199[c]	16.5	1,005[c]	83.5	9,468	7,720[c]	81.5	1,748[e]	18.5
2000	1,894	199[c]	10.5	1,695[c]	89.5	10,668	7,720[c]	72.4	2,948[e]	27.6
2030	2,609	199[c]	7.6	2,410[c]	92.4	11,912	7,720[c]	64.8	4,192[e]	35.2

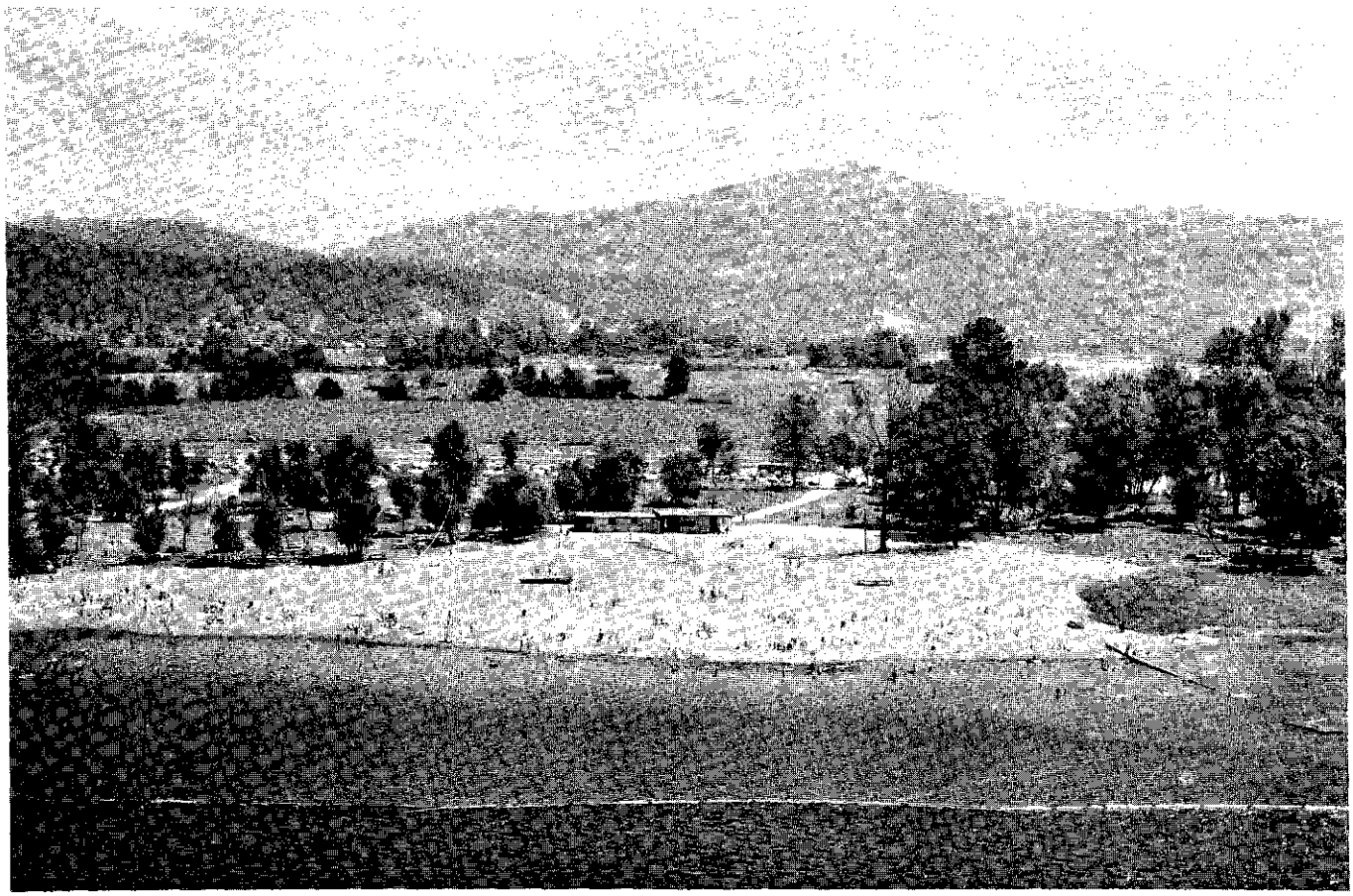
[a]*From Table 10-3.*

[b]*From Table 10-8.*

[c]*10-year average from Table 10-3.*

[d]*Based on 2.3 pounds per capita per year.*

[e]*Based on 4.0 pounds per capita per year.*



SAMMAMISH STATE PARK at the south end of Lake Sammamish is one of many public beaches currently in use in the Lake Washington drainage basin. Recreational use of lakes in the basin has been increasing in recent years.

Biological Response

Response of a lake to fertilization may be manifested in several ways. Microscopic plants and animals (plankton) utilize the available dissolved nutrients to produce cell tissue. Increasing fertilization is obviously accompanied by an increased production of plankton cell tissue. In addition, as the degree of fertilization increases, a change frequently occurs in the type of plankton being produced, especially algae. Waters low in nutrient concentrations usually contain plankton of the diatom species, while those high in nutrient concentrations usually contain large quantities of blue-green algae. The former rarely create nuisance conditions, while the latter frequently are accompanied by such conditions as excessively turbid waters and masses of odorous decaying scum.

During periods of high biological activity, available nutrients are rapidly utilized in the production of plankton cell tissue. As the plankton cells die, they settle to the lower waters, taking with them both living cells and other organic matter. Eventually, much of this material is deposited on the lake bottom. Since all cells contain phosphorus and nitrogen, much of the available nutrient material is removed from the surface waters. Some will be released again in usable form by the respiration of bottom life, chiefly bacteria, but much of the material will remain on the bottom and be sealed off by subsequent deposits. During active plankton blooms, the surface waters may become devoid of some critical nutritional substance, usually inorganic phosphate.

The respiration of bottom life living on deposited organic matter results in an uptake of oxygen from the surrounding water. When a lake becomes thermally stratified during part of the year, as Lake Washington does, a noticeable decrease occurs in the dissolved oxygen concentration in the hypolimnic (lower) water. In general, a decrease in dissolved oxygen concentration in the hypolimnium is attributable to increased biological activity.

Because of the many facets of biological response which may be observed, it is not surprising to find limnologists describing lake fertility in terms of several phenomena. It has become common practice to evaluate productivity in terms of the weight or volume and the identity of the plankton present. Generally, this information is obtained by direct microscopic examination. In addition, a number of chemical measures are in common use, chief among them being the oxygen deficit in the hypolimnion and the

Table 10-14. Plankton Quantities and Types in Lake Washington

Date	Total plankton	Blue-green species			
		Oscillatoria rubescens	Oscillatoria agardhi	Phormidium sp.	Aphanizomenon flos-aguae
1950					
13 May	2,140	0	84	2	0
24 June	794	0	16	8	0
21 July	211	0	30	4	0
4 August	219	0	5	2	0
21 August	3,069	0	3	0	0
1 September	567	0	9	13	0
15 September	762	0	1	105	0
Mean	935	–	–	–	–
1955					
1 July	2,895	2,783	0	1	1
14 July	1,407	893	0	8	0
18 August	1,755	397	0	610	493
22 September	1,314	255	0	125	727
Mean	1,725	–	–	–	–

Source: Edmundson, W. T., Anderson, G. C., and Peterson, D. R., "Artificial Eutrophication of Lake Washington," Limnology and Oceanography, 1, 47, (1956).

Calculated on basis of cell number and cell volume in epilimnion only.
Values expressed as u ^{3}/ml x 10^{-3}.

concentration of nutritional elements, principally nitrogen and phosphorus, in the epilimnion. Since the bulk of the plankton are photosynthetic organisms, the chlorophyll concentration has been used increasingly as a measure of biological activity.

Lake Washington's Behavior. Many studies have been conducted in recent years to determine the response of Lake Washington to the increasing supply of nutrients. Edmondson conducted surveys during the summers of 1950 and 1955, during which the quantity and types of plankton were determined. Results of these surveys (Table 10-14) indicate that the biological activity of the lake has increased approximately two-fold during the five-year period. Perhaps of greater interest is the large increase in blue-green algae which has occurred. At present, the algae population is comprised largely of troublesome blue-green forms as opposed to less troublesome diatom species of earlier years. Investigations conducted by Scheffer and Robinson[2] in 1933 showed that the major components of the summer plankton in the lake included a number of diatoms with relatively small quantities of blue-green algae. In 1950, the blue-green forms, particularly *Oscillatoria agardhi*, were present in greater numbers, while in 1955 the blue-greens *Oscillatoria rubescens* comprised the major portion of the plankton population of the lake. Qualitative examinations of lake samples taken in 1957 indicate that species of the blue-green algae, *Oscillatoria*, were the predominant organism.[6] Other species of blue-green algae, *Anabena* and *Aphanizomenon*, were frequently observed in large numbers. Fig. 10-4 shows a photomicrograph of a sample of Lake Washington water collected during August, 1957. At that time, the plankton population was comprised largely of the blue-green algae, *Oscillatoria*, *Anabena*, and *Aphanizomenon*. Also shown in the figure is an *Oscillatoria* by itself.

Hypolimnic oxygen deficit values based on data obtained by Scheffer and Robinson during their 1933 study, and by Edmondson in subsequent investigations, are shown in Fig. 10-5. These deficits represent the amount of oxygen utilized in biological decomposition of organic matter in the hypolimnion over a two-month summer period divided by area of the hypolimnion and time elapse in days. Such data have been used by limnologists in comparing different lakes as well as evaluating changing conditions in the same lake. As shown in Fig. 10-5, the oxygen deficit gradually increased during the 1933-1950 period from 1.5 milligrams per square centimeter per month (mg/cm^2/month) to 1.9 mg/cm^2/month. Since 1950, the deficit values have increased sharply to about 3.2 mg/cm^2/month in 1957. Comparison of these deficit values with commonly reported lower limits of 1.0 to 1.5 mg/cm^2/month for eutrophic lakes[7] indicates that Lake Washington is rapidly approaching a state of complete eutrophication.

The relationship between algal growths, nutrient supply and environmental conditions in Lake Washington (Fig. 10-6) was studied by the State Pollution Con-

[6] *Private communication from Dr. W. T. Edmondson.*

[7] An Investigation of Pollutional Effects in Lake Washington, *Technical Bulletin No. 18, Washington State Pollution Control Commission (1955).*

trol Commission during their limnological survey in 1952-1953. At the beginning of the survey in June 1952, the plankton population, as evidenced by the chlorophyll concentration, was at a minimum. With increasing water temperatures, environmental conditions became favorable for growth and a bloom commenced in August and persisted until early December. During the fall and winter period of mixing, at which time minimum water temperatures obtained, biological activity showed a sharp decrease. During this same period, however, supplies of phosphates and nitrates were brought from the hypolimnion to the epilimnion and the concentrations of these nutrients rose sharply to a maximum in March. With increasing temperatures in March, thermal stratification began and biological activity again commenced, reaching a peak in May. Rapid growths of algae during this period brought about a rapid decline in the nutrient concentrations until only traces could be found in the surface waters by the first of June. By the end of June, the bloom was essentially over and the midsummer minimum chlorophyll concentration remained fairly constant until the end of July. The spring bloom was greater in magnitude than the fall bloom but appar-

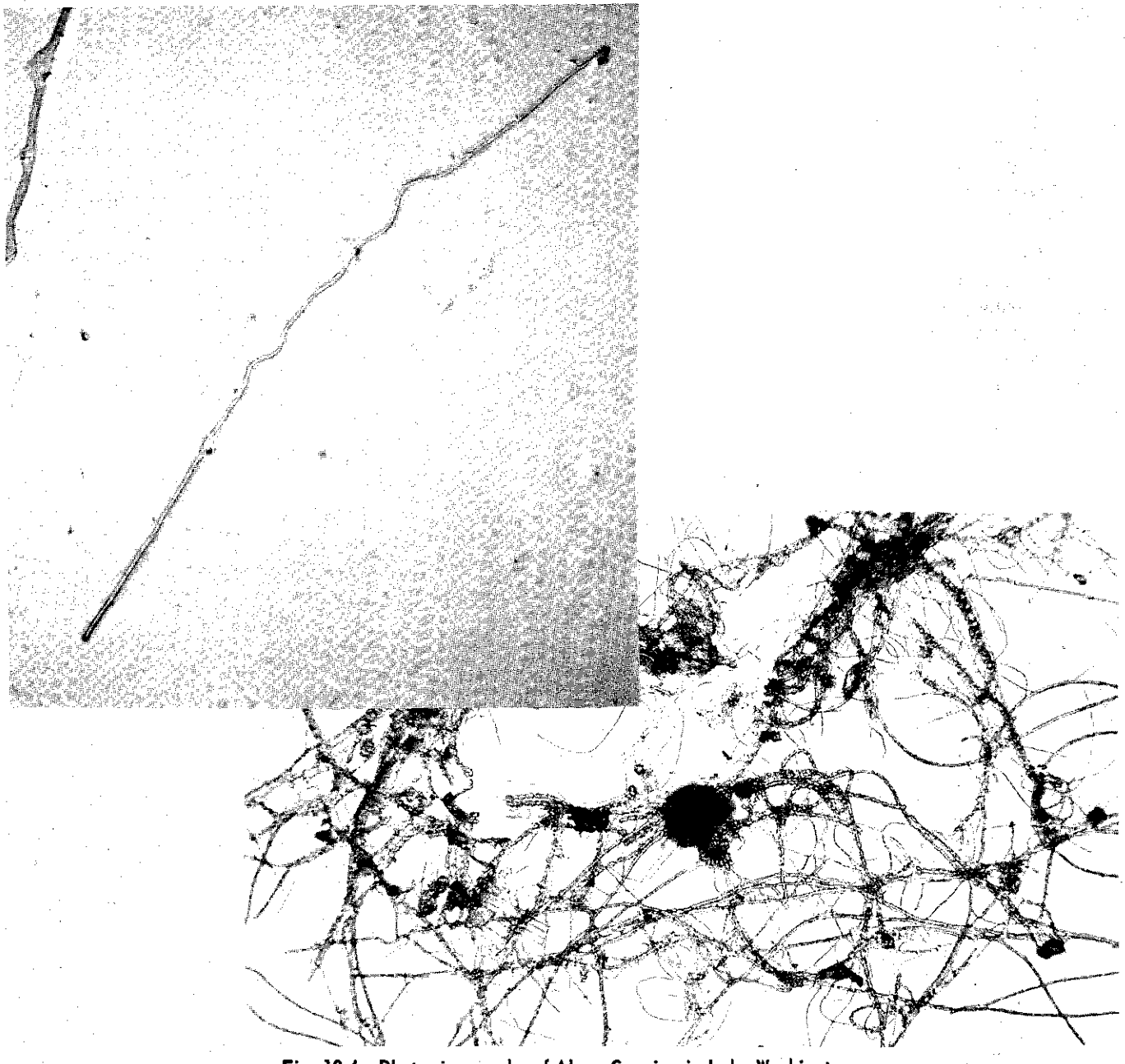


Fig. 10-4. Photomicrographs of Algae Growing in Lake Washington

Top photograph shows typical algal growths occurring in Lake Washington during the summer of 1957. Bottom photograph shows a blue-green alga, *Oscillatoria*, which is one of the principal bloom-producing forms. Organisms shown are 216 times natural size.

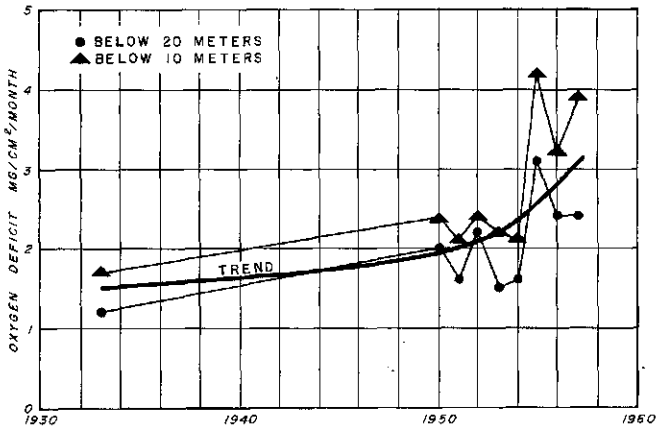


Fig. 10-5. Trend in Hypolimnic Oxygen Deficit in Lake Washington

The hypolimnic oxygen deficits shown represent the amount of oxygen utilized in the biological decomposition of organic matter in the hypolimnion (lower water) over a two month summer period divided by the area of the hypolimnion and the time lapse in days. Based on data obtained by Scheffer and Robinson and by W. T. Edmundson.

ently of shorter duration. It is interesting to note that, as the nutrients in the surface waters were exhausted during the spring bloom, the rate of biological activity appeared to decrease. Depletion of available nutrients through utilization by the plankton undoubtedly acted as a limiting growth factor during these studies.

Comparative Studies of Other Lakes. The biological response of Lake Washington to fertilization may be better understood when compared to the biological behavior observed in other lakes subject to sewage enrichment. Sawyer has reported upon the findings of an extensive investigation of sewage eutrophication of the lakes around Madison, Wisconsin.[4,8] In an effort to evaluate the biological significance of various nutritional substances and levels, observations were made of a number of lakes in addition to those in the Madison area. It was found that nitrogen and phosphorus were the critical elements which could act as limiting factors in the plankton productivity of lakes. Of these, phosphorus was held to be the key element in determining biological activity. Table 10-15, which is reproduced from one of Sawyer's reports,[4] shows the relationship which existed between biological production and inorganic nitrogen and phosphorus levels in 17 lakes in Wisconsin. Based on the data obtained, Sawyer concluded that nuisance conditions could be expected when the concentration of inorganic phosphorus equals or exceeds 0.01 ppm. Similarly, Sawyer indicated that the critical level for inorganic nitrogen is 0.30 ppm. Evidence was obtained also which indicated that extensive algae growths could develop with plentiful supplies of phosphorus and deficient

[8] *Lackey, J. B., and Sawyer, C. N.,* Plankton Productivity of Certain Southeastern Wisconsin Lakes as Related to Fertilization, *Sewage Works Journal, 17, 573 (1945).*

ANNUAL PARADE OF SMALL BOATS features opening of Seattle's yachting season.

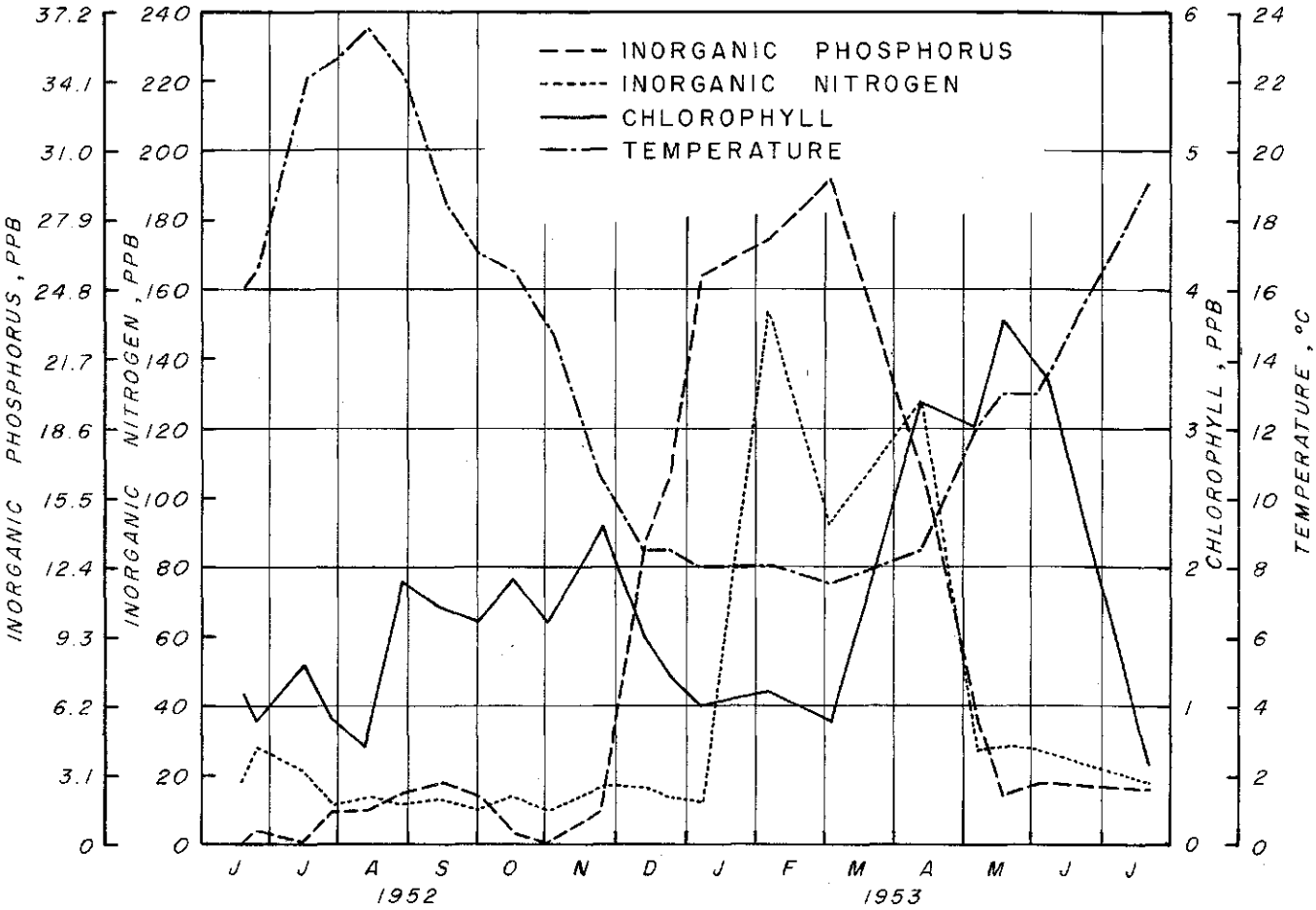
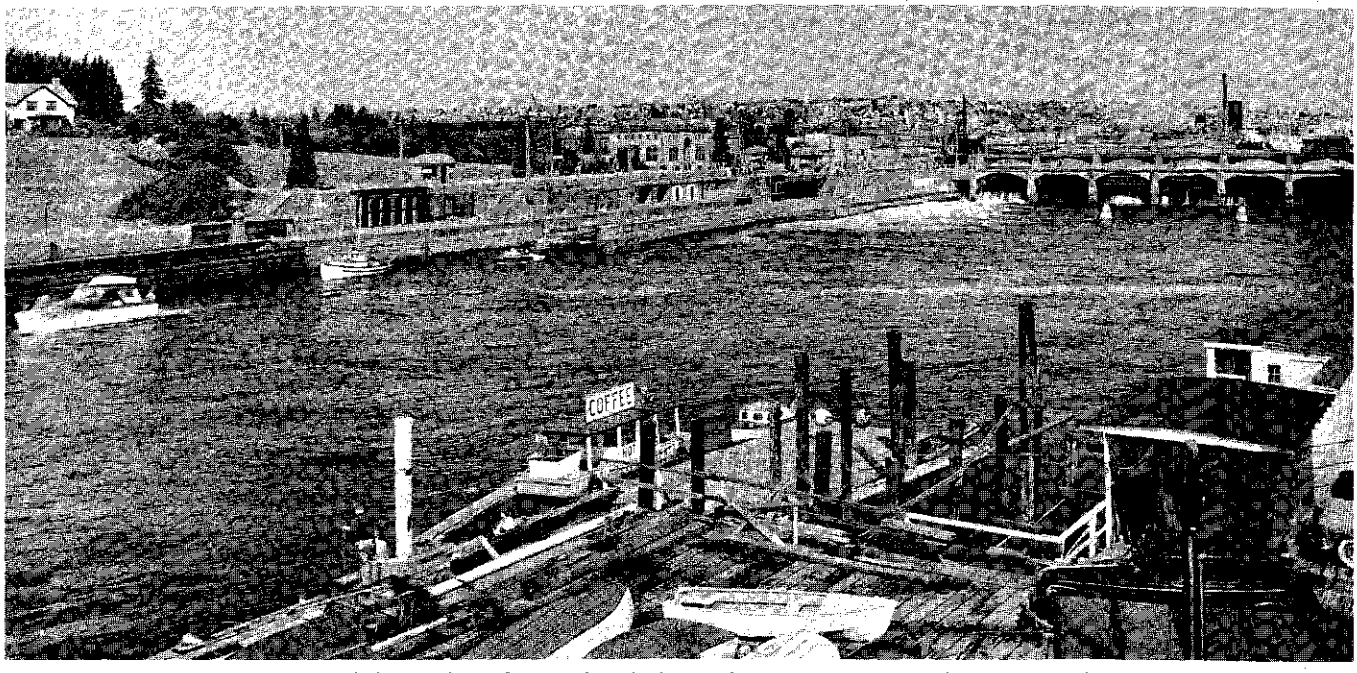


Fig. 10-6. Relationship between Algal Growths, Nutrient Supply and Environmental Conditions in Lake Washington

Source: "An Investigation of Pollution Effects in Lake Washington", Technical Bulletin No. 18, 1955, Washington State Pollution Control Commission.



GOVERNMENT LOCKS control the outflow of water from Lake Washington. For two to four months of each year during periods of minimum runoff, the use for lockage exceeds the input from tributary sources.

Table 10-15. Comparison of Effluent Waters from 17 Southeastern Wisconsin Lakes

Lake	Inorganic nitrogen, ppm			Inorganic phosphorus, ppm		
	Minimum	Maximum	Average	Minimum	Maximum	Average
Mendota	0.05	0.31	0.17	a	0.025	0.018
Monona[b]	0.07	0.50	0.33	a	0.07	0.041
Waubesa[b]	0.07	1.24	0.79	0.27	0.54	0.38
Kegonsa[b]	0.08	0.93	0.35	0.26	0.49	0.33
Wingra	0.07	0.53	0.26	a	0.02	0.012
Koshkonong	0.12	1.32	0.39	a	0.06	0.019
Delevan[b]	0.06	0.87	0.31	a	0.07	0.023
Geneva	0.05	0.15	0.10	a	a	a
Como	0.09	0.22	0.14	a	0.01	a
Lauderdale (Mill)	0.06	0.18	0.13	a	a	a
Pewaukee[b]	0.06	0.22	0.15	a	–	a
Nagawicka[b]	0.08	0.88	0.37	a	0.035	0.016
Upper Nemahbin[b]	0.05	0.65	0.24	a	0.02	0.013
Okauchee	0.05	0.19	0.13	a	0.01	a
Oconomowoc	0.04	0.15	0.12	a	0.01	a
Lac La Belle	0.07	0.30	0.19	a	0.01	a
Rock	0.07	0.15	0.10	a	0.01	a

Source: Sawyer, C. L., "Fertilization of Lakes by Agricultural and Urban Drainage," Journal New England Water Works Association, LXI, 109 (1947).

Results based on at least 10 samplings over 12-month period.

[a]*Less than 0.01 ppm.*

[b]*Produce nuisance blooms regularly.*

supplies of nitrogen. Bacterial or algal fixation of atmospheric nitrogen was held to be capable of satisfying nitrogen deficiencies. In the absence of adequate supplies of phosphorus, nitrogen fixation was found to be unimportant.

Phosphorus Limits for Lake Washington

The capacity of Lake Washington to receive sewage is dependent entirely upon its capacity to receive nutritional substances without producing nuisance conditions due to excessive biological activity. In turn, the biological activity of the lake is dependent upon the inorganic phosphorus supply.

During the Wisconsin studies, Sawyer found that there was a general similarity between optimum land fertility resulting from agricultural fertilizing practices and aquatic productivity resulting from mineral enrichment. Normal applications of phosphorus to farm lands seldom exceed 3 to 4 pounds per acre per year.[8] According to data obtained during this survey, the amount of readily available phosphorus entering Lake Washington is approximately 50 per cent of the total dissolved phosphorus. On that basis, a dissolved phosphorus supply rate of 6 to 8 pounds per acre per year would correspond to good farming practice aimed at optimum crop production. While the similarities between land and aquatic productivity are only general, nutrient threshold levels for biological response should be of a comparable order of magnitude.

As previously discussed, investigations conducted by Sawyer have indicated that nuisance conditions due to excessive biological activity occur in a lake when the inorganic phosphorus concentration equals or exceeds 0.01 ppm. As indicated in Table 10-15, however, one lake which produced nuisance blooms regularly (Pewaukee) contained less than an average of 0.01 ppm of inorganic phosphorus, while this concentration was exceeded in a number of other lakes which did not produce nuisance blooms. It is obvious, therefore, that lakes will react differently to fertilization by phosphorus, and that use of the 0.01 ppm limit without regard to other considerations may lead to erroneous conclusions. In the final analysis, specific information concerning the biological behavior of each lake is perhaps the surest basis for arriving at a reasonable estimate of its ability to tolerate the supply of fertilizing elements.

Lake Washington has been subject to a steadily increasing load of nutrients over recent years. Until about 1950, the major supply of nutrients came from natural sources, but since that time sewage sources have become increasingly important. As has been shown by various studies conducted in the past, Lake Washington was definitely oligotrophic in 1933, whereas all evidence indicates that by 1957 the lake was on the borderline of being eutrophic. Based on the total inflow of nutrients to the lake from both natural and sewage sources and on a total volume in the lake of

102 x 10^9 cubic feet, the calculated concentrations of inorganic phosphorus, even during periods when the lake was unquestionably oligotrophic, exceeded Sawyer's limit of 0.01 ppm (Table 10-16).

A number of factors are involved in evaluating a lake's ability to withstand fertilization with phosphorus. These factors, which would tend to preclude a direct comparison with other lakes, include the total mineral content of the water, the extent of thermal stratification, and the rate of water replacement. In general, waters containing high concentrations of dissolved minerals, such as calcium and magnesium, will exhibit a higher basic biological productivity for a given phosphorus concentration than will waters with low concentrations of dissolved minerals. Since the waters of Lake Washington have relatively lower total dissolved mineral concentrations than do the waters of the Wisconsin lakes studied, it seems probable that Lake Washington can tolerate greater concentrations of phosphorus than the lakes in Wisconsin.

In lakes in which thermal stratification occurs twice a year, that is, during the summer months when the surface waters are warmed and during the winter months when the surface waters are frozen, two periods of complete mixing of the waters also occur. During these periods of complete mixing, large quantities of nutrients are brought from the hypolimnion to the epilimnion where they become available for utilization by plankton. In waters such as Lake Washington, where thermal stratification and subsequent complete mixing of the waters occur only once a year, the amount of nutrients released to surface waters from the hypolimnion is probably decreased.

Perhaps the factor of greatest importance in evaluating Lake Washington is the rate of water replacement. Not only is the total rate important, but the time of the year when the maximum replacement occurs is also of significance. This factor was recognized by Sawyer who stated in one of his reports:[4] ". . . . the concentration of nutrient elements which exists in biologically treated sewage (inorganic nitrogen 15-25 ppm, inorganic phosphorus 2-4 ppm) is 10-100 times greater than that of normal agricultural drainage. When such wastes enter a lake, they cause outflow of an equal volume (less evaporation) of relatively good water. Such a change can only result in an appreciable increase of the concentration of nutrient elements in the receiving water. On the other hand, the composition of normal agricultural drainage is more comparable to lake waters in the concentration of nutrient elements carried. Consequently, when it enters a lake, it displaces water with more nearly an equal nutrient content and the net gain to the lake is small. This factor is of tremendous importance in Lake Waubesa, where the sewage flow approximates 15 per cent of the total annual inflow to the lake and often exceeds 50 per cent of the flow during the summer months."

Although the outflow from Lake Washington is controlled to some degree by water requirements for operating the Government Locks, conditions in the lake are similar to those reported for Lake Waubesa. As previously indicated, the contribution of phosphorus from sewage sources to Lake Washington in 1957 amounted to 43 per cent of the total annual inflow to the lake and exceeded 60 per cent of the total during June of 1957. Further, the maximum rate of outflow from the lake occurs during the months when the inflow from natural sources is the greatest. These factors would seem to indicate that a large part of the nutrient elements brought in by natural sources is flushed out rather rapidly from the lake, while a major part of those brought in by sewage is retained therein and is utilized in the production of plankton.

From the foregoing discussion, it appears that Lake Washington can assimilate a total inflow of nutrients in excess of the amount required to establish a concentration equaling Sawyer's limit of 0.01 ppm for inorganic phosphorus. While the exact limit is difficult to determine, it appears, on the basis of the behavior of the lake as well as the concentrations resulting from total inflow in the past years, that the limit would approach 0.02 ppm.

Based on a lake area of 21,600 acres and a volume of 102 x 10^9 cubic feet, and neglecting any removal by outflow or enrichment from stored sources, a dissolved inorganic phosphorus supply of 5.8 pounds per acre per year would establish a concentration of 0.02 ppm in Lake Washington. Assuming that 50 per cent of the total dissolved phosphorus is present in the biologically usable inorganic form, the inflow thereof which could be tolerated by the lake would amount to 11.6 pounds per acre per year.

Table 10-16. Calculated Concentrations of Phosphorus in Lake Washington

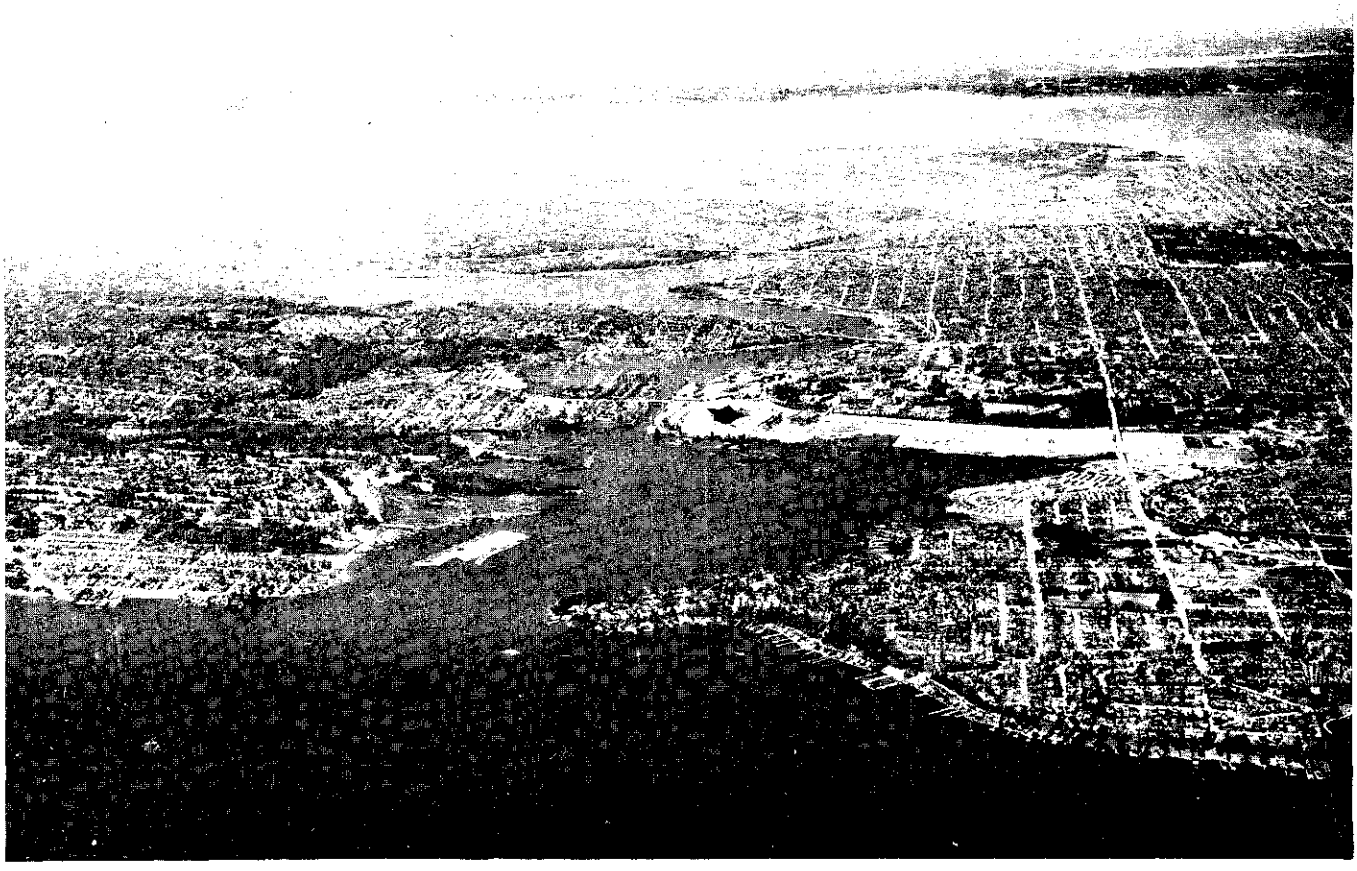
Period	Phosphorus concentration, ppm	
	Total	Inorganic[a]
Pre-1916[b]	0.022	0.011
1916 - 1930	0.039	0.019
1931 - 1940	0.041	0.021
1941 - 1950	0.044	0.022
1957	0.042	0.021

Calculated on basis of total inflow of phosphorus from Tables 10-3 and 10-8 and neglecting any removal by outflow or any enrichment by stored sources.

Based on lake volume of 102 x 10^9 cubic feet.

[a]*Assumed 50 per cent of total phosphorus.*

[b]*Before Cedar River was diverted to Lake Washington. Assumed contribution of Cedar River equals 30 per cent of total from natural sources.*



OUTLET FROM LAKE WASHINGTON is through a series of ship canals and Lake Union to Shilshole Bay (background). The ship canals and Lake Union are used primarily for navigation and boat moorage.

According to the findings of this survey, approximately 30 per cent of the dissolved phosphorus reaching Lake Washington escapes through the Ship Canal. Using this value and neglecting enrichment from bottom deposits, the dissolved phosphorus input which could be tolerated would thus amount to 16.6 pounds per acre per year. If the nutrient contribution from stored sources amounts to 40 per cent of the total annual phosphorus supply, which is about the value observed during 1956-57, the threshold value would be reduced to 10 pounds per acre per year. Thus, depending on the significance of phosphorus liberated from bottom deposits, the capacity of Lake Washington to receive phosphorus from outside sources without producing excessive algal blooms appears to range between 10 and 17 pounds per acre per year. As a matter of comparison, the total phosphorus input to three of the lakes in Wisconsin which were found to exhibit nuisance blooms by Sawyer (Monona, Waubesa and Kegonsa) varied from 19 to 89 pounds per acre per year.

REQUIREMENTS FOR DISPOSAL OF SEWAGE IN THE LAKE WASHINGTON DRAINAGE BASIN

Future effects of sewage disposal in the Lake Washington drainage basin can be evaluated only by an interpretation of past and present effects as related to the beneficial uses of the waters in the basin. In the preceding discussion, it has been shown that the primary consideration with regard to continued sewage disposal in the basin is the prevention of nuisance conditions resulting from excessive biological activity in the waters. Excessive biological activity is in turn dependent upon the supply of phosphorus which is discharged to the waters from both natural and sewage sources. In addition, prevention of bacterial contamination of beach areas by such practices as combined sewer overflows is a factor which must be considered. In effect, therefore, requirements for sewage disposal in the Lake Washington drainage basin fall into two principal categories. These comprise (1) the prevention of nuisance conditions resulting from mineral enrichment of the waters, and (2) the prevention of bacteriological contamination of recreational areas.

Prevention of Nuisance Conditions

The practice of sewage disposal by dilution in Lake Washington has resulted in a continuing deterioration of the lake water with respect to its bacteriological, physical and chemical characteristics. Conventional

sewage treatment processes can produce effluents of a quality which will meet all bacteriological and physical requirements, but no economically feasible process is presently available for removing fertilizing substances from sewage to an extent sufficient to control lake enrichment.

As set forth in the preceding section, Lake Washington appears to be capable of tolerating a maximum phosphorus input of between 10 and 17 pounds per acre per year. Since the diversion of Cedar River to Lake Washington in 1916, the quantity of phosphorus supplied to the lake by natural sources amounts to about 9 pounds per acre per year (Fig. 10-7). Meanwhile, phosphorus supplied by sewage sources has increased steadily from about 2 pounds per acre per year in the 1916-1930 period to a current level of about 6 pounds per acre per year. Presently, therefore, the total annual input of phosphorus, amounting to approximately 15 pounds annually per acre, is approaching the maximum input the lake can tolerate. At the same time, plankton blooms have been produced in increasing numbers and magnitude and are approaching nuisance proportions.

In considering ways and means of preventing the further deterioration and possibly the ultimate ruination of Lake Washington as a community asset, it is obvious that the ideal solution would require the removal of phosphorus from all waters, natural and waste, entering the lake. Since it is equally obvious that phosphorus cannot be removed from the natural waters, the only practicable alternative is to minimize its total input by (1) eliminating discharges from sewage treatment plants and (2) reducing the frequency of overflows from combined sewers. Under such a program, the total phosphorus input to the lake would amount to about 9 pounds per acre per year.

How soon Lake Washington will recover if all sewage is removed therefrom cannot be determined at this time. Since all indications at present are that the lake is rapidly approaching eutrophication, it may be concluded that nutrient concentrations therein will be more or less self-sustaining and that complete recovery might take many years. *On the other hand, if sewage is not removed, complete eutrophication of the lake will unquestionably occur within a relatively few years.* Beyond any question, reduction of the phosphorus input by the removal of sewage will stop the present trend toward complete eutrophication and very probably will lead to a satisfactory recovery of the lake.

The rate of recovery of the lake to an oligotrophic condition will be dependent not only on the amount and effect of the continued supply of nutrients from natural sources but also on the degree to which the lake has become degraded before the nutrient supply from sewage sources is removed. To determine whether the trend in the lake is toward complete recovery and, if so, the rate of recovery, limnological studies and studies of nutrient input should be continued for an indefinite future period. If it is found that recovery is not occurring or that the rate of recovery is so slow as to be indeterminate, additional remedial measures should be initiated. Among the possibilities in that connection are diversion of other major sources of nutrient supply, and establishment of improved agricultural methods such as reduction of soil erosion and proper application of fertilizers.

Prevention of Bacteriological Contamination

To provide relief for combined sewers during periods of rainfall, Seattle has constructed overflow structures along the entire Lake Washington shoreline and along the Ship Canal (Fig. 10-1). Bacteriological contamination of recreational areas brought about by discharges from these structures is largely confined to the area within the city.

Although the discharge from a combined sewer overflow is a mixture of storm water and sewage, dilution of the sewage with storm water has no more than a negligible effect on bacterial concentrations. From that standpoint, therefore, such discharges are equivalent to raw sewage discharges. At present,

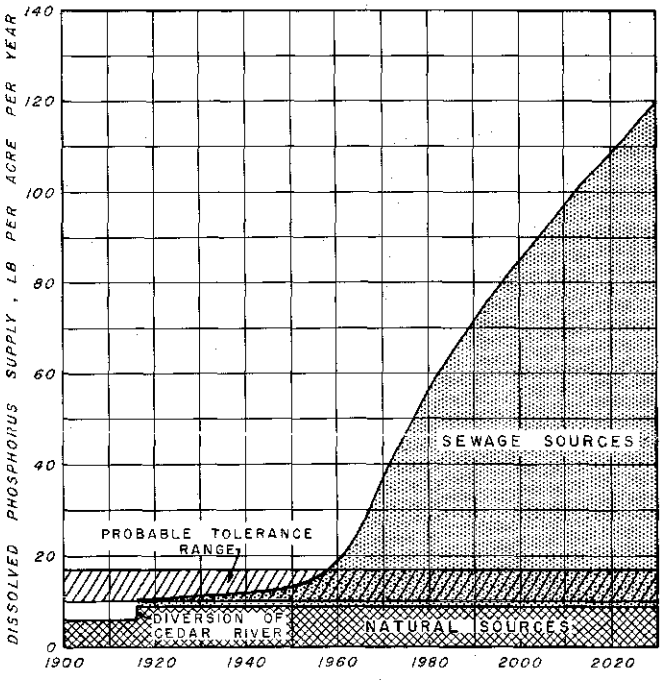
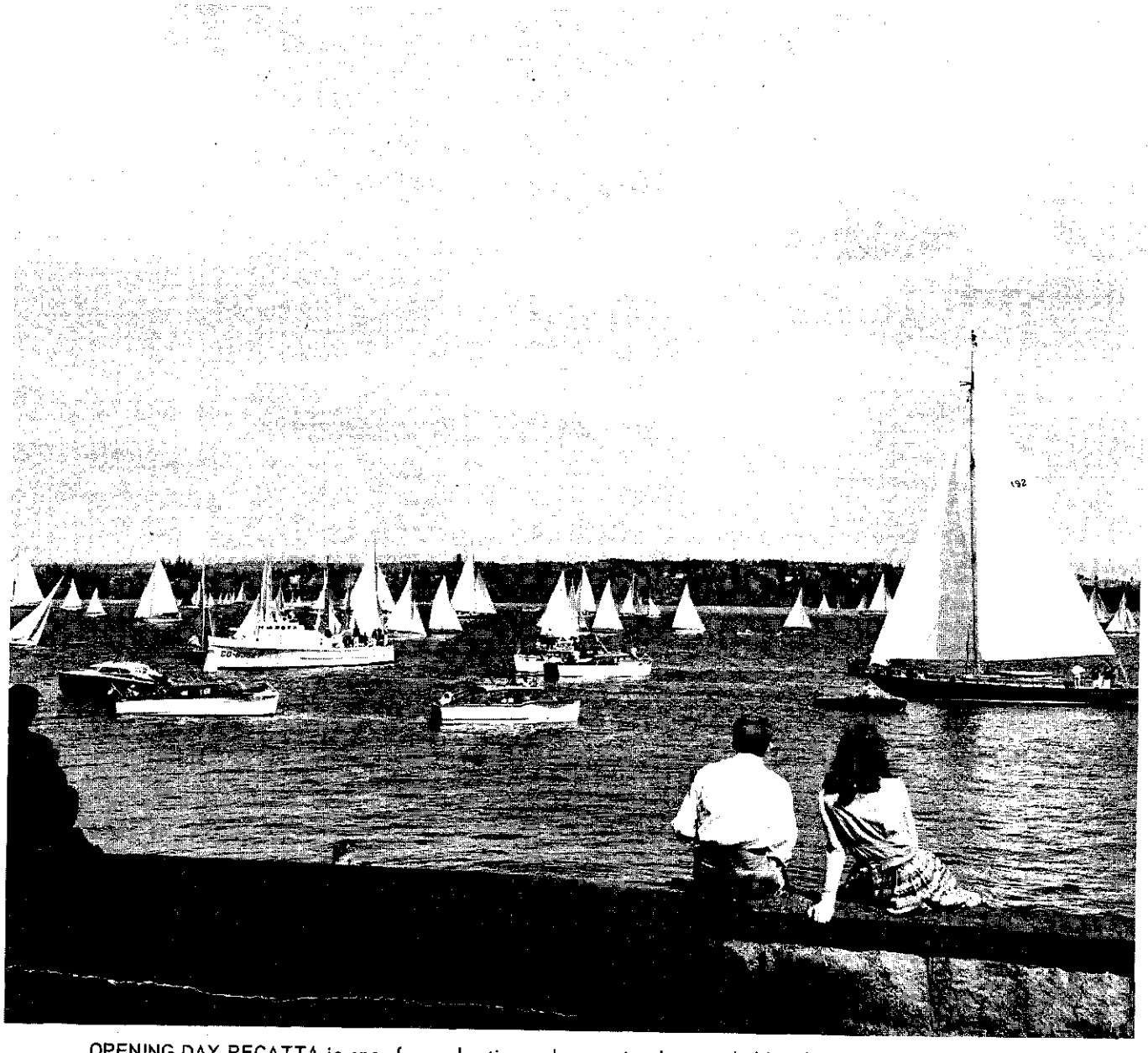


Fig. 10-7. Trend of Phosphorus Enrichment in Lake Washington

Sewage discharges have become an increasingly important source of phosphorus enrichment of Lake Washington. If present practices are continued, the ultimate contribution of phosphorus to the lake by sewage discharges is expected to reach 92 per cent of the total input of that element.



OPENING DAY REGATTA is one of many boating and recreational events held each year on Lake Washington.

overflows from combined sewers occur some 35 to 40 times during the recreational season, May to September, and are responsible for more or less continuous contamination of the beaches along the Lake Washington shore.

In order to prevent or minimize bacteriological contamination of the recreational areas in Lake Washington, it is evident that some degree of control will be required with respect to overflows from the combined sewer system of the city of Seattle. Since the extent and degree of contamination are dependent on the amount, duration and frequency of overflow, it will be necessary to reduce these factors to such a level that recreational uses are not adversely affected. Such a reduction can be achieved either by separation of domestic sewage from storm waters tributary to Lake Washington, or by provision of interceptor capacity sufficient to allow overflows only to the extent that they occur when recreational use of the waters is at a minimum.

According to available information, waters in the immediate vicinity of overflow structures are rendered unsafe for swimming on the day an overflow occurs. Obviously, then, overflows during the recreational season either should be eliminated or at least reduced to the lowest practicable minimum. On that basis, it is believed that reduction of the frequency to an average of once per recreational season constitutes a practicable solution. The reduction of the frequency of overflow will also reduce the amount and

duration of the overflow, so that the extent of bacteriological contamination during this one overflow will also be considerably reduced.

Combined sewer overflows in the ship canal system and in Lake Union present a somewhat different problem. As discussed previously, the principal uses of these waters are for navigation, boating and fish migration. Bacteriological standards applicable to recreational waters have no real meaning as regards such uses. At present, therefore, it appears that continued relief of combined sewers in these waters would not interfere with other water uses.

Miscellaneous raw sewage discharges, principally from houseboats and yachts, lead to some bacteriological contamination and physical impairment of the waters in the Lake Washington drainage basin. Where necessary, correction of this condition should be undertaken as a local problem by local agencies.

Chapter 11
SEWAGE DISPOSAL IN PUGET SOUND

In the disposal of sewage to tidal waters, the basic concept is that changed conditions brought about by sewage discharges shall not impair present beneficial uses of water or endanger anticipated future uses. As in all operations of this nature, the extent to which beneficial uses are affected depends on the ability of the receiving water to disperse and destroy organic matter and bacteria contained in the incoming sewage.

An analysis of the conditions which can be expected to prevail in a receiving water as a result of a particular sewage discharge requires a knowledge not only of the quantity and composition of the waste but of the physical, chemical and biological characteristics of the water. As related to Puget Sound, a knowledge is required also of the mechanics of both sewage effluent disposal and digested sludge disposal.

For convenience and clarity in presentation, this chapter is divided into two principal sections. Of these, the first is concerned with conditions affecting disposal of sewage in Puget Sound, while the second is concerned with an analysis of specific sites in terms of treatment requirements and submarine outfall performance.

CONDITIONS AFFECTING SEWAGE DISPOSAL

Information relating to the diverse conditions affecting sewage disposal in Puget Sound was gathered from various sources. While the determination of beneficial water uses and of water quality criteria pertaining to specific uses falls directly within the province of the Washington Pollution Control Commission, data on such physical characteristics as temperature, salinity, density, and tides and currents are collected routinely by the University of Washington Department of Oceanography and the U. S. Coast and Geodetic Survey.

Other conditions, such as the magnitude and direction of currents in the nearshore zone and the character of biologic life on the bottom adjacent to existing outfalls were investigated as a part of the field work during this survey. In addition, results of previous studies at specific locations were utilized where applicable.

Beneficial Uses of Water

Beneficial uses of water which must be protected in providing for waste disposal are generally listed in their approximate order of priority. This order depends upon legal decisions and local economic values. Two examples may be given to illustrate maximum and minimum ratings for waters of Puget Sound within the metropolitan Seattle area. Maximum priority is represented by a public beach which is used for wading, swimming, and recreational shell fishing purposes, and which may be jeopardized by an occasional discharge of untreated sewage at a nearby location. Minimum priority, on the other hand, is represented by a ship canal which, while otherwise capable of receiving a waste discharge, may become shoaled over a period of many years because of accumulated sludge deposits. Disposal of sewage by dilution is in itself a beneficial use which usually receives a low priority but is not necessarily incompatible with other uses.

Recreational Use. All of Puget Sound within the metropolitan area is used extensively for boating and sport fishing. Swimming and beach use, however, are restricted to the relatively few acceptable waterfront areas. Although water temperatures (54° F. in summer) obviously are not high enough to encourage very much swimming, long-range land use plans (Fig. 4-11) evision greatly increased development of waterfront areas for swimming and other recreational purposes. This outlook is illustrated in Fig. 11-1 which shows schematically both the present and planned future waterfront uses, as compiled by the Pollution Control Commission.[1]

While recreational use normally is considered to include water sports and similar activities involving a varying degree of direct personal contact with water, it also includes, in a broader sense, the somewhat intengible factor of esthetic enjoyment. Any condition which tends unduly to impair this enjoyment must be considered as undesirable, regardless of how inocuous it might be with respect to beneficial uses of a more tangible nature.

Effects of sewage and industrial waste disposal which must be avoided to preserve water for recreational use include impairment of physical appearance, odor, and general cleanliness, and contamination by coliform bacteria. Of these, physical appearance relates to turbidity and discoloration, and the presence of an oil or grease film on the water surface. Such a

[1] A Report on the Water Uses in the Seattle Metropolitan Area, *State of Washington Pollution Control Commission, October 1957.*

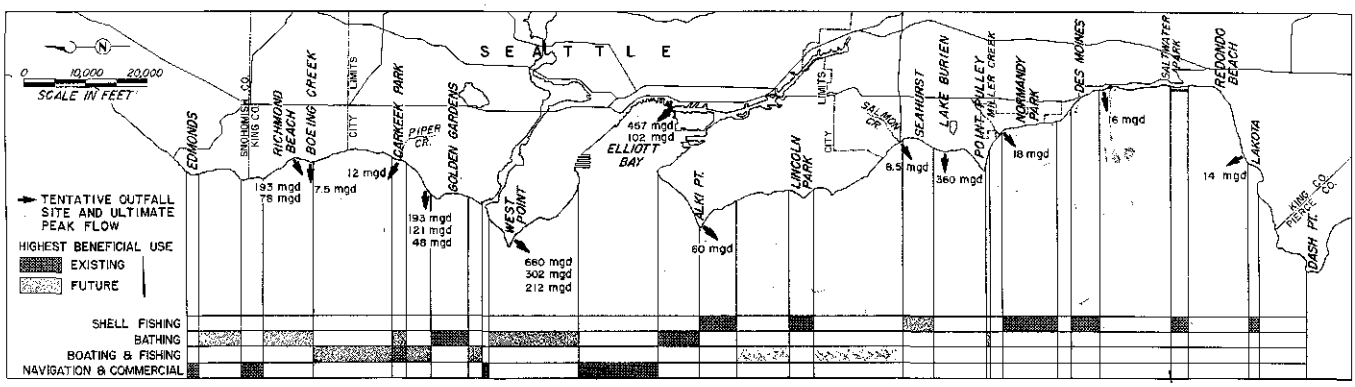


Fig. 11-1. Present and Future Waterfront Uses and Tentative Sewage Disposal Sites

In many cases, tentative outfall sites considered in the alternative sewerage plans are close to public waterfront areas having important beneficial uses. Protection of these uses requires a careful analysis of conditions affecting sewage disposal.

film, in addition to being unsightly, tends to adhere to swimmers and boats and to accumulate on beaches. Bacterial contamination, of course, may be present in the absence of any visual evidence of sewage and thus may well be the greatest hazard.

Fishing and Fisheries. Resident fish of Puget Sound which are important to commercial and game fishing include bass, flat fish, and salmon (Table 4-9). These species school and feed in large numbers in the waters of the sound bordering the metropolitan area. While commercial shell fishing is limited and of minor value, sport clamming is engaged in at several locations (Fig. 11-1).

An evaluation of sport fishing in Elliott and Shilshole bays has been made by the State Department of Fisheries for the year 1956.[1] Estimated catches in that year totaled 8,300 chinook and 7,200 silver salmon in Elliott Bay, and 12,000 and 9,000 respectively in Shilshole Bay. These catches, including allowances for tackle and boat facilities, were estimated to have a total value of $310,000 based on $130,000 for Elliott Bay and $180,000 for Shilshole Bay.

A small commercial shrimp harvest also is taken from Elliott Bay. In 1951, the year of the maximum harvest, the catch totaled over 30,000 pounds and had an estimated initial wholesale value of $22,500.

Maintenance of fish propagation requires a water environment not only in which commercial and game fish will survive but in which there is an abundance of food material in the form of smaller fish, crustacea, and minute animal plant life. Within limits dependent on local conditions, the discharge of sewage and nontoxic industrial waste matter can favor fish propagation by reason of the nutrients made available to the smaller organisms.

Conditions inimical to fish propagation are indicated in the list of state requirements given in Chapter 9. While all of these conditions are significant in the case of inland fresh waters, the dilution available in open salt waters in the vicinity of Seattle is such that only the items involving toxic matter and bottom deposits are of general importance.

Navigation. In addition to its use by recreational craft of all kinds, Puget Sound is used extensively by commercial vessels. Seattle harbor traffic (Table 4-5) totaled nearly 70,000 vessels in 1955, exclusive of recreational and commercial traffic in Lake Washington and the Ship Canal.

Commercial shipping involves little or no personal contact with the water. Obviously, therefore, this type of use ranks substantially lower than recreational use and permits less stringent requirements with respect to water conditions.

Industrial and Commercial Uses. Major industrial development in the metropolitan area is concentrated along the shores of Elliott Bay and in Duwamish River Valley (Fig. 4-11). In Elliott Bay, the entire shoreline from Duwamish Head to Smith Cove is taken up by industrial operations.

Salt water is used primarily for cooling purposes. Although temperature is of primary concern, cooling water should not contain excessive oils or turbidity nor should it be infested with slime-forming organisms. These deficiencies are significant in that they affect maintenance costs of cooling towers and other heat exchange equipment.

Probable Water Quality Criteria

Water quality criteria for specific beneficial uses have been established by the Pollution Control Commission (Chapter 9). As they relate to disposal of sewage in Puget Sound, the most important of these criteria are those pertaining to bacteriological conditions. Additionally, consideration must be given to criteria involving effects of a physical and esthetic nature.

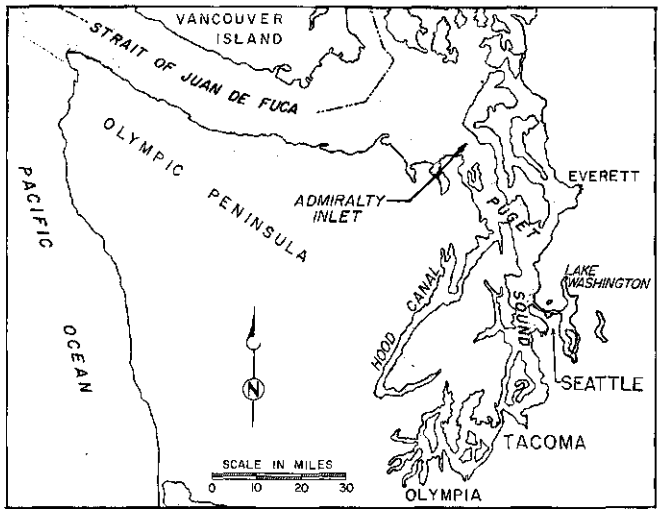


Fig. 11-2. Puget Sound

Criteria relating to bacterial concentrations are expressed in terms of the calculated MPN (most probable number) of coliform group organisms in a specific quantity of water (100 ml). For waters used exclusively for industrial and commercial shipping purposes, such as along the shore of Elliott Bay, there is no specific criterion. Cognizance must be taken, nevertheless, of areas used for log booming, which involves a degree of personal contact.

For waters farther offshore in Elliott Bay and in other places where recreational boating and fishing take place, an average MPN of 1,000 per 100 ml is applicable. Criteria applicable to areas used for swimming and bathing call for an average MPN of 240 per 100 ml. Along shores where shell fishing is a recognized use, a median MPN of 70 per 100 ml applies. Shell fish filter out and store up bacteria contained in water which they ingest. The latter criterion, therefore, is designed to safeguard the health of people who eat shell fish.

Criteria applicable to salt waters of the metropolitan area stipulate that, after reasonable dilution and mixing in the receiving body, there will be no floating suspended or settleable solids or sludge deposits which interfere with other beneficial uses of higher priority. Similar criteria apply to taste or odor producing substances and to phenolic compounds, oils, toxic, colored, and other deleterious materials.

Characteristics of Puget Sound

Puget Sound comprises a complex system of bays and channels (Fig. 11-2) which extends about 80 miles from the Strait of Juan de Fuca on the north to Olympia, the state capital, on the south. It is one of the deepest salt water basins in the United States, with depths to 930 feet in the northern portion and to 550 feet in the portion south of Tacoma Narrows. With a total area of about 650,000 acres and a mean tidal range of 9.3 feet, the mean tidal volume is 6,050,000 acre-feet.

Puget Sound has 1,330 miles of shoreline and encompases drainage basins totaling about 11,000 square miles. Drainage from these areas contributes a total mean flow of 50,000 cfs to the sound, of which 80 per cent enters via the major rivers.

Dynamics of the tidal prism are complicated by a tidal time difference of over 80 minutes from the Strait

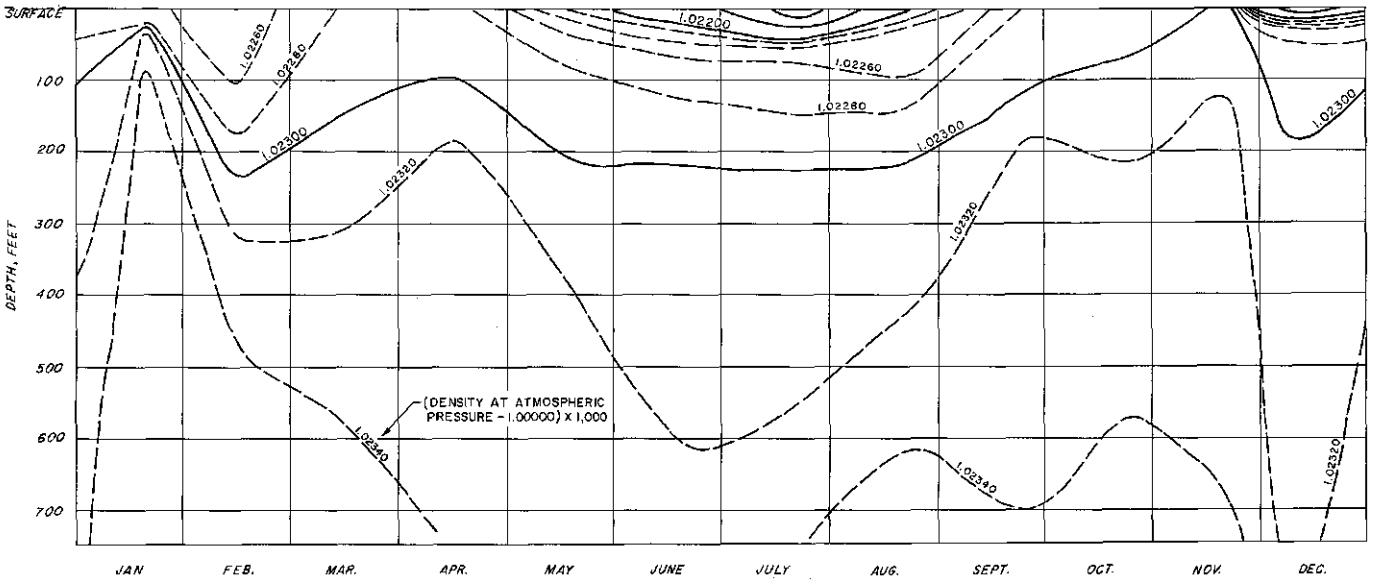


Fig. 11-3. Monthly Variation in Density Structure

Densities were calculated from temperature and salinity observations made by the Department of Oceanography of the University of Washington.

of Juan de Fuca to Olympia, and by strong and irregular currents. An unusual phenomenon of deep current movement is produced by an abrupt rise in the bottom which forms a dam 240 feet below the surface at the head of Admiralty Inlet. Ocean water flowing over the dam on flood tides is heavier than the less saline waters of the sound and continues its motion as a deep and distinct current as far south as Commencement Bay.

Temperature and Chemical Characteristics. Variations in certain characteristics of the waters of Puget Sound with season, location, and depth have been well defined by studies of the University of Washington Department of Oceanography.[2] These studies include thousands of determinations of temperature, salinity, and dissolved oxygen. While dissolved oxygen concentrations as low as 4.5 mg/1 have been found in the northern portion of the sound, they occur only at great depth and result from the oxygen demand of natural organic bottom deposits in pockets of dense sea water. Judging by the dissolved oxygen record, these pockets are flushed on the average of about once a year. Surface concentrations in the same areas rarely fall below 6.5 mg/1.

Density. By utilizing temperature data and concurrent salinity observations, as reported by the University of Washington,[2] sea water densities were calculated and are illustrated in Fig. 11-3 for the sound off West Point. Conditions indicated by the curves in this figure are generally typical for the part of Puget Sound bordering the metropolitan area.

[2] *Department of Oceanography, University of Washington,* Physical and Chemical Data - Puget Sound and Approaches, *Technical Report No. 45, March 1956.*

Tides and Currents. At most coastal locations, local currents in sea water bodies are affected by many factors, including tides, winds, thermal structures of the water mass, and oceanic currents. Currents in Puget Sound, however, are almost wholly induced by tides and differences in water density, with some modification at the surface brought about by river inflow and wind conditions.

Tides in Puget Sound are subject to diurnal inequality (Fig. 11-4), a characteristic reflected by pronounced differences in height of successive high and low water levels. This characteristic, which is greater in magnitude here than at any other location on the Pacific Coast of the United States, leads to unequal tidal time lags of flood and ebb currents. These lags lead in turn to changes in the duration of tidal flows and are particularly apparent in the lower or southern portion of the sound below Tacoma Narrows. At Seattle, the mean tidal range is 7.6 feet, while that at Olympia is 10.5 feet.

Variations in surface tidal current velocities in the vicinity of Seattle are shown in Fig. 11-5, which is reproduced from U.S. Coast and Geodetic current charts. One of the most striking features of the current pattern is the circulation which takes place around Vashon Island. Instead of reversing with a change in tide, current here is generally clockwise in direction. Even under maximum flood conditions, the current in Colvos Passage west of Vashon Island moves northward, or clockwise with respect to the island, at an average velocity of about 10 feet per minute (fpm). As a result, each tidal cycle is marked by a net water movement toward the south in the passage east of the island.

Wind Induced Currents. When wind blows across a body of water for a prolonged period of time, cur-

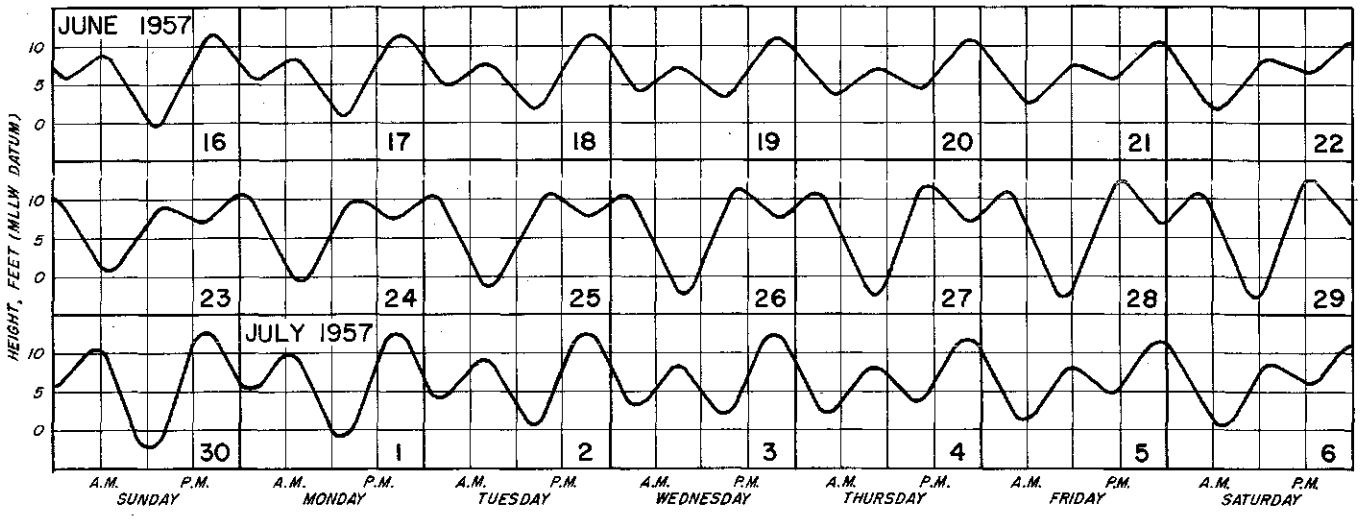
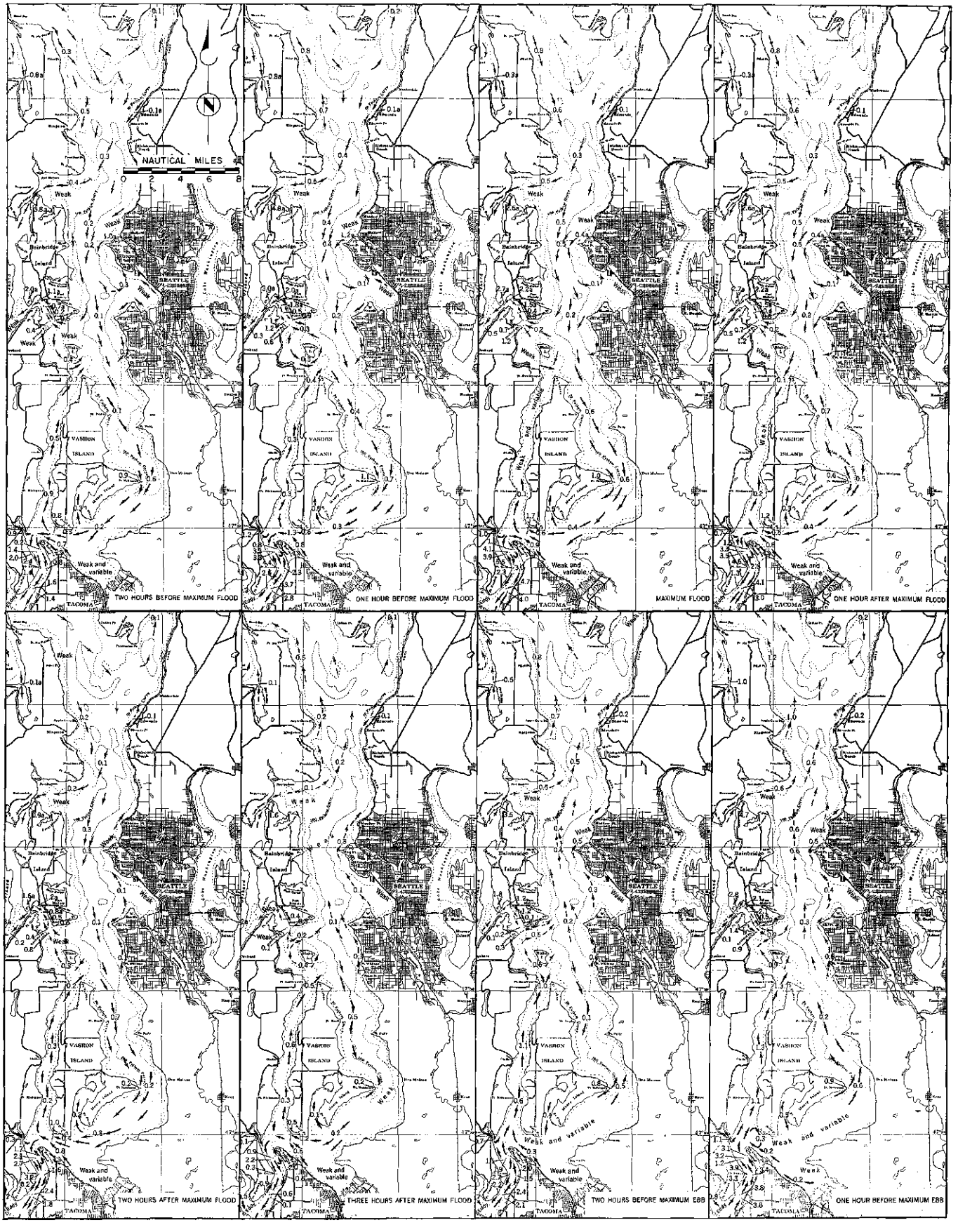


Fig. 11-4. Tidal Height Variation at Seattle

The two daily high waters may differ by over 5 feet and the two low waters by over 10 feet. Extreme variations during a day may exceed 15 feet, while the mean tidal variation is 7.6 feet.


NAUTICAL MILES
0 2 4 6 8
Weak
Bainbridge Island
SEATTLE
VASHON ISLAND
Weak and variable
TACOMA
TWO HOURS BEFORE MAXIMUM FLOOD
ONE HOUR BEFORE MAXIMUM FLOOD
MAXIMUM FLOOD
ONE HOUR AFTER MAXIMUM FLOOD
TWO HOURS AFTER MAXIMUM FLOOD
THREE HOURS AFTER MAXIMUM FLOOD
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ONE HOUR BEFORE MAXIMUM EBB

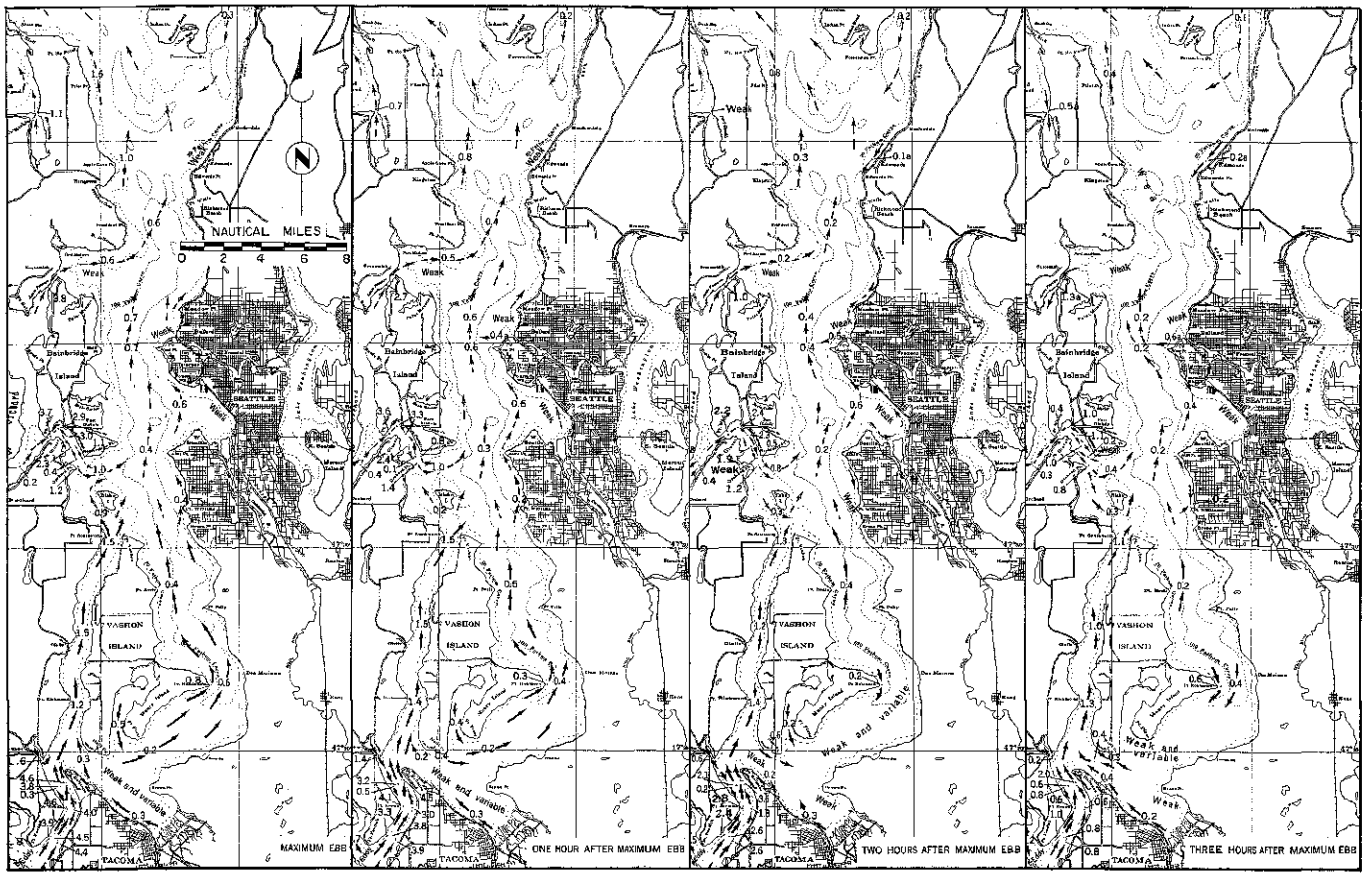


Fig 11-5. Generalized Tidal Currents in Surface Waters

Prepared from published charts of the U. S. Coast and Geodetic Survey, this figure illustrates the overall circulation patterns in the sound. During the sewerage survey, studies were made to define local current behavior at possible outfall sites.

rents are produced as a result of the friction between the air and water. Vertical mixing thus induced causes increased oxygen absorption and diffusion. In addition, a horizontal current may be set in motion. The magnitude of these induced currents is related directly to the length of the fetch, or unobstructed distance parallel to the wind direction, to the wind velocity, and to the duration of the wind. Movement in the surface layers of the water produces currents in the underlying water, although their magnitude rapidly diminishes at successively greater depths.

It is reported[3] that wind velocities less than about 13 miles per hour do not induce surface currents of an appreciable magnitude. Floating material, which projects above the water surface, obviously may be transported by winds of much lower velocity.

Observations and analyses of winds and wind-induced currents in Puget Sound have been made for the 31-mile fetch between Whidbey Island and Point Robinson.[4] Prolonged northerly winds, at the velocities frequently experienced, induce currents ranging from 0.08 knots in summer to 0.12 knots in winter. Currents induced by southerly winds, which generally are of greater velocity than those from the north, range from 0.10 knots in summer to 0.14 knots in winter.

Actual current velocities in the sound are many times greater than those attributable to wind action alone. It is apparent, therefore, that tidal action is the predominant factor in inducing current movement.

Mechanics of Effluent Discharge

The capacity of a receiving water to accept sewage and render it harmless depends on its ability to dilute or disperse the sewage, to destroy and otherwise reduce the concentration of sewage-borne organisms, and to oxidize and therefore stabilize the entering organic matter. In turn, the degree to which these functions can be performed naturally depends on the quantity and composition of the sewage. It also depends on (1) the extent of initial dilution in rising to the surface, (2) the extent of subsequent dilution after reaching the surface, (3) the rate of disappearance or

[3] *Sverdrup, H. V., Johnson, M. W., and Fleming, R. H.,* The Oceans, *1942.*

[4] *Harris, R. G.,* Surface Winds over the Puget Sound Area and Their Oceanographic Effects. *Technical Report No. 37, University of Washington, 1954.*

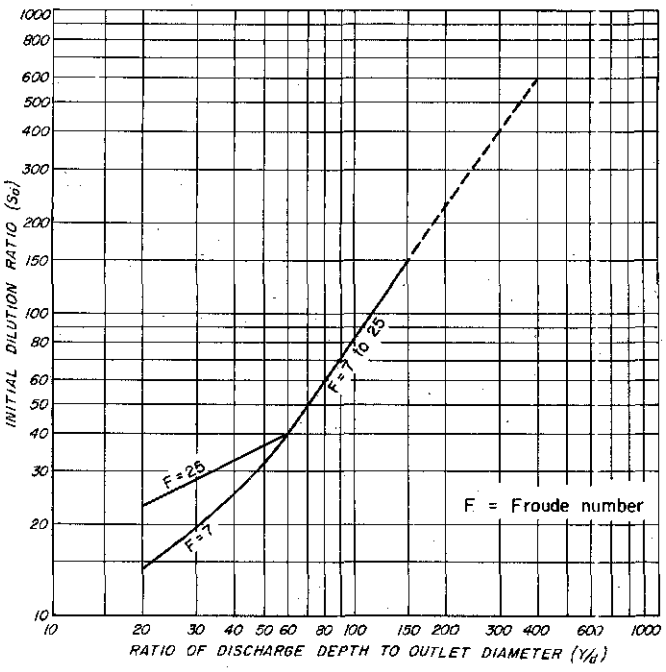


Fig. 11-6. Effect of Outlet Diameter and Depth on Initial Dilution of Sewage

The initial dilution (So) of sewage effluent as it reaches the surface or comes to equilibrium beneath the surface is related to the depth of discharge (Y) and to the diameter of the outlet (d). Froude numbers for most conditions encountered in outfall design fall in the 7 to 25 range. Curve based on analyses by N. H. Brooks.

reduction of coliform organisms, and (4) the behavior of local currents. While none of these four factors is subject to precise analysis, results of technical studies made elsewhere for similar purposes may be utilized for evaluating the first three. Local current behavior, however, had to be determined by field observation.

Initial Dilution. When sewage effluent is discharged below the surface of sea water, it is immediately subject to a buoyant force proportional to the difference in density between the sewage and the surrounding salt water. This force directs the discharge toward the surface and accelerates its ascent. Because, however, of the relative motion between the rising sewage column and the sea water, turbulence is generated and mixing takes place. Mixing is manifested first at the edge of the rising column and then progressively throughout the entire column. The extent to which initial dilution is achieved by the time the sewage-sea water mixture reaches the surface is dependent upon the discharge rate and velocity, the relative densities of the sewage and sea water, the depth at the point of discharge, and the current and turbulence conditions.

Experimental data and relationships for computing initial dilution were developed by Rawn and Palmer[5] and were reviewed and analyzed by Brooks[6]. Using the curve shown in Fig. 11-6, which was derived from result of Brooks' analysis, it is possible to estimate the initial surface dilution for various outfall situations.

Under favorable conditions of density and initial dilution, it is often possible to achieve a density of the mixture slightly greater than that of the surface layer of sea water. In such cases, the mixture will (1) remain below the surface of the receiving body and continue to spread and disperse, or (2) be carried to the surface under the action of the kinetic energy residing in the rising mixture, subsequently to plunge beneath the surface and disappear. Submergence or resubmergence of the mixture results from the discharge of sewage into sea water at a depth where the density is higher than that of the surface water.

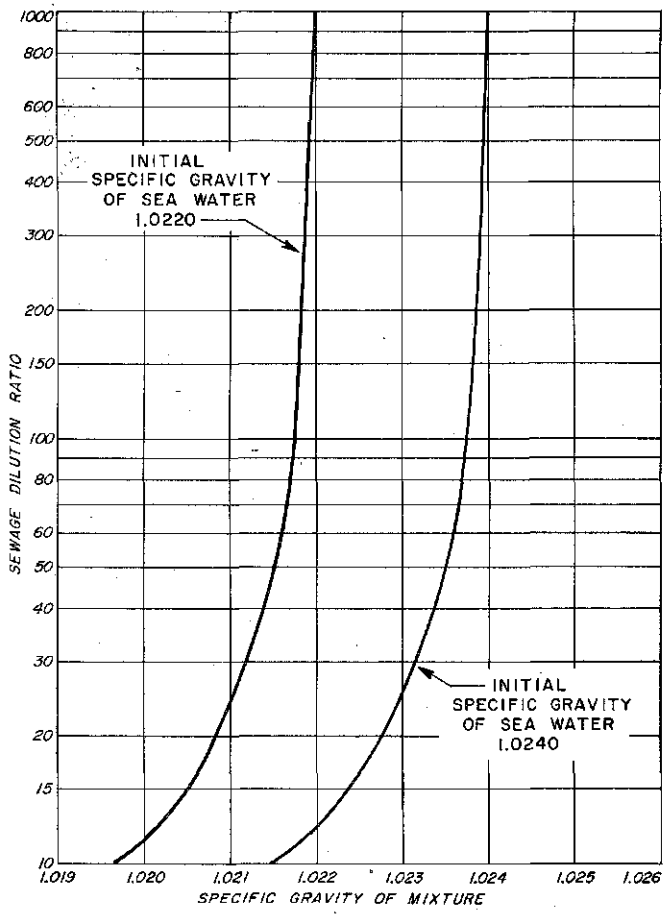


Fig. 11-7. Effect of Dilution on Density of Sewage-Sea Water Mixture

Specific gravity of salt water at depths where sewage can be discharged varies between 1.0220 and 1.0240. Submergence of the sewage-sea water mixture depends on the specific gravity of the surface water (Fig. 11-3).

[5] *Rawn, A M and Palmer, H. K.,* Pre-Determining the Extent of a Sewage Field in Sea Water, *Transactions A.S.C.E., 94 (1930).*

[6] *Brooks, N. H.,* Some Ocean Outfall Studies, *Report for Los Angeles County Sanitation Districts, September 1952.*

Fig. 11-7 illustrates the effect of initial dilution and of the density of diluting sea water on the density of the resulting mixture. Comparison of mixture densities for initial dilutions in excess of about 100 to 1 with the typical density structure (Fig. 11-3) indicates favorable conditions for submergence, particularly during the summer months. If this density condition is not achieved in the rising column, the sewage-sea water mixture will rise to the surface and float and spread as part of the surface layer.

Subsequent Dilution. After reaching equilibrium with its surroundings, whether submerged at depth or in the surface layer, the sewage-sea water mixture is further diluted by means of turbulent diffusion into the sea water. Except in extreme cases where strong vertical currents or turbulence exist, in Tacoma Narrows for example, diffusion is likely to be much greater in the horizontal direction than it is in the vertical. Hence, the area of the field of diluting sewage increases at a much more rapid rate than does its thickness. Compared with the initial dilution attained in rising from the point of discharge to the surface, subsequent dilution by turbulent diffusion is relatively slow.

Mathematical techniques developed by Brooks[7] were used for estimating subsequent dilution. For the reader with a technical interest in the calculations, it was assumed that the coefficient, c_1, which relates eddy diffusivity to the size of the sewage field, was in the range from 6 to 10, the higher value indicating a more rapid rate of diffusion. This range was chosen from the literature and from approximate values derived in the course of a similar study for the city of Tacoma in 1956.

Reduction of Coliform Organisms. In addition to the effects of dilution, the concentration of bacteria originally contained in sewage effluent is diminished through (1) sedimentation in the receiving water, (2) utilization as food by certain planktonic organisms normally present in sea water, (3) normal biological mortality or die-off, and (4) the effect of bacteriophage or antibiotic substances present in sea water[8]. As employed herein, the rate of disappearance of coliform bacteria, T, is defined as the time required for a 90 per cent reduction in coliform concentration due to the combined effects of these factors.

Normally, the disappearance rate is assumed to be constant for any given body of water. Recent studies of three Southern California outfall sites by the Hancock Foundation of the University of Southern California gave values of 2 to 4 hours. On the other hand, studies made by the city of Los Angeles in Santa Monica Bay yielded values of 6 to 8 hours. The lower values are indicative of a more rapid rate of disappearance. For the purpose of the present report, a range of 6 to 8 hours was assumed for the waters of Puget Sound. This is based on the relatively low temperature of the water and on other factors likely to retard the rate of coliform disappearance.

The combined effect on the concentration of coliform organisms resulting from subsequent dilution and natural disappearance is shown graphically in Fig. 11-8. As used in this figure, the coliform concentration ratio is the ratio of the coliform count in the sewage-sea water mixture after initial dilution to that at a given point in the receiving water. The ranges shown were obtained by making parallel calculations, using (1) the more rapid coefficients for diffusion and disappearance and (2) the slower coefficients. Beginning with known or estimated values for (1) the width of the sewage field as it reaches the surface, or comes to equilibrium beneath the surface, and (2) the time of travel to shore or other locations, the probable range in coliform concentration ratio can be readily estimated.

Mechanics of Digested Sludge Disposal

A number of cities on both the Atlantic and Pacific coasts are utilizing adjacent ocean water for disposal of digested sludge. Depending on location, this is accomplished either by barging to deep water or by discharge through a submarine outfall constructed to a suitable depth and distance offshore. In the case of the metropolitan Seattle area, the latter method is of particular interest.

In the digestion of sludge, the oxygen demand is greatly decreased, and except for residual grease and miscellaneous inert materials, sewage substances are reduced to a humus-like product. If required, residual grease and floatable inert solids can be removed effectively prior to discharge by means of a counter-flow type of washer. Washed digested sludge bears no resemblance to the solid and highly putrescible matter originally contained in sewage. In fact, digested sludge, as produced by the decomposition of organic matter, is quite similar in character to the material produced by the natural decomposition of dead plant and animal life. The latter action is taking place continuously in Puget Sound.

When discharged at a suitable depth, some of the digested material will settle and be deposited on the bottom, while the remainder will be transported and diluted by the prevailing currents. It must be recognized, however, that the upwelling of deep water,

[7] *Brooks, N. H.,* Methods of Analysis of the Performances of Ocean Outfall Diffusers with Application to the Proposed Hyperion Outfall, *Report to Hyperion Engineers, April 1956.*

[8] *California State Water Pollution Control Board,* An Investigation of the Efficiency of Submarine Outfall Disposal of Sewage and Sludge, *Publication No. 14, 1956.*

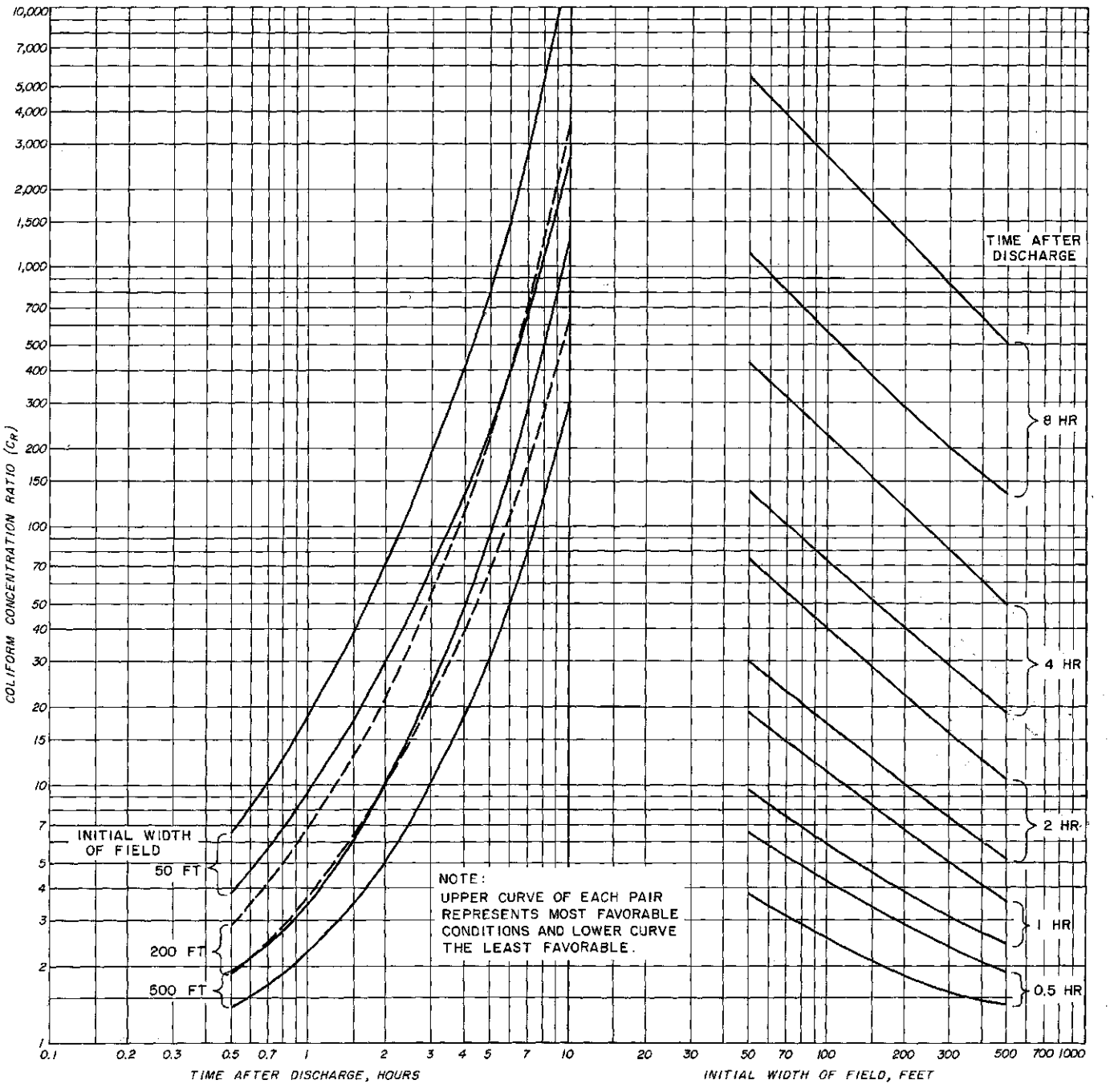


Fig. 11-8. Effect of Time and Size of Field on Coliform Concentration Ratio

The coliform concentration ratio (C_R) is defined as the ratio of the concentration after initial dilution to that at a given point in the receiving water. The upper and lower curves shown for initial width of field and for time after discharge are based on the ranges in diffusion and disappearance coefficients assumed to be applicable to the waters of Puget Sound.

which occurs because of seasonal changes in density structure, may result in a resuspension of previously deposited material. These conditions determine the biologic effects of submarine sludge disposal on fish and other aquatic life.

A comprehensive technical analysis of the deposition and dilution of digested sludge in sea water was made by Brooks[9] in connection with disposal to Santa Monica Bay from the Hyperion treatment plant of the city of Los Angeles.

Deposition and Dilution. The quantity of solids which will be deposited on the ocean floor is dependent on:

[9] *Brooks, N. H.* Predictions of Sedimentation and Dilution of Digested Sludge in Santa Monica Bay, *a report to Hyperion Engineers, August 7, 1956.*

(1) the type of outlet, the density of the discharge and the density structure of the water into which the sludge is discharged; (2) the physical characteristics of the digested sludge, primarily the distribution of particle sizes and their settling velocities; and (3) the magnitude of bottom or deep currents at and adjacent to the point of discharge.

As stated earlier, in discussing the mechanics of effluent disposal, a submarine discharge tends to rise toward the surface because its density is less than that of the sea water into which it enters. This tendency can be minimized by the relatively simple expedient of mixing digested sludge with sea water prior to delivery of the sludge to its submarine outfall. By adding enough sea water, the density of the sludge-sea water mixture can be increased to a level nearly equal to that prevailing at the point of discharge. When so diluted, only a relatively small amount of additional dilution in rising toward the surface will equalize the densities and cause the mixture to stabilize well beneath the surface. Deposition of a portion of the suspended particles will then occur.

Progressive dilutions which will occur while the sludge-sea water mixture rises toward the surface and the mixture density corresponding to various dilutions can be determined from Fig. 11-6 and Fig. 11-7. In using these figures, allowances must be made for changes in density brought about by premixing sea water with sludge prior to discharge. Under a given set of discharge conditions, the depth range in which the mixture will stabilize may be estimated from Fig. 11-3.

Because particles of varying sizes and characteristics settle at different rates, the time required for sludge particles to reach the bottom will vary considerably. By relating settling time to the distances that the sludge-sea water mixture is transported laterally by deep currents during the same time, it is possible to determine the proportion of particles which will settle, as well as their distribution or accumulation rate with respect to distance from the point of discharge.

Depending on the particular combination of influencing factors at each specific discharge point, the area of deposition may range from a relatively small section surrounding the outlet location to an area of several square miles. Under the first condition, which occurs generally when current velocities are extremely low or even negligible, deposition could possibly result in accumulations of sufficient thickness to cause both a localized reduction of dissolved oxygen and a change in the character of the bottom life. Because of the small size of the area thus degraded, the effect on the biology of the overlying water would be negligible. Under the second condition, which occurs when deep currents are fairly swift, the area of deposition can be so widespread that the amount of sludge deposited per square foot of bottom will be but a small fraction of the organic matter deposited from normal aquatic growths. Between these two extremes lie conditions which may or may not be amenable to sludge disposal and which, in each individual case, require local study and evaluation.

Diffusion. As the sludge particles are transported by currents, their concentration is reduced by deposition, as described above, and by dilution through the medium of eddy diffusion. Obviously, therefore, the concentration of particles remaining after a given time is a function of sedimentation and of the amount or extent of diffusion.

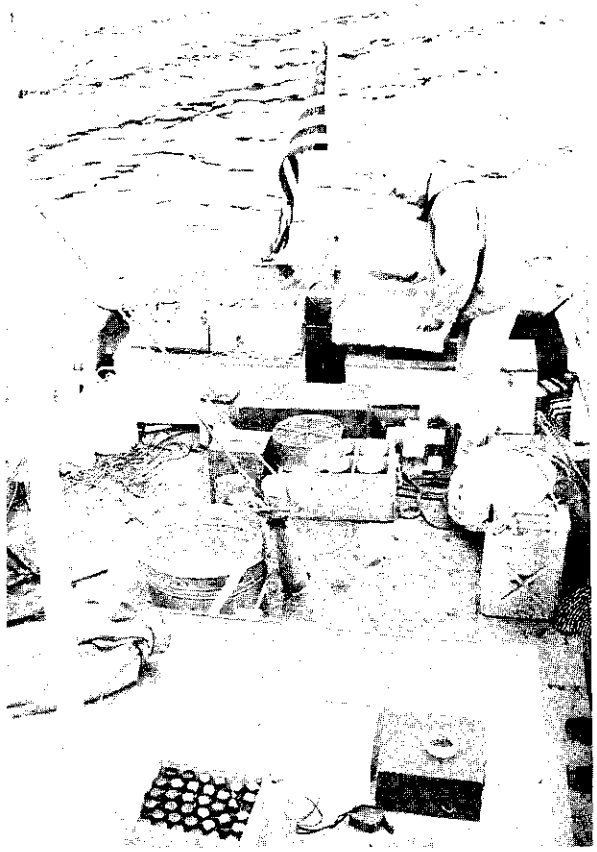
Resuspension. Since current velocity is one of the factors governing deposition, it is evident that increases in velocity would tend to pick up or resuspend particles which have previously settled to the bottom. This tendency is counteracted to some extent by the compaction which takes place after deposition. In the event, however, of either increased turbulence or seasonal upwelling of deep waters, a certain amount of resuspension is likely to occur.

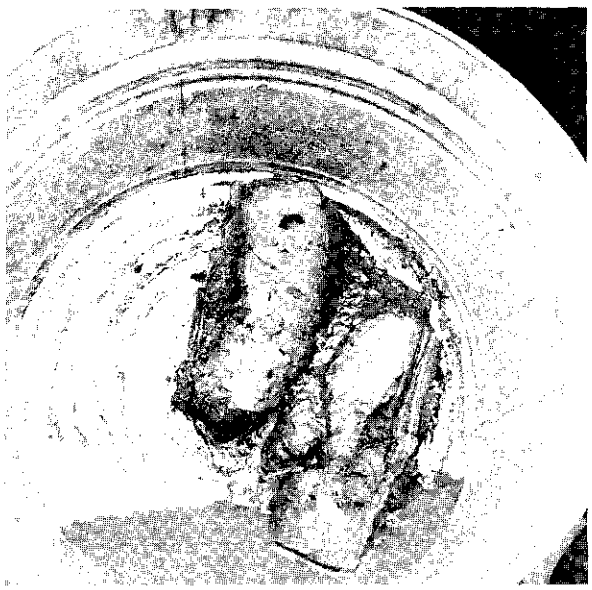
Initially, material will be resuspended from outer fringes of the deposition area where the least dense particles have settled. If conditions become increasingly favorable for resuspension, larger and heavier particles nearer the point of discharge will be picked up. In addition to particles of digested sludge, large quantities of other material will be resuspended from deposits of normal aquatic growths. These aquatic deposits are the source of the increased turbidity which occurs in Puget Sound during seasonal periods of upwelling.

Biological Effects. Submarine disposal of digested sludge can cause changes in the biota both of the deposit area and of the waters transporting suspended particles. The magnitude of these changes is related directly to the rate of deposition and accumulation, to the concentration of sludge particles dispersed in the water and, of course, to the nature or composition of the sludge itself.

With one exception, none of the existing treatment plants along the shore of the metropolitan area is discharging digested sludge to Puget Sound. Furthermore, the amount being discharged at the one plant (North Beach, Seattle) is not sufficient to produce measurable effects. It is not possible, therefore, to evaluate directly the biological effects brought about locally by discharges of digested sludge.

Raw sewage, on the other hand, is discharged to Puget Sound at numerous locations. Of these inputs, the largest is from the North Trunk sewer of the city





BOTTOM SAMPLES for biological examination were collected in Shilshole Bay, using a Petersen dredge (upper left, arrow). The biological studies, undertaken cooperatively by the survey and the United States Public Health Service, were aimed at assessing the effect of continuous raw sewage discharges on the biota of the bottom. Typical sample shown in lower left photo.

of Seattle which discharges to Shilshole Bay at a depth of 40 feet. Although deposition of sewage solids and resulting changes in bottom life observed at this location cannot be considered comparable to conditions which would result from the discharge of digested sludge through an outfall to deep water, they nevertheless serve to indicate the general nature of the over-all effects of such an operation.

In the course of a study by R. O. Sylvester during 1949,[10,11] a limited number of bottom samples were taken near the terminus of the North Trunk outfall and were examined for both physical and biological characteristics. Deposited materials of sewage origin, as evidenced by the presence of organic matter, creosote or oily substances, and the odor of hydrogen sulfide, were found in the immediate vicinity of the outfall. These materials were present in a zone parallel to the shore which had an average width of about 700 feet and

[10] *Sylvester, R. O.,* Puget Sound Pollution, Seattle Metropolitan Area, *a report to the Washington Pollution Control Commission, 1949.*

[11] *Letter from A. E. Bartch, Biologist, Division of Water Pollution Control, U. S. Public Health Service to E. F. Eldridge, Director and Chief Engineer, Washington Pollution Control Commission, dated October 7, 1949.*

extended 1,600 feet to the southwest and 1,000 feet to the northeast of the outfall.

As a part of the field work during the sewerage and drainage survey, an investigation of conditions at the Shilshole Bay outfall was undertaken in cooperation with the U.S. Public Health Service. Attention was directed in particular to sludge deposits and bottom life with a view to estimating the possible biologic effects which would result from the discharge of digested sludge to Puget Sound. Results of that investigation are set forth in a report from the U.S. Public Health Service, a copy of which is presented herein as Appendix C.

In the 1957 study, the observed limits of the zone in which evidence of deposition was found were roughly similar to those noted in 1949. As reported in Appendix C, intermittent deposits, which ranged in thickness from about 6 inches to a thin film, were found within a radius of 1,000 feet of the terminus of the outfall.

In both the 1949 and the 1957 studies, marine organisms found in bottom sediments within the zone of deposition were predominantly of fresh water origin and presumably were introduced along with the raw sewage. Inside the zone of deposition, the number of marine scavenger organisms, notably crustacea, was relatively high. Outside the zone of deposition, both the number of species and the total number of organisms were similar to those elsewhere in the sound.[12]

It should be noted that the number of species and of different organisms appears to decrease with increasing depth in the sound. In any case, results of both studies justify the conclusion that the existing sewage discharge from the North Trunk outfall has had no more than a local effect on bottom marine life.

The question next arises as to the possible effect of digested sludge discharges on the biota of the waters adjacent to an outfall. Of particular concern are a possible reduction in dissolved oxygen concentration and the introduction of chemically toxic materials. Studies reported by Pearson[8] indicate (1) that digestion for a period of about 30 days reduces the BOD of sludge by 85 to 90 per cent, and (2) that when digested sludge is discharged to the same body of receiving water as is effluent from a primary treatment plant, the resulting BOD load is increased in the order of 2 per cent. In view, therefore, of the high dilution ratios which can be achieved through the use of a properly designed outlet structure, such an increase would have little or no effect on the receiving waters.

A discussion of the many substances possibly toxic to fish and other aquatic life is beyond the scope of this survey. It is reasonable to assume, however, that sewerage agency control over industrial waste sources would limit the contribution of toxic substance to a level which would be rendered nontoxic after initial dilution with sea water.

Disposal in Puget Sound. The feasibility of discharging digested sludge to Puget Sound may be determined in two ways. Of these, the most direct, of course, would be to use such a disposal system for a period of time sufficiently long to determine the nature and magnitude of its effect on bottom and adjacent water biota. Fortunately, the situation in the Seattle area lends itself ideally to such an approach. This is because it will be at least 5 years before a conclusion will have to be reached in regard to the final design of sludge disposal facilities at the proposed West Point treatment plant. Meanwhile, disposal of digested sludge through an independent outfall into deep water could be undertaken on a trial basis at the new Alki Point treatment plant. Controlled disposal at that location, coupled with a comprehensive monitoring program for a period of 5 years, would provide ample information concerning biological and other effects, and thus would enable a final decision as to the practicability of sludge disposal in the waters of Puget Sound. The monitoring program should be undertaken jointly by the Pollution Control Commission and the city of Seattle and should be designed to achieve two objectives: first, it should establish existing environmental conditions in the vicinity of the discharge; and second, it should develop essential information concerning any biological, chemical or other changes, including possible beneficial effects, which may devolop as a result of the disposal operation.

The second way to determine the feasibility of discharging digested sludge to Puget Sound would be to estimate the effects of deposition and dilution under local physical conditions. Since the sludge can be conveyed to comparable depths at all of the alternative sites for treatment and disposal, it is logical to assume that its effects will be most pronounced at the site of the largest discharge. This will be the West Point site where the proposed treatment plant will serve an estimated ultimate population of roughly one million persons.

Based on (1) a suspended solids loading of 0.25 ppcd in raw sewage, (2) a solids removal of 60 per cent by primary sedimentation, (3) a 75 per cent volatile solids content of the raw sludge, (4) a 60 per cent reduction in volatile solids during digestion, and (5) a 93 per cent moisture content in the digested sludge, it is estimated that the average output of digested sludge solids ultimately will amount to 80,000 pounds per day at West Point and will be contained in a daily volume of 135,000 gallons. At this site, as at other possible sites along the sound, a relatively small

[12] *Department of Oceanography, University of Washington,* Oceanographic Survey on Submarine Portion of Snohomish-Kitsap, 25 KV Line, *1953.*

diameter submarine outfall would be required for digested sludge. Such a line could be laid to a depth of 400 feet or more, which depth is reached at West Point at a distance of 3,700 feet offshore.

The specific gravity of digested sludge, about 1.010, can be increased to about 1.017 by pre-mixing one volume of sludge with two volumes of sea water. Based on a specific gravity of sea water at the 400-foot depth of approximately 1.023, (Fig. 11-3), an additional dilution after discharge of less than 50 times would be sufficient to bring the sludge-sea water mixture into equilibrium with its surroundings. This additional dilution could be achieved without the use of multiple outlets at the end of the outfall, and the kinetic energy of the rising column would be dissipated in a distance of less than 100 feet above the bottom. Under certain favorable conditions, the mixture could stabilize after rising about 50 feet. In either case, deposition would tend to occur following stabilization and would be governed by deep currents, as well as by the relative settling velocities of the particles of digested sludge.

In a zone 3,000 to 5,000 feet off West Point, the direction of deep currents was found to be predominantly parallel to shore. Based on an average velocity of 70 feet per minute observed during an extreme tidal range, an average velocity of 40 feet per minute under all tidal ranges is considered appropriate for use in estimating the accumulation rate. It is worthy of mention in this connection that both the density stratification which exists in the sound most of the year and the prevailing current directions in the West Point area make it highly unlikely that a water mass would be transported onshore from an initial depth of 300 to 400 feet.

No information is available locally with respect to the distribution of settling velocities of digested sludge particles. Based on settling rate analyses made elsewhere by Brooks[9] and on a deep water current velocity of 40 feet per minute, Fig. 11-9 shows the accumulation rates which may obtain under the ultimate loading conditions at West Point. In developing this information, it was assumed that deposition would occur from a height of 50 feet above the bottom. If the discharge rose to a height of 100 feet before stabilizing, the rate of accumulation per unit area of bottom would be considerably less.

It should be emphasized that the rates indicated in Fig. 11-9 are not the result of a precise analysis. They are, however, believed to be indicative of the magnitude of deposition and are based on as reasonable an evaluation of local conditions as is presently possible.

Although shoreward movement of the deep waters is not likely to occur, it is of interest to calculate possible shore concentrations by employing the criteria for surface water movement. Assuming a sustained maximum onshore surface current of 20 feet per minute, a minimum of three hours would be required to reach shore. During this time, the mixture of digested sludge and sea water would be further diluted in the approximate ratio of 1:100 through the effects of diffusion and deposition. Travel to any other point along the shore would involve longer times, and therefore, still greater diffusion and deposition.

Assuming also that pre-mixing would provide a 3 times dilution and that the discharge would be diluted an additional 50 times while reaching density equilibrium, the concentration of digested sludge particles approaching shore would be decreased in the ratio of 1:15,000 compared to its initial level. In other words, the initial concentration of 70,000 ppm, which is equivalent to the 7 per cent solids content assumed for the digested sludge, would be reduced to 5 ppm even under the most adverse and unlikely onshore current conditions. Similar calculations, employing a minimum travel time to Golden Gardens beach of 4.5 hours, indicate that a concentration no greater than 1 ppm would occur there even under the most adverse current conditions.

Consideration must be given also to the effect of resuspension on the sludge particles which do settle out. Because of the lack of specific information on upward velocities connected with upwelling or other seasonal changes in deep waters of the sound, the

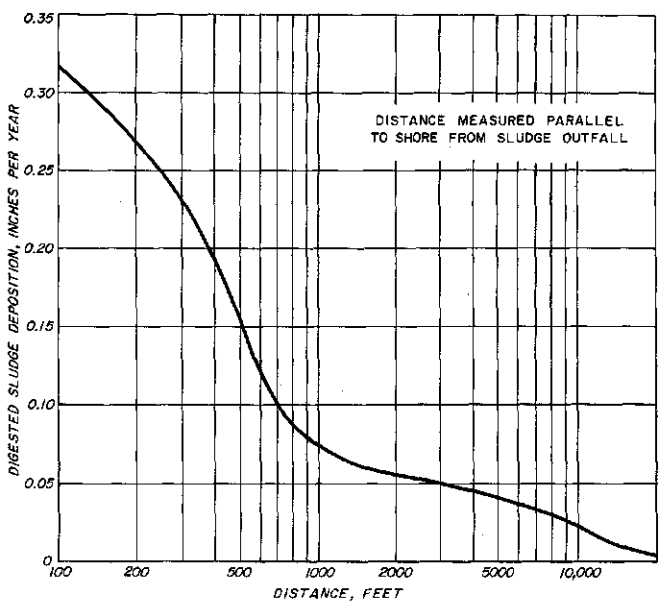


Fig. 11-9. Possible Magnitude of Digested Sludge Deposition from Discharge off West Point

Since precise information regarding many of the factors affecting deposition in Puget Sound is not available, the rates shown must be considered as illustrating only the general magnitude of this action.

magnitude of this effect cannot be estimated quantitatively in other than general terms.

The total weight of digested sludge which ultimately might be discharged to the sound from treatment plants on the shore of the metropolitan area amounts to about 40 million pounds per year. Estimates of the accumulation rate off West Point (Fig. 11-9) and the results of measurements of deep currents indicate that sludge dispersion would take place throughout an extremely large area. If this area, for purposes of illustration, extended from the north end of Vashon Island northward to Point Jefferson, it would have a volume of 4.4 cubic nautical miles or 10^{12} cubic feet. In such a volume, the annual loading of digested sludge, including resuspended and nonsettling portions, would produce a concentration of 0.5 ppm. Furthermore, concentrations approaching that value would occur only during seasonal upwelling. When upwelling occurs under present conditions, a concentration many times greater than 0.5 ppm develops due to resuspension of material from normal aquatic depositions.

In summary, it can be stated that the disposal of digested sewage sludge to Puget Sound is feasible for the following reasons:

1. Discharges will tend to stabilize at a height of 50 to 100 feet above the bottom, providing the sludge is pre-mixed with sea water and discharged into relatively dense water at a depth of 400 feet.

2. Sludge particles will settle out over a very large area. For example, at a distance of 1,000 feet from an outlet off West Point, the ultimate accumulation rate will be about 0.1 inch per year. At a distance of 5,000 feet the rate will be 0.05 inches per year.

3. Currents in the sound in the depth range where sludge particles would tend to stabilize are predominantly parallel to shore. Only under unusual conditions, therefore, would portions of the diluted and dispersed sludge-sea water mixture be transported toward shore. In such an event, and in the case of a discharge off West Point, the effects of dilution, deposition and diffusion will reduce concentrations to about 5 ppm directly onshore and to 1 ppm at Golden Gardens.

4. Resuspension of digested sludge particles due to seasonal upwelling will result in no more than a very slight increase in the concentration of material resuspended at the same time from normal organic deposits.

5. Biologic activity, at depths where sludge-sea water mixtures will tend to stabilize and where deposition of particles will tend to occur, has been observed to be of rather limited magnitude. This means that little, if any, impairment of the biota at these depths can be expected. In fact, increased biologic activity may occur in the deep waters.

Nature and Scope of Current Studies

Despite the vast amount of work which has been done in defining the chemical and physical characteristics of the water in Puget Sound, it was found that insufficient data were available regarding the effects of local currents at each of several possible disposal points. As a consequence, it became necessary to develop further and more specific information regarding current conditions. Attention was directed particularly to the action of local eddy currents and longshore currents potentially capable of carrying sewage effluent onshore or to nearby recreational areas.

Planning of the current measurement program was facilitated by data which had been developed at the University of Washington, using the hydraulic model of Puget Sound. Qualitative movements of water masses, as determined by photographing the path of a dye tracer in the model, are the subject of a thesis by Rogers.[13] In the course of this work, tracer dye was introduced and photographs were taken of the resulting conditions at several of the locations presently under consideration as alternative outfall sites. As a consequence, no formal use was made of the hydraulic model for the purpose of the sewerage survey.

Results of the current studies on the sound confirm the general qualitative results on the model reported by Rogers. Discrepancies with respect both to travel times, measured in the model in terms of tidal cycles, and to specific local conditions, measured in terms of hours, are readily explainable by such factors as surface tension and scale effects on the model. In addition, an average annual tide pattern was employed in the model, whereas actual studies in the sound were under a variety of tidal conditions, including those of extreme ebb and flood ranges. As a consequence, maximum current velocities and water mass movements in the sound tend to be greater than those determined by means of the model.

From the model data and from other current studies in Puget Sound, it was evident that generally similar local currents exist at locations which have similar shore and offshore topography. It was unnecessary, therefore, to conduct separate studies at each of the alternative outfall sites. Included in the seven areas selected for study (Fig. 11-10) were those with regular, fairly straight shorelines, those in the vicinity of points of land extending into mainstream, and the one large indentation, Elliott Bay.

It should be noted here that the term local currents means those which prevail between the mainstream or midchannel currents in the sound and the currents in the zone immediately adjacent to the shore. For the purposes of the survey, it is assumed that sewage-

[13] *Rogers, E. H.,* A Pollution Study of Puget Sound Using a Hydraulic Model, *University of Washington Master's Thesis, 1955.*

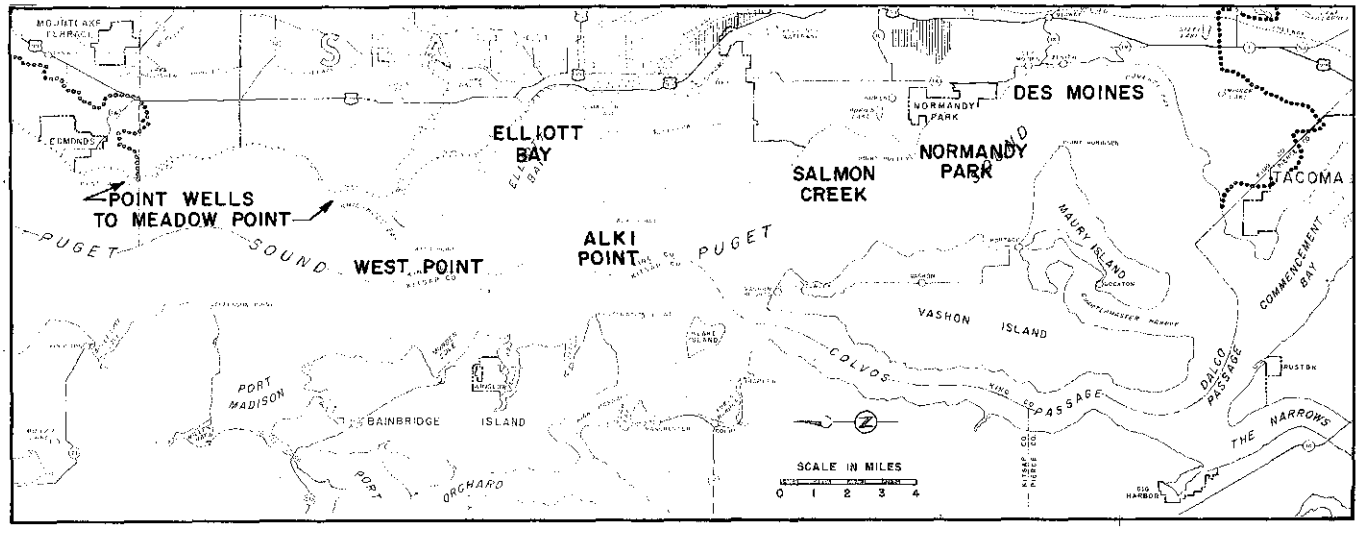


Fig. 11-10. Location of Current Study Areas

sea water mixtures are on the shore when they reach this zone.

Further field work in the sound included, as stated earlier, a cooperative study with the U.S. Public Health Service to assess the effects on bottom biota brought about by deposition of material from existing raw sewage discharges. Because ample information has been developed in the past by city, county and sta' agencies (Chapter 8), no samples were taken for ba. teriological examination.

Current Study Procedure

During the period from June 18 to July 30, 1957, a total of 22 current study cruises were made in the sound at locations ranging from Point Wells on the north to Des Moines on the south. Since the general currents in the sound are almost entirely the result of tidal action, cruises were scheduled so as to observe them under extreme, average, and minimum ranges of tidal variation.

A 40-foot patrol boat and its crew of three were generously provided by the U.S. Coast Guard for all the studies in the sound. Because of its shallow draft, high speed and maneuverability, this type of craft was especially well suited for the current study operations.

Free-floating biplane drags were constructed and used for current observations (Fig. 11-11). Of the two sizes shown, the larger ones were used at depths of 100 feet and more, while the smaller were used for relatively shallow depths of 10 to 20 feet. Sufficient ballast was attached to the bottom of each drag both to prevent planing and to cause the marker can to float half-submerged on the surface. By utilizing plastic laminated sheets of glass fiber for the vanes, the drags were relatively lightweight and easy to handle. A small hand winch, mounted on a boom, was used for releasing and retrieving the large drags, while the small drags, with their much shorter length of cable, were easily dropped and pulled in by hand.

Unlike wood, the laminated vanes do not absorb water and thus do not increase in weight during an extended period of submergence. Wooden drags, even when well painted, can absorb enough water to offset

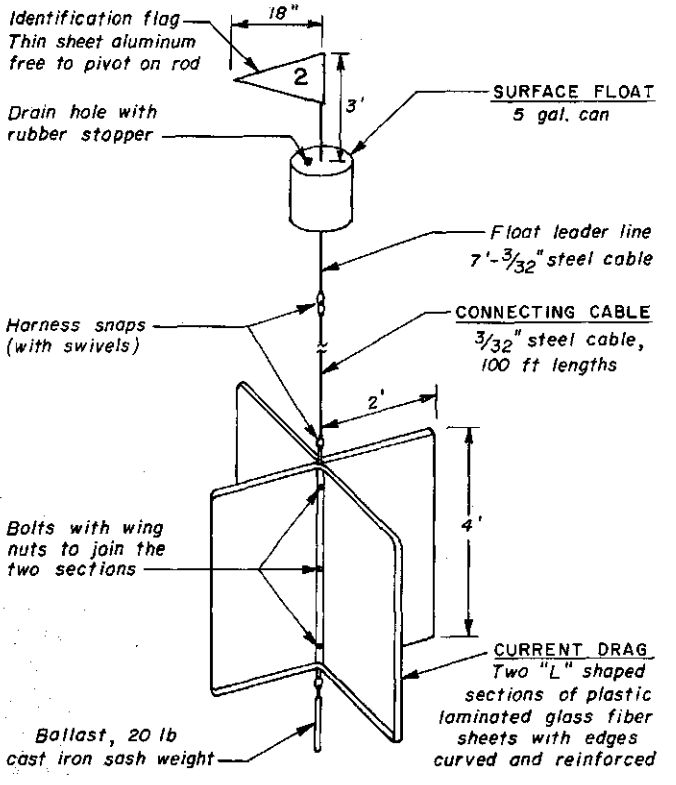
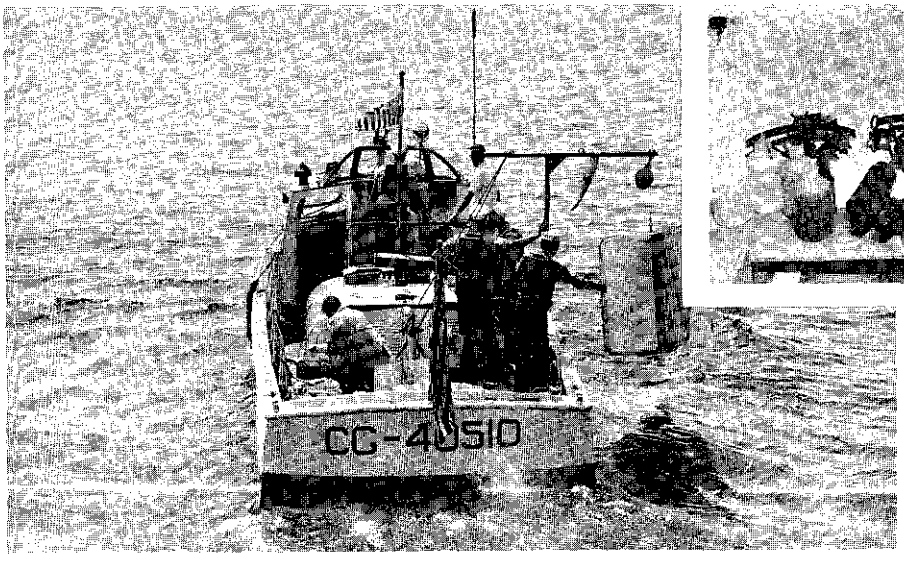


Fig. 11-11. Biplane Drags used to Measure Deep Currents

Drags of the type illustrated above were used to measure currents at depths to 400 feet. Drags for measuring currents at or near the surface were similar but had vanes 1 by 2 feet.

DEEP CURRENTS in offshore waters were measured by means of free-floating drags constructed of plastic laminated glass fiber. Large drags (left photos), used to measure currents at depths up to 400 feet, were lowered and raised by a small hand winch mounted on a boom.



SURFACE CURRENTS were measured with small drags (right photo) which were dropped and pulled in by hand. Both surface and deep drags were so weighted that the surface marker float was almost submerged (circle, lower right). After release, the position of each float was determined at intervals of approximately one hour.

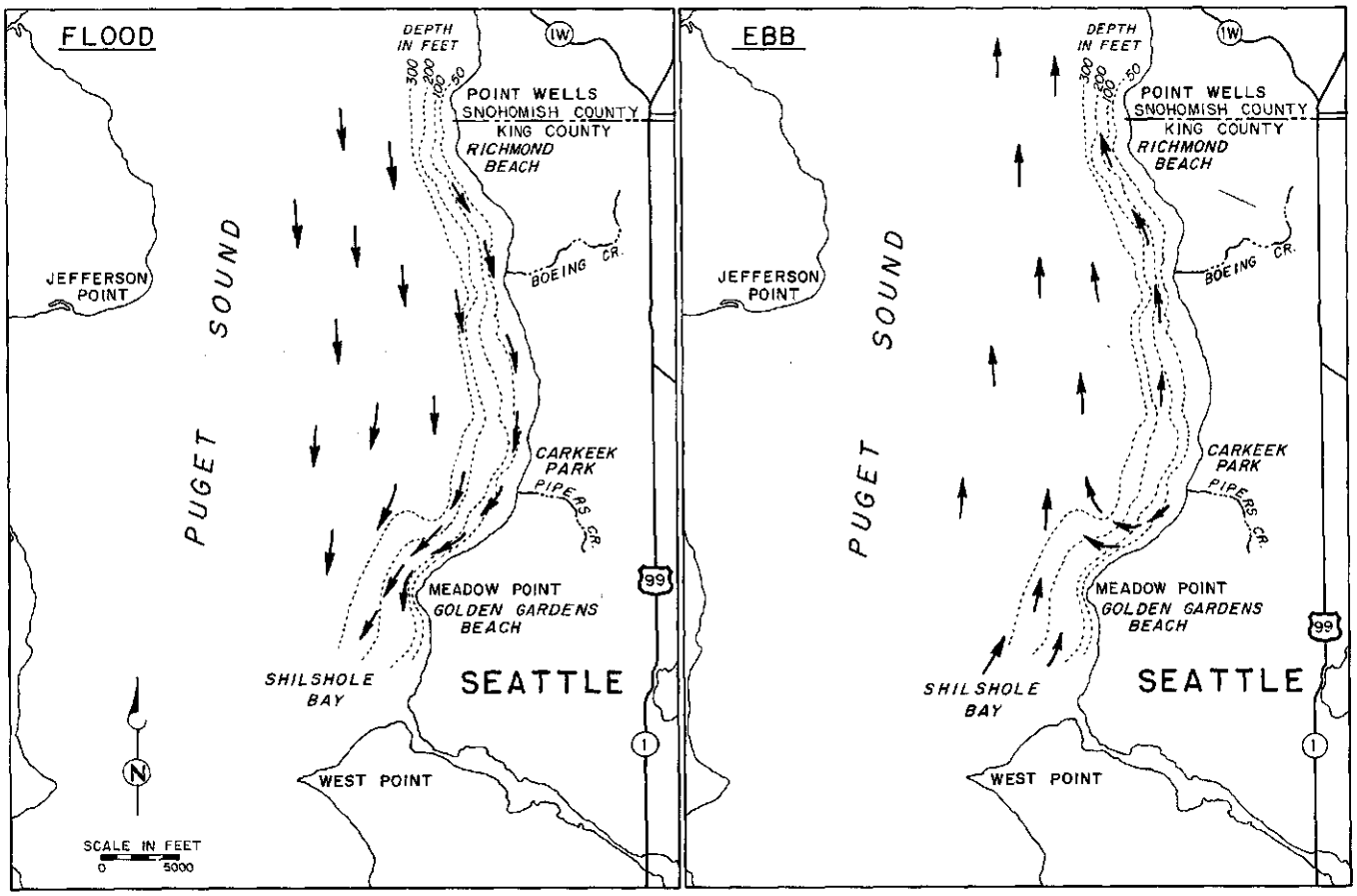


Fig. 11-12. Generalized Current Directions - Point Wells to Meadow Point

This, and similar figures which follow, illustrate the movements of prevailing local currents as determined both from previous investigations and from studies during the survey.

the positive buoyancy of the surface marker. This results in a gradual sinking of the entire assembly until the marker submerges and is lost.

Throughout the current studies, positions were determined by sextant observations from the boat to previously located control points on the shore. Float positions were plotted immediately on charts, using a three-arm protractor. To minimize the effects of drifting, it was found necessary to use two sextants and to read angles simultaneously to assure accurate plotting of position.

Generally, each series of observations was begun by releasing drags at points 500 to 1,500 feet apart on a line approximately normal to shore. In most cases, a total of six drags was released. After the last one was turned loose, the boat returned to the first drag, the location thereof was determined and plotted, and other necessary observations recorded. Following that, the remaining drags were located in turn and the entire procedure was repeated until the drags travelled out of the area of study.

Depending somewhat upon the area over which the drags were dispersed by the currents, a complete circuit of the group was made at approximately hourly intervals. Operating in this manner, it was possible to chart current movements accurately during a particular tidal condition. In most cases, drags were retrieved and a new series of observations was begun as closely as possible to the time of tidal current changes.

Interpretation of Current Study Results

Data obtained from the current studies and from other investigations at specific locations serve to define local current velocities and directions. Prevailing currents determine the general suitability of specific disposal sites. Maximum velocities determine the minimum period of time during which dilution, diffusion and disappearance of sewage and sewage-borne organisms may take place before the sewage-sea water mixture reaches a location of public health significance.

For reference purposes, detailed results of the current study cruises made during the survey are presented in Appendix D. In the following discussion, the results are summarized and interpreted for each of the general areas where the studies were made.

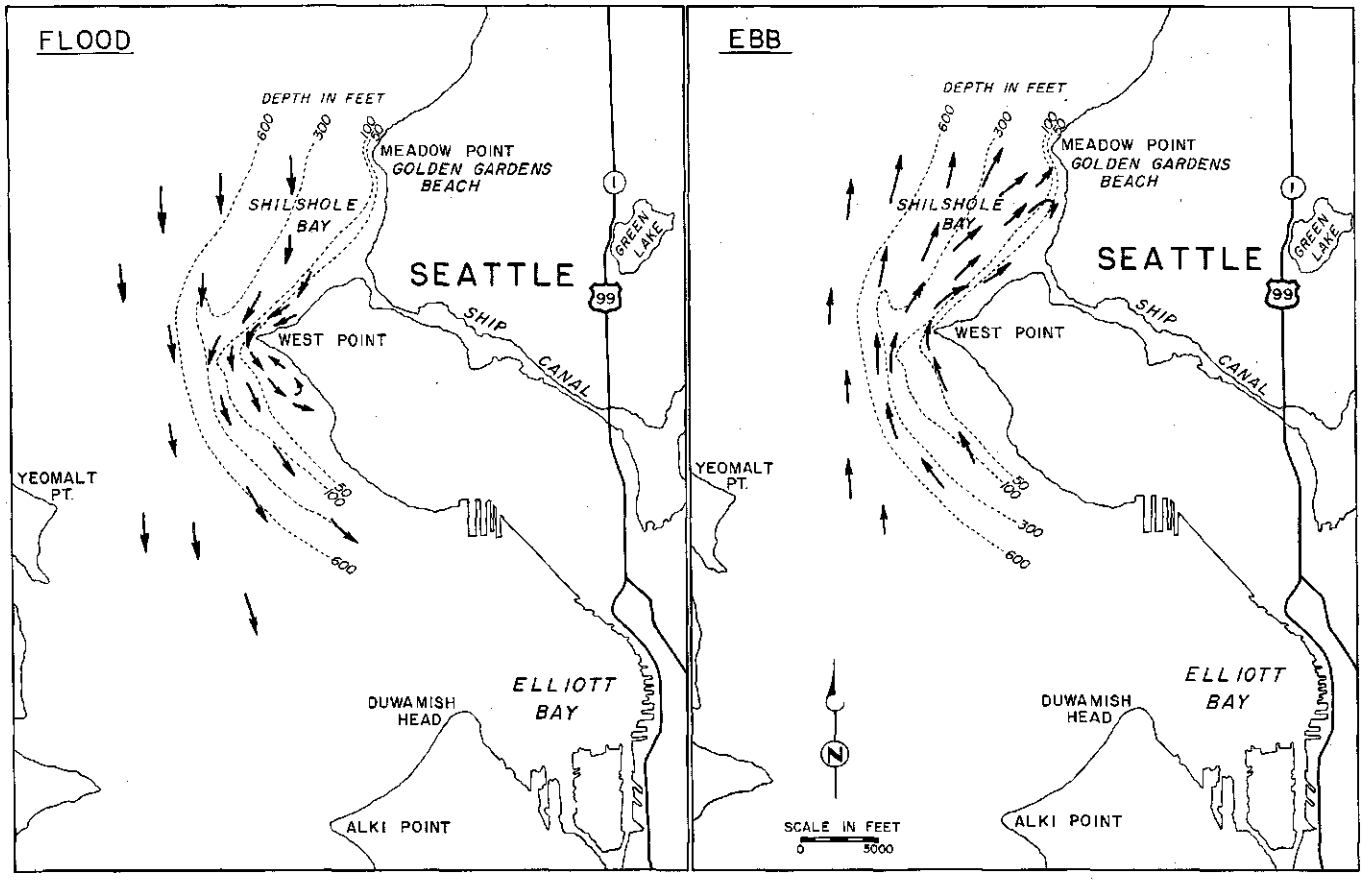


Fig. 11-13. Generalized Current Directions - West Point

Point Wells to Meadow Point. Currents in various parts of this six-mile stretch of the sound were observed under a variety of tidal conditions on four different days. Prevailing currents, both surface and deep, are parallel to shore (Fig. 11-12). Current reversals, due to the change in direction of tidal movement, take place within an hour. Accordingly, periods of slack water, during which there is little current action, are quite short. Onshore currents develop as a part of the reversal in direction.

Significant variations in current velocity occur with depth and tidal range. Maximum surface velocities, averaging 70 feet per minute (fpm), were observed parallel to shore on a flood tide having a 12-foot range. Under the same tidal condition, velocities averaging 35 fpm and 30 fpm were observed at depths of 100 and 200 feet. Currents directly onshore, as well as the onshore component of currents at an angle to shore, were in the 10 to 15 fpm range.

To compensate for higher velocities during tidal ranges greater than those observed, it is appropriate to increase the observed maximum velocities for use in calculating minimum travel times. With this factor of safety, representing approximately a 20 per cent increase, maximum velocities would amount to 83 fpm, or 5,000 feet per hour, parallel to shore and to 20 fpm, or 1,200 feet per hour, onshore.

West Point. Currents in the vicinity of West Point were observed during eight different cruises. At distances 5,000 feet and farther west of the point, prevailing current directions were parallel to the north-south axis of the sound during both ebb and flood tide conditions.

Closer to shore, the influence of West Point is quite pronounced (Fig. 11-13). Sea water, moving southward into the area on a rising or flood tide, is deflected southwesterly parallel to the shore of Shilshole Bay. Off West Point itself, as this moving mass comes under the influence of the prevailing southerly current in the sound, the major portion is deflected around the point and flows parallel to shore in a southeasterly direction toward Elliott Bay. Under certain tidal conditions, the momentum of the water mass deflected southwesterly is sufficient for some of it to maintain its direction for a considerable distance into the sound and toward Bainbridge Island. At the same time, an area of eddy currents develops off West Point. In any case, when velocities of the water mass moving around West Point are relatively high, an extensive onshore

eddy also develops in the shallow water along the south side of the point.

During the ebb, the water passing close to West Point swings northeasterly into Shilshole Bay, while the main current farther offshore continues in a northerly direction. Under such conditions, water passing close to West Point is directed toward Meadow Point and the Golden Gardens beach area. On reaching shallow water, the water mass divides, with some eddying southward close to shore and some continuing around Meadow Point.

Velocities in excess of 100 fpm were observed frequently in the surface layers under both ebb and flood conditions. During the latter stage, maximum currents of 160 fpm were observed in the surface layers as nearshore water was swept outward around West Point. At depths of 100 and 200 feet, velocities were generally lower and amounted to between 50 and 75 per cent of those in the surface layers. At times, however, the deep velocities were observed to be of the same magnitude as those of the surface layers.

The magnitude and direction of currents at a depth of 400 feet were observed during the course of ebb and flood tides having ranges of about 12 and 14 feet respectively. Velocities averaging 70 fpm were noted for both stages at this depth. Since similar velocities were indicated by drags at a depth of 100 feet during the same period, it appears that fairly uniform rates prevail throughout a considerable depth off West Point.

Relatively few onshore currents were observed in the surface layers. Those which were seen were generally associated with eddying conditions relatively close to shore both north and south of the point. Based on these observations and on the onshore components of currents at an angle to shore, the average onshore velocity was slightly less than 20 fpm. Use of this value for calculating travel time directly onshore, provides some allowance for the higher velocities which might occur on higher tidal ranges.

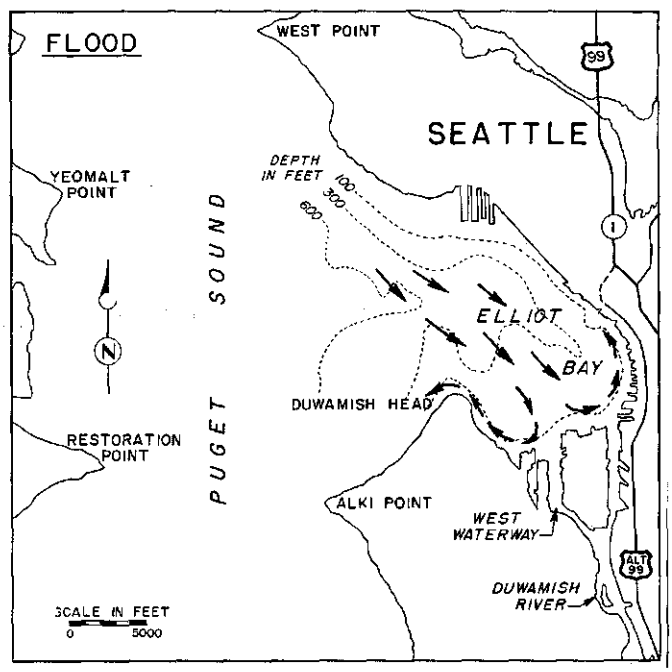


Fig. 11-14. Generalized Current Directions - Elliott Bay

Minimum travel times from directly west of the point to locations both north and south along the shore were observed not to vary greatly with respect to initial distance offshore. Within the distances to which an outfall sewer may extend, it appears that a minimum of 4.5 hours would be required for movement of the surface waters to Golden Gardens beach. For the North Trunk outfall in Shilshole Bay, the minimum travel time to Golden Gardens appears to be about 4 hours. Although observations indicate that such a possibility is rather remote, it should be noted that minimum travel time from the area directly west of West Point to Duwamish Head and Alki Point could be as low as 5.5 hours. Similarly, the time to Yeomalt Point on Bainbridge Island could be as low as 7 hours.

Elliott Bay. Cruises in Elliott Bay were made on two days and covered as wide a range as possible in tidal conditions. On the first day, the ranges in ebb and flood tides were 2.2 and 5.0 feet. Those on the second day were 13.5 and 14.5 feet. Observations on these two days served, therefore, to define the probable extremes of current velocity and water mass movement in the bay.

Under the maximum tidal range, nearshore currents observed at flood stage in both surface and deep waters were parallel to shore on each side of the bay and were directed away from the lower end of the bay (Fig. 11-14). Longshore flows appeared to originate off the mouth of West Waterway of Duwamish River.

Under the minimum tidal range, currents were extremely slow and erratic, although longshore movement away from the lower end of the bay was noted at flood tide. Deep currents in the central part of the bay were not observed. It appears, nevertheless, that they must be directed toward the lower end of the bay. In addition to the counter flow along the shore of the bay, some upwelling probably occurs at the lower end.

During the period of maximum tidal variation, longshore current velocities toward Duwamish Head were found to range from 20 fpm to 40 fpm, apparently irrespective of depth. Under these conditions, it would be possible for a water mass to travel from the end of the bay to the shore area between Duwamish Head and Alki Point during half a tidal cycle, or about 6 hours. Onshore current velocities of 20 fpm were observed during the maximum tidal range and are appropriate for calculating minimum travel times directly onshore.

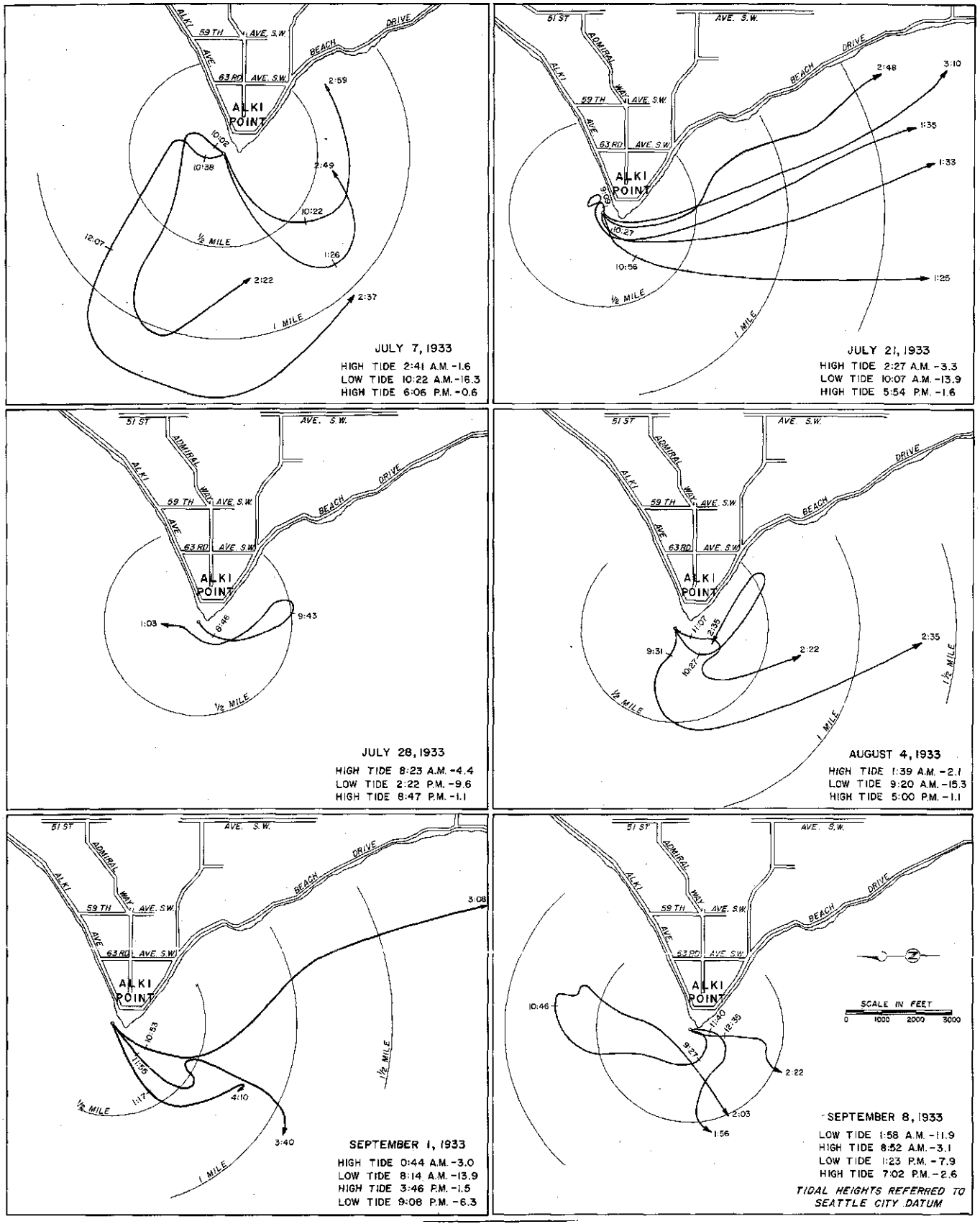


Fig. 11-15. Typical Results of 1933 Current Studies off Alki Point

Float paths shown are based on observations by city of Seattle personnel during the summer of 1933.

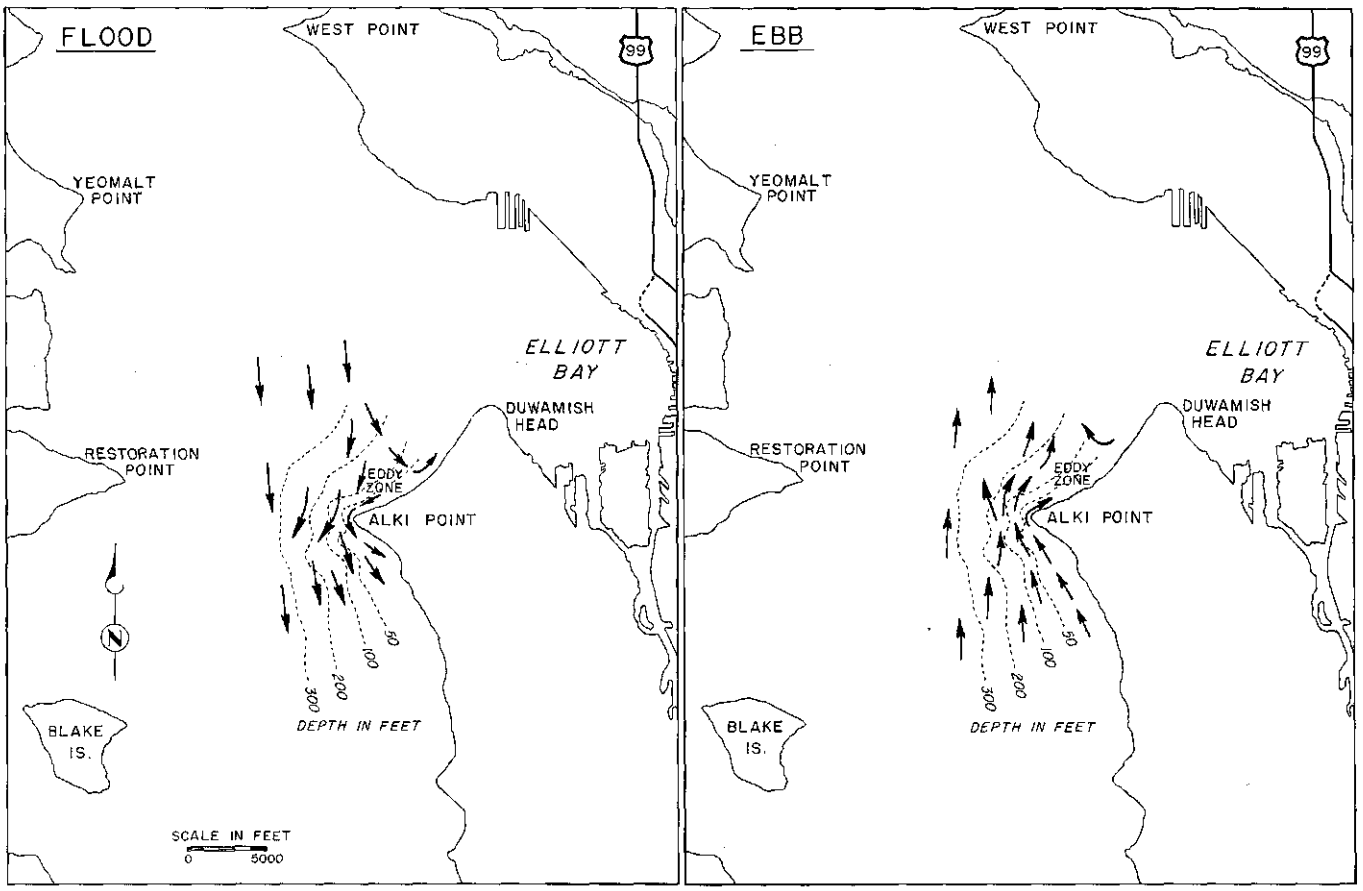


Fig. 11-16. Generalized Current Directions - Alki Point

Alki Point. Extensive studies of surface currents in the vicinity of Alki Point and Duwamish Head were made by the city of Seattle during the summer months of 1933. Of a total of 25 days of observation, 13 were in the immediate vicinity of Alki Point and were carried out under ebb and flood tides of both large and small ranges. Because of the information thus made available, only two current study cruises were made off Alki Point during the present survey. These were conducted under ebb and flood tides having fairly large ranges and covered areas farther offshore than the earlier studies. Results of six of the 1933 studies, as plotted in Fig. 11-15, are typical of conditions observed at that time. Generalized circulation patterns plotted in Fig. 11-16 are based on results of both the 1933 and 1957 studies.

Each of the two studies indicated the existence at certain times of onshore eddies north and south of Alki Point. While these eddies occur principally because the point deflects the main stream flow in the sound, the circulation pattern is further affected by Duwamish Head. On both tidal stages, eddies develop frequently in the onshore waters between these two locations. During flood tides, however, there is evidence of eddy action south of Alki Point. Water masses which are more than 1,500 feet offshore as they pass Alki Point do not swing toward shore or become involved in the eddying action.

Onshore current paths indicate that travel times to shore from possible submarine outfalls in the vicinity of Alki Point will be governed not only by current velocity and distance offshore but by their location with respect to Alki Point. Travel times from two locations are shown in Fig. 11-17 for various distances offshore. The two locations are (1) directly west of Alki Point, and (2) southwest of the site of the sewage treatment plant which is now being constructed just south of the point. While considerable scatter exists in the plotted points, it is evident from the trend lines that the time to shore from equal distances offshore is considerably greater from locations west of the point than it is from locations southwest of the treatment plant site.

Salmon Creek. Observations of current conditions in the vicinity of Salmon Creek were made on two days, each of which had moderate ebb and flood tides, ranging from 6 to 10 feet. Salmon Creek discharges into a cove which, to some extent, is isolated from the midstream currents in the sound by Brace Point

to the north and Point Pulley to the south. As a consequence, current velocities are lower than those in midchannel. Current directions may be erratic, particularly during periods of reversal. Onshore currents were observed during these periods.

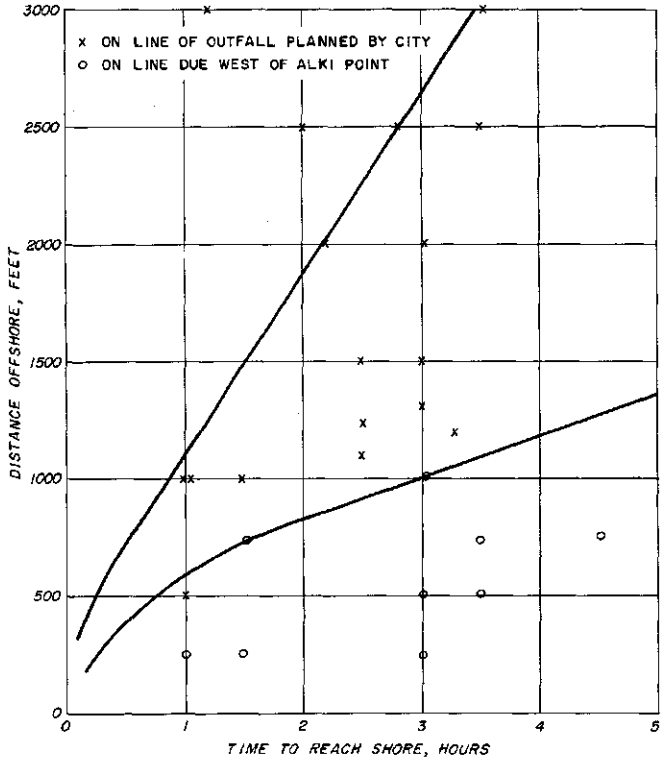


Fig. 11-17. Minimum Travel Times to Shore from Locations off Alki Point

Time to shore is considerably greater from locations due west of Alki Point than it is from locations southwest of the point.

Circulation patterns in this part of the sound are known to be greatly affected by the nearly continuous northerly flow in Colvos Passage. To compensate for this flow, a net transport toward the south must occur in the East Passage. As might be expected, therefore, observed longshore currents (Fig. 11-18) were much stronger in a southerly direction on the flood than in a northerly direction on the ebb. Velocities in the surface layer averaged about 30 fpm on the flood, as compared to about 5 fpm on the ebb. At depths of 100 to 200 feet, velocities averaged about 10 fpm during both tidal stages. Onshore currents, confined to the time of slack water, were observed to average about 13 fpm in the surface layer and 10 fpm at depths of 100 to 200 feet.

For the purpose of calculating minimum travel times from the point of discharge of a submarine outfall to locations along the shore, it is appropriate to increase longshore velocities to 33 fpm, in a southerly direction and to 6 fpm in a northerly direction. In view, furthermore, of the important beneficial water uses along this stretch of the sound, it is reasonable to provide a somewhat greater factor of safety for onshore currents than for longshore currents. A velocity of 20 fpm, therefore, is considered appropriate for onshore currents in this area.

Point Pulley. Current studies in the vicinity of Point Pulley were made on three days and covered flood tide ranges from 14.2 feet to 4.9 feet and ebb tide ranges from 13.2 feet to 2.8 feet.

Eddy activity similar to that noted at both West Point and Alki Point was found to occur on each side of Point Pulley. Currents observed off the point itself

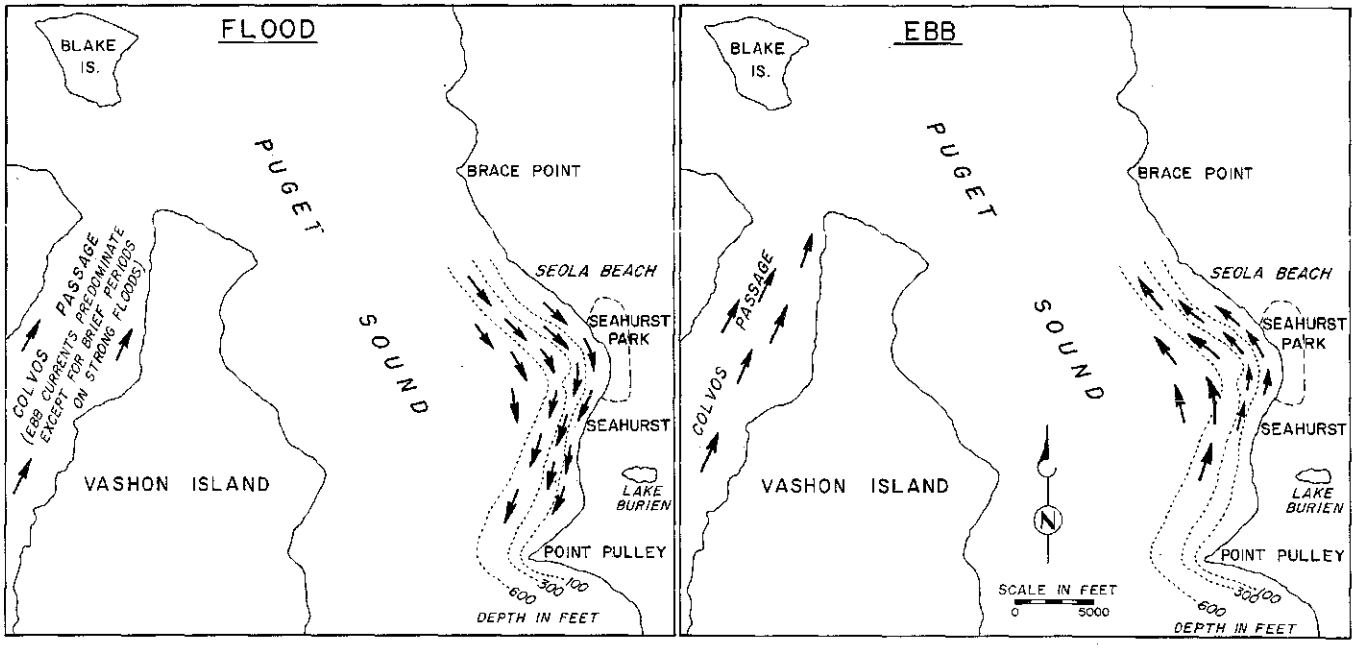


Fig. 11-18. Generalized Current Directions - Salmon Creek

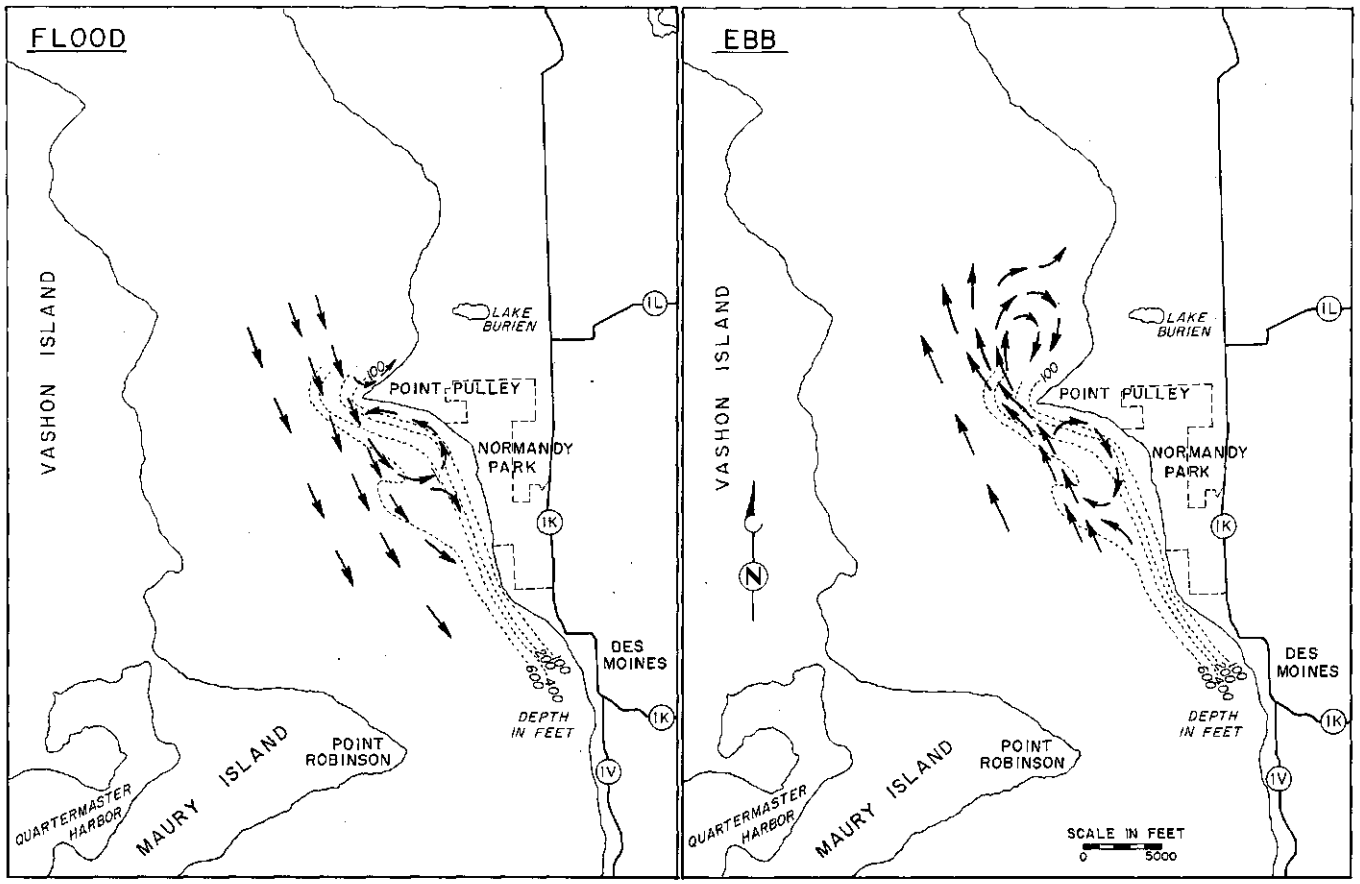


Fig. 11-19. Generalized Current Directions – Point Pulley

were rather erratic, particularly during lower tidal ranges. At these times, as might be expected from the general circulation pattern in this part of the sound, it appears that currents toward the south, which develop on flood tides, persist throughout a considerable length of the subsequent ebb. Because of the erratic conditions during low tidal ranges, the generalized current directions plotted in Fig. 11-19 are typical only of conditions during moderate to high ranges.

In the vicinity of Point Pulley, the time required to reach shore varies greatly. An outfall extending offshore from Miller Creek or Normandy Park would discharge into an area where onshore eddies develop during the flood stage and where, as a result, sewage effluent could be carried directly onshore in a rather short time. On the other hand, an outfall due west of Point Pulley would discharge into waters nearer midchannel. While some of these waters swing easterly on the flood and enter the eddy area south of the point, the tendency thereto decreases with increasing distance off the point and the time of travel to shore increases correspondingly.

Observed shoreward velocities in the eddy zone south of Point Pulley averaged less than 20 fpm, indicating that a velocity of 20 fpm is appropriate for

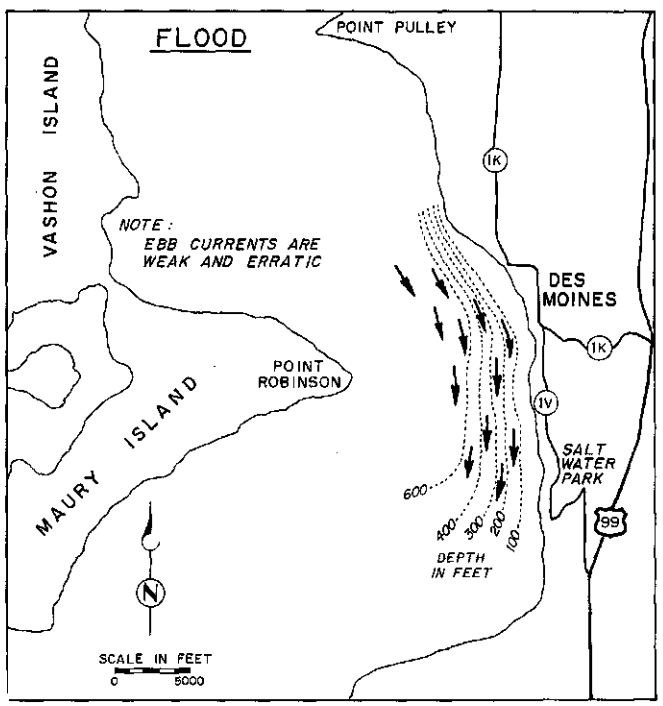


Fig. 11-20. Generalized Current Directions - Des Moines

Table 11-1. Sample Calculations of Outfall Performance

	Case I		Case II	
Distance offshore, feet (L)	1,350		2,750	
Depth, feet (Y)	100		250	
Peak flow, mgd, Fig. 11-1	193		193	
Diffuser ports, number (n)	8		8	
Diameter, inches (d)	24		24	
Spacing, feet (Y/4)	25		62	
Ratio of depth to outlet diameter (Y/d)	50		125	
Initial dilution, Fig. 11-6 (S_o)	32		115	
Onshore velocity, fpm (v_s)	20		20	
Time to shore, hours ($t = L/60v_s$)	1.1		2.3	
Initial field width, feet ($b = nY/4$)	200		500	
	High C_R	Low C_R	High C_R	Low C_R
Coliform concentration ratio, Fig. 11-8 (C_R)	8.2	4.3	14	6.5
Shore count, coliform organisms per 100 ml (Eff. count[a]/S_oC_R)				
After primary treatment				
Without disinfection	190,000	362,000	31,000	67,000
With disinfection[b]	950	1,810	155	335
After secondary treatment				
Without disinfection	19,000	36,200	3,100	6,700
With disinfection[b]	95	181	15	33

[a]*Concentration of coliform organisms assumed to be 5 x 10^7 per 100 ml in primary effluent without disinfection and 5 x 10^6 per 100 ml in secondary effluent without disinfection.*

[b]*Disinfection assumed to achieve a 99.5% reduction in coliform organisms.*

calculating minimum travel times. Current paths observed west of the point indicated that the time to shore may vary from about 2 hours from a position 1,000 feet offshore to over 4 hours from 2,000 feet offshore.

Des Moines. Current studies in the area from Des Moines south to Salt Water State Park were made on one day only. During that day, the tidal ranges were 10.1 feet for the ebb and 12.5 feet for the flood stage.

Shoreline topography in this area is roughly comparable to that in the vicinity of Salmon Creek, indicating that somewhat similar longshore current patterns can be expected (Fig. 11-20). On the other hand, the area offshore from Des Moines is not quite as isolated from the main current stream in the sound. For that reason, and also because the cross-sectional area of the East Passage is smaller at this location, current velocities can be expected to be somewhat higher.

Although a slight northerly transport develops on the ebb stage, currents are sluggish and tend to be erratic. Offshore movement seen during the last of the ebb appeared to be part of a gradual reversal in direction between ebb and flood stages. On the flood, however, a more definite longshore movement was noted. Longshore velocities both in the surface layers and in deeper water averaged a little less than 50 fpm. This value, therefore, is appropriate to use in calculating minimum travel times.

Directly onshore currents were not observed in this area. It is reasonable, nonetheless, to assume that they do develop. For the purpose of calculating minimum travel times, a velocity of 20 fpm, as derived for other locations on the East Passage, is appropriate.

ANALYSIS OF POSSIBLE DISPOSAL SITES

Preceding sections of this chapter have been concerned with the effects of various factors controlling the disposal of sewage to Puget Sound. At any given site, the magnitude of many of these effects is dependent on additional factors such as submarine topography, outfall location, and quantity of waste to be discharged. In order, therefore, to determine the degree of treatment required at each site, as well as the appropriate length of submarine outfall, it is necessary to make a separate analysis of conditions at each possible location.

Methods of Analysis

In appraising the effectiveness of an outfall system, the calculated concentration of coliform organisms at shore locations was used as the primary criterion. After establishing a suitable diffuser section, based on the ultimate peak wet weather flow at each possible site (Fig. 11-1), calculations, as illustrated in Table

Table 11-2. Summary of Treatment and Disposal Requirements

Site[a]	Ultimate peak flow, mgd	Degree of treatment[b]	Outfall length, feet	Discharge depth, feet	Diffuser ports		
					Number	Diameter, inches	Spacing, feet
Richmond Beach	193	S	1,200	90	8	24	22
	78	S	1,000	70	8	18	18
Boeing Creek	7.5	P	1,400	180	4	8	45
Piper Creek	12	P	2,400	265	4	10	66
South of Piper Creek	193	S	2,800	120	8	24	30
	121	S	2,500	100	6	24	25
	48	S	1,900	50	4	18	12
West Point	660	P	3,900	210	20	24	52
	302	P	3,700	150	24	18	38
	212	P	3,500	120	8	18	30
Elliott Bay	457	P	1,400	190	8	24	48
	102	P	1,000	175	6	21	44
Alki Point, due west	60	P	1,100	210	10	12	52
existing site	60	P	2,700	310	10	12	78
	60	S	1,350	85	10	12	21
Southwest Suburban	8.5	P	2,000	300	5	8	75
	8.5	S	800	85	5	8	21
Seahurst	360	P	3,200	260	8	24	65
	360	S	1,200	80	8	24	20
Miller Creek, off creek	18	P	2,900	200	4	12	50
	18	S	1,400	40	4	12	10
off Pt. Pulley	18	P	1,300	170	4	12	42
Des Moines	16	P	2,600	320	4	12	80
	16	S	1,300	60	4	12	15
Redondo Beach	14	P	1,500	120	4	10	30

[a]*See Fig. 11-1 for locations of sites.*

[b]*P indicates primary and S indicates secondary. Effluent disinfection required at all locations.*

11-1, were made for a number of outfall lengths as follows:

1. A determination of the initial dilution which will occur while the sewage rises toward the surface (Fig. 11-6). It was assumed here that the spacing of the individual outlets or ports of the diffuser would be such that there would be no mutual interference between rising columns. It was further assumed that the most critical conditions of shore contamination would occur with a sea water density structure permitting the mixture to rise to the surface. It should be recognized, nevertheless, that the density structure and the relatively high initial dilutions available in Puget Sound often will favor submergence or resubmergence of a sewage field (Fig. 11-7).

2. A determination of the probable range of subsequent dilution and coliform organism reduction as the sewage-sea water mixture diffuses and spreads in the surface layer of sea water (Fig. 11-8). For the diffuser section at each site, the width of the sewage field is dependent upon the length of the path which the rising mixture follows in reaching the surface. For the relatively deep discharges which can be achieved in the sound, the path length is only slightly greater than the depth. For present purposes, therefore, the two were assumed equal. Time after discharge was determined by the time of travel from the offshore point of discharge to the particular onshore location for which the calculations were made.

3. A determination of the probable range in shore count or onshore concentrations of coliform organisms. This was accomplished by dividing the concentration of organisms in the effluent by the product of initial dilution and coliform concentration ratios. In these calculations, it was assumed that the concentration of coliform organisms contained in an undisinfected effluent from plants providing primary treatment would be 5×10^7 per 100 cc. It was further assumed that a 90 per cent reduction in this concentration would occur if secondary treatment were provided prior to disposal. While higher values are fairly common, a 99.5 per cent kill of coliform organisms

contained in primary effluent was assumed as being the maximum reduction which may be obtained by disinfection in routine operation. By assuming a similar reduction for disinfection of secondary effluent, an additional measure of protection is provided because more uniform kill can be expected in this type of treatment.

After completing the calculations, the probable range in shore counts resulting from discharges at various distances from shore was plotted graphically and compared with the probable governmental requirements for both present and anticipated future beneficial water uses (Fig. 11-1). In determining the appropriate outfall length, the upper limit of the range in shore counts was used because it was based on the least favorable conditions with respect to diffusion, disappearance and mortality. In other words, shore counts will tend to be less than the applicable limit.

Table 11-2 summarizes treatment and disposal requirements at each of the possible sites herein considered for sewage treatment plant locations.

Richmond Beach

At Richmond Beach, where current studies between Point Wells, to the north, and Meadow Point, to the south, indicated comparable onshore and longshore current movements, the selection of an outfall site was based on the treatment plant location utilized in the development of alternative sewerage plans (Chapter 15).

Two shore locations are of importance from the standpoint of contamination. These are: (1) Carkeek Park, situated about 3 miles southward along the shore, where a limiting coliform concentration of 70 per 100 ml is applicable to the area used for recreational shell fishing; and (2) directly onshore where a limit of 240 per 100 ml is applicable to waters used for swimming.

Travel time to Carkeek Park, as determined from the current studies, was based on a maximum longshore velocity of 83 fpm and would be about 3.2 hours from the outfall site, regardless of outfall length. Travel times directly to the shore were based on a maximum shoreward velocity of 20 fpm.

It will be seen from Table 11-2 that the Richmond Beach study involved two treatment plant alternatives, the first with an ultimate peak flow capacity of 193 mgd and the second with a capacity of 78 mgd. For the first, it will be necessary to provide for secondary treatment and for disinfection of the effluent. The required outfall would be 1,200 feet in length and would terminate with a diffuser section laid at a depth of 90 feet. Such an installation would result in satisfactory conditions both onshore and at Carkeek Park (Fig. 11-21). With the same degree of treatment, a peak flow of 78 mgd could be discharged 1,000 feet offshore at a depth of 70 feet.

Disposal of disinfected effluent from a primary type treatment plant is not considered feasible because outfalls would be required at depths in excess of 300 feet. For large diameter outfalls (about 8 feet at this site), construction costs rise markedly with depth because of restrictions on the working time of divers. At 300 feet, for example, the optimum working time

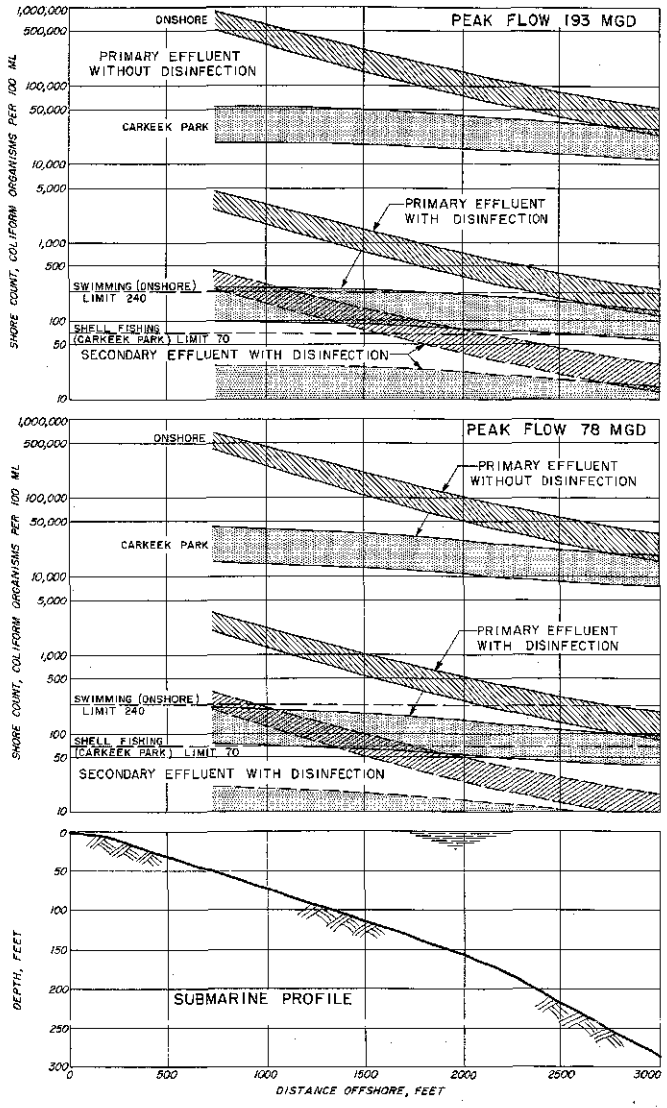


Fig. 11-21. Estimated Maximum Coliform Counts Richmond Beach Site

Calculated shore counts shown above and on the following figures are based on estimated minimum travel times to shore. In determining the required outfall length and degree of treatment to meet applicable quality standards, the upper limit of the range shown for each critical shore location was used since it reflects the least favorable conditions with respect to diffusion, disappearance and mortality. It is reasonable to expect, therefore, that actual shore counts will be lower and that the applicable standard will be approached only under adverse or extreme conditions.

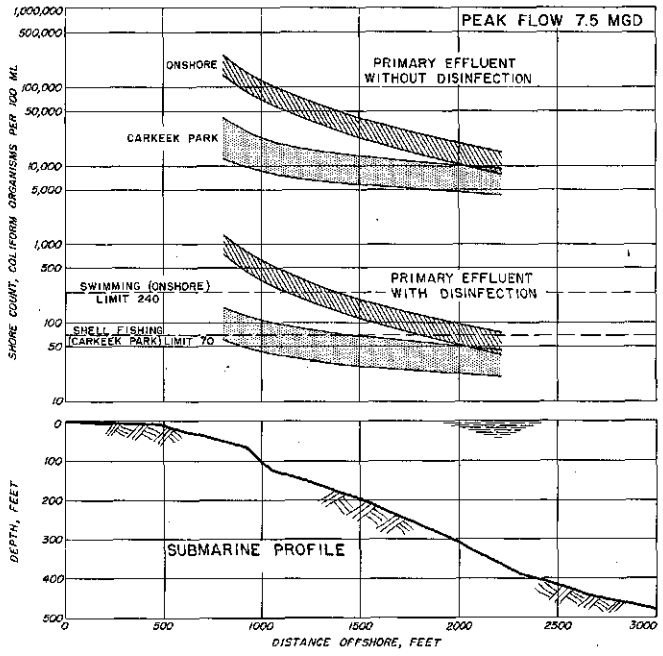


Fig. 11-22. Estimated Maximum Coliform Counts Boeing Creek Site

is 20 minutes, following which the diver must spend 160 minutes in decompression. Moreover, only one descent in 12 hours is recommended.

Boeing Creek

Since current conditions along the shores in this portion of Puget Sound are reasonably similar, it follows that outfall performance would not be affected particularly by slight variations in location. In other words, this means that the outfall can be directly offshore from the treatment plant.

Bacteriological conditions resulting from a treatment plant operation at the Boeing Creek site would be of importance both at Carkeek Park and directly onshore. For these locations, the applicable limits for coliform concentrations are 70 per 100 ml at Carkeek Park and 240 per 100 ml directly onshore. The latter limit is based on the fact that long-range plans for recreational facilities call for development of a public swimming beach adjacent to the mouth of Boeing Creek.

Minimum travel time to Carkeek Park, about 2.5 miles southward, was based on a longshore velocity of 83 fpm and would be about 2.6 hours from the outfall site regardless of outfall length. Minimum onshore travel time was based on a maximum shoreward velocity of 20 fpm.

A sewage treatment plant at the Boeing Creek site would have an ultimate peak flow capacity of 7.5 mgd and would have to provide primary treatment plus disinfection of the effluent. Maintenance of satisfactory conditions both at Carkeek Park and directly onshore would require disposal of effluent through an outfall having a length of 1,400 feet and terminating in a diffuser section at a depth of 180 feet (Fig. 11-22).

Piper Creek

An outfall from a treatment plant site adjacent to Piper Creek could be laid directly offshore from the mouth of the creek. Slight variations in location would not affect its performance.

Areas of importance from the standpoint of bacteriological contamination are Carkeek Park, directly onshore, and Golden Gardens beach, which is southward around the tip of Meadow Point. Applicable limits for coliform concentrations are 70 per 100 ml at Carkeek Park and 240 per 100 ml at Golden Gardens.

Travel times to Carkeek Park from various distances offshore were calculated on the basis of a maximum onshore velocity of 20 fpm. Conditions observed during the current studies indicated that water masses possibly could travel from the area offshore of Piper Creek to the vicinity of Golden Gardens beach in a minimum time of 1 hour.

For a sewerage plan involving disposal of the peak flow of 12 mgd offshore from the mouth of Piper Creek, primary treatment and effluent disinfection would be required. Disposal of the effluent through an outfall having a length of 2,400 feet and terminating in a diffuser section at a depth of 265 feet would result in satisfactory conditions both onshore at Carkeek Park and to the south at Golden Gardens beach (Fig. 11-23). An outfall to accommodate this flow would be about

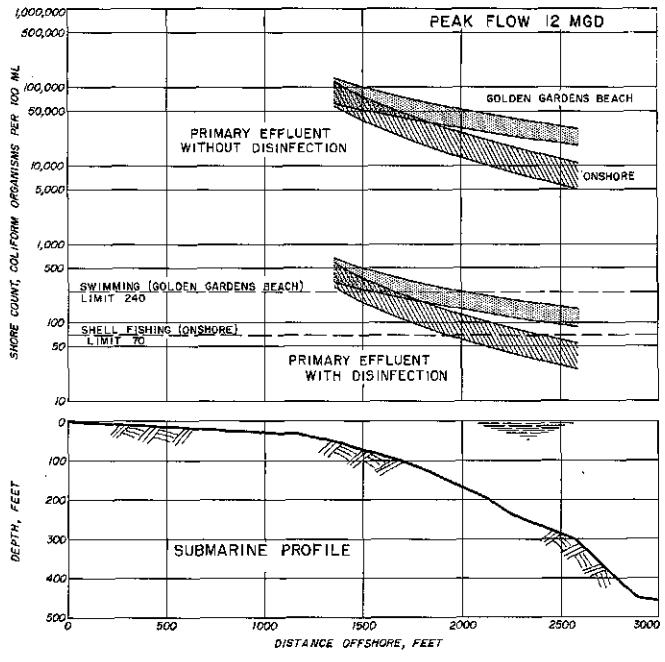


Fig. 11-23. Estimated Maximum Coliform Counts Piper Creek Site

3 feet in diameter and could be pulled into place, thus minimizing diver time.

South of Piper Creek

Several of the sewerage alternatives for various sections of the northern part of the Lake Washington watershed envision construction of a treatment plant within that area. Effluent from such a plant would be conveyed westward through a tunnel and discharged to Puget Sound at a site south of the mouth of Piper Creek. This means that Carkeek Park and Golden Gardens beach again would be the areas of importance from the standpoint of bacteriological quality. As before, the applicable limits for coliform concentrations are 70 per 100 ml at Carkeek Park and 240 per 100 ml at Golden Gardens.

Travel times to Carkeek Park were calculated by using components composed of a longshore velocity of 50 fpm and an onshore velocity of 20 fpm. A travel time to Golden Gardens beach of 1 hour was considered to represent the minimum time to that area.

For any of the three alternatives, secondary treatment and effluent disinfection would be required to assure satisfactory conditions at Both Carkeek Park and Golden Gardens (Fig. 11-24). Required outfall lengths and depths of the diffuser sections would be as follows:

Peak flow, mgd	Length, feet	Depth, feet
193	2,800	120
121	2,500	100
48	1,900	50

Disposal of disinfected effluent from a primary treatment plant would not be feasible (Fig. 11-24). This is because outfalls would be required at depths which would be excessive from the standpoints of construction and cost.

West Point

In the vicinity of West Point, the most suitable location for an outfall is in an area southwest of the point itself. There, relatively shallow water extends a greater distance offshore than at any other potentially useable location, thus making it possible to approach the midstream currents in the sound with a minimum of deep water construction. For comparable depths of discharge, the travel time to shore would be greater here than anywhere else in the West Point area.

Two shore locations are of importance from the standpoint of bacterial contamination. These are Golden Gardens beach and the area directly onshore at West Point. In the former, where swimming is the highest beneficial water use, the limiting coliform concentration is 240 per 100 ml. In the onshore location, a limit of 1,000 per 100 ml is presently applicable. Treatment and outfall requirements, however, were based on a limit of 240 per 100 ml. This is because long-range planning for park facilities (Fig. 4-11) envisions ultimate development of a swimming area at the onshore site.

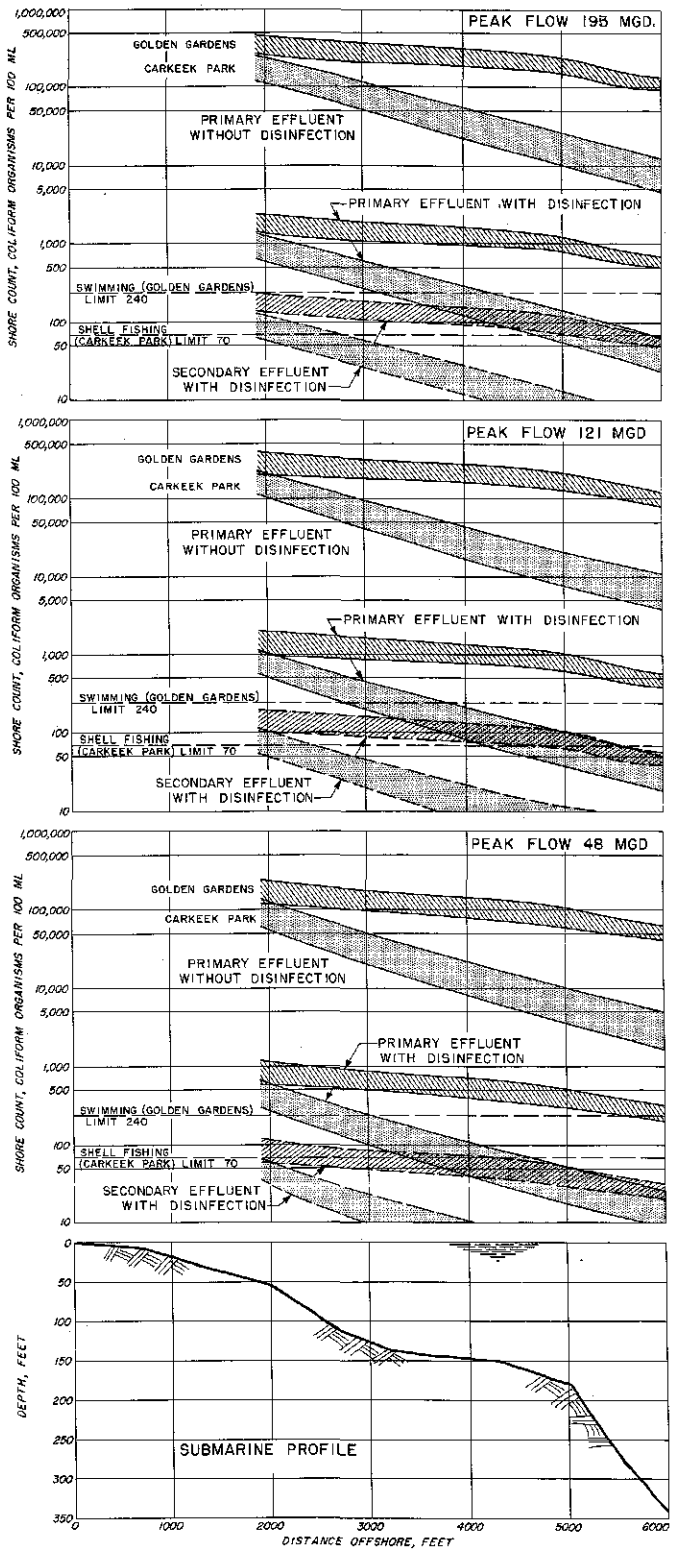


Fig. 11-24. Estimated Maximum Coliform Counts Site South of Piper Creek

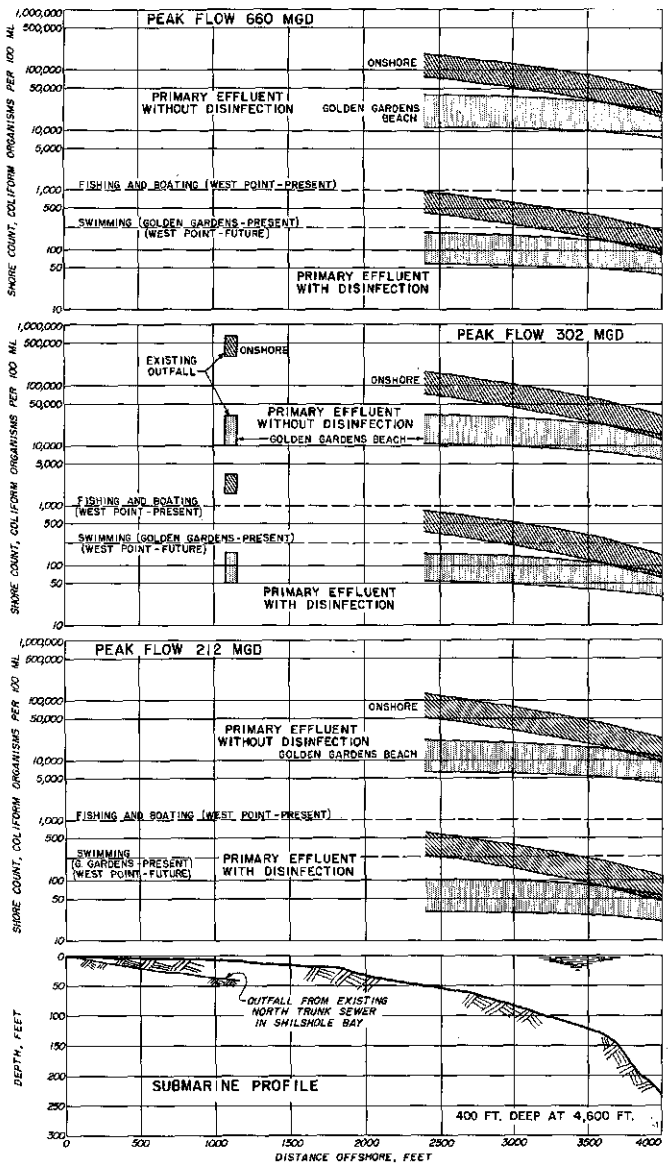


Fig. 11-25. Estimated Maximum Coliform Counts West Point Site

Travel times directly onshore were computed for a velocity of 20 fpm. As reported earlier in discussing results of the current studies, the travel time to Golden Gardens beach was taken as 4.5 hours.

Coliform counts for each of the three alternative plans at the West Point site were calculated for each of the two locations in question. As illustrated in Fig. 11-25 for various outfall lengths, acceptable shore counts at Golden Gardens could be achieved by using shorter outfalls than will be needed to satisfy the anticipated future limit of 240 per 100 ml for the onshore location. The latter criterion, however, was used in determining outfall requirements.

Primary treatment, coupled with effluent disinfection, would be required in each of the three cases. Outfall lengths and depths of the diffuser sections would be as follows:

Peak flow, mgd	Length, feet	Depth, feet
660	3,900	210
302	3,700	150
212	3,500	120

Fig. 11-25 also shows results of calculations based on using the existing outfall of the North Trunk sewer in Shilshole Bay. At the indicated peak flow of 302 mgd, primary treatment and disinfection would not be capable of reducing the shore concentration of coliform organisms to even the presently acceptable limit.

Elliott Bay

Two of the alternative sewerage plans involve disposal to Elliott Bay at a site immediately north of the East Waterway. Since the highest beneficial use of the bay is for fishing and boating, the applicable limit

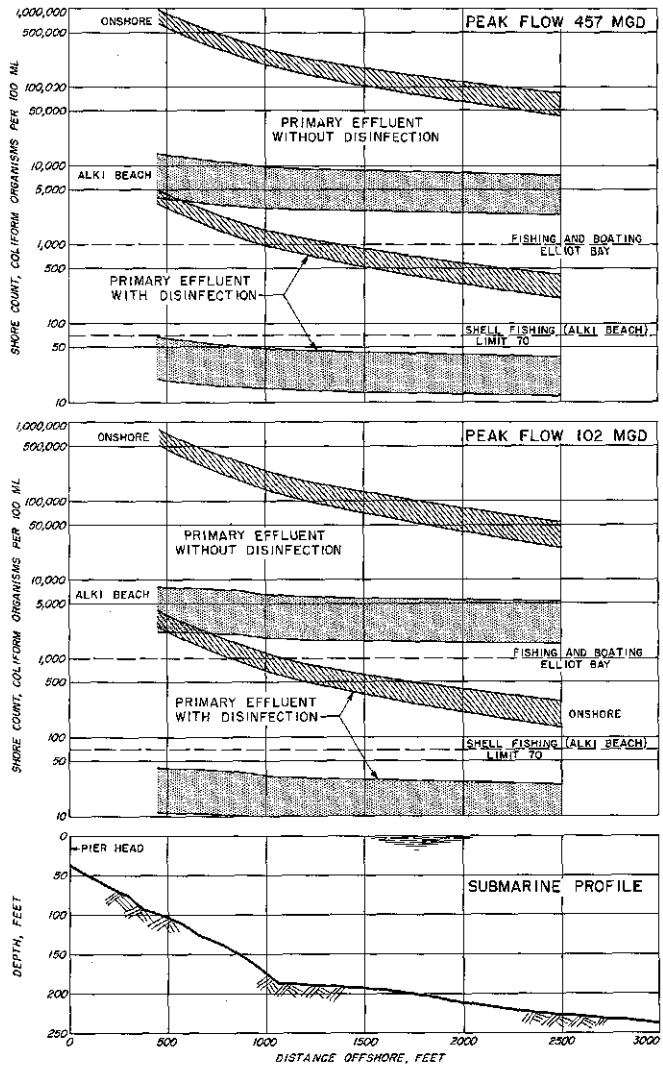


Fig. 11-26. Estimated Maximum Coliform Counts Elliott Bay Site

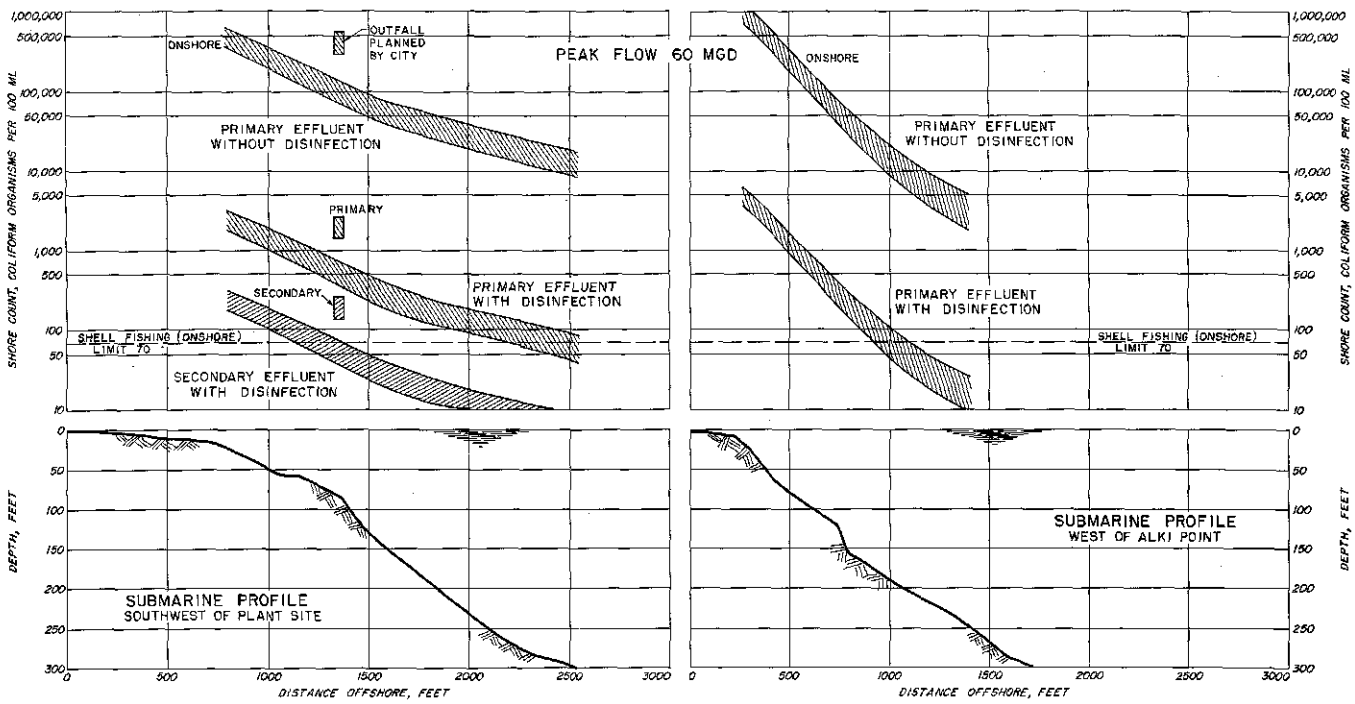


Fig. 11-27. Estimated Maximum Coliform Counts - Alki Point Site

for the concentration of coliform bacteria is 1,000 per 100 ml.

Under certain tidal conditions, the sewage-sea water mixture possibly could be carried to the vicinity of Duwamish Head. Between Duwamish Head and Alki Point, where the shore waters are used for swimming, the applicable coliform limit would be 240 per 100 ml.

Travel times toward the shores of Elliott Bay were calculated using a velocity of 20 fpm. In a direction parallel to shore, it was considered possible that sewage effluent could be carried to Duwamish Head or beyond in 6 hours, which is approximately half of a tidal cycle.

Primary treatment and effluent disinfection would be necessary for either of the two alternatives (Fig. 11-26). A peak flow of 457 mgd would require an outfall 1,400 feet long, terminating in a diffuser section at a depth of 190 feet. An outfall 1,000 feet long terminating at a depth of 175 feet would be sufficient for a peak flow of 102 mgd.

Alki Point

As determined by the current studies, the most suitable location for an outfall in the vicinity of Alki Point would be directly west of the point itself. For comparable distances from shore, the travel times to shore would be considerably longer from this location than they would from a site offshore from the treatment plant now under construction (Fig. 11-17).

Between Alki Point and Duwamish Head, the limiting onshore coliform concentration is 240 per 100 ml. South of Alki Point, where the waters are used for recreational shell fishing, the applicable limit is 70 per 100 ml. Shore counts in the latter area, therefore, would determine the suitability of disposal operations in the vicinity of Alki Point.

Shore counts resulting from an ultimate peak flow of 60 mgd are plotted in Fig. 11-27 for each of the two possible outfall locations. Directly west of Alki Point, primary treatment and effluent disinfection, together with an outfall 1,100 feet in length and terminating in a diffuser section at a depth of 210 feet, would be sufficient to assure satisfactory onshore conditions. At the site southwest of the treatment plant, an outfall 2,700 feet long and terminating with a diffuser at a depth of 310 feet would be required. Provision of secondary treatment, along with effluent disinfection, would reduce the required length at this site to 1,350 feet.

Effluent from the primary type treatment plant now under construction at Alki Point is to be disposed of offshore from the treatment plant through an outfall having a diameter of 42 inches, a length of 1,400 feet and a terminal depth of 85 feet. As now proposed, final discharge will be through the open end of the outfall rather than through a diffuser. As a consequence, the shore counts will be higher (Fig. 11-27) than they would be for a diffuser-equipped outfall.

Southwest Suburban

At this site, consideration was given to an outfall in the same location as that of the existing outfall from

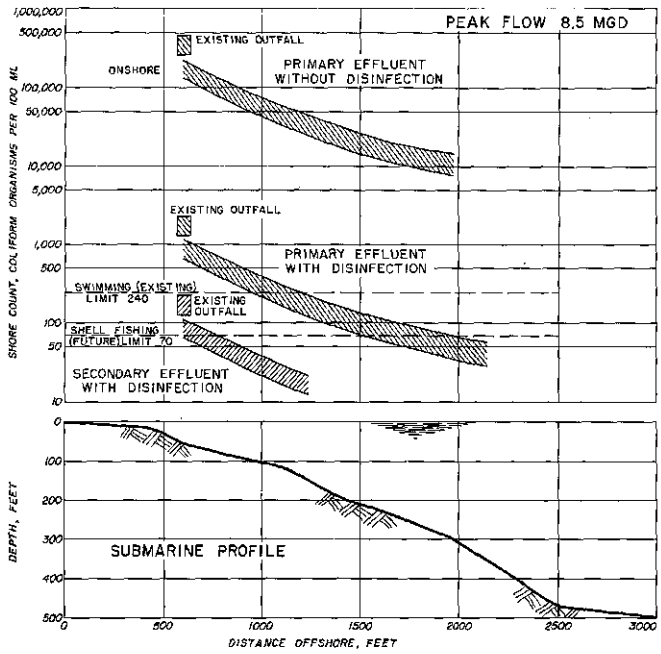


Fig. 11-28. Estimated Maximum Coliform Counts Southwest Suburban Site

the treatment plant of the Southwest Suburban Sewer District. With respect to bacteriological quality, the area directly onshore is of primary concern. Although presently utilized for swimming, long-range plans for development onshore of Seahurst Park include recreational shell fishing. For that reason, a coliform concentration of 70 per 100 ml was utilized in evaluating outfall performance.

Travel times from various distances offshore were based on a maximum onshore velocity of 20 fpm.

Data plotted in Fig. 11-28 indicate that, for the ultimate peak flow of 8.5 mgd, satisfactory shore conditions would be assured with (1) disposal of disinfected primary effluent through an outfall 2,000 feet in length terminating in a diffuser section at a depth of 300 feet, or (2) disposal of disinfected secondary effluent from an outfall 800 feet in length terminating in a diffuser section at a depth of 85 feet.

Since the existing outfall at the Southwest Suburban location is not equipped with a diffuser, results of calculations based on continued use of the outfall (Fig. 11-28) reflect shore counts higher than those for a diffuser-equipped outfall. Even if it were so equipped and provision were made for secondary treatment and disinfection, performance of the outfall would not be satisfactory at the present distance of 600 feet from shore.

Seahurst

Under one of the alternative sewerage plans laid out to serve various sections of the southern part of the Lake Washington watershed, effluent from a treatment plant located in that area would be conveyed westward and discharged to Puget Sound at a site offshore from the community of Seahurst, which is situated about one mile north of Point Pulley.

Although there are no public recreational facilities directly onshore, it is reasonable to assume a limiting coliform concentration of 240 per 100 ml. This is because the shores are accessible and a certain amount of bodily contact with the water is almost certain to occur. About two miles southward along the shore, recreational shell fishing in the vicinity of the community of Normandy Park indicates a limit of 70 per 100 ml.

Onshore travel times for various outfall lengths were based on a maximum velocity of 20 fpm in that direction. At a longshore velocity of 33 fpm, a minimum of 5 hours would be necessary to reach Normandy Park.

For the ultimate peak flow of 360 mgd, satisfactory shore conditions directly onshore and at Normandy Park would be assured by (1) disposal of disinfected primary effluent through an outfall 3,200 feet long terminating in a diffuser section at a depth of 260 feet, or (2) disposal of disinfected secondary effluent through an outfall 1,200 feet long terminating in a diffuser section at a depth of 80 feet (Fig. 11-29).

Miller Creek

Current studies in the vicinity of Miller Creek indicated that outfall performance would vary considerably with location. Two locations, therefore, were considered, one extending west of Point Pulley and the other extending southwest from the mouth of the creek.

Recreational shell fishing takes place onshore in the vicinity of the community of Normandy Park. For

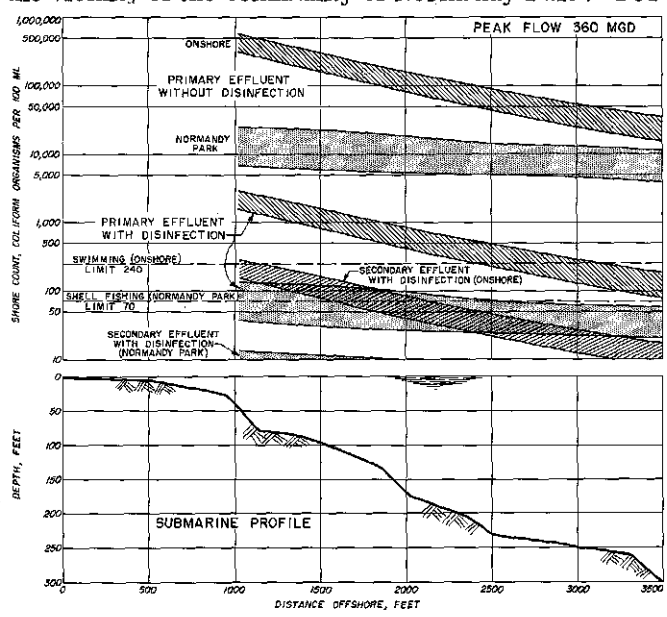


Fig. 11-29. Estimated Maximum Coliform Counts Seahurst Site

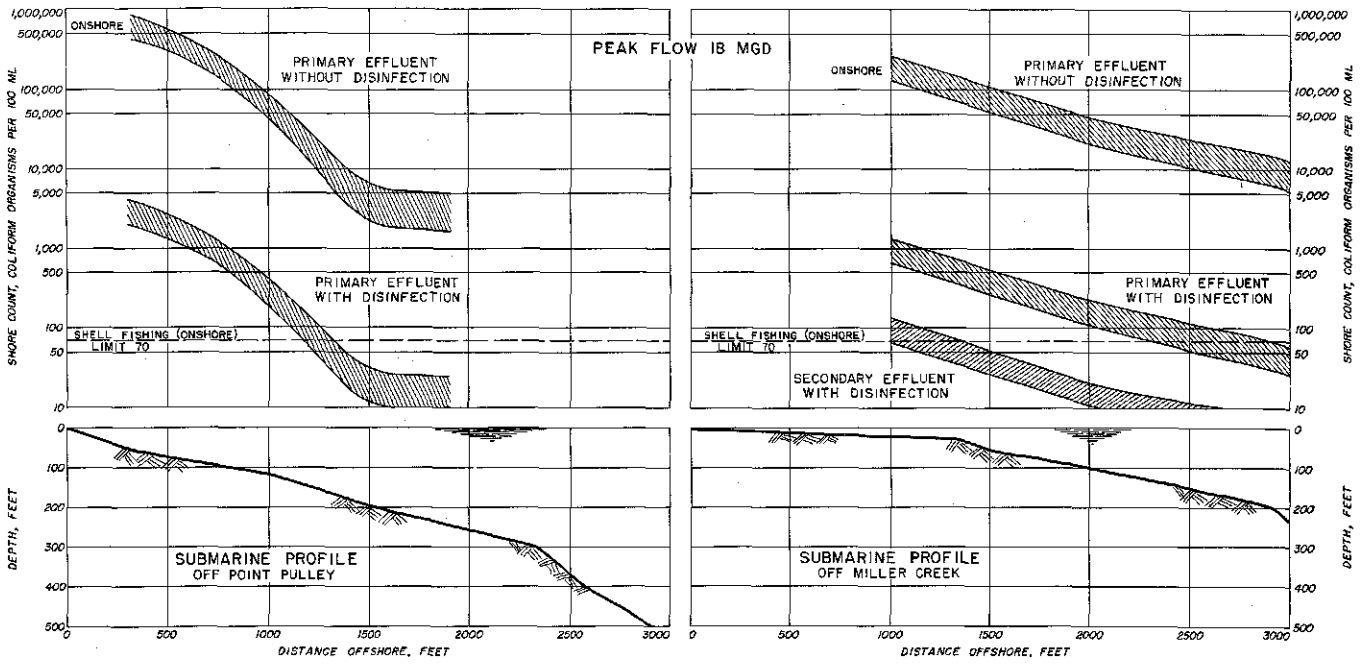


Fig. 11-30. Estimated Maximum Coliform Counts - Miller Creek Site

that reason, the applicable limit for coliform concentration would be 70 per 100 ml.

Current studies reported herein indicated that minimum travel times to critical shore areas from the location west of Point Pulley range from 2 hours to about 4 hours between 1,000 and 2,000 feet offshore. For distances of less than 1,000 feet, the estimated travel times assume a shoreward velocity of 20 fpm. At the Miller Creek site, the estimated travel times also assume a velocity of 20 fpm.

Primary treatment and effluent disinfection would be required at each of the two outfall locations (Fig. 11-30). Off Point Pulley, satisfactory conditions at the ultimate peak flow of 18 mgd could be obtained by discharge through an outfall having a length of 1,300 feet and terminating in a diffuser section at a depth of 170 feet. Off Miller Creek, a longer outfall, 2,900 feet in length and terminating with a diffuser at a depth of 200 feet, would be required. Provision of secondary treatment and effluent disinfection at the Miller Creek site would reduce the outfall length of 1,400 feet. Such an outfall would terminate at a depth of 40 feet and would be equipped with a diffuser.

Des Moines

Because of the general similarity of currents in the Des Moines area, the outfall could be constructed at a location directly offshore from the site of a sewage treatment plant. Since the shores in this area are used for recreational shell fishing, the applicable coliform limit would be 70 per 100 ml. A similar use and, as a consequence, a similar limit apply at Salt Water State Park, nearly two miles to the south.

Travel times directly onshore assume a maximum shoreward velocity of 20 fpm. Assuming a longshore velocity of 50 fpm, the minimum travel time to Salt Water Park would be about 3 hours. For a discharge less than 3,600 feet from shore, it is evident that the time of travel would be less to onshore locations

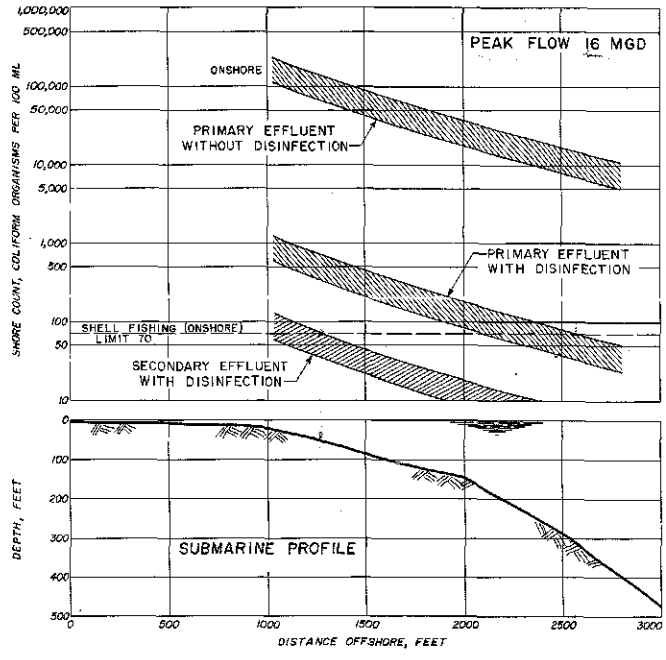


Fig. 11-31. Estimated Maximum Coliform Counts Des Moines Site

than it would be to the park. Accordingly, bacteriological conditions onshore are the more critical.

For the ultimate peak flow of 16 mgd, an outfall 2,600 feet in length and terminating in a diffuser section at a depth of 320 feet would be required for disposal of disinfected primary effluent (Fig. 11-31). A shorter outfall, 1,300 feet in length and terminating in a diffuser section at a depth of 60 feet would suffice for disposal of disinfected secondary effluent.

Redondo Beach

One of the alternative sewerage plans for the extreme southwestern portion of the metropolitan area calls for a treatment plant on the shore north of Dumas Bay near the community of Redondo. Because of generally similar current conditions in this area, the outfall could be offshore from the plant site.

Two locations are of importance from the standpoint of bacteriological contamination. Directly onshore, where swimming is the primary beneficial use, the applicable limit for coliform concentration would be 240 per 100 ml. At Salt Water Park about four miles north, where recreational shell fishing is the primary use, the limit would be 70 per 100 ml.

Travel times directly onshore assume a maximum velocity of 20 fpm. Regardless of distance from shore, the travel time to Salt Water Park, based on a longshore velocity of 50 fpm, would be about 3.5 hours.

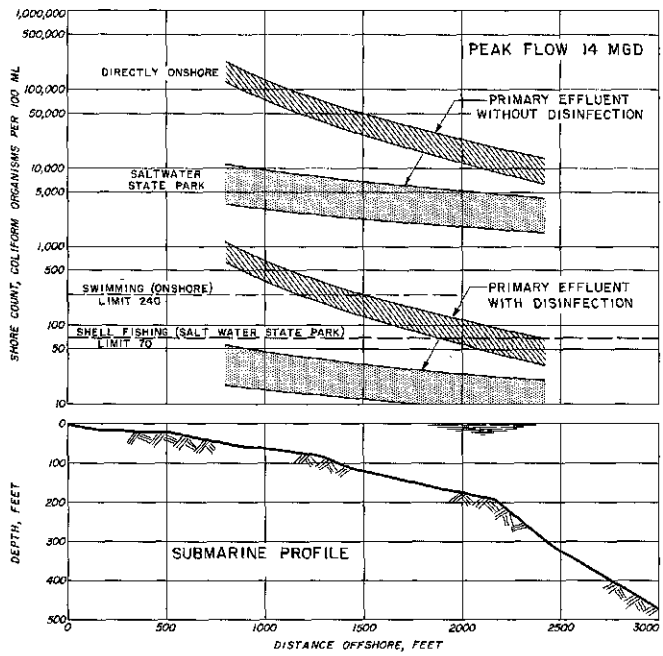


Fig. 11-32. Estimated Maximum Coliform Counts Redondo Beach Site

For disinfected primary effluent and an ultimate peak flow of 14 mgd, an outfall 1,500 feet in length and terminating in a diffuser section at a depth of 120 feet would be required (Fig. 11-32).

Chapter 12

SEWAGE DISPOSAL IN GREEN-DUWAMISH RIVER

Because the Green-Duwamish River is near to potential future industrial and residential developments, use of its waters for waste receiving purposes is of paramount importance to the metropolitan Seattle area. Its ability to act in this capacity is dependent on the beneficial water uses to be protected, the quantity and characteristics of the wastes to be disposed of, and the self-purification capacity of the river.

PHYSICAL CHARACTERISTICS OF GREEN-DUWAMISH RIVER

The Green-Duwamish river system bisects the southern portion of the metropolitan Seattle area. (Fig. 12-1). Rising in the Cascade Mountains about 40 miles east of Auburn, Green River flows westward to its confluence with Black River in the Renton-Tukwila area. Duwamish River, formed by the confluence of the Green and Black rivers, flows north for a distance of about 12 miles through the heart of Seattle's industrial area to Elliott Bay.

The last five miles of the Duwamish have been dredged for navigation and industrial shipping. To aid further development, the Port of Seattle is currently planning both for additional channel improvements and for industrial expansion in the lower Green-Duwamish basin.[1,2] Under this program, the existing dredged waterway will be extended south at a depth of 30 feet below mean lower low water (MLLW) for a distance of approximately 9 miles. A turning basin will be constructed in the vicinity of Tukwila, and a barge canal having a depth of 15 feet at MLLW will extend southward for an additional 1.6 miles. Control works, consisting of a salt water barrier, spillway and stilling basin, will be provided at the southern end of the barge channel.

According to data obtained from the Seattle District office of the U.S. Army Corps of Engineers, tidal effects in the existing system extend approximately 90,000 feet upstream from the mouth of the Duwamish River at Harbor Island. This distance, of course, will be shortened upon construction of the proposed waterway extension and salt water barrier in the vicinity of Orillia. Although Orillia is presently about 77,000 feet from the river mouth, channel realignment will reduce the distance to about 58,000 feet.

Mean monthly flows of Green River at the U.S. Geological Survey gaging station about one mile upstream from Auburn are shown in Fig. 12-2. Flows are usually at a maximum from late autumn to early summer and at a minimum in August or early September. For the period of record (22 years), the minimum monthly flow of 110 cfs was recorded in a three month period from September to November, 1952, and occurred during an exceptionally dry autumn. Operation of the proposed Eagle Gorge Dam at the head waters of the Green will increase the minimum dry weather flow to approximately 180 cfs.[3]

WATER USES AND WATER QUALITY REQUIREMENTS

As stated earlier, the conditions to be maintained in any given body of water are dependent upon the beneficial uses of that water. Disposal of wastes by dilution in river systems, while itself a beneficial use, must not have a deleterious effect on other uses, either present or potential.

Water Uses

Water uses in the Green-Duwamish river system are diversified and involve such activities as fish propagation and migration, shipping and navigation, irrigation, industrial, recreation, and waste disposal.

Fisheries is perhaps the most important resource of the Green-Duwamish drainage basin. This activity is estimated to have a present annual value of almost $800,000, of which $216,000 represents the amount spent each winter in steelhead and cutthroat sports fishing.[4] In addition, the upper reaches of Green-Duwamish provide an important spawning and nursery area. Periods of upstream and downstream migration of the several species overlap to the extent that some migration occurs almost throughout the year. Upstream migration of steelhead generally occurs from November to May, while that of salmon takes place from late September through December.

[1] *Knappen, Tippetts, Abbett, and McCarthy,* Development Plan for the Duwamish and Lower Green River Valley, *September, 1954.*

[2] *Tippetts, Abbett, McCarthy and Stratton,* Report on the Master Plan for the Port of Seattle Industrial Development District-Duwamish Valley, *June 1957.*

[3] Preliminary Examination and Survey for Flood Control of the Green-Duwamish Waterway, *U.S. Corps of Engineers, Seattle District Office, 1948.*

[4] *State of Washington Pollution Control Commission, September 1957.*

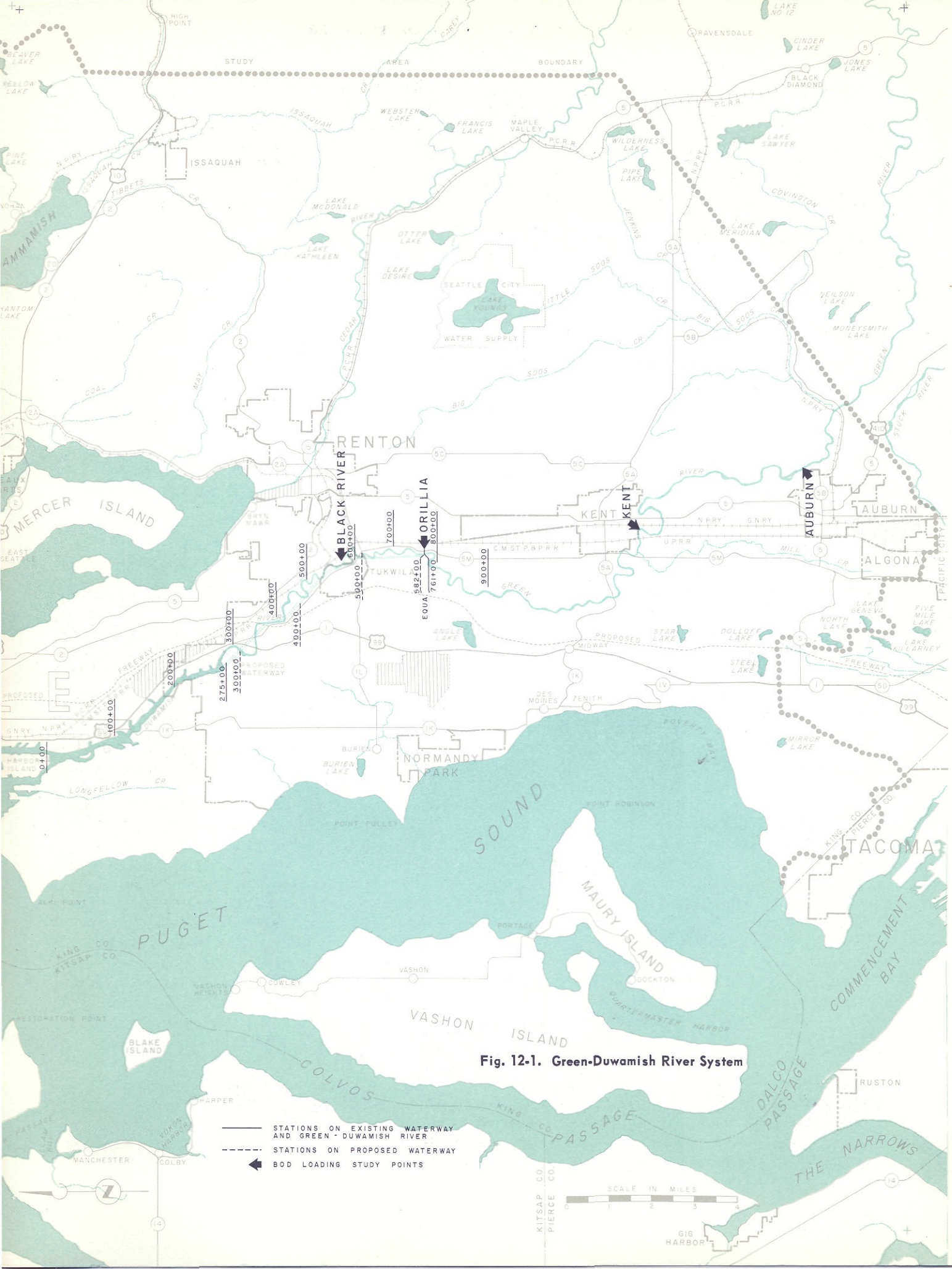


Fig. 12-1. Green-Duwamish River System

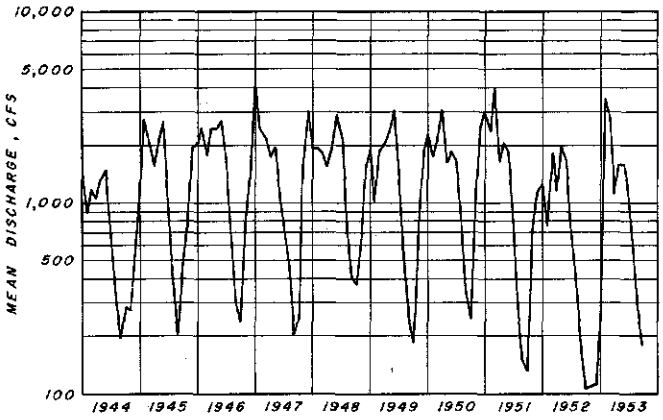


Fig. 12-2. Green River Discharge

Green River discharge has been measured since 1936 by U.S. Geological Survey at gaging station one mile upstream from Auburn.

Steelhead migrate downstream in April and May and salmon from April through July.

Extension of the dredged waterway to Orillia will extend industrial shipping and navigation uses accordingly. Recreational boating, though often referred to as a major activity in the lower Duwamish, is hardly desirable in an industrial waterway. Conditions above the waterway, of course, are such that boating can be, and actually is, one of the principal uses of the river.

Upstream from sections affected by tidal action, Green-Duwamish waters are used for crop irrigation and livestock watering. It is considered likely that much of the adjoining land in that area will continue to be used for agricultural purposes despite an expected expansion of industrial activity (Fig. 4-11). This implies, of course, that irrigation use will be a continuing requirement.

Although relatively minor at present, some use is

GREEN-DUWAMISH RIVER flows through the heart of metropolitan Seattle's industrial area. Green River originates in the Cascade Mountains about 40 miles east of Auburn. Duwamish River is formed by the confluence of the Green and Black rivers (arrow).

made of Duwamish water for industrial cooling. An increase in this use can be expected with further development of the area, particularly in the lower Duwamish.

At present, industrial wastes and sanitary sewage, both treated and untreated, are being discharged to Green-Duwamish River. Discharges of industrial waste are concentrated largely along the dredged portion of the Duwamish, while raw sewage discharges are principally at locations within the city of Seattle. Effluents from five sewage treatment plants are also disposed of in the river system (Fig. 6-1).

Water Quality Requirements

Of the beneficial water uses to be protected, those relating to fish propagation and migration and to irrigation of crops are of paramount importance. Sewage and industrial waste disposal practices which will satisfy the quality requirements for these two uses will, at the same time, satisfy all other requirements with respect to beneficial use.

Conditions inimical to fish propagation and migration have been described in Chapter 9. Controlling water quality criteria in this category are concerned with five basic factors, namely, dissolved oxygen concentration, temperature, pH, presence of toxic or other deleterious substances in harmful amounts, and the formation of organic or inorganic bottom deposits.

At present, the maintenance of a minimum dissolved oxygen level at 5.0 ppm is the controlling factor with respect to waste disposal practices. Future industrial expansion may, however, cause toxicity to become equally important. In such an event, strict control of toxic discharges will be required and, where necessary, offending industries will have to provide facilities for the pretreatment of wastes containing abnormal quantities of toxic materials.

The bacterial quality of surface waters used for irrigation of crops intended for human consumption, particularly of foods which may be eaten raw, is important from the standpoint of public health. A consistently high degree of disinfection is necessary, therefore, where sewage or sewage-laden water is used for general irrigation. For this type of use, the limiting coliform median of 50 per 100 ml applies, providing a sanitary survey indicates that the organisms could be of human origin.

EFFECTS OF PRESENT WASTE DISCHARGES

In recent years, the University of Washington, the State Pollution Control Commission and the city of Seattle have made pollution surveys and studies of the Green-Duwamish river system.[5,6] These investigations have indicated that the present BOD load entering the system is approximately 26,000 pounds

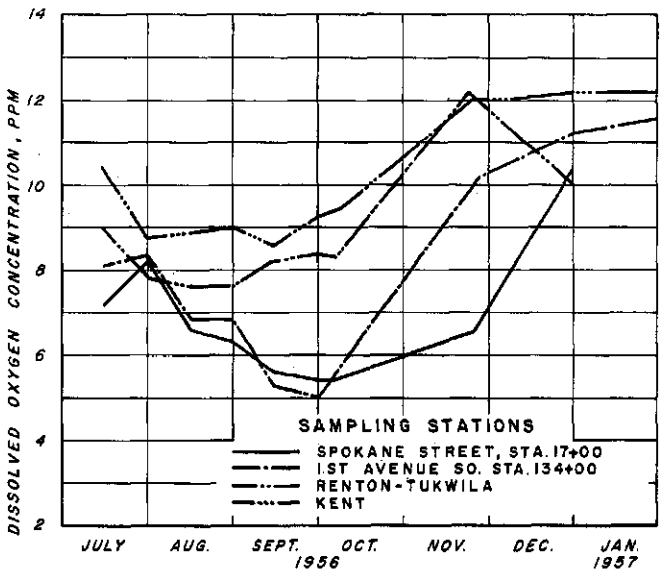


Fig. 12-3. Dissolved Oxygen Concentrations in Green-Duwamish River

Source: Okey, R. W., "A Study of Present and Future Pollutional Effects in the Green-Duwamish River", M.S. Thesis, University of Washington, 1957.

per day. Of this total, about 3,000 pounds enter the river between Auburn and Tukwila and nearly all of the remainder enters in the dredged Duwamish waterway.

Dissolved oxygen concentrations were measured at four stations during the 1956 survey by Okey (Fig. 12-3). Two of the four sampling stations, Spokane Street and First Avenue South, were in the dredged channel, while the remaining two were upstream from the channel. Comparison of Fig. 12-3 with Fig. 12-2 shows that the seasonal fluctuation in dissolved oxygen corresponds to the fluctuation in flows in Green River, with minimum concentrations occurring at the same time as minimum river flows. Although the analyses indicate no serious oxygen depletion during most of the year, the minimum value obtained in September of 1956 at the First Avenue South sampling station was 5.0 ppm, the minimum prescribed by the Pollution Control Commission for fish life. Similarly, during September and October, minimum concentrations were only slightly above the required level at the Spokane Street and First Avenue South sampling stations. These findings indicate that the river, even under the existing condition of dispersed loading, has about reached its limit for the safe disposal of sewage.

[5] *State of Washington, Pollution Control Commission,* An Investigation of Pollution in the Green-Duwamish River, *Technical Bulletin No. 20, 1955.*

[6] *Okey, R. W.,* A Study of Present and Future Pollution Effects in the Green-Duwamish River, *M.S. Thesis, University of Washington, 1957.*

CAPACITY OF GREEN-DUWAMISH RIVER TO RECEIVE SEWAGE

In a body of water such as Green-Duwamish River, the conditions required for satisfactory and innocuous disposal of sewage are:

1. A flushing action sufficient to prevent both build-up of the pollution load and deposition of suspended matter. This action results from fresh water inflow and tidal displacement or diffusion.
2. A supply of oxygen in the water sufficient to assure oxidation of organic matter in the sewage without objectionable depletion of dissolved oxygen. This supply is obtained from fresh water inflow, tidal movement, atmospheric reoxygenation, and algal photosynthesis.

Receiving water conditions are least favorable each year during August and September, at which time fresh water inflow is usually at its lowest and water temperatures are at their highest. These conditions accelerate biochemical oxygen demand and reduce dissolved oxygen.

In determining the waste receiving capacity of the Green-Duwamish river system, analyses must be made of two separate portions, each of which has a different waste receiving capacity. Of the two portions, the first is concerned with that part of the river which is above tidal influence and thus receives no oxygen supply from tidal waters. In subsequent discussion, this portion is referred to as Green River. The second portion, which will be referred to as the Duwamish estuary, is in the zone of tidal influence, which means that both oxygen resources and flushing action are affected by tidal movement. Since the physical character of the estuary will be changed by construction of the proposed industrial waterway extension, analyses must be made of both the existing and proposed estuaries.

Green River

The self-purification capacity of a stream varies with its flow. Where low flow conditions develop during the summer, waste receiving capacities are even further reduced because of increased rates of oxygen demand resulting from higher water temperatures. Pollution studies, therefore, are concerned generally with evaluating the waste receiving capacity of a stream during the period of minimum flow and maximum water temperatures.

Oxygen Resources. In the process of oxidation of organic material by bacteria, oxygen is taken from solution in the surrounding water. Usually, the quantity removed is but a small portion of that available and the depletion is quickly replenished. If, however, there is an abnormal concentration of organic material requiring oxidation, as occurs when sewage is discharged in a localized area, oxygen depletion may exceed the available supply. To avoid such a possibility, it is necessary, in planning for disposal, to consider the various sources of dissolved oxygen.

In a river such as the Green, there are three primary sources of oxygen: (1) that contained in the receiving water and in the sewage itself at the point of sewage discharge; (2) that absorbed at the water surface from the atmosphere whenever the dissolved oxygen content of the water falls below its saturation value (reaeration); and (3) that released by plankton under the influence of sunlight.

In the Green River, the largest source of oxygen is that brought in with fresh water from upland flow. According to data obtained during recent field investigations, the minimum concentration at Auburn during August and September was 8.8 ppm and maximum water temperatures were 15° to 16° C.[6] For purposes of determining waste receiving capacity, minimum values of 150 cfs and 8.5 ppm were assumed for flow and dissolved oxygen in the river at Auburn.

All natural bodies of water absorb oxygen from the atmosphere. The rate at which absorption takes place is largely dependent on oxygen deficiency and the turbulence and intimacy of contact between the oxygen deficient waters and the atmosphere. This rate can be described in terms of oxygen deficiency by the formula:

$$D_t = 10^{-k_2 t} D$$

where D_t is the dissolved oxygen deficit at any time t, D is the original oxygen deficit, and k_2 is the reaeration constant. The reaeration constant varies from stream to stream, and even from section to section of a given stream, and is influenced by many factors, including the depth-surface area relationship, vertical and horizontal mixing, and wind action.

In Green River, where typical flow conditions prevail and where the cross-section and slope are fairly uniform throughout the length in question, a value for the reaeration constant can be calculated from available information and from methods developed by Streeter and Phelps,[7] and O'Connor and Dobbins.[8] Based on stream survey data,[6] calculated reaeration constants for the river from Auburn to Kent varied from 0.22 to 0.27 during the August to November period when flows varied from 186 to 1,302 cfs. A reaeration constant of 0.20 was used, therefore, in calculating the waste receiving capacity of the Green River.

Although photosynthetic organisms may add appreciable quantities of oxygen to natural bodies of water,

[7] *Phelps, E. B.* Stream Sanitation, *John Wiley and Sons, Inc. New York, 1944.*

[8] *O'Connor, D. J., and Dobbins, W. E.* The Mechanism of Reaeration in Natural Streams, *Journal of Sanitary Engineering Division, ASCE, 82, Proc. SA6, 1115, December 1956.*

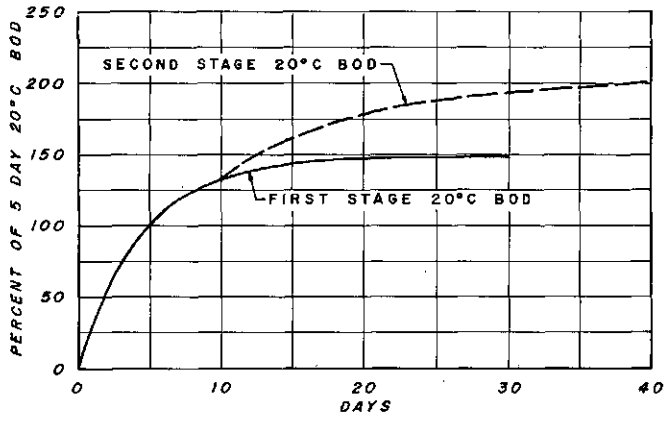


Fig. 12-4. Typical Biochemical Demand Reactions for Domestic Sewage

Stabilization of putrescible organic matter in sewage is reflected by reduction of the biochemical oxygen demand, an action which takes place in two distinct stages. Of these, the first lasts for about ten days, during which time carbonaceous matter is oxidized. In the following or second stage, which continues until all organic matter is destroyed, both the nitrogenous matter and the remaining carbonaceous matter are oxidized.

previous studies of Green-Duwamish River have indicated that photosynthesis is apparently a negligible source of supply. As a consequence, this type of addition is neglected in all subsequent discussion.

Oxygen Balance. Domestic sewage and many industrial wastes contain dissolved and suspended substances which serve as an excellent source of food for aquatic microorganisms. When such matter is placed in a natural body of water, it is consumed by microorganisms which utilize dissolved oxygen from the surrounding water in their metabolic processes. This biological consumption is referred to as biochemical oxygen demand, or more commonly as BOD.

Table 12-1. Effect of Temperature on BOD Reaction Rate Constants

Temperature, °C	First stage BOD	Second stage BOD
15	0.07	0.02
20	0.10	0.03
25	0.14	0.04

During the past 25 years, the BOD reaction has been the subject of much research. Sufficient information has been developed concerning the BOD of domestic sewage to establish its reaction rate. As shown in Fig. 12-4, the reaction takes place in two stages or steps. During the first stage only carbonaceous organic matter, such as fats and sugars, is oxidized, whereas nitrogenous substances, chiefly ammonia, are attacked during the second stage. Temperature has a definite effect on the reaction rate, with the rate increasing as the temperature increases. A comparison of BOD reaction rate constants for both the first and second stages is presented in Table 12-1. The greater the value of the reaction rate, the greater is the amount of oxygen consumed per unit of time.

Because BOD tests are normally made and reported in terms of the first stage 5-day demand at a temperature of 20° C, all analysis results presented herein are expressed on that basis. As indicated in Fig. 12-4, the ultimate first stage BOD is 146 per cent of the 5-day BOD, whereas the ultimate second stage BOD is 200 per cent of the 5-day.

The oxygen deficit, or conversely, the amount of oxygen consumed as a result of sewage discharges into Green River, was calculated by the method developed by Streeter and Phelps.[7] This method takes

Table 12-2. Dissolved Oxygen Concentration in Green River at Various BOD Loads Applied at Auburn

	Distance upstream from mouth,[a] 1,000 ft	Travel time,[b] days	DO and BOD concentrations at given BOD loadings					
			5,000 lb per day[c]		10,000 lb per day[c]		20,000 lb per day[c]	
			DO, ppm	BOD,[d] ppm	DO, ppm	BOD,[d] ppm	DO, ppm	BOD,[d] ppm
Auburn	145	–	6.9	6.8	6.9	13.6	6.9	27.2
Kent	100	0.38	6.9	6.3	6.5	12.7	5.7	25.4
Estuary end[e]	90	0.40	6.9	6.3	6.5	12.6	5.6	25.2
Orillia[f]	77	0.52	6.9	6.1	6.4	12.3	5.2	24.6

Based on river flow of 150 cfs and sewage discharge of 50 cfs at Auburn. Assumptions: river DO - 8.5 ppm; sewage DO - 2.0 ppm; temperature - 16° C; reaction constant (k_1) - 0.08; reaeration constant (k_2) - 0.20; constant DO deficit of 2.7 ppm.

[a]*See Fig. 12-1.*

[b]*From Auburn; river velocity at flow of 200 cfs - 1.5 feet per second.*

[c]*5-day 20° C BOD. 5-day BOD equals 68 per cent of ultimate 1st stage BOD; see Fig. 12-4.*

[d]*Ultimate 1st stage BOD.*

[e]*Upper limit of existing estuary.*

[f]*Upper limit of proposed estuary.*

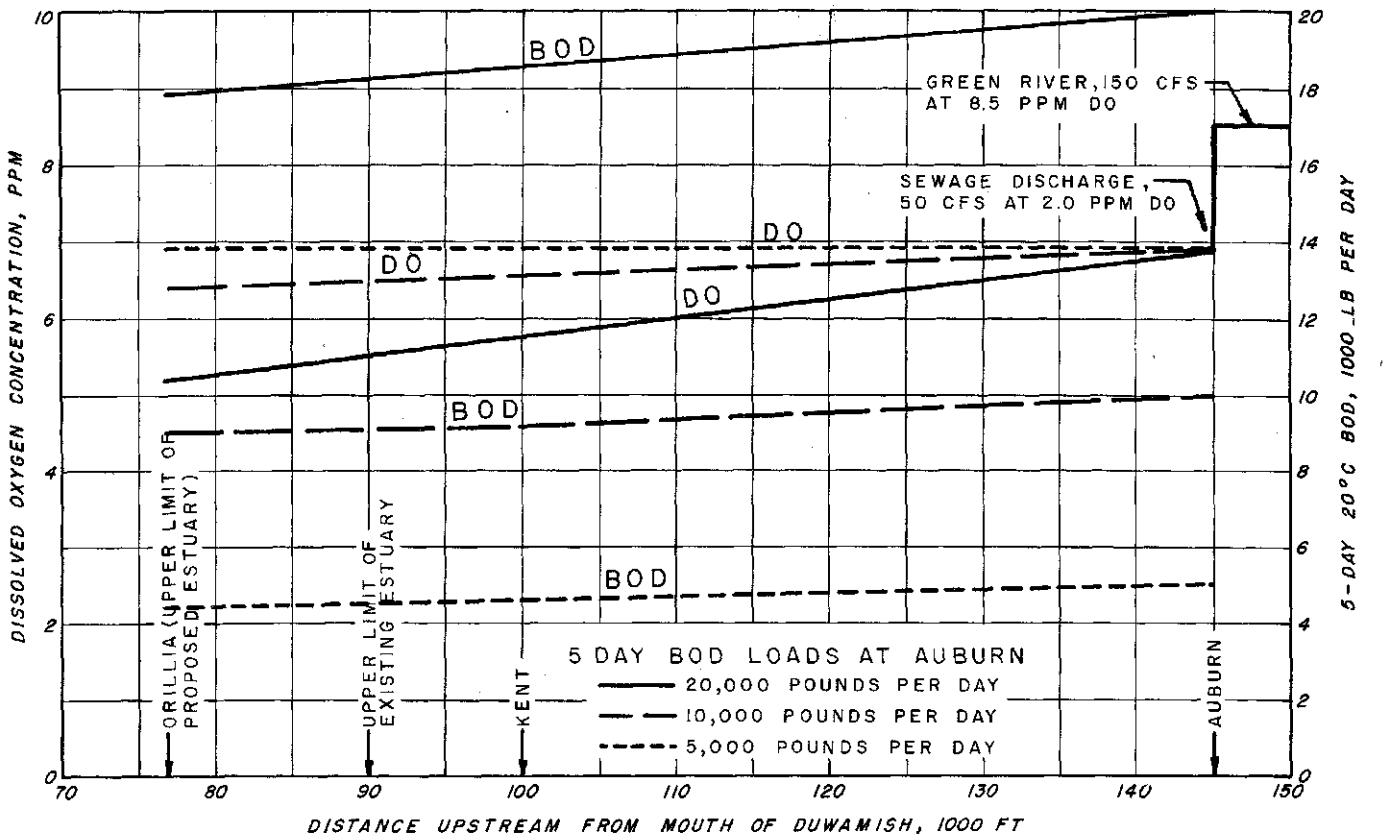


Fig. 12-5. Dissolved Oxygen Concentrations in Green River at Various BOD Loads Applied at Auburn

into account all of the factors previously discussed and permits the calculation of oxygen dificiencies that will result during different times of flow when a certain BOD load is placed upon the water.

Table 12-2 and Fig. 12-5 present the results of calculations for various BOD loads discharged to the river at Auburn. All calculations assume that 50 cfs of sewage effluent containing 2.0 ppm of dissolved oxygen will be discharged to the river at this point. As indicated by the figure and table, 20,000 pounds of 5-day BOD per day could be discharged to Green River at Auburn while still maintaining the dissolved oxygen level above the minimum permissible limit of 5.0 ppm. Of this total load, about 18,000 pounds would still remain in the river as it enters the Duwamish estuary. The extent to which this residual BOD can be tolerated downstream is dependent on the self-purification capacity of the estuary.

Duwamish Estuary

Estuaries may be classified generally as either positive or neutral. When the net tidal outflow is greater than the inflow, an estuary is said to be positive. Under this condition, the estuary water is diluted by upland inflow of fresh water and the salinity is less than that of the entering sea water. A neutral estuary receives little or no fresh water inflow, on which basis the tidal outflow and inflow are approximately equal. A neutral estuary may be characterized by a water of equal or higher salinity as compared with that of the entering sea water.

In a positive estuary, fresh water tends to stratify above the heavier saline water. During flood tide, fresh water does not move upstream as fast as the underlying saline water and, in extreme cases, may even move outward on an incoming tide. On an ebb tide, fresh water moves faster than the salt water, thus promoting rapid flushing.

In a neutral estuary, fresh water inflow is negligible and stratification rarely occurs. Water moves back and forth with the tides much like a piston. Little difference is noted between bottom and surface currents, and the only interchange results from diffusion. Flushing action, therefore, depends more upon mixing and diffusion that it does upon net outward movement. Sewage disposal is most critical under these conditions.

Since appreciable quantities of fresh water enter the Duwamish even at times of minimum flow in Green River, the estuary no doubt functions as a positive estuary. During each tidal cycle, a volume of salt water enters the estuary from Elliott Bay, mixes with fresh water flowing seaward, and ultimately returns to Puget Sound. On ebbing tide, surface waters move

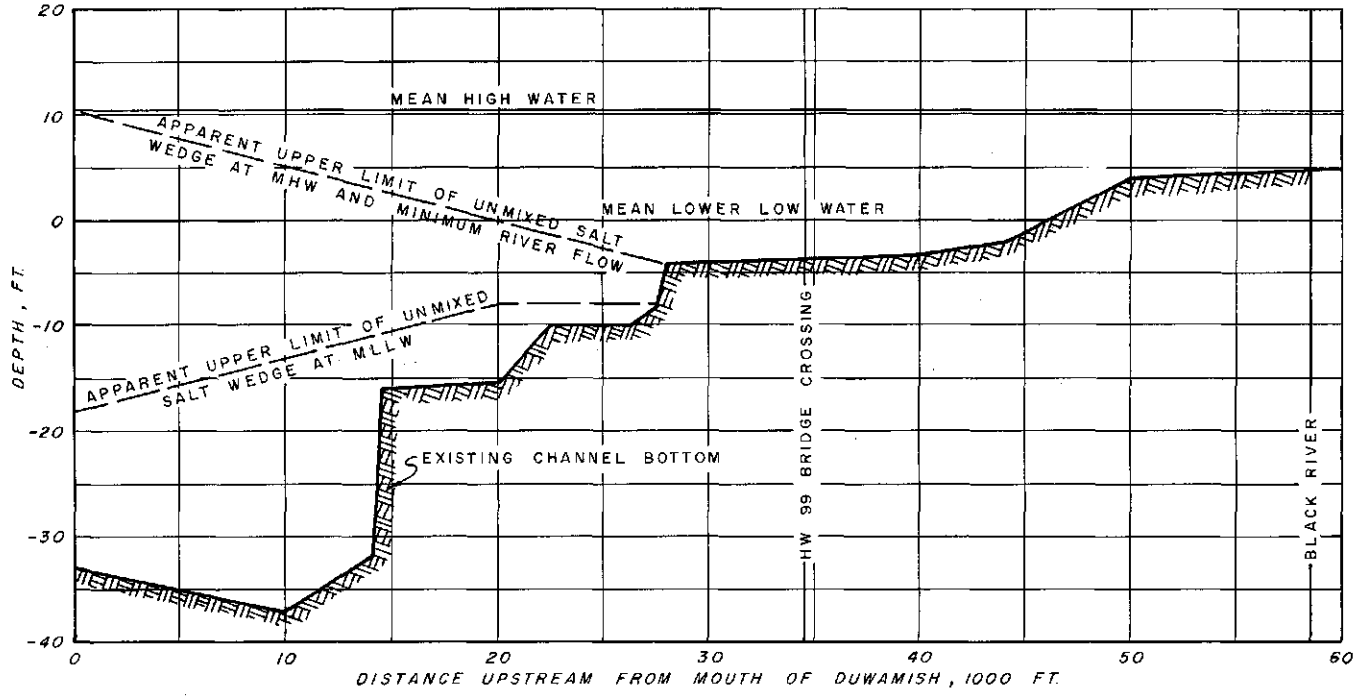


Fig. 12-6. Location of Salt Water Wedge in Existing Duwamish Estuary

out into Elliott Bay where they are either swept away by the surface currents in the bay or returned to the estuary by the subsequent flood tide. As noted in Chapter 11, surface waters are swept away from the estuary mouth by currents in Elliott Bay.

As stated previously, the ability of a body of water to absorb sewage innocuously depends on available dilution and oxygen resources. In an estuary, dilution results from fresh water inflow and also from tidal ebb and flow. Flushing action denotes the combined effect of all factors which contribute to the ability of the estuary water to disperse sewage. Each of these factors, as it applies to the Duwamish estuary, is discussed in the following sections.

Mixing and Diffusion. Flow in positive estuaries is characterized by a layer of fresh water flowing out over a layer of heavy underlying salt water. Vertical mixing between incoming salt water and fresher surface waters is often incomplete for considerable distances upstream. Because of its characteristic shape, the underlying mass of salt water is frequently referred to as a salt wedge. The salt wedge-fresh water interface obviously will fluctuate with the tide and fresh water runoff. Waters lying above the salt wedge represent the zone of mixed flow wherein sewage discharges are dispersed and diluted.

The apparent location of the salt wedge in the existing estuary, as reported from field measurements taken during low fresh water inflow conditions, is shown in Fig. 12-6. Under these conditions, the apparent upstream limit of unmixed salt water at high tide extends about 28,000 feet from the mouth of the estuary.

A theoretical salt wedge was calculated by the methods of Farmer and Morgan[9] and Linder[10] for an idealized channel of the same dimensions as the proposed Duwamish estuary (Fig. 12-7). This wedge would extend to the salt water barrier at the end of the proposed dredged channel.

Fig. 12-7 also shows the limits of the salt wedge as estimated from the slope and location of the wedge in the existing estuary. As there indicated, the estimated upstream limit of the salt wedge at high tide and minimum fresh water inflow extends to about Black River junction, or 46,000 feet from the mouth of the estuary along the proposed realignment.

As salt water moves up an estuary, it mixes gradually with overlaying fresh surface water. Hence, the seaward-moving fresh water becomes increasingly saline as it moves out of the estuary. At any point, the volume of outgoing water is equal to the accumulated fresh water volume plus the admixed salt water volume.

Based on the apparent location of unmixed salt water (Figs. 12-6 and 12-7), salt water concentrations at various locations in both the existing and proposed

[9] *Farmer, H. G. and Morgan, G. W.,* The Salt Wedge, *Proceedings of the Third Conference on Coastal Engineering, Cambridge, Massachuttes, pp. 54-64, October 1952.*

[10] *Linder, C. P.,* Intrusion of Sea Water in Tidal Sections of Fresh Water Streams, *ASCE Proc., Separate 358, November 1953.*

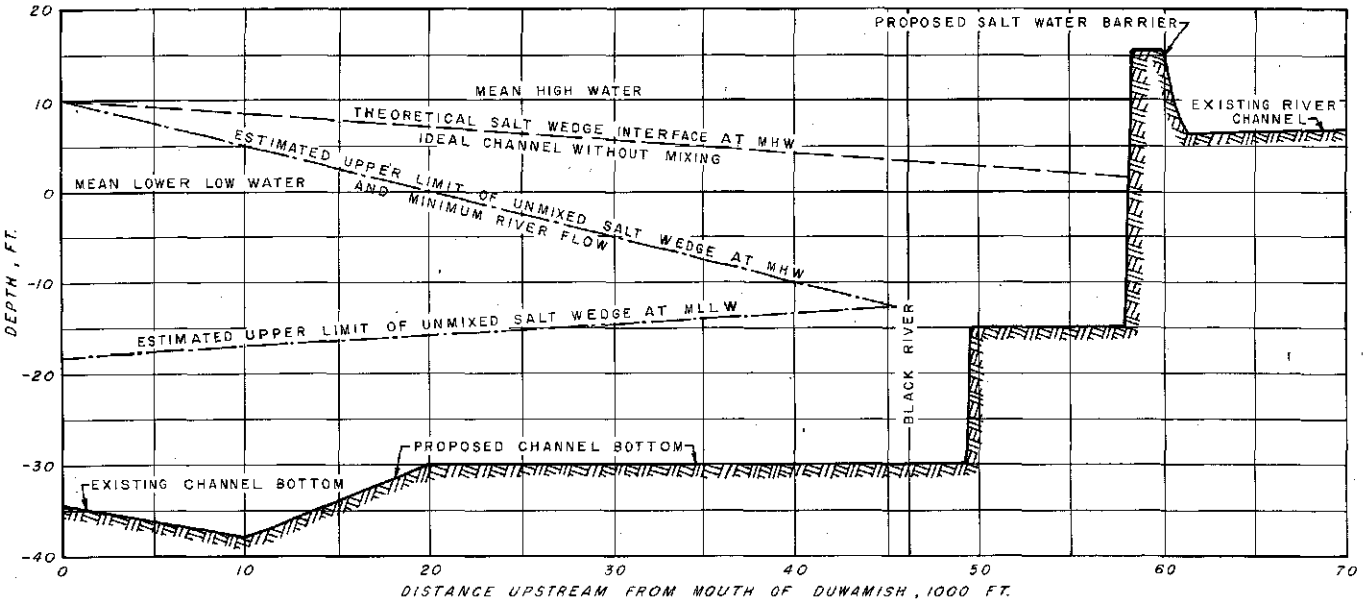


Fig. 12-7. Calculated Location of Salt Water Wedge in Proposed Duwamish Estuary

estuaries were calculated and are shown in Figs. 12-8 and 12-9. Concentrations are given for the hypothetical condition of complete vertical mixing and for the most probable condition of partially mixed flows. As indicated, salt water flushing and dilution apparently are insignificant upstream from the Black River junction. Downstream from this point, available dilution increases rapidly.

Flushing Time. Since tidal currents are oscillatory, the water leaving on ebb tide may return to nearly its original position on subsequent flood tide. In an estuary where fresh water added at the upper end must flow out the lower end, average ebb currents are always either more rapid or of longer duration than average flood currents. Daily average excursion velocities tend to vary throughout the length of an estuary, being slowest near the mouth and greatest at the upper or head end.

The average time required for river water and its contained pollution load to move through an estuary is defined as the flushing time. Calculations thereof may be made by either of two methods: (1) from field observations of salinities and flow directions and velocities; or (2) from the known tidal cycle, the dimensions of the estuary, and the fresh water flow.[11] Because of the lack of sufficient field data, flushing characteristics of both the existing and proposed Duwamish estuaries were calculated by the latter method.

Expressed as the mean age in days, the length of time for a particle of fresh water to flow from the head end to the mouth of the existing estuary is shown

[11] *Ketchum, B. H.,* The Exchanges of Fresh and Salt Waters in Tidal Estuaries, *Woods Hole Oceanographic Institution, Collected Reprints, Contribution No. 516 (1951).*

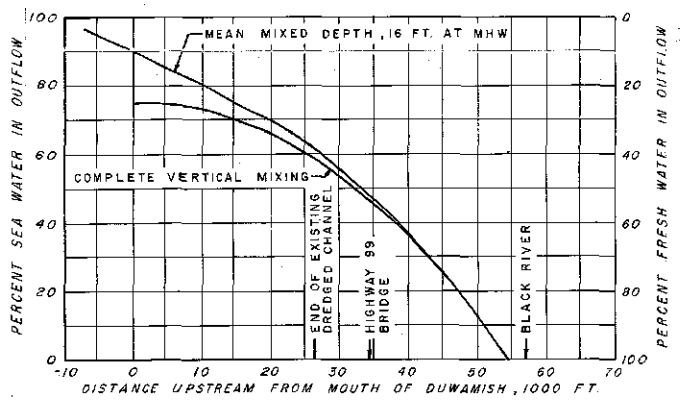


Fig. 12-8. Calculated Salinity in Existing Duwamish Estuary

Based on upland fresh water flow of 200 cfs.

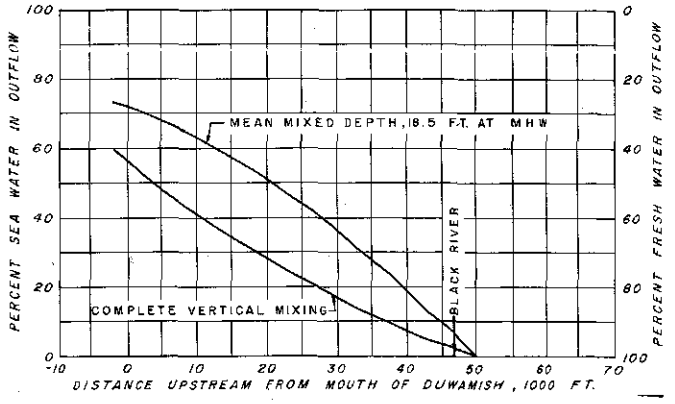


Fig. 12-9. Calculated Salinity in Proposed Duwamish Estuary

Based on upland fresh water flow of 400 cfs.

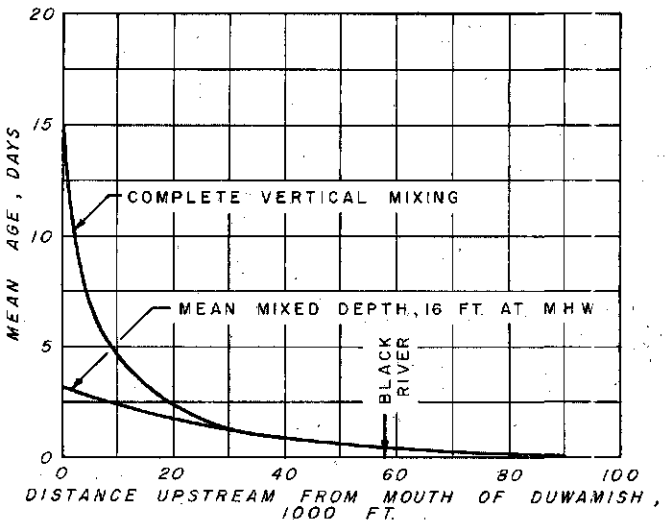


Fig. 12-10. Mean Age of Fresh Water in Existing Duwamish Estuary during Passage Downstream

Based on upland fresh water flow of 200 cfs.

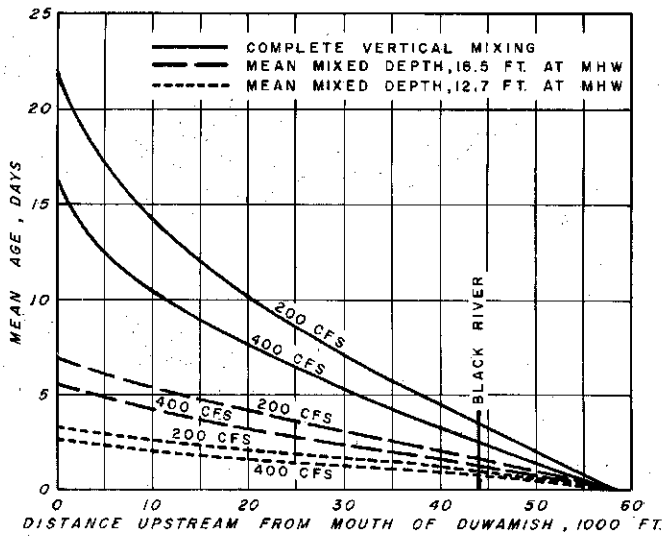


Fig. 12-11. Mean Age of Fresh Water in Proposed Duwamish Estuary during Passage Downstream

The figure shows the influence of mixing depth and fresh water flow on flushing time.

in Fig. 12-10 for two conditions of mixing. If the estuary waters were completely mixed, a particle of fresh water entering at the head end would stay in the estuary for over 15 days. If, however, the waters were mixed to a depth of 16 feet at mean high water, the travel time would be decreased to three days.

Results of similar calculations for fresh water deliveries of 200 and 400 cfs to the head end of the proposed Duwamish estuary are shown in Fig. 12-11. A comparison of this figure with Fig. 12-10 shows that, other things being equal, it will take longer to flush a particle of water from the proposed estuary than it does at present.

Oxygen Resources. Oxygen resources in the Duwamish estuary include (1) that contained in the incoming upland waters, (2) that in tidal waters entering from Elliott Bay, and (3) that obtained from the atmosphere. Contributions from these sources depend in part upon river flow, tidal exchange, mean velocity, depth, and surface area, all of which were determined for the critical period. Two other important factors are the dissolved oxygen concentration and the temperature both of the incoming water and of the water within the estuary.

The Department of Oceanography of the University of Washington is engaged in a continuous program of sampling and studying the waters of Puget Sound. Data obtained from this department for sampling stations just outside Elliott Bay at Alki Point on the south and at Point Jefferson on the north show that minimum dissolved oxygen concentrations of about 6 ppm occur in deep water during the summer and early autumn (Fig. 12-12). As indicated by float studies, waters entering the Duwamish estuary from Elliott Bay during this period apparently are from depths greater than 20 meters and are therefore low in dissolved oxygen. For the purpose, therefore, of calculating the waste receiving capacity of the estuary during this critical period, the dissolved oxygen of the water entering from Elliott Bay was assumed to be 6.0 ppm.

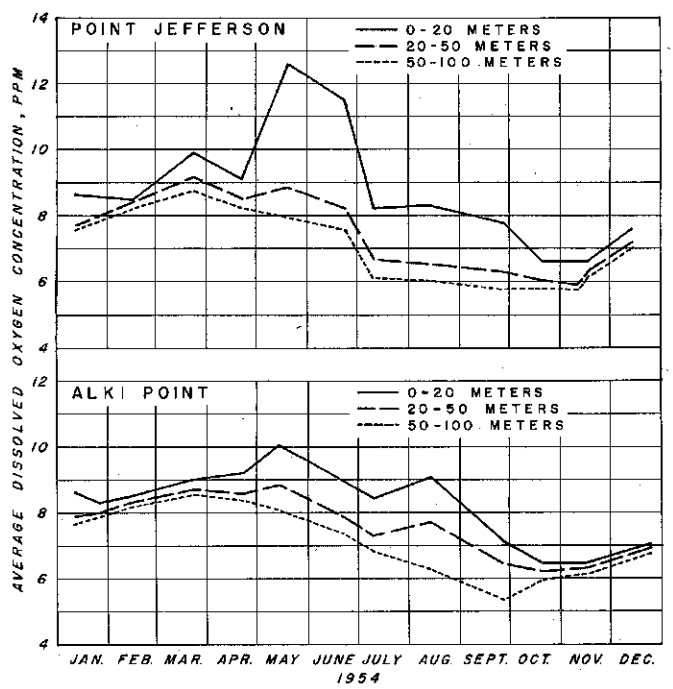


Fig. 12-12. Dissolved Oxygen Concentrations in Puget Sound

Source: Department of Oceanography, University of Washington, "Physical and Chemical Data, Puget Sound and Approaches", Technical Report No. 46, January to December, 1954.

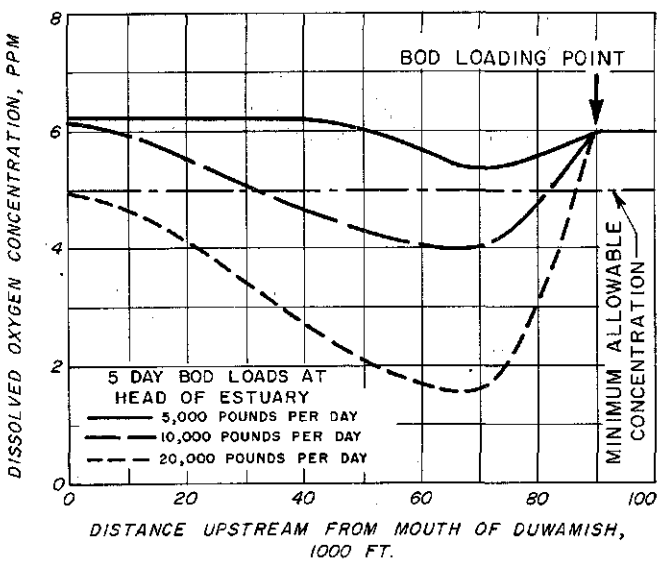


Fig. 12-13. Dissolved Oxygen Concentrations in Existing Duwamish Estuary for Various BOD Loads Applied at Head of Estuary

Based on minimum fresh water flow of 200 cfs, initial dissolved oxygen concentration of 6.0 ppm in both salt and fresh waters, and a mean mixed depth of 16 feet at mean high tide.

In determining the waste receiving capacity of the Duwamish estuary, it was assumed (1) that the minimum river flow would be 150 cfs, (2) that the sewage inflow would be between 50 and 250 cfs, and (3) that the dissolved oxygen concentration of sewage discharged to the estuary would be such that the initial concentration in mixed river water and sewage would be not less than 6.0 ppm.

Atmospheric reaeration in the estuary was calculated by the same method as that used for calculating reaeration in Green River. A reaeration constant of 0.20, or the same as for Green River, was used for the section of the existing estuary upstream from the dredged channel. Since the surface area which comes in contact with the atmosphere is considerably reduced, the amount of oxygen which can be absorbed in the water is less in the dredged channel than in the river and a lower reaeration constant results. Based on methods developed by O'Connor and Dobbins,[8] a value of 0.05 was calculated for the reaeration constant of the dredged channel. This value was used for the proposed estuary, as well as for the dredged portion of the existing estuary.

Oxygen Balance. If the BOD loading and dissolved oxygen supply rates are known, it is possible to calculate the dissolved oxygen concentration present in any segment of an estuary. The BOD load exerted at various points in an estuary is determined by the amount being added upstream and the length of time it has been held in the water at the point in question. The amount of oxygen carried by the fresh water can be calculated from its volume and dissolved oxygen concentration. Likewise, the amount of oxygen contributed by the salt water can be calculated from the volume penetrating any segment and from its dissolved oxygen concentration.

Calculations of oxygen balance were made for various BOD loadings at selected points along both the existing and proposed estuaries. Sample calculations for a loading of 10,000 pounds per day at the head end of the existing estuary are given in Table 12-13. The dissolved oxygen concentration which will result from this loading, as well as those which will result from loadings of 5,000 and 20,000 pounds per day, are shown in Fig. 12-13.

It will be seen that the maximum possible loading at the head end of the existing estuary is slightly over 5,000 pounds of 5-day BOD per day. If the receiving point were moved downstream approximately 32,000 feet to the Black River junction, the BOD load which could be assimilated safely without reducing the dissolved oxygen concentration below 5.0 ppm would be increased to 10,000 pounds per day (Fig. 12-14).

Because of the increased holdup time in the proposed estuary and the resulting probability of a higher temperature, calculations of the oxygen balance therein were made on the basis of a maximum summer temperature of 20° C. Further, since considerations with respect to sewage disposal should logically be based on future conditions, minimum fresh water inflow was taken as 400 cfs. This flow comprises a

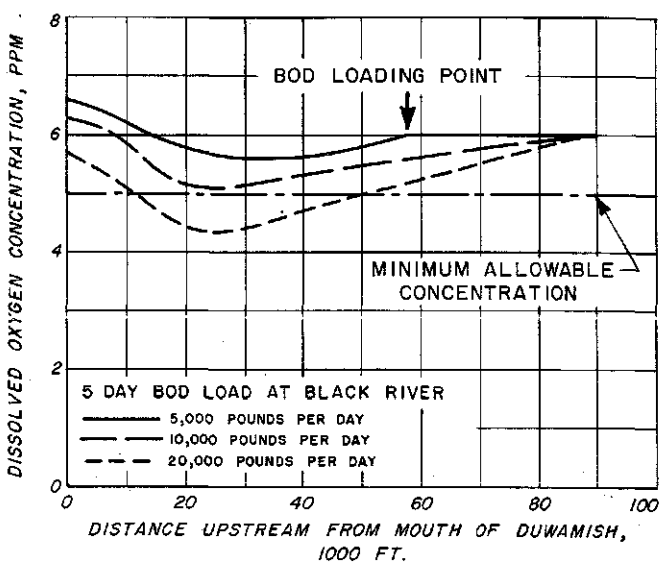


Fig. 12-14. Dissolved Oxygen Concentrations in Existing Duwamish Estuary for Various BOD Loads Applied at Black River

Based on minimum fresh water flow of 200 cfs, initial dissolved oxygen concentration of 6.0 ppm in both salt and fresh waters, and a mean mixed depth of 16 feet at mean high tide.

Table 12-3. Oxygen Balance in Existing Duwamish Estuary

	Segment				
	O	I	II	III	IV
River flow (R) - 200 cfs or 9 x 10^6 cu ft per tide					
Station along river[a]					
From	900+00	550+00	290+00	150+00	45+00
To	550+00	290+00	150+00	45+00	-30+00
Length, feet	35,000	26,000	14,000	11,500	7,500
Volume per segment, 10^6 cu ft					
At MLLW (V)	0	9	38	92	195
Tidal prism (P)	9	29	54	103	–
At MHW (P+V)	9	38	92	195	–
Exchange ratio (P/P+V)	1.0	0.765	0.588	0.528	0.5[b]
Fresh water volume per segment (RxP+V/P), 10^6 cu ft	9	12	15	17	18
Accumulated volumes, 10^6 cu ft					
At MLLW (V_n)	0	9	47	139	334
Tidal prism (P_n)	9	38	92	195	–
At MHW (P_n+V_n)	9	47	139	334	–
Fresh water (Q_n)	9	21	36	53	71
Fresh water fraction ($F=Q_n/P_n+V_n$)	1.0	0.45	0.26	0.16	0.10[b]
Mean age, of fresh water, days					
Within segment[c] (D)	0.52	0.67	0.88	0.98	1[b]
Leaving segment	0.52	1.19	2.07	3.05	4
Escape volume, 10^6 cu ft					
Per tide (R/F)	9	20	35	56	90
Per day (B)	17	39	68	108	174
Net (B x D)	9	26	60	106	174
Daily salt water inflow[d], 10^6 cu ft	0	22	51	91	157
Dissolved oxygen from salt water (S),[e] lb per day	0	5,600	10,400	17,200	24,300
Dissolved oxygen from reaeration (A),[f] lb per day	400	1,900	1,700	3,900	6,900
Total dissolved oxygen (S+A), lb per day	400	7,500	12,100	21,100	31,200
BOD exerted per segment[g]					
Per cent[h]	15	14	21	20	11
Pounds per day (L)[i]	1,500	1,400	2,100	2,000	1,100
Escape dissolved oxygen[j], lb per day	2,200	8,300	18,300	37,400	67,500
Dissolved oxygen concentration,[k] ppm	4.0	4.9	4.9	5.7	6.2

Based on river plus sewage flow of 200 cfs with dissolved oxygen concentration of 6.0 ppm. Assumed mixing to depth of 16 feet at MHW.

[a]*See Fig. 12-1.*

[b]*Assumed.*

[c] $\frac{12.4\ (P+V)}{24\ P}$

[d]*Volume of salt water that is mixed with fresh water.*

[e]*Per segment. Based on dissolved oxygen concentration of 6.0 ppm in incoming salt water.*

[f]*Based on constant oxygen deficit of 3.0 ppm and reaeration constant (k_2) of 0.20 for segments O and I and 0.05 for segments II, III and IV.*

[g]*5-day 20° C BOD.*

[h]*From Fig. 12-4 adjusted to reaction rate constant (k_1) of 0.09 for temperature of 18° C.*

[i]*Based on load of 10,000 pounds per day discharged at head end of estuary.*

[j]*Dissolved oxygen moving downstream from segment to segment. Equals E+S+A–L where E is dissolved oxygen entering segment from upstream.*

[k]*Escape dissolved oxygen divided by weight of net escape volume.*

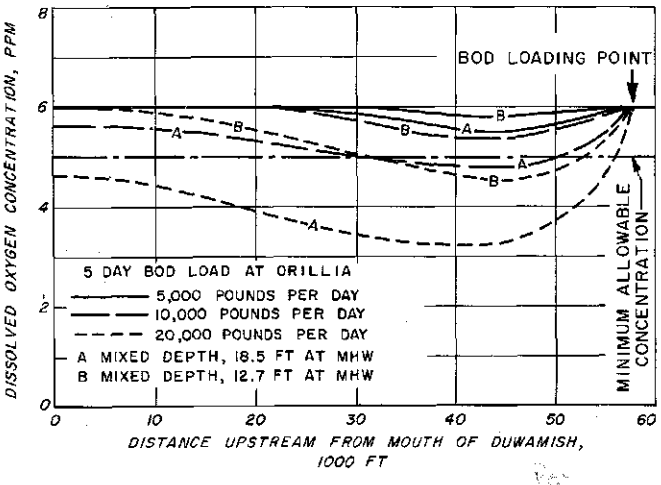


Fig. 12-15. Dissolved Oxygen Concentrations in Existing Duwamish Estuary for Various BOD Loads Applied at Orillia

Based on minimum fresh water flow of 400 cfs and initial dissolved oxygen concentration of 6.0 ppm in both salt and fresh waters.

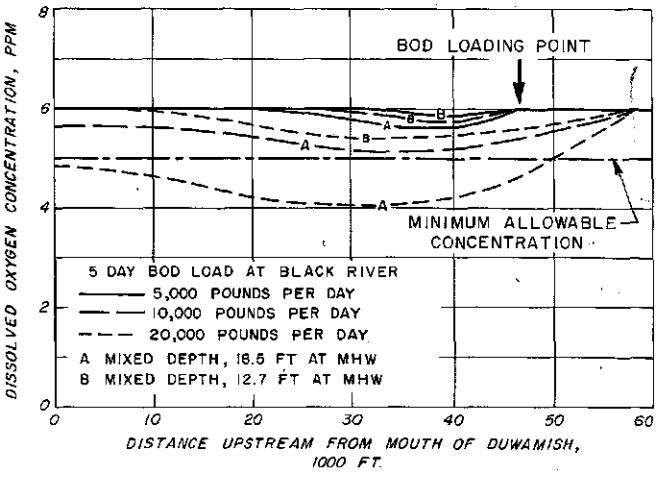


Fig. 12-16. Dissolved Oxygen Concentrations in Proposed Duwamish Estuary for Various BOD Loads Applied at Black River

Based on minimum fresh water flow of 400 cfs and initial dissolved oxygen concentration of 6.0 ppm in both salt and fresh waters.

river flow of 180 cfs, which will be the minimum upon completion of the Eagle Gorge dam, and a sewage flow of 220 cfs, which will be the ultimate likely to be tributary to the estuary.

Since the exact location of the salt wedge is indeterminate at present and could lie within a relatively wide range (Fig. 12-7), two conditions of mixing were used in determining the waste receiving capacity of the proposed estuary. Under these conditions, the mean mixed depths were assumed to be 18.5 feet and 12.7 feet respectively, both at mean high water. In all probability, the actual mixed depth will be somewhere between the assumed limits.

Fig. 12-15 shows dissolved oxygen concentrations which will result from various BOD loads discharged at Orillia, the head end of the proposed estuary. At a mean mixed depth of 18.5 feet, minimum concentrations will be 5.5 ppm for an applied load of 5,000 pounds of 5-day BOD per day, and 4.7 ppm for a load of 10,000 pounds per day. At a mean mixed depth of 12.7 feet, similar values for corresponding loads will be 5.8 and 5.4 ppm respectively. Apparently, therefore, the maximum load which can be discharged at Orillia amounts to about 10,000 pounds of 5-day BOD per day

Estimated effects of discharging sewage at a downstream location, Black River junction, are shown in Fig. 12-16. With a mean mixed depth of 18.5 feet, the maximum load which can be discharged at this point is 10,000 pounds of 5-day BOD per day. This value increases to 20,000 pounds at a mean mixed depth of 12.7 feet. It can thus be assumed that the proposed estuary will be capable of accommodating a load somewhere between these two limits, or about 15,000 pounds of 5-day BOD per day. Moving the point of discharge 10,000 feet downstream from Black River junction does not appear to offer much in the way of further increases in permissible BOD loads (Fig. 12-17).

MAXIMUM PERMISSIBLE BOD LOADINGS

Since sewage discharges at various points in the Green-Duwamish river system will naturally affect downstream water conditions, it is necessary to assess the effects of such discharges in order to deter-

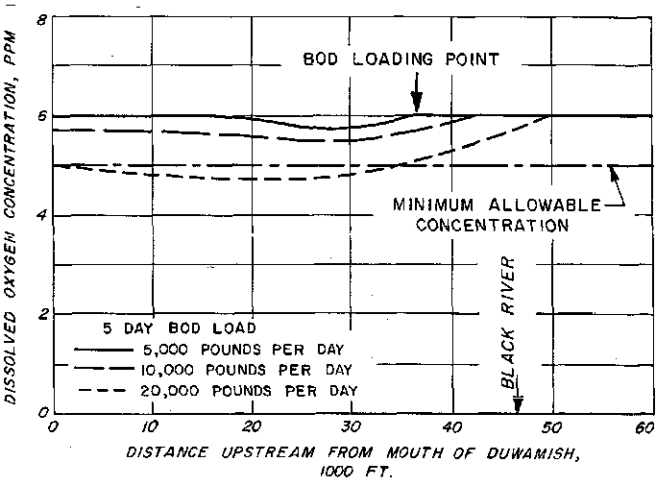


Fig. 12-17. Dissolved Oxygen Concentrations in Proposed Duwamish Estuary for Various BOD Loads Applied Downstream from Black River

Based on minimum fresh water flow of 400 cfs, initial dissolved oxygen concentration of 6.0 ppm in both salt and fresh waters, and mean mixed depth of 18.5 feet at mean high water.

mine the maximum permissible load which may be applied at any given location. In other words, if sewage is being discharged at one or more points upstream, the downstream receiving capacity is reduced and BOD computations for downstream locations must be modified accordingly.

In the preceding calculations of BOD loadings, which were made for four locations in the Green-Duwamish system, it was assumed that all of the receiving capacity would be completely utilized at a single point of discharge. On that basis, of course, there could be no discharge either upstream or downstream from the location in question. As indicated above, however, the problem is to develop some sort of a yardstick whereby the effects of upstream discharges can be evaluated and the receiving capacity at any given point can be properly determined. Although any number of combinations of possible loadings can be developed from the data previously presented, the five given in Table 12-4 for both the existing and proposed estuaries provide an adequate basis for sewerage planning purposes.

While 5-day BOD loads of 20,000 pounds per day could be applied to Green River at Auburn, the residual BOD load entering the Duwamish estuary from such a discharge would reduce the dissolved oxygen concentration therein below the minimum allowable level of 5.0 ppm. Based on the self-purification capacity of the estuarial portion of the Green-Duwamish, the maximum BOD loads which could be satisfactorily discharged at Auburn would be 7,000 pounds per day for the present estuary and 12,000 pounds per day for the proposed estuary (Table 12-4). If any additional BOD loads were introduced downstream, the allowable load which could be discharged at Auburn would be reduced accordingly.

It is evident from the data in Table 12-4 that the head waters of both the existing and proposed estuaries are the poorest of the possible locations for sewage discharges in the estuary. In both estuaries, the capacity to receive sewage is increased markedly as the point of loading is moved downstream toward the Black River. At the head end of the existing estuary, the maximum permissible load which could be applied amounts to 6,000 pounds of 5-day BOD per day, whereas 10,000 pounds per day could be discharged at Black River. Similarly, in the proposed Duwamish estuary, the maximum BOD loads which could be discharged are 10,000 pounds per day at Orillia and 15,000 pounds at Black River. As indicated by Table 12-4, the amount of BOD which could be safely introduced into either the existing or proposed estuary would be reduced by any upstream addition in Green River.

Table 12-4. Maximum Permissible BOD Loadings in Green - Duwamish River

Condition	Load[a] discharged at			
	Auburn	Estuary end[b]	Orillia[c]	Black River
Existing Duwamish estuary				
1	7,000	0	–	0
2	5,000	2,000	–	0
3	5,000	0	–	7,000
4	0	6,000	–	0
5	0	0	–	10,000
Proposed Duwamish estuary				
1	12,000	–	0	0
2	5,000	–	6,000	0
3	5,000	–	0	12,000
4	0	–	10,000	0
5	0	–	0	15,000

[a] *5-day 20° C BOD in pounds per day.*

[b] *Upper limit of existing estuary.*

[c] *Upper limit of proposed estuary.*

Chapter 13
DESIGN CRITERIA AND BASIS OF COST ESTIMATES

In the evaluation of alternative projects designed to perform a given function, each project must be laid out in sufficient detail to permit comparisons of performance and cost, both construction and operation. To make such layouts it is necessary, first, to develop criteria applicable to the preliminary design of all major sewerage and drainage facilities, and second, to develop basic cost data for each facility and each type of construction.

PRELIMINARY LAYOUTS

Design criteria and basic cost data presented herein, apply to preliminary design or layout of major sewerage and drainage facilities. In such layouts, detailed construction drawings and specifications are not required. Instead, it is necessary only that a reasonably close approximation of the size, location, route and cost of the various facilities be developed and that this information be given in sufficient detail to permit comparisons between alternative plans. Obviously, therefore, relocation and resizing of some of the facilities may be required at a later date as a result of the detailed engineering analysis which is made during the preparation of construction drawings and specifications. Furthermore, preliminary layouts are limited to trunk and intercepting sewers, trunk storm drains, pumping stations, and sewage treatment and disposal works. Local sewers and storm drains are considered only as they relate to separation of sanitary sewage and storm water presently picked up in combined systems. Definitions of trunk sewers and storm drains, as distinguished from local sewers and drains, are given in subsequent sections of this chapter.

DESIGN PERIOD

All plans or projects are laid out to serve ultimate development of the tributary area. In this report, the word "ultimate" refers to conditions which are expected about 70 years in the future, or about the year 2030. These conditions, it is believed, will represent approximate saturation of the area as related to major sewerage and drainage improvements. It is entirely possible, however, that complete saturation will never be attained, especially in the outlying or fringe areas.

Although plans are laid out to provide for ultimate development, it does not follow that all facilities need to be constructed immediately. In some cases, a particular facility will not be required until a future date while in other cases, because a facility may be enlarged readily, only a portion will require immediate construction. As a general rule, maximum economy in construction costs is achieved by construction of trunk sewers and storm drains with sufficient capacity for the ultimate needs of the tributary area. This is especially true with large lines located in waterfront areas or other areas of difficult construction conditions. In some cases, however, slower rates of development of the tributary areas, coupled with other factors, make it desirable to construct trunk sewers initially with a capacity sufficient for only a portion of that ultimately needed. In such cases, a parallel line would be laid when required at a future date.

Storm drains for long-term needs are constructed normally with a capacity sufficient to accommodate flows anticipated at ultimate development of the tributary area. Pumping stations and treatment works, on the other hand, are suited to a program of stage construction whereby basic structures only are built initially for ultimate flow requirements. Under such a program, sedimentation units, sludge digestion facilities, and pumping and other mechanical equipment may be added incrementally to accommodate future increases in flows and loadings.

USE OF EXISTING FACILITIES

Throughout the survey every effort was made to ascertain the capacity and condition of existing facilities with the view to their incorporation in the final recommended plan. In general, existing systems of trunk sewers and storm drains are utilized fully. In a few cases, however, some of the smaller sewers, which are designated as trunk sewers within several of the sewerage agencies, are of insufficient capacity or are improperly located for inclusion in the general plan. Furthermore, only 2 of the 25 existing treatment plants can be incorporated effectively in the recommended sewerage program.

DESIGN CRITERIA

Experience in many localities has demonstrated that separate sewers are desirable where sanitary sewage must be conveyed a long distance for treatment and disposal. Combined sewers are subject to a number of disadvantages (Chapter 9). Of these, the two

most significant in relation to local conditions are: (1) the cost of interception, conveyance and treatment of combined sewage is many times greater than that of separate sanitary sewage; and (2) the provision of even extremely large interceptor sewers and of excessive hydraulic capacity in treatment and disposal works would not prevent periodic overflows of diluted sewage and storm water and a resultant pollution of adjacent waters. For these reasons, the design criteria presented herein for trunk sewers, storm drains, intercepting sewers, pumping stations and treatment plants are based on the assumption that all new areas will be developed with separate sanitary systems. At present, new areas east of Lake Washington, as well as new developments north and south of the Seattle central area, are being provided with separate systems. It is essential that this policy be continued in the future and that, in some older areas, the existing combined systems be separated.

Design Loadings - Separate Sanitary Systems

Design loadings may be defined in terms of the significant characteristics, such as volume and strength, of the waste to be conveyed and treated. For a particular structure, design loading is determined by multiplying the unit design quantity by the number of units tributary to it. Thus, the total volume of sanitary sewage to be carried by a trunk sewer is calculated by multiplying the peak per capita sewage flow by the contributary population. Similarly, storm drain capacity is determined by multiplying the rainfall intensity during the period of concentration by the tributary area and by the coefficient of runoff.

Unit design quantities for sanitary sewerage systems are given in Table 13-1 and are based on the studies of sewage characteristics reported in Chapter 7. They take into account expected future variations, and anticipate also the establishment of appropriate regulations by responsible agencies. It is assumed that the latter will set limits on physical and chemical characteristics of the sewage which would produce undue loadings, or would lead to deleterious effects either on the collecting sewers and treatment works or on the treatment and disposal processes.

Design Loadings - Storm Drainage Systems

Unit design quantities for storm drainage systems depend upon the frequency, duration, and intensity of rainfall and on the character of the surface with respect to the proportion of rainfall which runs off. For any given area, the rate of storm water runoff is commonly determined by the so-called rational method. This method is represented by the formula $Q = ciA$, wherein Q is the runoff rate in cubic feet per second, c is a selected coefficient of runoff expressed as the ratio of runoff to rainfall, i is the mean intensity of rainfall in inches per hour, and A is the tributary area in acres.

The rational formula expresses the value of Q in cubic feet per second by virtue of the fact that an inch of depth of rainfall per hour over an area of one acre is substantially equivalent to a rate of flow of one cubic foot per second. Values of c must be estimated from a study of the soil, the slope and condition of the surface, the imperviousness of the surface, and a consideration of the probable future changes in surfaces within the drainage area. Values of i depend on the rainfall characteristics in the particular area as determined from a study of weather records. Selection of an appropriate value for i involves a considera-

Table 13-1. Design Loadings for Separate Sewerage Facilities

Volume		Peak flow ratios	
Sanitary sewage, gallons per capita per day	60	Sanitary sewage	
Industrial wastes, gallons per acre per day		Within major sewerage areas	1.75
Heavy industrial areas less than 1,000 acres	4,000	Two or more major sewerage areas	1.50
Heavy industrial areas greater than 1,000 acres	2,000	Industrial wastes	
Light industrial areas	2,000	Heavy industry	2.0
Ground water infiltration, gallons per acre per day		Light industry	3.0
Summer conditions			
Existing construction	300	Biochemical oxygen demand (BOD)	
Future construction	300	Sanitary sewage, pounds per capita per day	0.20
Winter conditions		Industrial wastes, pounds per capita (equivalent)	
Existing construction	1,200	per day	0.20
Future construction	600	(equivalent population based on average volume)	
Storm water inflow, gallons per acre per day			
Summer conditions		Suspended solids	
Existing construction	500	Sanitary sewage, pounds per capita per day	0.25
Future construction	0	Industrial wastes, pounds per capita (equivalent)	
Winter conditions		per day	0.25
Existing construction	2,000	(equivalent population based on average volume)	
Future construction	500		

Table 13-2. Runoff Coefficients as Reported by Babbitt

Type of surface	Range in coefficients
Roof surfaces assumed to be impervious	0.70 - 0.95
Asphalt surfaces in good order	0.85 - 0.90
Macadam roadways	0.25 - 0.60
Gravel roadways and walks	0.15 - 0.30
Parks, gardens, lawns, meadows depending on surface slope and subsoil character	0.05 - 0.25
Wooded areas or forest land depending on surface slope and subsoil	0.01 - 0.20
Most densely populated or built-up portion of a city	0.70 - 0.90

Source: Sewerage and Sewage Treatment - Babbitt. 7th Edition, page 43, Table 14.

tion of the frequency of recurrence of storms of given intensity, as well as a determination of the time of concentration.

Coefficient of Runoff. Expressed as a decimal, the coefficient of runoff represents the fraction of rainfall on a given area that flows off as free surface water. Seldom, of course, are conditions such that all the rain falling on an area runs off, even when the entire surface is composed of pavement, roofs, or other impervious surfaces. Some of the rain is always absorbed in wetting the surfaces and some is held back in small depressions and irregularities. Evaporation also takes place, even during a storm.

At the beginning of a rainstorm, the runoff coefficient will be quite small but will increase gradually as the storm progresses until the soil has become saturated, impervious areas have been thoroughly wetted, and all depressions have been filled. From then on, the coefficient will remain substantially constant, affected only slightly by the intensity of rainfall. Other factors which also affect the coefficient are the type of soil, the slope and condition of the surface, and the size of the area. It is evident, therefore, that successful use of the rational method depends largely on the skill and judgement of the engineer in estimating suitable coefficients.

In practice, the design of urban storm drainage systems is usually based on constant values of the coefficient of runoff. For the purpose of the survey, average coefficients for different types of surfaces were developed from typical areas similar to those of Seattle and from results of studies reported by various authorities (Tables 13-2 to 13-4). A summary of typical values, as applied to conditions at Seattle, is given in Table 13-5. Coefficients listed in the last column of this table were used in all calculations of storm runoff.

Rainfall Intensity and Frequency. As here used, the term "design frequency" defines the period, or recurrence interval, during which each section of a given facility will be called upon at one time or another to carry a storm flow equal to or in excess of its capacity. At this frequency, on the average, surcharge of a section or local flooding will result. General flooding would result only in the event of a prolonged high intensity rainfall which exceeds the intensity for the design frequency.

Table 13-3. Runoff Coefficients as Reported by Various Authorities

Type of surface	Range in coefficients
For the most densely built-up portions of a district	0.70 - 0.90
For the adjacent well built-up portions	0.50 - 0.70
For the residential portions with detached houses	0.25 - 0.50
For the suburban portions with few buildings	0.10 - 0.25
Froehling - For the densely built center of the city	0.7 - 0.9
- For densely built residence districts	0.5 - 0.7
- For residence districts, not densely built	0.25 - 0.5
- For parks and open spaces	0.1 - 0.3
Imhoff - Very thickly built up, 140 persons per acre	0.80
- Closely built up, 100 persons per acre	0.60
- Well built up, 60 persons per acre	0.25
- Suburban, 40 persons per acre	0.15
- Unsettled, 6 persons per acre	0.05

Source: American Sewerage Practice, Vol. I - Metcalf and Eddy. Page 290, Table 93.

Note: All values cited by Metcalf and Eddy were taken from data given by Bryant and Kuichling, Froehling, Imhoff, and others.

Table 13-4. Runoff Coefficients Used in Tacoma Design

Type of area	Population density per acre	Calculated runoff coefficient
50 per cent development		
New residential	6	0.24
Old residential	9	0.30
100 per cent development		
New residential	12.5	0.39
Old residential	18.5	0.50
Multiple family	40	0.55
Strip commercial		0.56
Outlying shopping centers		0.85
Downtown commercial		0.90
Industrial, exclusive of railroad yards		0.65
Parks and rough terrain		0.10

Source: Metropolitan Tacoma Sewerage and Drainage Survey - Brown and Caldwell, June 1957. Chapter 7, page 127, Table 7-4 — Calculated Runoff Coefficients.

Table 13-5. Runoff Coefficients Used in Seattle Design

Zone type	Runoff coefficient
Industrial	0.75
Commercial	0.80
Residential - high density	0.60
Residential - medium density	0.50
Residential - low density	0.40
School grounds with buildings	0.30
Public parks	0.20
Playgrounds	0.25
Mixed industrial and residential	0.70

Selection of a design frequency is governed by such related considerations as (1) economic implications of local flooding; (2) type, nature and extent of areal development which might be subject to damage by flooding; (3) magnitude of applicable rainfall intensities; (4) size or extent of tributary area; and (5) economics of construction. In considering the problem of design frequency, it is obvious that the interval selected for a major flood control project must be much longer than that for urban drainage of the type under consideration in this report. Hence, with all factors taken into account and evaluated in relation to each other, a 10-year design frequency is recommended for trunk storm drainage facilities in the metropolitan Seattle area. Local drainage for areas not critically affected by flooding may well be designed for a shorter recurrence interval.

For a given frequency or recurrence interval, the intensity of rainfall varies inversely with the duration of the storm. In other words, heavy showers do not last as long as rains of lower intensity. This relationship can be illustrated as a curve, with the rate

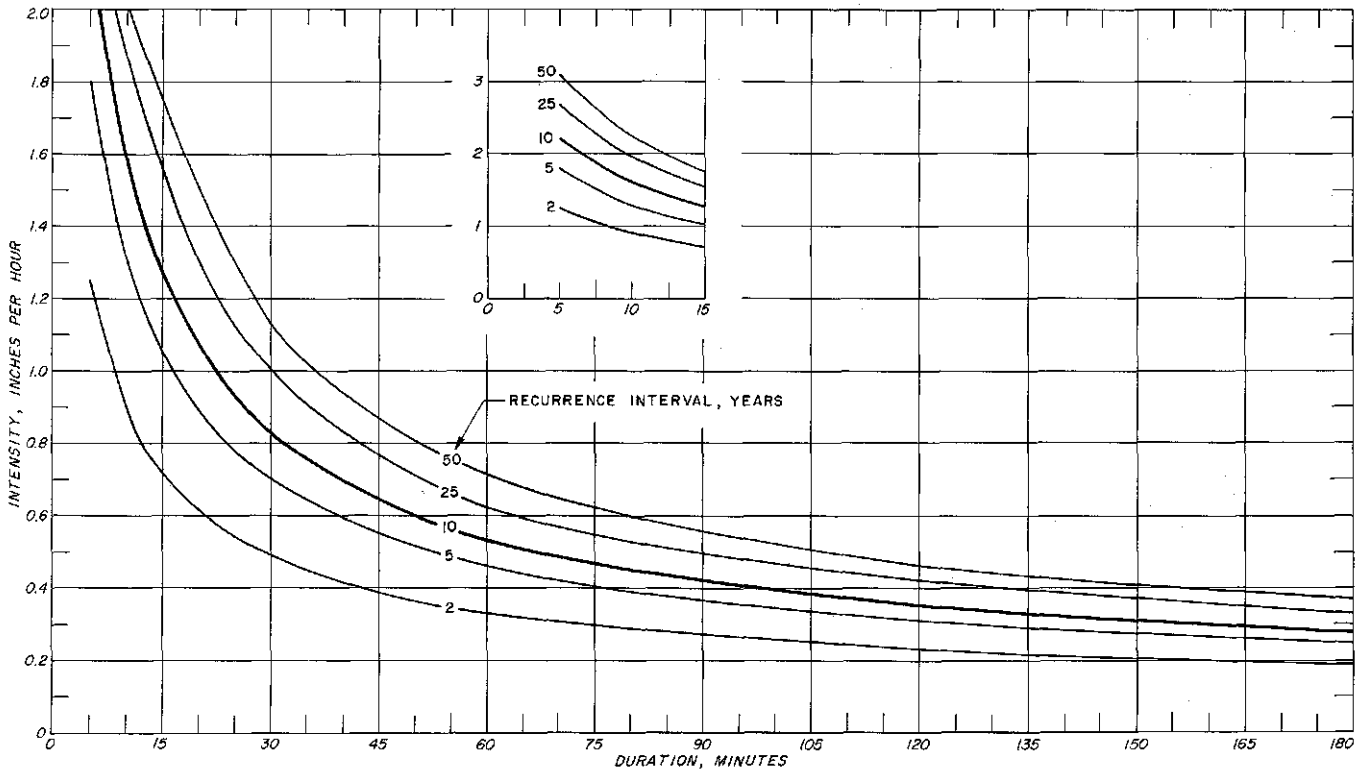


Fig. 13-1. Rainfall Frequency-Intensity-Duration for Seattle, January through December

Based on curves developed for Seattle by U.S. Weather Bureau and published in "Rainfall Intensity-Duration-Frequency Curves", Technical Paper No. 25. Design of trunk storm drainage facilities is based on use of the 10-year frequency curve.

of rainfall expressed in inches per hour plotted against the duration of the rain in minutes.

Calculations required to construct curves showing intensity-duration relationships for rainfalls of different frequency include an analysis of records of recording rainfall gages. Analyses of this type, as made by the U.S. Weather Bureau, are illustrated by the curves in Fig. 3-15, Chapter 3. For convenience in calculating runoff quantities, these curves were replotted in Fig. 13-1 to indicate the rate of rainfall in inches per hour for time intervals between 5 and 150 minutes. Intensities for longer durations, also determined by the Weather Bureau, are listed in Table 13-6. For use in claculating pipe sizes, the 10-year frequency-intensity record is given in Table 13-7.

It should be pointed out that the data on which Fig. 13-1 and Table 13-6 are based were derived from the annual series rainfall intensities (maximum value for each year). In this method, the possibility is ignored that the second highest intensity of some year might exceed the highest of some other year. If the partial duration series, which includes all maxima, were used, the actual intensities would have to be multiplied by the following factors:

2-year recurrence interval -- 1.13
5-year recurrence interval -- 1.04
10-year recurrence interval -- 1.01

For longer periods, the differences between the two series are regarded as insignificant.

Time of Concentration In accordance with the basic assumptions of the rational method, the maximum discharge at a particular point in a drainage system normally occurs when (1) the entire area tributary to the point is contributing flow, and (2) the rainfall intensity corresponds to that taken from the intensity-frequency curve at a rainfall duration equal to the time required for water to flow from the most remote point of the tributary area. As thus used, the most remote point refers to the point from which the time of flow is greatest.

Table 13-6. Rainfall Intensities with Durations of 2 to 24 Hours

Duration, hours	Recurrence interval, years				
	5	10	25	50	100
2	0.31	0.35	0.42	0.46	0.52
3	0.25	0.28	0.33	0.37	0.41
6	0.18	0.20	0.24	0.26	0.29
12	0.13	0.15	0.17	0.19	0.21
18	0.11	0.12	0.14	0.16	0.18
24	0.10	0.11	0.13	0.14	0.16

Intensities expressed as inches per hour.

Source: Rainfall Intensity-Duration-Frequency Curves, *U.S. Department of Commerce, Weather Bureau Technical Paper No. 25, December 1955.*

Table 13-7. Ten-Year Rainfall Intensity-Duration at Seattle

Duration, minutes	Intensity, inches per hour	Duration, minutes	Intensity, inches per hour
5	2.23	34	0.75
6	2.10	36	0.72
7	1.97	38	0.70
8	1.85	40	0.68
9	1.73	42	0.65
10	1.62	44	0.63
11	1.53	46	0.63
12	1.46	48	0.61
13	1.40	50	0.59
14	1.35	52	0.58
15	1.30	54	0.57
16	1.25	56	0.56
17	1.20	58	0.55
18	1.15	60	0.53
19	1.10	65	0.50
20	1.05	70	0.48
21	1.01	75	0.45
22	0.98	80	0.44
23	0.96	85	0.42
24	0.94	90	0.41
25	0.92	95	0.40
26	0.90	100	0.39
27	0.88	105	0.38
28	0.86	110	0.37
29	0.85	115	0.36
30	0.83	120	0.35
32	0.79		

Known as the time of concentration, the greatest time of flow to a particular point is composed of two parts, "inlet time" and "time of flow". Inlet time is that required for water to flow overland from the most remote point to the first inlet. Time of flow is that time during which the water flows in the drainage system to the point in question.

Inlet time varies with surface characteristics such as slope, distance to inlet, type of soil, vegetation cover, surface irregularities, and similar factors. A detailed analysis of inlet time is quite involved and is not justified in municipal storm drainage work. Instead, an abbreviated graph, such as that shown in Fig. 13-2, may be employed to obtain an adequate estimate of this time. Time of flow in the drainage system can be determined from hydraulic computations.

Design Loadings - Combined Systems

In the case of combined systems, design is based usually on storm flow considerations exactly the same as those used for the design of drainage systems. Until very recently, however, design of combined

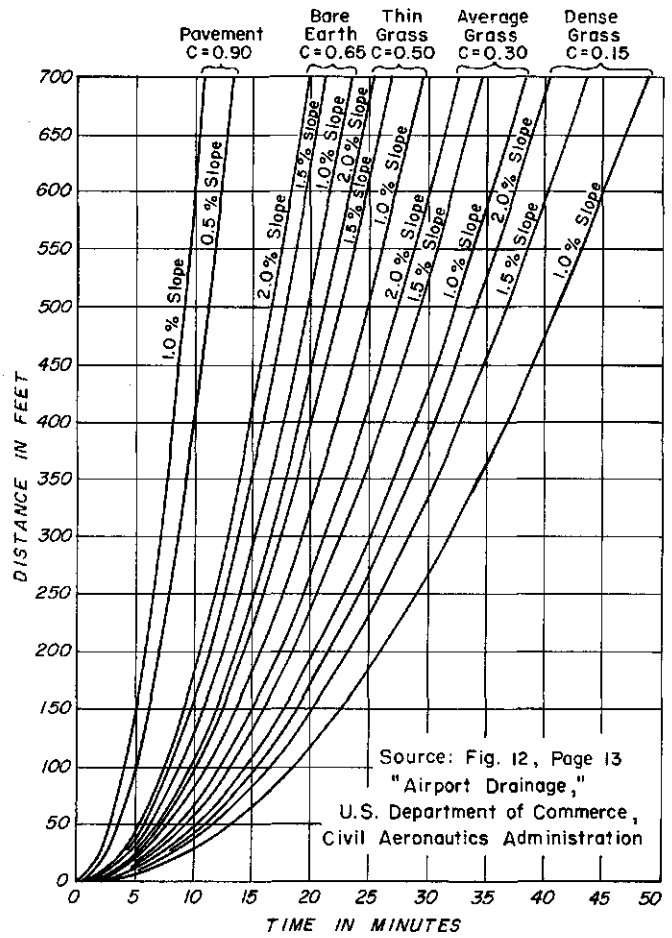


Fig. 13-2. Storm Water Inlet Time

sewers in Seattle was based on a fixed rainfall rate of 1.0 inch per hour and a fixed runoff coefficient of 0.25. Application of these values results in a calculated runoff of 0.25 cubic feet per second per acre, regardless of the size, slope or character of the tributary area. For average drainage areas, the rate of runoff thus determined is slightly greater than one-half that which would obtain from the use of the rational method with a 10-year storm. In fact, the Seattle method is roughly equivalent to the rational method, using a storm intensity occurring on the average of once every two years. On the average, therefore, areas of Seattle served with a combined system designed on the standard Seattle basis can expect to experience flooding difficulties once in each two year period.

Design Loadings - Intercepting Sewers

An intercepting sewer receives sanitary sewage from a number of transverse sewers or outlets and, in some cases, predetermined quantities of storm water (from a combined system), and conducts such wastes to a selected point for treatment and disposal. In the case of separate systems, peak flows delivered to an intercepting sewer, or interceptor, consist of a combination of peak sanitary flow plus an allowance for storm water infiltration and inflow. In combined systems, peak flows picked up by an interceptor includes only a portion of the total flow in the sewer being intercepted. The remainder overflows and is discharged at upstream points to adjacent waters.

In a combined system, the intercepted flow is commonly expressed as a ratio of the dry weather flow. This ratio can be expressed also in terms of the number of times per year, or per summer, that excess flow would have to be bypassed. If the interceptor is designed on the basis of picking up only the sanitary flow plus the allowance for storm inflow used in sanitary design, nearly every rainfall will result in overflow and a consequent discharge of sewage diluted with storm water. On the other hand, an intercepting sewer large enough to carry storm flows occurring at the frequency of 10 years used in drainage design may be as much as 50 times larger than that required for sanitary sewage only and would be prohibitive in both size and cost.

To analyze problems of interceptor design in terms of the frequency and amount of overflow, both on a yearly basis and for the critical summer months, it is necessary to utilize data on the intensity and duration and on the total quantity of rainfall. Intensity-duration curves for storms on a yearly basis have been given already in connection with design criteria for storm drainage systems (Fig. 13-1). By replotting the summer rainfall data given in Fig. 3-17 to show the intensity-duration relationship, a family of curves for summer conditions can be developed (Fig. 13-3). By using these curves and assuming average values for time of concentration and per capita sewage flow, it is possible to calculate the frequency of overflow from interceptor sewers of various capacities. Frequencies calculated for concentration times of 30 to 120 minutes and interceptor capacities of 1 to 70 times the average dry weather sanitary flow are plotted in

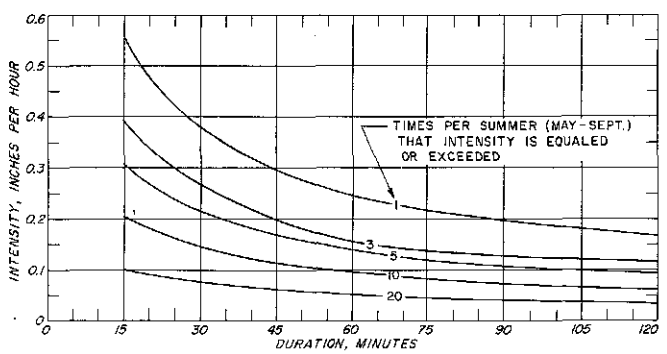


Fig. 13-3. Summer Rainfall Frequency-Intensity-Duration for Seattle, May through September

Based on analysis of summer rainfall intensities during the period 1940-1949, inclusive (Fig. 3-17).

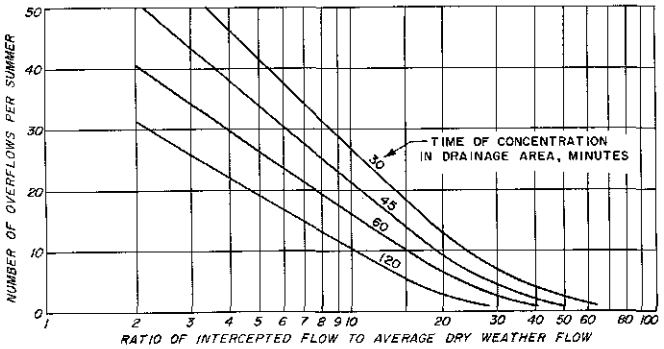


Fig. 13-4. Effect of Interceptor Capacity on Frequency of Overflow from Combined Sewers

Based on (1) summer rainfall intensity-duration at Seattle, (Fig. 13-3); (2) average dry weather flow (DWF) of 1,500 gallons per acre per day; and (3) average coefficient of runoff of 0.33 (summer).

Fig. 13-4. These curves are used later in Chapter 18 in developing plans for separation of combined sewers in the city of Seattle.

Where contamination of recreational waters or pollutional effects of overflows are of primary concern, it is necessary to know the quantity, as well as the frequency of overflow. Based on a method used by J. E. McKee,[1] a statistical study was made of summer and yearly rainfall at Seattle for the rainfall period 1946 to 1950. With the exception of rearranging records to cover a summer period of May through September, necessary data for this phase of the work were compiled and results were calculated by Mr. H. N. Phillips assisted by Mr. D. M. Fine, both of the Seattle engineering department. Results of that study are summarized in Fig. 13-5 which shows the per cent of sewage overflow, occurrence of rainfall, and per cent of total sewage not intercepted for the full year. It is evident that the total quantity of sewage not intercepted is relatively small, even with an interceptor designed to carry only two times the average dry weather flow of sanitary sewage and ground water. It is evident also that the increase in percentage of sewage intercepted with larger interceptors is very slight. By increasing the interceptor capacity from two to five times the dry weather flow, the quantity of sewage intercepted will be increased under summer conditions only from 98.4 to 99.4 per cent while the interceptor capacity is increased 250 per cent.

In the design of flood basins and holding tanks to be used on interceptors of limited capacity, or where overflow cannot be permitted, it is necessary to utilize information on the quantity of runoff from typical summer storms. Such information can be obtained only through a detailed analysis of individual storms occurring over a sufficiently long period to eliminate yearly variations. For this purpose, a special study was undertaken, using records of the U.S. Weather Bureau for the period 1940 to 1949, inclusive.

To develop the required information, hourly precipitation data for the Seattle station were analyzed by

[1] *McKee, J. E.,* Loss of Sanitary Sewage through Storm Water Overflows, *Journal of the Boston Society of Civil Engineers, 34-55-1947).*

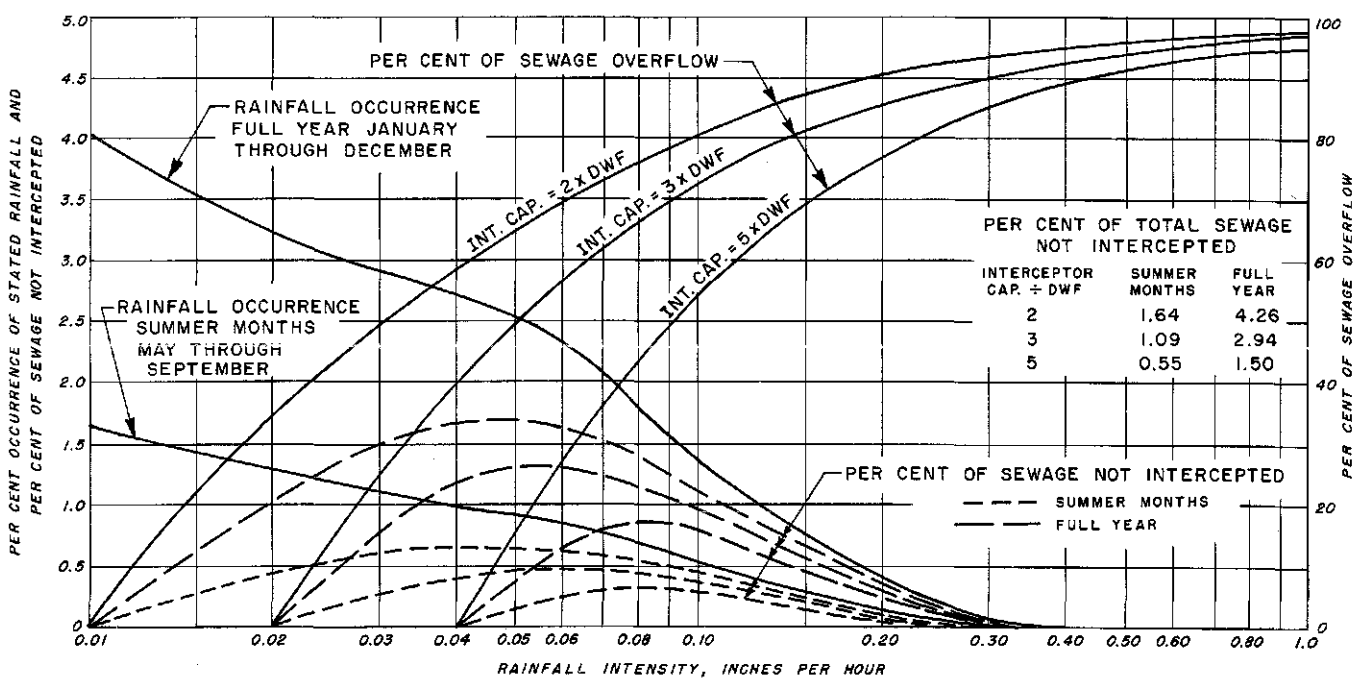


Fig. 13-5. Effect of Interceptor Capacity on Quantity of Sewage Not Intercepted from Combined Systems

Based on an analysis of Seattle rainfall data for the period 1946 to 1950, using method developed by J. E. McKee (Journal of the Boston Society of Civil Engineers, 34-55-1947).

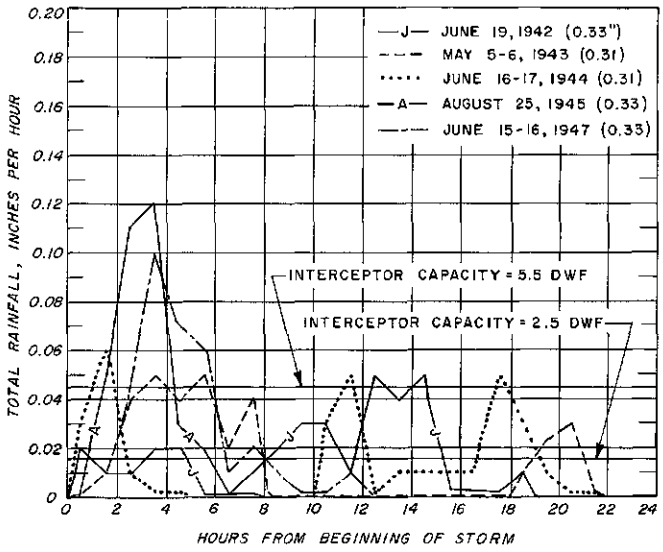


Fig. 13-6. Analyses of Typical Storms in the Six Times per Summer Group

tabulating the total precipitation from each storm occurring during the May through September period of each year. A storm was defined as the maximum total rainfall occurring during a 24-hour period, and two consecutive 24-hour periods were considered as two separate storms. Precipitation data thus obtained were then tabulated by groups according to depth and in ascending order of magnitude. Following that, the depth groups corresponding to selected storm frequencies were determined by counting the number of occurrences.

Precipitation depths corresponding to frequencies of once per summer to six times per summer were found to be as follows:

Number of occurrences per summer May through September	Total depth of precipitation during storm, inches
1	0.80 - 0.84
2	0.65 - 0.69
3	0.45 - 0.49
4	0.40 - 0.44
5	0.35 - 0.39
6	0.30 - 0.34

In addition to the above frequency data, which cover a 10-year period, Weather Bureau records indicate that the maximum summer rainfall over a 65-year period amounted to 1.91 inches in 24 hours. In effect, this figure represents a storm which would be expected to occur, on the average, once in 65 years and is significant to the extent that it establishes the maximum runoff likely to occur during the summer season.

Having determined the frequency grouping of summer storms, typical occurrences in each group were further analyzed by plotting each storm to show total hourly depth of rainfall. Fig. 13-6 shows the results obtained for the six times per summer group, 0.30 to 0.34 inches per 24 hours. Similar graphs were made for storms falling in each frequency group.

To determine the quantity of runoff resulting from summer storms, as well as the portion picked up and the portion overflowing from interceptors of various sizes, it was necessary next to express runoff, interception and overflow in terms of hourly depth of precipitation. For this purpose, it was assumed that the runoff from 0.01 inches per hour of rainfall equals the average dry weather sanitary sewage flow. Based on a runoff coefficient of 0.33, the runoff from 0.01 inches per hour is equivalent to 1,500 gallons per acre per day, which in turn is equivalent to the average dry weather flow from 15 persons per acre, with allowance for some infiltration.

If interception rates are expressed as multiples of dry weather sanitary flow, they can be expressed also as equivalent rainfall. For example, if an interceptor has a capacity of five times the average dry weather sanitary flow, it has an equivalent capacity to intercept storm flow equal to the runoff from 0.04 inches per hour of rainfall in addition to the average sanitary flow (0.01 inches per hour).

For any given interceptor capacity, expressed as equivalent hourly rainfall, the amount intercepted is the area under the rainfall curve (Fig. 13-6) or under the interceptor capacity, whichever is the lesser. Overflow is indicated by that area under the rainfall curve which is above the interceptor capacity.

Based on a similar analysis of each rainfall frequency group, the quantity of rainfall intercepted for several different interceptor capacities is shown in Fig. 13-7. By subtracting the indicated depths given in the figure from the total rainfall per storm for the

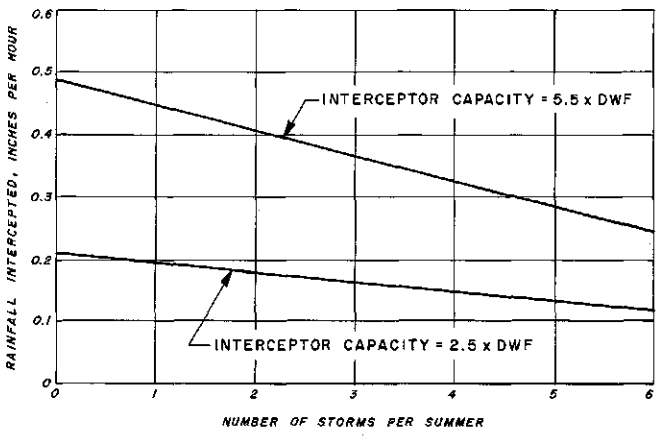


Fig. 13-7. Effect of Interceptor Capacity on Average Quantity of Rainfall Intercepted for Typical Summer Storm

Based on (1) analysis of summer storms at Seattle (May through September, 1940-1049, inclusive); (2) average dry weather sewage flow (DWF) assumed equivalent to runoff from rainfall of 0.01 inch per hour; and (3) average runoff coefficient of 0.33 (summer).

indicated frequency group, the capacity of a holding tank can be computed for any unit of tributary area. Fig. 13-8 shows the capacities of tanks per acre of tributary area as related to the number of overflows per summer and the capacity of the interceptor.

Because a holding tank must be designed for a definite storm frequency group, it is obvious that the tank itself will overflow during storms of greater intensity than that for which it was designed. It follows, therefore, that the average quantity of storm water overflowing a tank of any given capacity is the total non-intercepted flow of all storms greater than the one for which the tank is designed, divided by the number of years included in the record. Results of such an analysis of all summer rainfalls for the 10-year study period, 1940-1949, are shown in Fig. 13-9. Information given in this figure is useful in determining the frequency and degree of contamination of receiving waters in the vicinity of a point of overflow disposal.

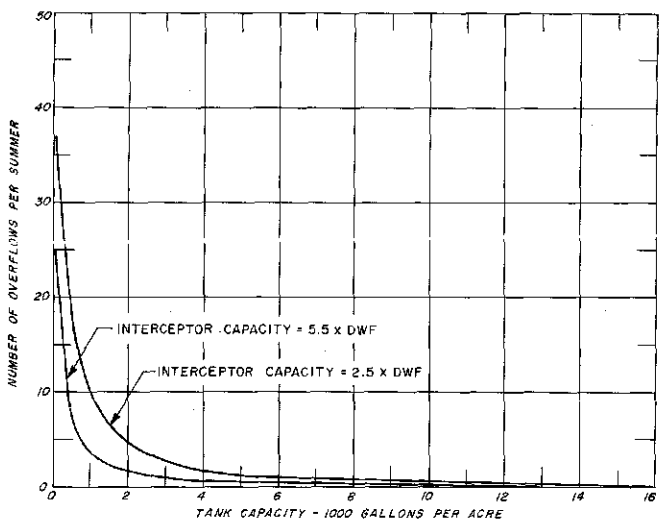


Fig. 13-8. Effect of Interceptor Capacity on Capacity of Storm Water Holding Tanks for Summer Storms of Various Frequencies

Based on Fig. 13-7.

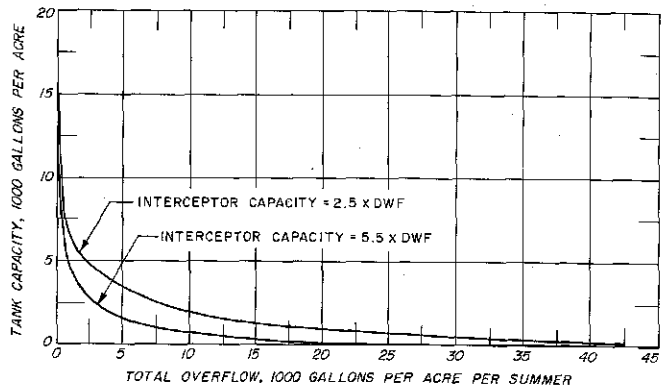


Fig. 13-9. Effect of Interceptor Capacity on Quantity of Overflow from Storm Water Holding Tanks

Based on Fig. 13-7

Trunk and Intercepting Sewers

For purposes of this report, the extent of trunk and intercepting sewer facilities is limited to minimum local service areas of approximately 1,000 acres. That is, trunk service is provided for each tributary natural drainage area to a point where not more than 1,000 acres remain beyond the upper end of the trunk. Local drainage areas smaller than 1,000 acres may, of course, be served along the route of the trunk sewer. Based on average population densities and ground slopes, this limitation results in a minimum trunk sewer size in the range of 12 to 15 inches in diameter. In establishing this definition of a trunk sewer, it is assumed that local sewerage service, i.e., local service trunk lines, laterals and house connections, will be provided by local agencies.

Trunk sewer capacity must be adequate to convey the ultimate peak rate of flow. Peak flow occurs generally during wet weather and is the sum of several loadings calculated separately for each section of sewer, namely: (1) peak flow of sanitary sewage; (2) peak flow of industrial waste; (3) winter infiltration; and (4) winter storm inflow (Table 13-1).

Ground water infiltration rates applicable to new trunk sewer construction are predicated on the use of reinforced concrete pipe equipped with rubber gasket type joints. These rates assume also that lateral and branch sewers, and house connections will be constructed with permanently tight joints. To achieve such an objective it will be necessary: (1) to provide for rigid inspection and testing of all new lateral and trunk sewers; (2) to adopt rigid specifications for the installation and testing of house connections; (3) to inspect premises periodically for possible roof leader and yard drain connections to the sewers; and (4) to prohibit direct connection of house sewers to trunk sewers.

The ability of a sewer to transport suspended solids contained in sewage depends on the velocity of flow in the sewer. A velocity of 2 feet per second for pipes flowing full is considered to be the minimum which will present deposition and keep the pipe clean. Diameters of trunk sewers were determined by means of Manning's formula, using a roughness coefficient of 0.013. Although modern reinforced concrete pipe in larger sizes may exhibit lower values of the roughness coefficient, use of the single value provides some allowance for contingencies inherent in preliminary design.

Storm Drains

For the same reasons as those set forth for sanitary sewers, the extent of trunk storm drainage facilities

is limited to minimum local service areas of approximately 160 acres. Under conditions of minimum slope and average runoff, this limitation results in minimum trunk storm drain sizes in the range of 48 to 60 inches in diameter.

Storm drains are sized to carry the maximum flow from a given tributary area which results from a storm of a specified frequency. As indicated earlier, the procedure used in determining the capacities of all proposed storm drains, as well as in checking the adequacy of existing lines, is based on the rational method and a storm recurrence interval of 10 years. For average conditions with respect to area, slope and development, and for a storm recurrence frequency of 10 years, the resulting runoff amounts to approximately 0.5 cubic feet per second per acre.

As distinguished from sanitary sewers, storm drains do not have to be constructed so as to exclude ground water infiltration. Lower cost pipe employing tongue and groove joints sealed with mortar may therefore be used. Diameters of storm drains were determined, as for sanitary sewers, using Manning's formula and a roughness coefficient of 0.013. Because of the quantities of grit which are inevitably carried by storm drainage, a minimum full velocity of 3 feet per second was adopted.

Storm Channels

Open channels now carrying storm drainage will, in many cases, continue to be used for many years before it becomes necessary to replace them with a closed pipe. In fact, it is unlikely in the case of some creeks that the natural channel will ever be replaced with pipe. Carrying capacities of open channels were calculated by means of Manning's formula, using the following values for the coefficient of roughness:

Earth, smooth, free from weeds	0.020
Earth, good condition, some weed growth	0.025
Earth, stones and weeds	0.030
Earth, bad condition, weeds and brush	0.035
Concrete lined	0.015

The necessity for lining was determined separately for each section, based on velocity of flow and on the characteristics of the adjacent area. Similarly, decisions with respect to fencing were made for each individual section, taking into account existing and probable future uses of adjacent land.

Combined Sewers

Although no combined sewers are proposed under any of the recommendations contained in this report, it has been necessary to check the adequacy of existing lines and to determine where the capacity of such lines should be augmented. For that purpose, capacities were calculated by adding the peak storm runoff, as derived for storm drains, to the peak sanitary flow, as derived for sanitary trunk sewers. As in all combined systems, the problem of grit deposition is of particular concern. To alleviate this problem, which appears to be particularly severe in the Seattle area, continued use of existing grit accumulators is essential. It is essential also that new accumulators be constructed where necessary to prevent the entrance into intercepting sewers of excessive loads of gravel and sand.

Inverted Siphons and Force Mains

Inverted siphons and force mains, unlike gravity sewers and storm drains, always flow full and must be designed with proper velocities to prevent the deposition of solids. To insure an adequate minimum velocity, it is necessary in many cases to resort to the use of two or some times three lines.

For inverted siphons, the inlet structure provides for use of one line until the flow increases to the point where adequate velocities can be maintained in two or more lines. Force mains, which are used in conjunction with pumping stations, can be similarly employed in sequence to maintain necessary minimum velocities.

Preliminary plans developed for this report call for the use of welded steel pipe, concrete lined and coated. In many cases, of course, other types of pipe could be used. Diameters of inverted siphons and force mains were calculated by means of the Williams and Hazen formula, using a coefficient of roughness of 110 and 120 respectively.

Bypass Structures

Bypass structures are used on trunk and intercepting sewers serving combined areas and are designed to limit the quantity of sewage picked up by intercepting sewers during storms. Although not always possible, it is desirable to locate such structures on the trunk line being intercepted rather than on the interceptor itself. This is particularly desirable in situations where the interceptor also serves areas with separate sewers. With the bypass on the combined trunk, a smaller quantity of sanitary sewage is permitted to overflow than would be the case if the bypass were located on the interceptor.

Several types of bypass or regulating devices can be used, ranging from a simple overflow weir to complicated mechanical gates controlled by water level. In general, because of the difficulty in keeping mechanical devices operating properly, it is best to use only the simplest form wherever possible. Some times, however, due to controlling conditions of elevation and other factors, a float-operated or power-operated device is required.

Actually, there are nearly as many variations in

design of overflow structures as there are trunk lines to be intercepted. Each installation, therefore, requires special consideration and study during final design.

Storm Water Pumping Stations

Storm water pumping stations are required in locations where the elevation of the area being drained is such that gravity flow of storm runoff to the receiving body of water is not possible at all times. This condition may occur only when high tides or high river flows coincide with peak storm water runoff from the tributary area. In any case, stations are designed for peak runoff of a specific frequency from the tributary area. All stations are laid out with propeller pumps driven by diesel engines. Because it is necessary that such stations be able to operate at any time and under practically any condition, they are designed both to be as completely automatic as possible and to be completely independent of outside sources of power.

Sewage Pumping Stations

Sewage pumping stations on trunk or intercepting sewers are designed for the ultimate peak wet weather flow of the tributary area. All stations are laid out with centrifugal or mixed flow pumping units, as required by capacity and discharge head requirements. At each one, the number of units is dictated by the range of flow to be provided for, from present minimum to future peak. In some cases, as described later under specific projects, design calls for mechanical equipment to be installed incrementally as required by flow increases due to development of the tributary area.

Normally, pumps are driven by variable speed electric motors, with the speed controlled by the required rate of pumping. Standby engine drives would be installed at stations where bypassing cannot be allowed. For pumping heads greater than approximately 125 feet, the standard non-clog centrifugal pumps normally employed in sewage pumping must be used in pairs, so arranged that one pump discharges into the suction of the second. By this means, the total discharge head may be increased to approximately 200 feet, or with special pump casings, to 250 feet.

All pumping stations are laid out with easy access to both the wet and the dry sides, and adequate ventilation is provided to prevent condensation on mechanical and electrical equipment. In addition, design calls for control and metering devices and all other necessary appurtenances.

Sewage Treatment Plants

The capacity of a sewage treatment plant is defined on the basis of average dry weather flow. This implies that proper allowances are made for peak rates of flow due not only to normal variations in dry weather flow but to storm water inflow and ground water infiltration, both of which occur at a maximum rate during winter months.

Two types of sewage treatment are considered herein, namely, primary and secondary. Primary treatment consists of (1) coarse screening to remove large objects which would damage pump and other equipment; (2) grit removal; (3) sedimentation which removes substantially all settleable and floatable material; (4) sludge digestion; and (5) disposal of digested sludge. Secondary treatment includes all the steps of primary treatment and provides in addition for oxidation of organic material still contained in the sewage after sedimentation. Secondary treatment provides also for an additional sedimentation step in which humus sludge formed in the oxidation step is removed. Although there are several types of oxidation processes, only two types are here considered, namely, activated sludge and trickling filtration.

Unless otherwise noted under the specific projects, all treatment plants are designed on the following basis:

1. Pretreatment facilities, to accommodate peak flow.
2. Preaeration and primary sedimentation tanks, two hours detention at average flow.
3. Trickling filters, 75 pounds of BOD per 1,000 cubic feet per day.
4. Activated sludge aeration tanks, 35 pounds of BOD per 1,000 cubic feet per day.
5. Sludge digestion tanks, 0.20 pounds of dry suspended solids per cubic foot per day.
6. Secondary sedimentation tanks, two hours detention at average flow.

Treatment plants considered under the various alternatives are laid out to provide for incremental or stage construction under which such components as sedimentation tanks, trickling filters, and sludge digestion facilities will be added as the load on the plant increases. Provision is made also for incremental installation of equipment as needed. Structures not readily adaptable to future enlargement, such as inlet works, channels, pipelines, and buildings to house pumps and other equipment are sized for ultimate design flow requirements.

Digested sludge may be disposed of in one of the several ways described in Chapter 9. Because conditions affecting sludge disposal vary between the alternative treatment plant locations, the method to be employed at each is discussed separately in connection with each specific treatment operation (Chapter 15).

Treatment plants are laid out to provide for first class construction in all respects and take into account the fundamental requirement that the design must be of such nature as to permit the utmost flexibility and

ease of operation. Flexibility of operation implies the provision of all necessary bypasses, drains, duplicate equipment, gates and other devices to the end that individual units may be taken out of service for cleaning, repair, or for any other purpose. Ease of operation implies the development during design of an efficient plant layout and a program of treatment requiring a minimum of travel on the part of plant attendants. It also implies provision of such features as good lighting, safe stairways, and necessary railings. Suitable landscaping and fencing are considered to be essential features of adequate design. In general, layouts of treatment plants follow the applicable design standards recommended by the Pollution Control Council of the Pacific Northwest Basin.

CONSTRUCTION COSTS

As stated previously, construction and operating costs are based on preliminary layouts of the proposed sewerage and drainage facilities. For estimating purposes, therefore, prices of comparable work were obtained from all available sources of current information. Manufacturers, suppliers of material and equipment, and local contractors were consulted on specific questions. Costs of large sewers, pumping stations and sewage treatment plants, as derived from actual projects designed by the firm of Brown and Caldwell, were relied upon heavily but were adjusted to local conditions as far as possible. Wherever possible, use was made of costs of large sewers and interceptors recently constructed by the city of Seattle.

In considering the estimates, it is important to realize (1) that changes during final design quite possibly will alter the totals to some degree, and (2) that future changes in the cost of material, labor and equipment most certainly will be reflected in any costs here presented. On the other hand, since the relative economy of alternative projects can be expected to change only slightly with an increase or decrease in component prices, decisions based on present comparisons will remain valid more or less indefinitely.

All costs in this report are based on an Engineering News-Record (ENR) Construction Cost Index of 800. Although this index now stands at 743 (January 2, 1958), all contractors, in bidding a job, allow to some extent for an increase in prices during the construction period. It is assumed, therefore, than an index of 800 properly reflects current (spring 1598) bidding conditions.

As indicated by the curve in Fig. 13-10, which portrays the trend of the ENR index since 1940, costs have been increasing for many years. Since 1931, for example, the index has increased steadily at approximately 5.5 per cent per year, and now stands at 743 as compared with 500 early in 1950.

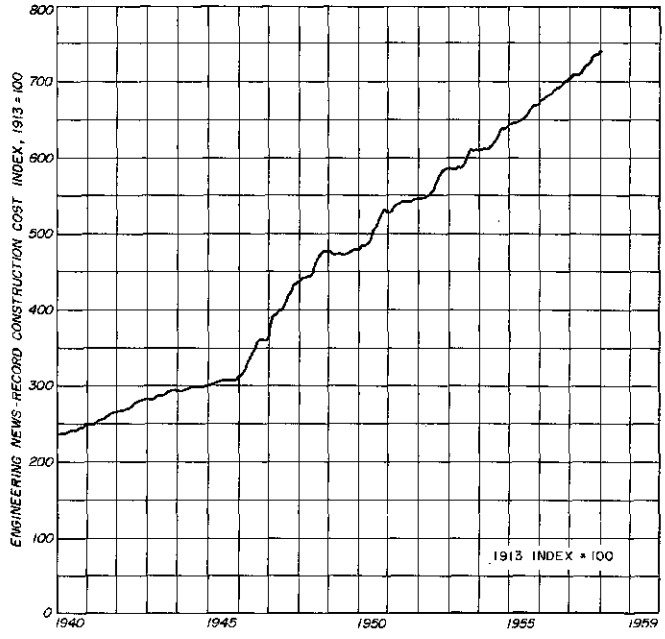


Fig. 13-10. Engineering News-Record Construction Cost Index

Present day costs are more than double those prevailing in 1946. All cost data used in this report have been adjusted to an index of 800, which properly reflects the current (spring 1958) bidding index.

If costs continue to rise, appropriate allowances must be made for the increases which can be expected by the time construction is started. The amount to be added to the estimates presented herein will vary with the date of bidding and will amount roughly to five per cent per year.

As here developed, unit costs do not include allowances for land acquisition, engineering and contingencies, or legal services. Where rights-of-way or plant sites are required for specific structures, such as pumping stations and treatment plants, appropriate allowances are included in the estimated costs of each particular unit. Likewise, engineering and contingency allowances, expressed as a percentage of the construction cost, are added as a specific item to each estimate. These allowances are further discussed in a subsequent section of this chapter.

Trunk Sewers and Storm Drains

Unit costs of trunk sewers and storm drains (Table 13-8) are based on actual construction costs in the Puget Sound area and on pipe costs obtained from local manufacturers. Figures are given for dry to moderately wet trench conditions and include the cost of pipe, pipe laying and jointing, excavation, backfill with compacted granular material to a height of 6 inches above the crown of the pipe, manholes at standard spacings, and cleanup. Prices are based on the use of vitrified clay or reinforced concrete pipe with plas-

Table 13-8. Basic Cost Data for Sewers and Storm Drains

Pipe size, inches	Cost, dollars per lineal foot for stated depths of dry or moderately wet excavation							Imported gravel backfill, add dollars per lineal foot per foot of depth	Storm drains, deduct dollars per lineal foot	Repaving and interference with utilities, dollars per lineal foot
	6-foot	9-foot	12-foot	15-foot	18-foot	21-foot	24-foot			
12	9.30	9.80	10.40	11.00	11.90	13.20	14.80	0.50	1.80	1.10
15	10.80	11.30	12.20	12.90	13.90	15.30	17.10	0.50	1.80	1.10
18	12.10	12.60	14.10	14.90	16.00	17.30	19.20	0.60	1.90	1.20
21	13.50	14.10	14.80	16.70	17.90	19.60	21.70	0.60	1.50	1.20
24	16.10	16.60	17.40	18.10	19.00	21.90	24.10	0.70	2.40	1.30
27	18.10	18.80	19.60	22.60	23.90	25.80	28.30	0.70	2.70	1.30
30	20.00	20.70	21.60	23.60	25.00	27.20	29.90	0.80	2.80	1.40
33	21.10	21.90	23.00	25.50	27.20	29.70	32.70	0.90	2.90	1.50
36	22.00	23.00	24.20	27.30	29.20	32.00	35.40	1.00	3.00	1.60
39	25.70	26.70	28.00	30.20	32.10	35.10	38.70	1.10	3.10	1.70
42	28.40	29.50	30.80	32.90	34.90	38.00	41.80	1.10	3.50	1.80
45	30.70	31.80	33.20	35.50	37.60	40.80	44.80	1.20	3.60	1.80
48	33.40	34.60	36.00	39.40	41.60	45.00	49.20	1.30	3.90	1.90
51	–	37.20	38.70	40.60	45.30	48.80	53.10	1.30	4.00	2.00
54	–	39.70	41.30	43.20	48.30	51.90	56.40	1.40	4.40	2.00
57	–	42.20	43.80	45.80	51.30	55.10	59.80	1.40	4.60	2.10
60	–	45.40	47.10	49.20	54.70	58.70	63.50	1.50	4.60	2.20
63	–	48.00	49.80	52.00	57.90	62.00	67.00	1.50	4.80	2.20
66	–	50.60	52.50	54.70	61.00	65.20	70.50	1.60	4.90	2.30
72	–	58.00	59.90	62.30	69.70	74.20	79.70	1.70	5.00	2.40
78	–	63.90	66.00	68.50	77.30	82.10	88.00	1.80	5.50	2.50
84	–	–	72.90	75.60	79.00	91.70	98.00	1.90	6.00	2.60
90	–	–	82.50	85.40	89.00	102.40	109.10	2.00	6.30	2.70
96	–	–	90.10	93.00	96.70	111.40	118.30	2.10	6.70	2.80
102	–	–	103.90	107.10	111.00	123.60	130.90	2.20	7.10	2.90
108	–	–	–	117.80	121.80	137.00	144.60	2.30	7.60	3.10
114	–	–	–	129.00	133.50	140.50	160.00	2.60	8.00	3.40
120	–	–	–	137.40	142.10	149.30	172.60	2.60	8.40	3.50

Costs are based on ENR Construction Cost Index of 800 and provide for use of either vitrified clay or reinforced concrete pipe to 21 inches in diameter, and reinforced concrete rubber gasket pipe for 24 inches in diameter and larger. Costs include excavation, backfill, laying and jointing, manholes, cleanup, paving and contractor's overhead and profit. Initial backfill is imported granular material to a depth of 6 inches above top of pipe. Imported material for complete backfill is provided by additional item. Deduction for storm drains provides for use of tongue and groove reinforced concrete pipe instead of that specified for sewers. Cost of sheeting is included for trench depth greater than 12 feet. For wet construction of ordinary difficulty, add $4.00 per lineal foot plus $0.40 per lineal foot per foot of depth over 10 feet. For very difficult wet construction, the costs are to be estimated specially for each case. Costs do not include allowance for engineering and contingencies or rights-of-way.

tic or rubber gasket joints for sewers and on tongue and groove pipe for storm drains. Imported gravel backfill, use of tongue and groove joints, repaving, and allowance for wet trench conditions are provided for by the use of additive and subtractive items. Special estimates were made for extremely wet or otherwise difficult construction conditions. In each such case, details of construction and estimated costs are given under the project estimates.

Force Mains and Inverted Siphons

Unit costs of force mains (Table 13-9) include the cost of pipe, pipe laying, trench excavation, backfill and cleanup. Pipe costs are based on welded steel, cement lined and coated, and having an appropriate thickness for the pressures involved. Other types of pipe, such as cast-iron or asbestos-cement, are acceptable, however, and may be substituted for the lined and coated steel in any particular installation.

Costs for repaving (Table 13-8) are added separately where applicable. Separate estimates are given also for inverted siphons, including inlet and outlet structures. In all cases, details of construction and estimated costs are given under the specific project.

Intercepting Sewers

Because intercepting sewers are relatively large and are usually located in areas of difficult construc-

Table 13-9. Unit Costs for Force Mains

Diameter, inches	Cost, dollars per lineal foot
10	6.10
12	6.80
14	7.50
18	9.60
21	11.30
24	13.25
27	15.20
30	17.60
33	19.80
36	22.30
42	26.80

Costs are based on an ENR Construction Cost Index of 800 and provide for use of welded steel pipe, concrete lined and coated. Costs include excavation, backfill, laying and jointing, clean-up, and contractor's overhead and profit. Costs do not include engineering and contingencies, rights-of-way, or repaving. For costs of repaving, see Table 13-8.

tion, it is not practicable to develop unit costs for estimating purposes. Instead, each line is estimated separately, section by section, based on conditions determined from careful field inspection. In developing the individual estimates, consideration was given to the effect of such factors as: (1) foundation conditions, as revealed by an analysis of available soil borings; (2) presence of ground water and the precautions necessary for its control; (3) interference with existing utilities such as water, gas, electric, and telephone lines; and (4) surface conditions as related to proximity of buildings and traffic interference. Details of construction conditions and final cost estimates for each section of interceptor are given under specific projects.

Tunnels

Because of the extremely variable geological conditions in the Seattle area (Chapter 3), the detailed estimation of tunnel costs for specific locations is subject to many uncertainties. Soil conditions may change from block to block horizontally and within a few feet vertically, with the result that information available at a particular location may not be characteristic of the area as a whole. Along any route the probability is that some good and some bad tunneling conditions will be encountered. To determine the relative amounts of each would, of course, require a detailed program of soil investigation. Such a program, while a definite necessity prior to final design, is neither economically feasible nor absolutely necessary for the purpose of a preliminary study.

It is apparent that present knowledge of geological conditions in the Seattle area is insufficient to permit a detailed estimation of tunnel costs. Nevertheless, estimates must be made in order to evaluate the various projects and must necessarily be of a generalized type. Moreover, they must be liberal enough to allow for the high degree of uncertainty.

Because of the uncertainty with respect to geology, the application of general costs to a specific location may result in an estimate which may appear either unreasonably high or unreasonably low. On the average, however, it is believed that use of the unit tunnel costs presented herein will afford a reasonably close approximation to the final construction cost.

Probably the best method of developing the necessary unit costs is that of utilizing the costs of tunnels already constructed in the Seattle area. Those costs undoubtedly represent average conditions, that is, some good and some bad tunneling. Tunnel costs in other localities, though not as useful as local costs, also indicate the general range to be expected.

Costs of local tunnels adjusted to a common ENR index of 800 are given in Table 13-10. These costs, as well as those of tunnels constructed by the Los Angeles County Sanitation Districts, are plotted in Fig. 13-11. This figure also includes a cost curve

Table 13-10. Costs of Existing Tunnels in Seattle

Tunnel location	Length, feet	Size, inches	Year constructed	Cost, dollars per lineal foot	ENR Index, date of construction	Cost, dollars per lineal foot, adjusted to ENR Index of 800
West Hanford Street	5,607	108	1928	69.20	205	270
Laurelhurst	1,501	42	1935	20.20	196	82
Charlestown Street	2,437	42	1931	16.24	185	70
North Trunk	–	60	1907	14.50	80	145
North Trunk	–	48	1907	13.50	80	135
North Trunk	–	54	1907	13.50	80	135
North Trunk	–	144	1907	51.00	80	510
North Trunk	–	108	1910	35.00	80	350
North Trunk	–	114	1907	36.00	80	360
North Trunk	–	32 x 48	1907	16.00	80	160
Lake City	1,292	84	1953	194.80	590	264

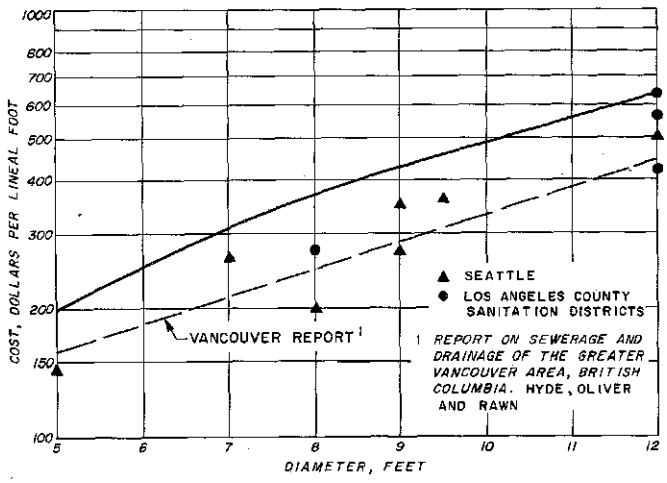


Fig. 13-11. Estimated Unit Costs of Tunnels

Solid curve is used for estimating tunnel costs in the Seattle area and is based on an ENR construction cost index of 800. Costs include excavation, bracing, lining, cleanup, and contractor's overhead and profit. They do not include engineering and contingencies.

based on considerable experience with sewer and water tunnels in the Vancouver, B.C. area. For the reasons previously stated, the curve in Fig. 13-11, which shows the costs used in this report, is placed higher than any of the plotted points for previously completed construction.

Submarine Outfalls

Unit costs for shallow and deep submarine outfalls (Fig. 13-12) are based on a comprehensive study made in 1955 by E. A. Pearson[2] for the California State Water Pollution Control Board. In his report on that work, the author presents in convenient form all known information concerning the construction and operation of submarine outfalls both on the Pacific Coast and on the Atlantic Coast.

Based on an analysis of the cost data contained in the Pearson report, it is evident that the cost of submarine outfalls is affected by (1) current velocity in the vicinity of the outfall, (2) range of tidal variation, (3) type and steepness of bottom, and (4) depth of pipe, including depth of terminal section. These factors were taken into account in developing the cost data plotted in Fig. 13-12 for the two ranges of depth expected along the shore of Puget Sound from Browns Point near Tacoma to the Snohomish County line north of Seattle. Allowances for diffuser sections located at the terminal end of submarine outfalls were made by assuming a unit cost for that section of twice that shown in the plotted data.

[2] *Pearson, E. A.,* An Investigation of the Efficacy of Submarine Outfall Disposal of Sewage and Sludge, *Publication No. 14, State Water Pollution Control Board, Sacramento, California.*

Storm Water Pumping Stations

Unit costs for storm water pumping stations depend on the total capacity provided and vary from $750 per cfs for stations with about 300 cfs capacity to $650 per cfs for stations with about 1,200 cfs capacity. Costs are predicated on the provision of facilities capable of fully automatic operation and include allowances for such landscaping and fencing as may be necessary around the structure itself. They do not, however, include allowances for additional facilities, such as holding basins, which may be required at a particular site. Costs for additional facilities are estimated separately for each applicable situation.

Sewage Pumping Stations

Unit costs for pumping stations (Fig. 13-13) are based on the design criteria set forth earlier in this chapter. Costs are expressed in terms of ultimate peak wet weather pumping capacity and allow for a reserve or standby capacity such that the peak flow can be handled with any one pump out of service. In addition, the unit costs recognize the desirability of reducing operator attendance to the fullest possible extent through the provision of facilities for automatic control of the pumping functions. Allowances are included also for suitable landscaping and fencing.

Where lifts in excess of 125 feet require the use of

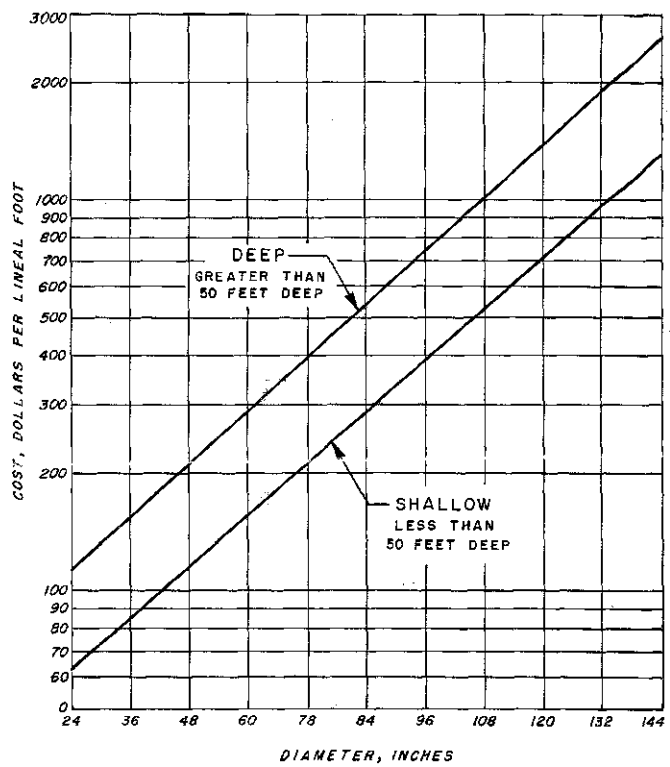


Fig. 13-12. Unit Costs of Submarine Outfalls

Based on analysis of costs of existing outfalls adjusted to ENR construction cost index of 800. Estimates do not include engineering and contingencies.

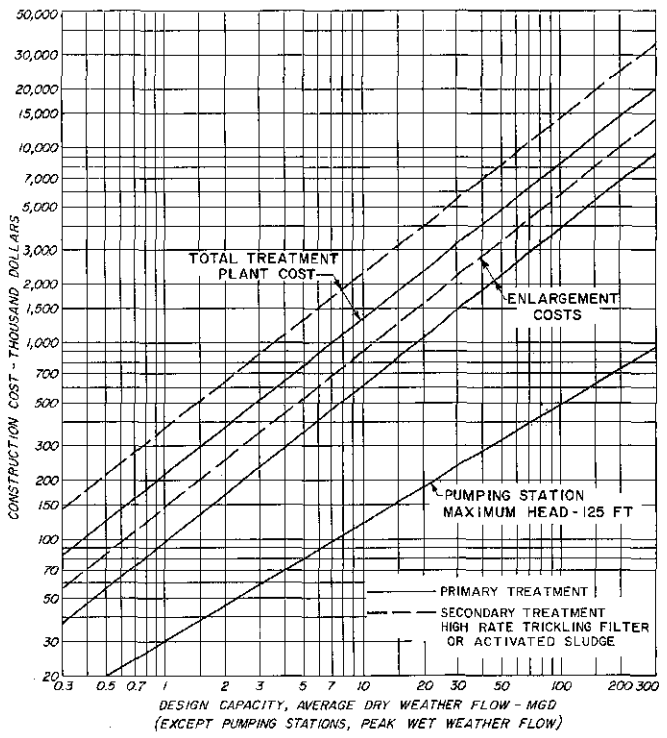


Fig. 13-13. Construction Costs of Sewage Treatment Plants and Pumping Stations

Costs are based on an ENR construction cost index of 800 and provide for raw sewage pumping, screening, grit removal, preaeration, sedimentation, secondary treatment (where applicable), sludge digestion, sludge drying on underdrained beds, plant operation and control facilities, plant grading and landscaping, and contractor's overhead and profit. Costs do not include land, special foundations, or engineering and contingencies. The curves for sewage treatment plants provide for initial dry weather flow, with later additions of facilities to accommodate twice the initial design flow. In addition to sedimentation and sludge digestion and disposal facilities for the given dry weather flow, initial costs provide for basic structures, influent pumping, channels, major pipelines, and service and control facilities which can be used without enlargement when the capacity is increased. Enlargement costs provide only for additional sedimentation and digestion facilities. Pumping station costs provide for a structure suitable for ultimate requirements and are based on ultimate peak wet weather flow.

two-stage pumping, stations are estimated to cost 125 per cent of the comparable single stage installation. For a pumping station constructed as an integral part of a sewage treatment plant, the estimated costs, as compared with a separate single-stage station, are 75 per cent for influent (raw sewage) and 25 per cent for effluent (treated sewage) pumping. Costs of influent pumping stations are included in the treatment plant cost curves, whereas effluent pumping station costs are not included.

Lower costs for stations constructed integrally with a treatment plant are possible for several reasons. These include (1) concurrent construction as part of a much larger project, (2) inclusion in the treatment plant of many items of equipment which otherwise would have to be provided as part of the pumping station, and (3) reduction in structural and space requirements. Effluent pumping stations are less costly than influent stations primarily because they are less complicated both structurally and mechanically.

Sewage Treatment Plants

Unit costs of sewage treatment plants (Fig. 13-13) are based on design criteria outlined previously in this chapter. In developing the plotted data, an analysis was made of the costs of nearly all west coast treatment plants built since the war. Figures were obtained both for primary and for secondary type plants, with the latter employing either trickling filtration or activated sludge, and were plotted to show the total adjusted contract cost, exclusive of the costs of land, and of engineering and legal services.

Unit costs reflected by the curves in Fig. 13-13 were compared with costs derived by application of the Velz[3] cost curves which were developed in 1948 from an analysis of 185 plants in northeastern and central United States. In addition, a comparison was made with cost curves recently developed by A. N. Diachishin.[4] While substantially the same as Velz' curves, the curves in Fig. 13-13 are somewhat higher, depending on plant type and capacity of plant, than those prepared by Diachishin.

As stated in the note under Fig. 13-13, treatment plant costs are based on initial construction of facilities to accommodate a given dry weather flow, with later additions to accommodate twice the initial flow. In addition to sedimentation and digestion facilities, initial costs provide for inlet structures, channels, major pipelines, control and service facilities, and other basic units which can be used without enlargement when the capacity is increased. Enlargement costs provide for additional sedimentation and digestion facilities to a capacity twice that of the initial dry weather flow.

As here presented, the unit cost curves represent, as precisely as possible, average costs in the Seattle area. While they are used later to evaluate the relative costs of alternative projects, they are not intended to supplant the detailed estimates which will have to be developed in connection with the preparation of construction plans and specifications.

In using the curves, it is necessary to allow properly for extraordinary design problems associated with such factors as high maximum hydraulic capacity, excessive BOD and suspended solids loading condi-

[3] *Velz, C. J.,* How Much Should Sewage Treatment Cost, *Engineering News-Record, 141:16, October 1948.*

[4] *Diachishin, Alex N.,* New Guide to Sewage Plant Costs, *Engineering News-Record, 159:15, October 1957.*

tions, and special structural and foundation conditions. When these factors are taken into account, the curves provide a useful medium for estimating costs of treatment plants of various capacities.

Separation Costs

As part of the job of developing total costs for the various projects analyzed herein, it has been necessary to include the costs of separating storm water runoff from sanitary sewage in areas now served with combined sewers. For this purpose, two degrees of separation are considered, as follows: (1) complete separation whereby street inlets, roof leaders and foundation drains are picked up in an entirely new storm water system; and (2) partial separation, whereby only street inlets are picked up in a new system, with roof leaders and foundation drains remaining connected to the sanitary sewers.

Because it would be completely beyond the intended scope of this survey to undertake the detailed design which is required for ascertaining the exact cost of storm water separation in all combined areas, it has been necessary to resort to a unit area method of approach. Using this method, average costs for each type of separation were worked out and found to be as follows:

Complete separation, per acre	$3,890
Partial separation, per acre	$1,860

These costs are based on detailed analyses of typical areas, using criteria previously presented for storm frequency-duration and sewer design. They include allowances for engineering and contingencies, and cover all pipelines, street catch basins, and connections required to make a complete system. Based on average results for the various areas analyzed, separation of street drainage from combined systems will remove approximately 65 per cent of the total storm runoff during a storm of 10-year design frequency and somewhat more than that during the summer storms of low intensity.

It is believed that the application of the unit cost method to larger areas will result in total estimated costs agreeing reasonably well with those based on a more detailed layout. In any case, the costs are sufficiently liberal to provide for considerable latitude in working out variations during final design.

Engineering and Contingencies

Unit construction costs presented in this chapter do not include allowances for engineering and contingencies. Engineering services include (1) preliminary investigations and reports; (2) site and route surveys and foundation explorations; (3) preparation of construction drawings and specifications; (4) preparation of quantity and construction cost estimates; (5) resident engineering and inspection of construction; (6) surveying, sampling, and materials testing during construction; and (7) final inspection and submission of report relative to completion and acceptance of the project. Contingent costs allow for uncertainties unavoidably associated with preliminary design. Such factors as foundation conditions, necessity for special construction methods, and variations in final lengths of pipelines are a few of the many contingency items for which allowances must be made.

Depending on the size and type of the project, total engineering costs may range from 7 to 12 per cent of the contract cost. Lower percentages apply to relatively large projects which do not require a great amount of preliminary work, such as pumping stations and sewage treatment plants. Higher percentages apply to smaller, more complicated projects requiring a great deal of preliminary investigation. For the purposes of this report, an average value of 9 per cent is applied to all estimates as an allowance for all engineering. For contingent items, an allowance of approximately 15 per cent is applied to all projects. Taken together and applied in sequence, the total allowance for engineering and contingencies thus amounts to 25 per cent of the estimated basic contract cost.

ANNUAL COSTS

Annual costs comprise interest on invested capital, depreciation, and charges for administration, operation and maintenance. Any engineering study involving a comparison of alternative projects designed to perform a specific function must include an analysis of such projects on the basis of total annual cost. This is because the true economy of each one individually is best reflected not by its construction cost but by what it costs per year to finance, operate and maintain. In some instances, however, factors other than annual cost must also be taken into account in reaching a final decision as to which is the most suitable of several possible alternatives.

Interest and Depreciation

An average interest rate of 5 per cent, which is slightly higher than that currently applicable to municipal general obligation bonds, was used in computing annual interest charges. Depreciation was computed by the sinking fund method with interest at 5 per cent. This method is frequently used in comparing annual costs of public works projects. The combination of interest costs and depreciation costs is commonly referred to as fixed cost and may be computed by the use of capital recovery factors found in most interest tables.

For the present study, the economic life of all sew-

ers and pipelines, including outfalls, was taken as 50 years, whereas a 30-year life was used for pumping stations and treatment plants. In the latter case, the shorter period results from the fact that mechanical equipment therein normally has to be replaced before the structures have fully depreciated.

With reference to replacement of mechanical equipment, it should be noted that ordinary day-to-day maintenance, the cost of which is included under operation and maintenance, is assumed to keep equipment operating until it has to be replaced due to obsolescence, inadequacy or major deterioration. It should be noted also that the economic life assigned to the various structures does not necessarily reflect their true effective life. In fact, sewers are presently in use which are more than 100 years old. Similarly, large modern treatment plant structures can be expected to serve more than 30 years.

Operation and Maintenance

Operation and maintenance costs include all charges for operational services, power, supplies, and administration incidental to operation of the various facilities. Administration costs include supervision, engineering, office overhead, legal fees, special consultants fees, accident and liability insurance, and miscellaneous items. Operation and maintenance costs presented herein are based on a study of records of various agencies operating facilities similar to those considered for the metropolitan Seattle area. In particular, a study by R. O. Sylvester[5] of various plants in the state of Washington was utilized in developing costs for smaller installations.

All costs are based on present (January, 1958) prices of materials and labor. If prices continue to rise, an adjustment in operating costs will have to be made accordingly.

Trunk and Intercepting Sewers. Annual operation and maintenance costs for trunk sewers, interceptors, inverted siphons, force mains, outfalls and appurtenant structures are taken at 0.25 per cent of the total construction cost. This allowance represents an average value for careful inspection and maintenance practices and can be expected to vary from year to year.

Sewage Treatment Plants. It is difficult to develop a common basis on which to compare operating costs of treatment plants. For a given dry weather flow, a plant operating at or near design capacity would be expected to be less costly than one operating at one-half its capacity. On the other hand, a comparison based on design capacity is inadequate in that it fails to reflect basic differences in maintenance practices and power requirements. For example, the skill of the operator or superintendent very markedly affects the maintenance cost of mechanical equipment, especially at fully loaded plants. In the case of purchased power, the cost thereof is related to such factors as the total pumping lift in the plant, the use of preaeration as a treatment process, and the extent to which power is developed from sewage gas.

All things considered, it appears most equitable to relate operating costs to average dry weather flow. In using data thus developed, it must be realized that, in specific cases, adjustments are necessary to allow for (1) unusual conditions such as excessive pumping lift, (2) either very low or very high average flow in relation to design capacity, (3) costly sludge disposal, and (4) the degree to which power is developed from sludge gas.

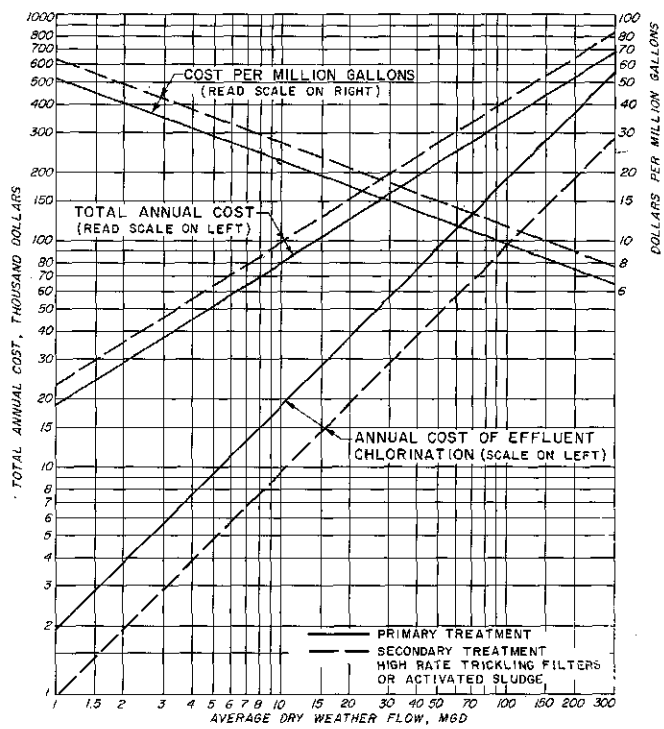


Fig. 13-14. Operating Costs of Sewage Treatment Plants

Costs provide for first-class operation and include all necessary charges for operational and maintenance services, power, supplies, replacement parts and administration. The curves are based on the assumption that all plants having a capacity of 5 mgd or greater are provided with gas engines utilizing sludge gas either for generation of electric power or for driving pumps, blowers, or other equipment. Operating costs allow for digested sludge drying on open air beds. For other methods of disposal, costs have to be adjusted as follows: disposal by dilution, subtract $3.00 to $10.00 per dry ton depending on plant size; disposal by wet hauling, add $10.00 per dry ton; heat drying, add $20.00 per dry ton; disposal in lagoons, subtract $2.25 to $7.50 per dry ton depending on plant size.

[5] *Sylvester, R. O.,* A Critical Appraisal of Sewage Works Operation and Design, *Sewage and Industrial Wastes, 27-6-759 (June 1955).*

Average operating costs for sewage treatment plants, expressed on the basis of average dry weather flow, are presented in Fig. 13-14. These costs assume first class operation and include all necessary charges for operational services, power, supplies, replacement parts and administration. They assume also that all plants having a capacity of 5 mgd or more are equipped with gas engines which utilize sludge gas either for generation of electrical power or for direct driving of pumps, blowers or other equipment. A third assumption is that digested sludge is dried on open air beds. For disposal of sludge either by dilution in the plant effluent or through a separate disposal line, a deduction from the indicated operating cost may be made, varying from $10.00 per ton dry weight for small plants to $3.00 per ton for large plants. If the digested sludge is disposed of in lagoons, a deduction may be made from the operating cost amounting to 75 per cent of that for disposal by dilution. Where wet digested sludge must be hauled away, disposal costs are greater than bed drying and $10.00 per dry ton must be added. Heat drying is estimated to cost $20.00 per dry ton in addition to the operating costs based on bed drying.

On the same figure are shown two curves giving the annual cost of effluent chlorination both for primary treatment and for secondary treatment using trickling filtration or the activated sludge process. Chlorination costs, as related to average dry weather flow, are based on dosages of 8 ppm and 4 ppm for primary and secondary effluent, respectively, and a total chlorine cost of 8 cents per pound applied.

Sewage Pumping Stations. In analyzing operation and maintenance costs of sewage pumping stations, it is desirable to separate costs of power from those of services and supplies. Power costs calculated for average discharge heads of 25 to 200 feet and for average dry weather flow are illustrated by the curves in Fig. 13-15. These curves are based on the cost of electric power in the Seattle area and allow properly for daily, weekly and yearly variations in flow. Power costs consist of an electrical energy charge computed for average dry weather flow, and of a maximum demand charge based on twice the average dry weather flow during ten months each year and on four times the average flow during two months of peak demand.

Operation and maintenance costs, exclusive of power charges, are also illustrated in Fig. 13-15. These costs include all necessary charges for operational and maintenance services, supplies, replacement parts, and administration. Operational services are based on the assumption that the stations will be substantially automatic and that full use will be made of automatic control equipment. The given costs also assume that the stations will be operated and maintained in first class condition at all times. For two stage stations, as dictated by discharge heads greater than 125 feet, operation and maintenance costs are to be increased 35 per cent above those indicated by the curve.

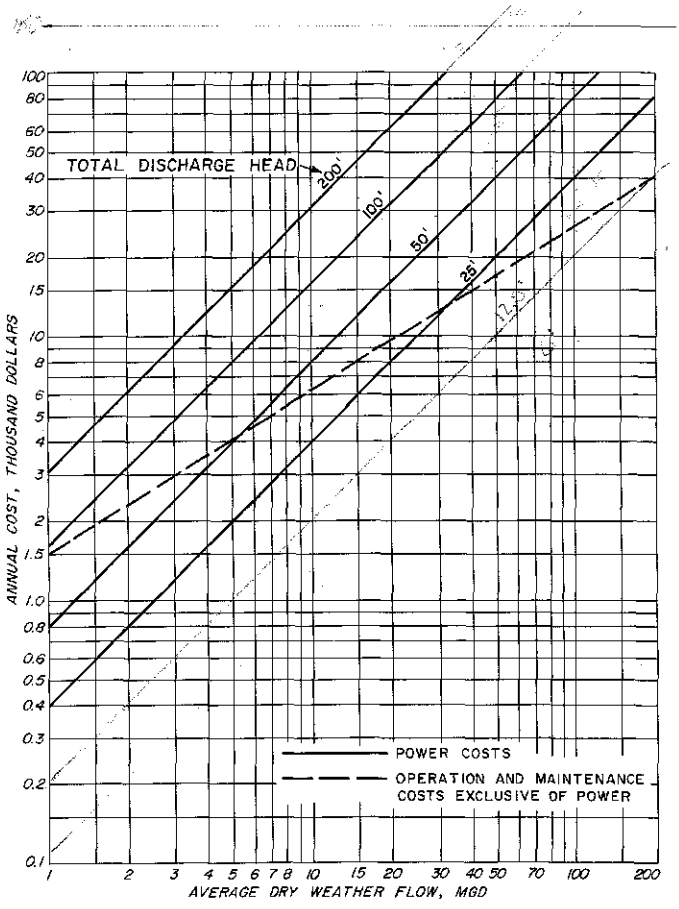


Fig. 13-15. Operating Costs of Sewage Pumping Stations

Costs provide for first-class operation. Power costs represent average charges for industrial electrical service in the Seattle area. Operation and maintenance costs include all necessary charges for operational and maintenance services, supplies, replacement parts and administration. Operational services are based on the assumption that stations will be substantially automatic in operation. For two stage stations, operation and maintenance costs are to be increased by 35 per cent.

Chapter 14
SEWERAGE AND DRAINAGE SUBAREAS

In planning for sewerage and drainage of a large area, such as metropolitan Seattle, one of the first considerations is that of dividing the area into units which logically will lend themselves to detailed study of the various phases of sewerage and drainage design. This division is based on such factors as topography, geographical and political boundaries, population distribution and land use, and the extent, nature, capacity and location of existing facilities.

Because of inherent differences in principles of design, drainage service areas usually differ from service areas for sewerage systems. Drainage areas are limited almost exclusively by topography and require division into relatively small areas, each of which is tributary to a watercourse or body of water which can be utilized for the disposal of storm water. Sewerage areas are limited and defined not only by topography but by economic and political considerations as well. Political considerations, however, while often significant in planning for the sewerage of a small local entity, usually have very little bearing on long-term sewerage plans for a metropolitan area. Obviously, therefore, the controlling factors with respect to division of the metropolitan Seattle area are geographic and economic in character.

SEWERAGE SERVICE AREAS

Boundaries of the metropolitan Seattle sewerage study area have been established previously on the basis of the topographic and growth and development studies presented in Part I. This area (Fig. 14-1) is bounded on the west by Puget Sound, and on the north and northeast by the boundaries of the Snohomish River and local watersheds. Because the watersheds of the Cedar and Green rivers and of Issaquah Creek extend beyond any probable urban development, an arbitary boundary, based on the limit of probable development, has been established along the southeast edge of the area. Local watershed boundaries define the limits of the area to the south.

Lack of a natural boundary on the southeast does not significantly affect the preliminary design of major trunk sewers and of sewage treatment and disposal facilities. These can be planned for the projected development of each watershed as a whole without regard to the exact location of future peripheral communities.

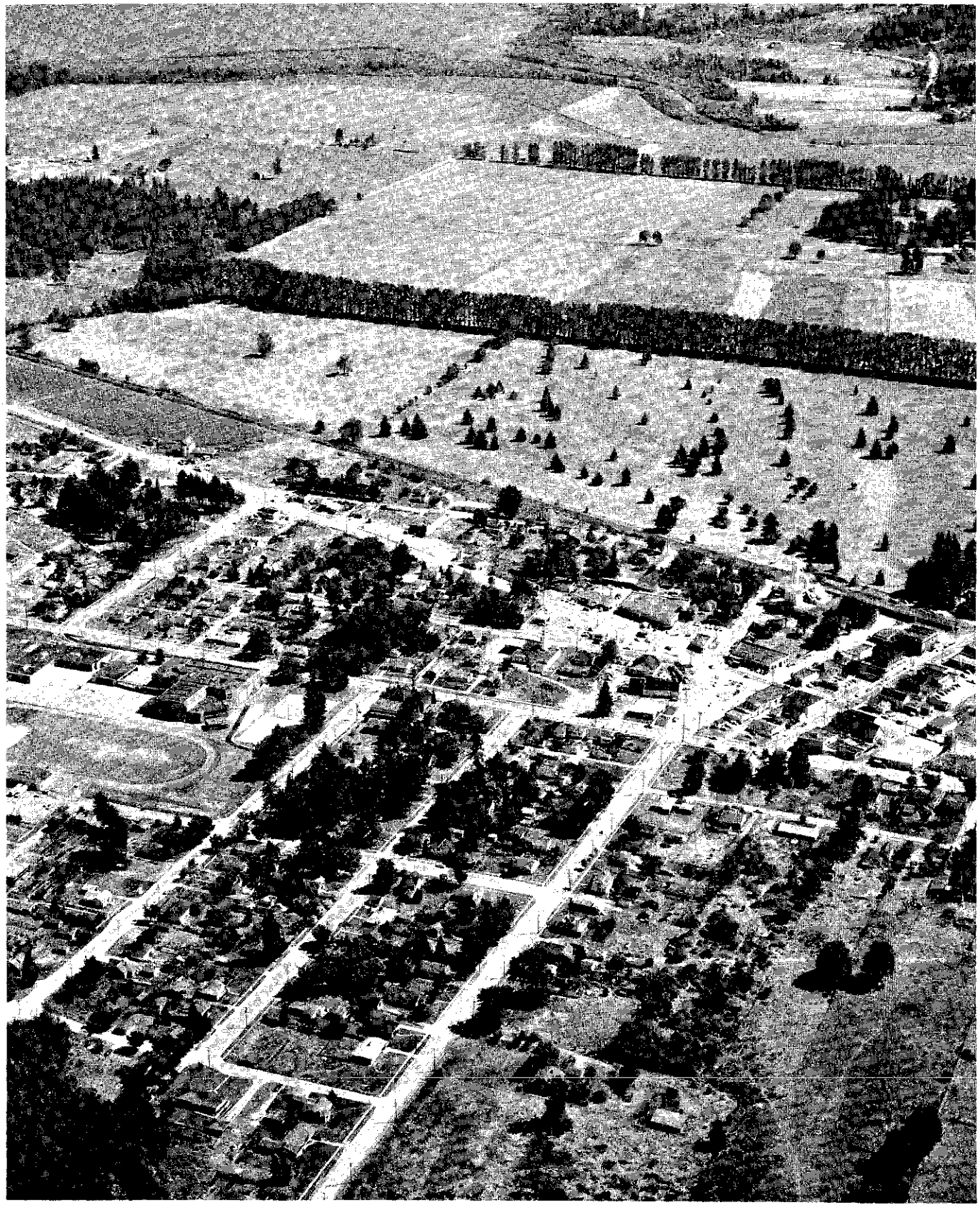
As set forth in Chapter 3, the study area consists of four primary watersheds, namely, Lake Washington, Puget Sound, Green-Duwamish and Lake Union - Ship Canal. For the purposes of sewerage planning, these watersheds are divided into twelve major areas, the divisions conforming generally to natural topographic boundaries and to natural points of sewage concentration (Fig. 14-1 and Table 14-1). Each of the major areas is further subdivided into a number of smaller units, which hereinafter will be referred to as local service areas.

Table 14-1. Sewerage Areas

Sewerage area	Area in acres	Population in thousands				Pop/Acre
		1957	1980	2000	2030	
North Lake Sammamish	49,400	20	49	99	158	3.2
South Lake Sammamish	28,900	10	23	50	95	3.3
East Lake Washington	36,640	62	108	159	202	5.5
North Lake Washington	48,140	26	100	173	250	5.2
Northwest Lake Washington	19,270	86	126	153	165	8.55
South Lake Washington	34,230	31	64	110	170	5.0
Green River	71,580[a]	25	76	195	320	4.5
Southwest Lake Washington	8,890	77	86	90	94	10.6
Elliott Bay	20,450	157	182	202	215	10.5
Lake Union	16,300	263	268	278	284	17.4
South Puget Sound	30,020	83	134	187	238	7.9
North Puget Sound	6,350	24	41	45	47	7.4
Total	370,260	864	1,257	1,741	2,238	

See Fig. 14-1 for location of sewerage areas.

[a]*Excluding Lake Youngs reservoir and water department property, 2,560 acres.*



THE CITY OF REDMOND lies near the northern end of Lake Sammamish in the broad valley of the Sammamish River. All sewage of the North Lake Sammamish sewerage area can be delivered to a point near Redmond.

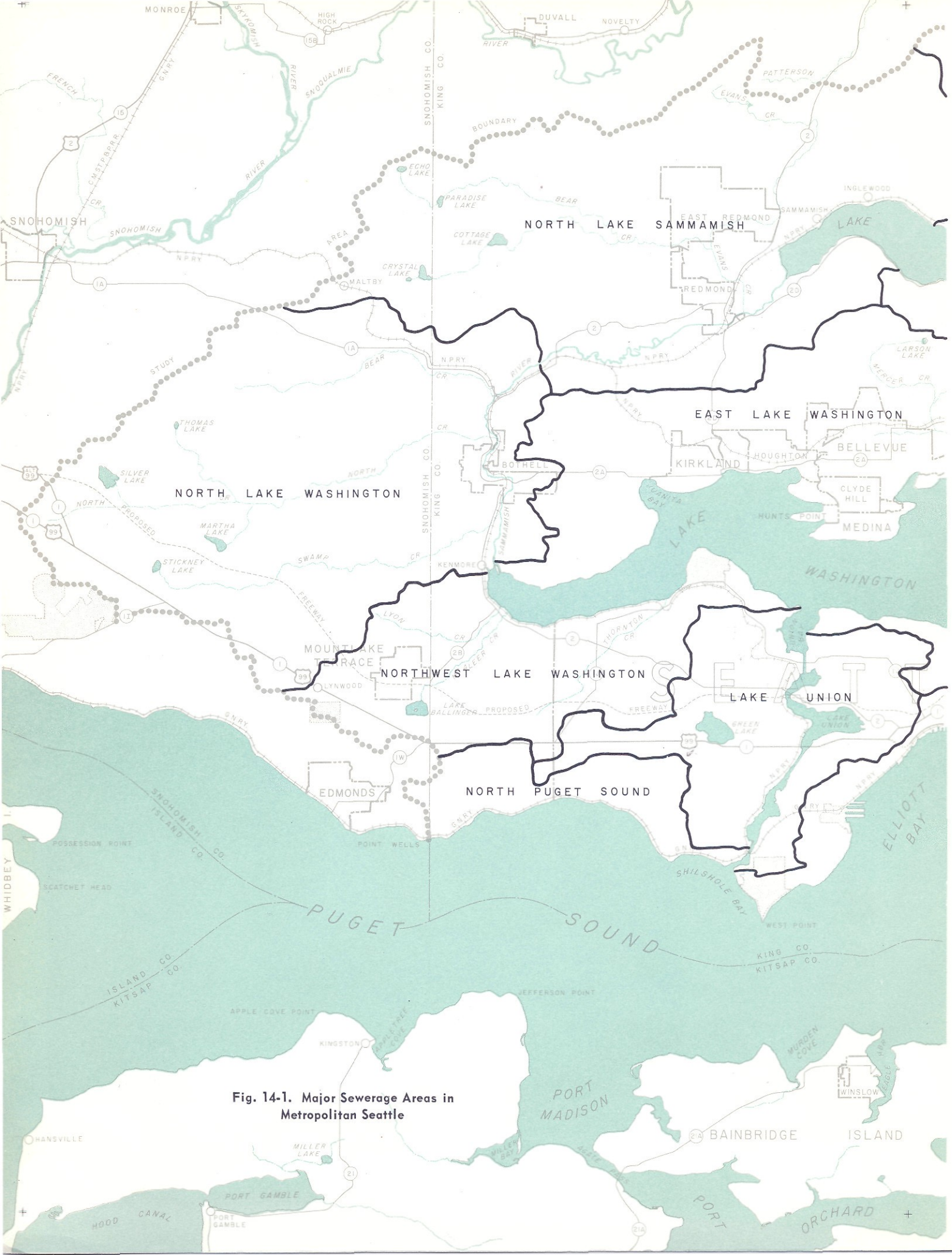
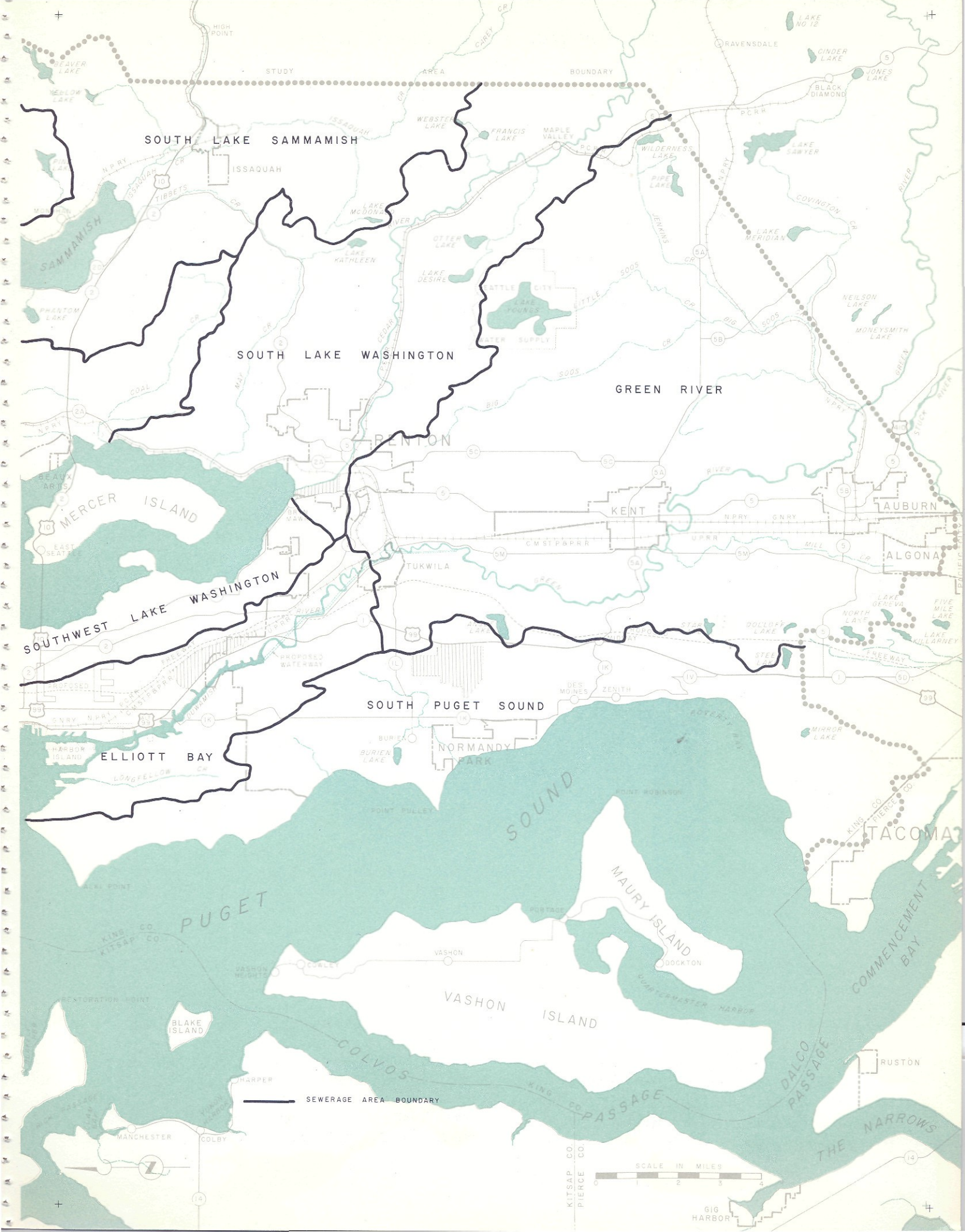


Fig. 14-1. Major Sewerage Areas in Metropolitan Seattle


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
MAURY ISLAND
VASHON ISLAND
COMMENCEMENT BAY
DALCO PASSAGE
COLVOS PASSAGE
THE NARROWS
MERCER ISLAND
BLAKE ISLAND
RENTON
KENT
AUBURN
ALGONA
TUKWILA
NORMANDY PARK
TACOMA
ISSAQUAH
SAMMAMISH
RAVENSDALE
BLACK DIAMOND
MAPLE VALLEY
HIGH POINT
DES MOINES
ZENITH
BURIEN
VASHON
DOCKTON
RUSTON
MANCHESTER
COLBY
GIG HARBOR
KING CO.
KITSAP CO.
PIERCE CO.
SEWERAGE AREA BOUNDARY
SCALE IN MILES
0 1 2 3 4

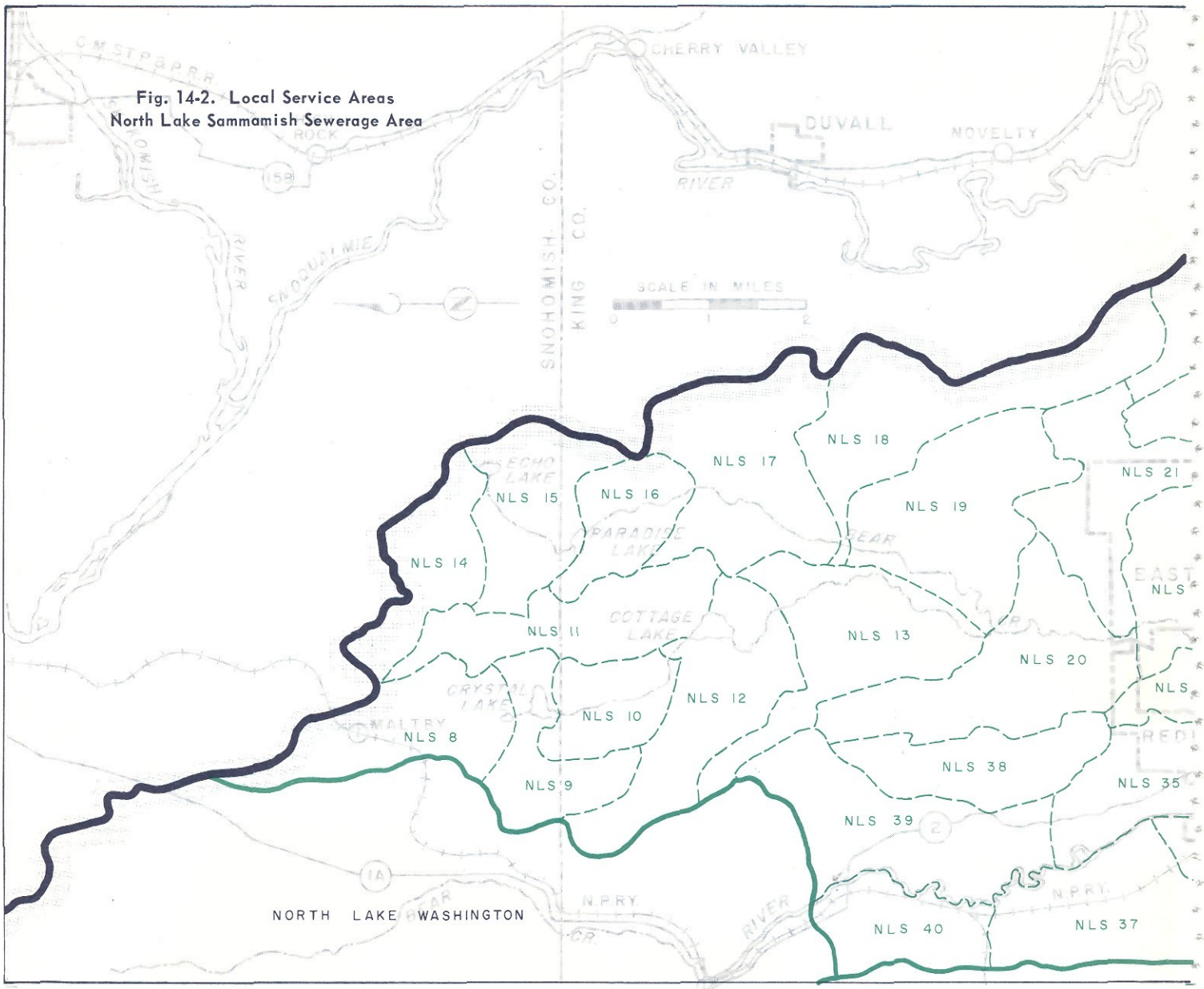


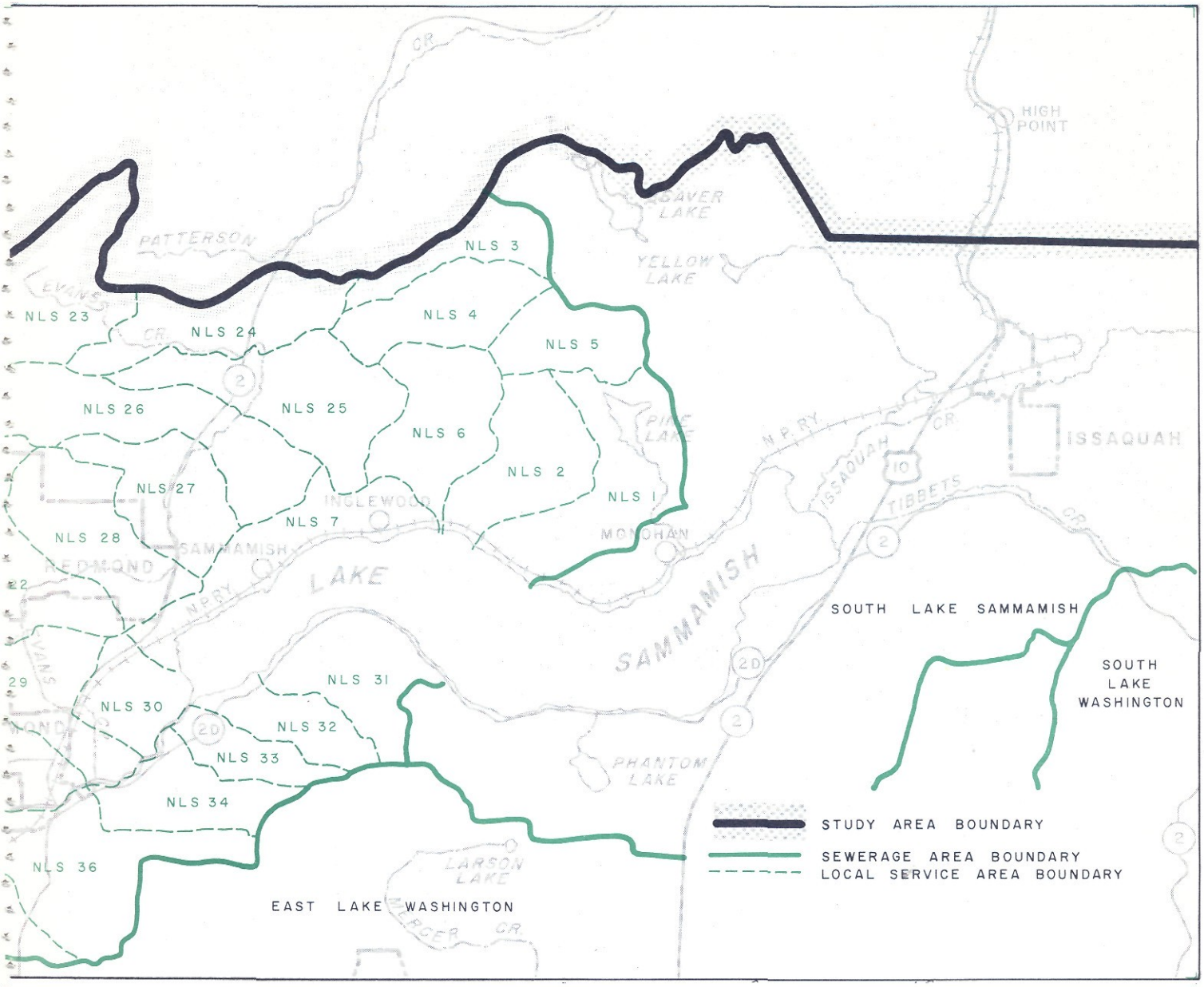
Fig. 14-2. Local Service Areas
North Lake Sammamish Sewerage Area

North Lake Sammamish Sewerage Area

This area contains 49,400 acres, of which 4,930 acres are in Snohomish County. It includes part of the watershed tributary to Sammamish River, as well as a portion of the watershed directly tributary to Lake Sammamish. Within its boundaries are the cities of Redmond and East Redmond. Although predominantly agricultural at present, residential developments are taking place and undoubtedly will be accelerated upon completion of the proposed second Lake Washington crossing and of freeways now under construction. No public sewerage facilities are presently available but the city of Redmond has recently awarded a contract for the construction of collection, treatment and disposal facilities.

Drainage from the northwest portion of the area is generally southward to Sammamish River, which has a flat, northward sloping valley. Sections tributary to Lake Sammamish slope steeply to the lake, with the result that all sewage therefrom will have to be collected by lake front interceptors. Concentration of flows from the entire area can be achieved by gravity at a point about two miles north of Redmond. From there, the alternatives are either gravity flow northward to the North Lake Washington sewerage area or pumping over the divide to the west into the East Lake Washington sewerage area.

For purposes of sewerage planning, the North Lake Sammamish sewerage area is subdivided into 40 local service areas (Fig. 14-2 and Table 14-2).



South Lake Sammamish Sewerage Area

Comprising a total of 28,990 acres, the South Lake Sammamish area includes all of the Issaquah Creek watershed within the metropolitan area, plus several minor watersheds directly tributary to the southerly portion of Lake Sammamish. This area is predominantly agricultural or undeveloped at present but, with completion of the eastside freeways, is expected to develop appreciably within the next two decades.

Issaquah, the only incorporated city in the area, now has sewage collection, treatment, and disposal facilities. Sewerage service is provided also in the north end by the Lake Hills Sewer District. The remaining sections are unsewered and depend on private disposal systems.

Most of the area drains to Issaquah Creek, which in turn empties into Lake Sammamish north of the city of Issaquah. A major portion of its sewage, therefore, can be delivered by gravity flow to a point north of Issaquah. Westward from Issaquah, mountain ranges exceeding 1,000 feet in elevation preclude economic conveyance in that direction. Sewage can, however, be transported north along the shore of Lake Sammamish to a point east of Phantom Lake, from which point it can be pumped westward over a relatively low divide into the East Lake Washington sewerage area.

For sewerage planning, South Lake Sammamish sewerage area is subdivided into 28 local service areas (Fig. 14-3 and Table 14-3).

DEVELOPMENT in the South Lake Sammamish sewerage area is largely concentrated in the city of Issaquah. Urban development in the remainder of the area, which is agricultural or undeveloped at present, is expected to increase appreciably within the next two decades.

Table 14-2. Local Service Areas within the North Lake Sammamish Sewerage Areas

Area designation	Area in acres	Area designation	Area in acres
NLS-1	1,440	NLS-21	1,450
NLS-2	1,090	NLS-22	1,400
NLS-3	620	NLS-23	1,410
NLS-4	910	NLS-24	840
NLS-5	780	NLS-25	1,460
NLS-6	1,030	NLS-26	1,670
NLS-7	1,060	NLS-27	980
NLS-8	1,410	NLS-28	1,200
NLS-9	950	NLS-29	1,120
NLS-10	820	NLS-30	630
NLS-11	1,280	NLS-31	1,020
NLS-12	1,760	NLS-32	610
NLS-13	1,800	NLS-33	510
NLS-14	1,040	NLS-34	890
NLS-15	1,360	NLS-35	1,300
NLS-16	880	NLS-36	1,440
NLS-17	1,900	NLS-37	1,590
NLS-18	2,070	NLS-38	1,130
NLS-19	2,060	NLS-39	1,810
NLS-20	1,910	NLS-40	770
Total			49,400

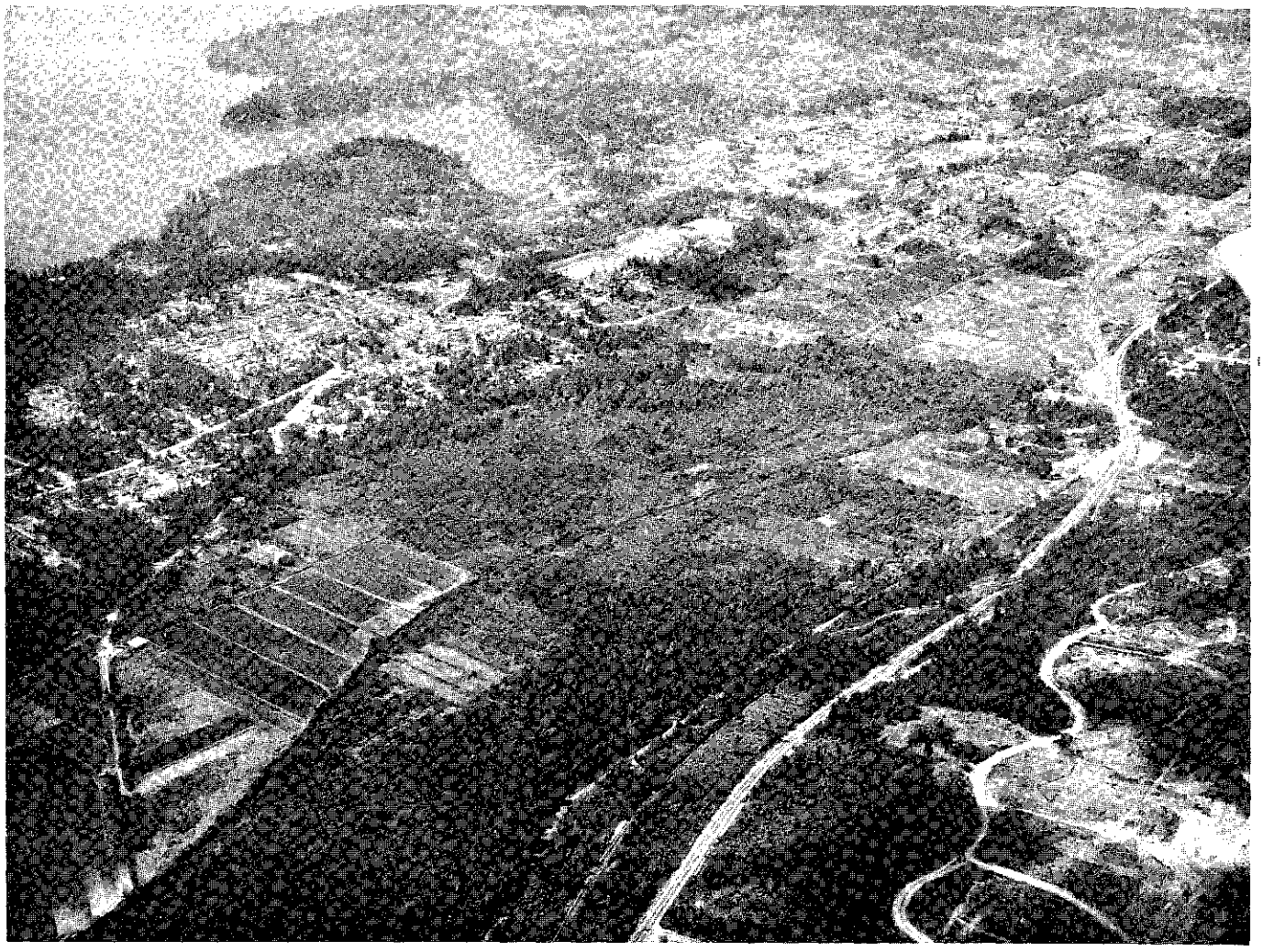
See Fig. 14-2 for location.

East Lake Washington Sewerage Area

This area contains 36,640 acres and includes Mercer Island. Extensive urban development has taken place on Mercer Island and along the east shore of Lake Washington, particularly in the incorporated cities of Beaux Arts, Bellevue, Clyde Hill, Houghton, Hunts Point, Kirkland and Medina. Rapid growth is expected to continue, with urbanization moving eastward and southward and continuing on Mercer Island.

Public sewerage service is provided by the Bellevue and Lake Hills sewer districts and by the city of Kirkland on the mainland, while Mercer Island is served by the East Mercer and Mercer Island sewer districts. Several densely populated areas are as yet unsewered and now depend on individual septic tank systems.

Drainage of the area is generally westward to Lake Washington, although a natural grade to the south exists along the Northern Pacific Railroad right-of-way. Collection of sewage along Lake Washington and its delivery to a central point is complicated by numerous small ravines and creeks which are perpendicular in direction to the natural drainage pattern. This means, of course, that extensive pumping will be re-



RAPID RESIDENTIAL DEVELOPMENT is occurring in the East Lake Washington sewerage area north of the Lake Washington floating bridge. The photograph shows the five-city area of Beaux Arts, Bellevue, Clyde Hill, Hunts Point and Medina.

Table 14-3. Local Service Areas within the South Lake Sammamish Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
SLS-1	840	SLS-15	1,230
SLS-2	1,200	SLS-16	900
SLS-3	870	SLS-17	1,170
SLS-4	1,090	SLS-18	1,180
SLS-5	830	SLS-19	1,000
SLS-6	1,340	SLS-20	780
SLS-7	1,400	SLS-21	1,380
SLS-8	990	SLS-22	910
SLS-9	1,140	SLS-23	1,160
SLS-10	740	SLS-24	1,240
SLS-11	1,450	SLS-25	530
SLS-12	1,120	SLS-26	1,120
SLS-13	1,000	SLS-27	1,040
SLS-14	490	SLS-28	850
Total			28,990

See Fig. 14-3 for location.

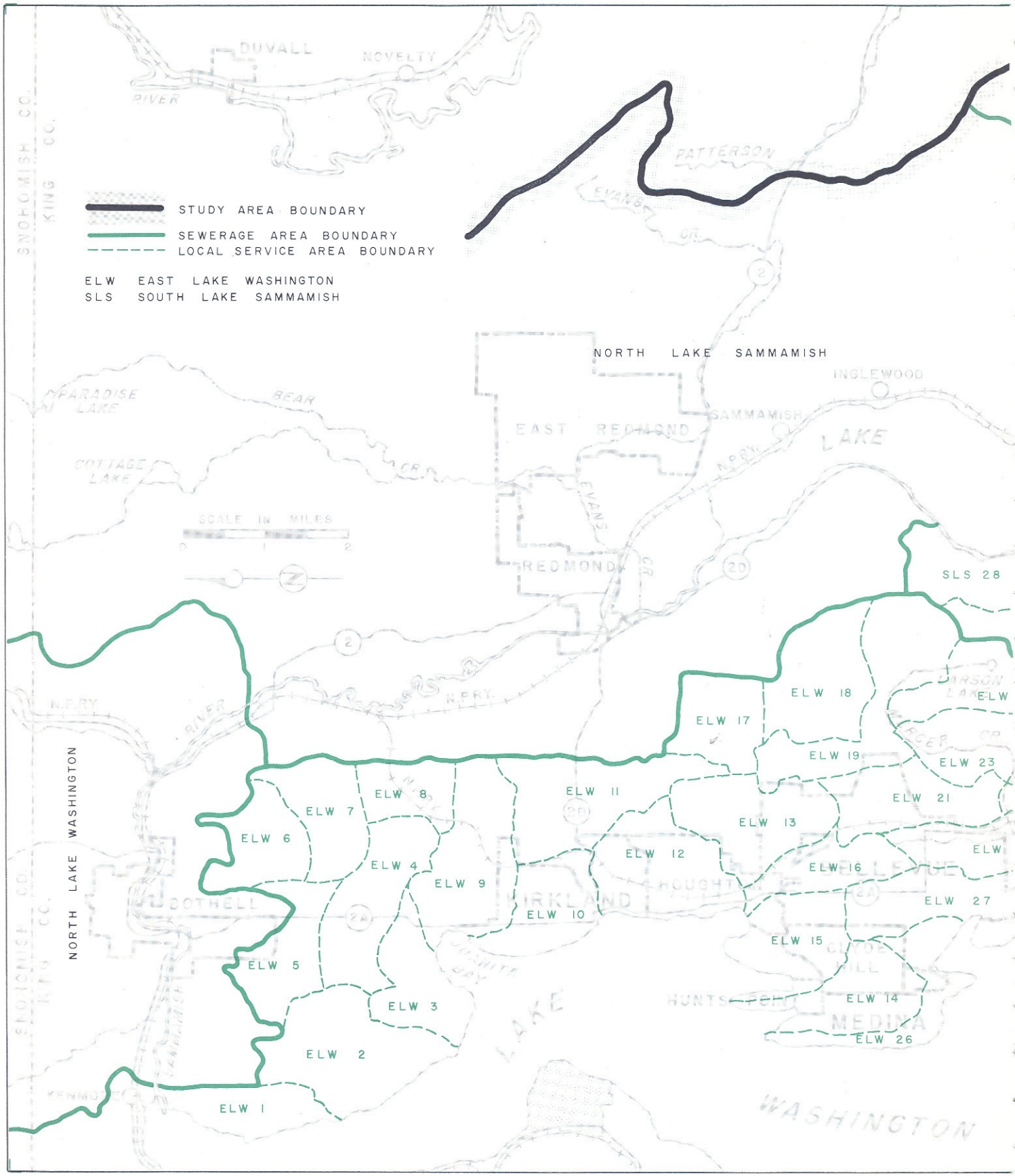
quired. In any event, flows can be transported from the area either southward along the railroad right-of-way to the South Lake Washington sewerage area, or westward across Lake Washington to the Southwest Lake Washington sewerage area.

For sewerage planning, the East Lake Washington area is subdivided into 40 local service areas (Fig. 14-3 and Table 14-4).

North Lake Washington Sewerage Area

Lying entirely within the Sammamish River watershed, this area contains 48,140 acres, including 39,410 acres in Snohomish County. It includes the incorporated city of Bothell and the communities of Woodinville and Alderwood Manor, as well as part of the highly developed Lynwood area.

Except for the southwest portion, the area is largely agricultural or undeveloped. Following completion of proposed freeways, however, rapid residential development is expected to take place northward from


STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
ELW EAST LAKE WASHINGTON
SLS SOUTH LAKE SAMMAMISH
NORTH LAKE SAMMAMISH
NORTH LAKE WASHINGTON
SNOHOMISH CO.
KING CO.
DUVALL
NOVELTY
PATTERSON
INGLEWOOD
SAMMAMISH
EAST REDMOND
REDMOND
BOTHELL
KIRKLAND
HOUGHTON
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
WASHINGTON
SCALE IN MILES
ELW 1
ELW 2
ELW 3
ELW 4
ELW 5
ELW 6
ELW 7
ELW 8
ELW 9
ELW 10
ELW 11
ELW 12
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ELW 23
ELW 26
ELW 27
SLS 28

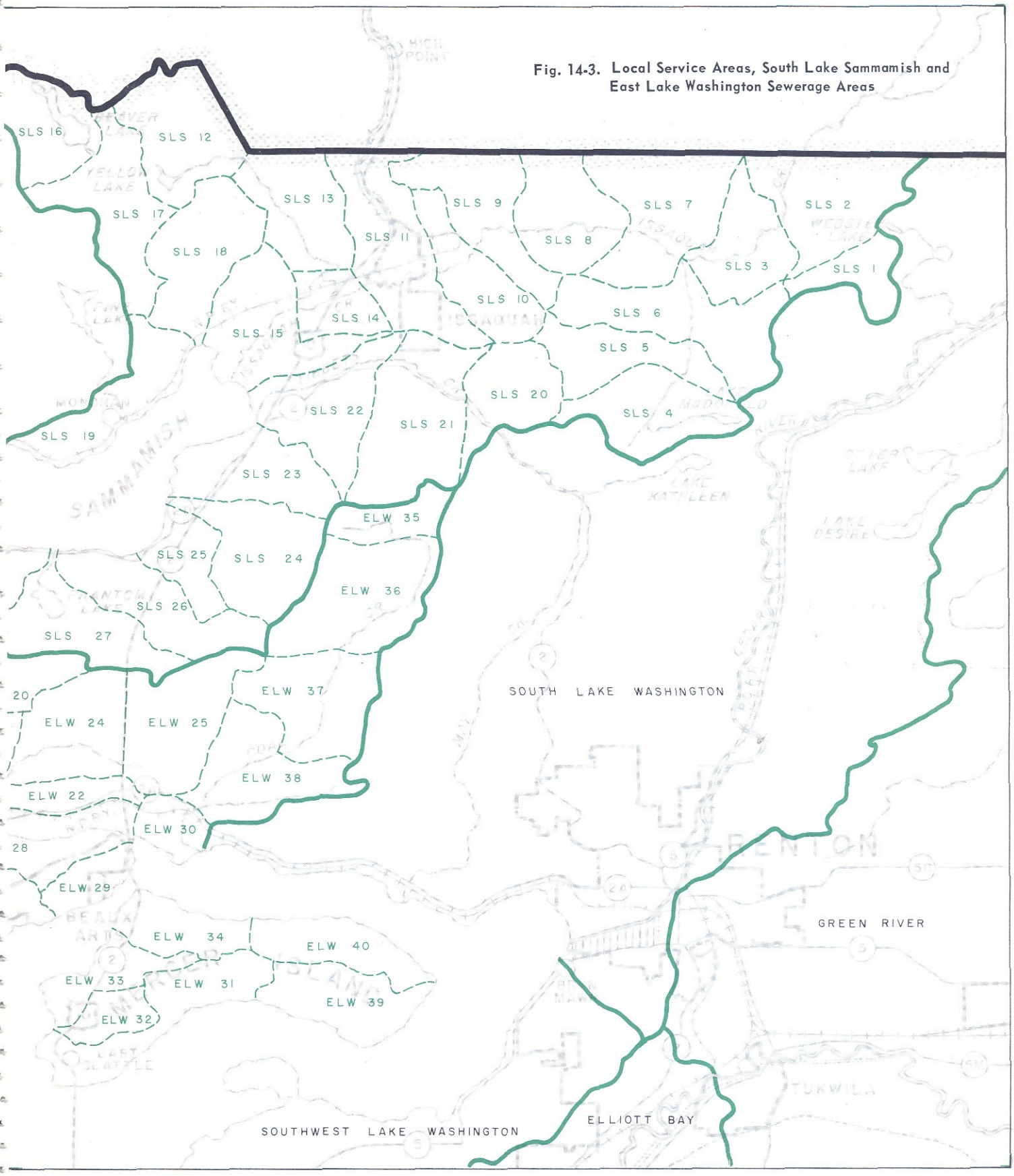


Fig. 14-3. Local Service Areas, South Lake Sammamish and East Lake Washington Sewerage Areas

Table 14-4. Local Service Areas within the East Lake Washington Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
ELW-1	930	ELW-21	980
ELW-2	1,060	ELW-22	480
ELW-3	950	ELW-23	680
ELW-4	1,120	ELW-24	1,010
ELW-5	810	ELW-25	1,190
ELW-6	820	ELW-26	590
ELW-7	910	ELW-27	1,090
ELW-8	790	ELW-28	1,220
ELW-9	1,360	ELW-29	370
ELW-10	910	ELW-30	340
ELW-11	1,400	ELW-31	960
ELW-12	1,020	ELW-32	370
ELW-13	1,150	ELW-33	470
ELW-14	980	ELW-34	630
ELW-15	910	ELW-35	610
ELW-16	680	ELW-36	1,350
ELW-17	880	ELW-37	1,200
ELW-18	1,280	ELW-38	1,080
ELW-19	1,170	ELW-39	880
ELW-20	1,250	ELW-40	760
Total			36,640

See Fig. 14-3 for location.

Seattle and southward from Everett. No public sewerage facilities are presently available but a system for Bothell is under design.

Natural drainage is to Sammamish River. All sewage can be delivered by gravity flow to a point about two miles west of the city of Bothell, from which point it can be pumped into the Northwest Lake Washington sewerage area.

For study purposes, the North Lake Washington sewerage area is subdivided into 44 local service areas (Fig. 14-4 and Table 14-5).

Northwest Lake Washington Sewerage Area

Situated entirely within the Lake Washington watershed, this area contains 19,270 acres, of which 4,940 are in Snohomish County. Extensive urban development has taken place at a rapid rate and undoubtedly will continue at the same pace. Within its boundaries, the area includes a portion of the city of Seattle, the city of Mountlake Terrace in Snohomish County, and the residential communities of North City and Lake Forest Park. It also includes part of both Lynwood and Richmond Highlands and Seattle Heights.

Although sewerage service is provided in much of the area by the city of Seattle, the heavily populated sections in the north are unsewered. Planning is now in progress for sewers in Mountlake Terrace and in the Echo Lake section of the Ronald Sewer District.

Drainage is to Lake Washington through a number of creeks, the most prominent of which are Lyon,

Table 14-5. Local Service Areas within the North Lake Washington Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
NLW-1	1,220	NLW-23	1,580
NLW-2	710	NLW-24	1,240
NLW-3	870	NLW-25	1,530
NLW-4	970	NLW-26	1,300
NLW-5	2,080	NLW-27	700
NLW-6	1,100	NLW-28	890
NLW-7	1,110	NLW-29	1,270
NLW-8	500	NLW-30	960
NLW-9	1,610	NLW-31	1,040
NLW-10	1,360	NLW-32	970
NLW-11	580	NLW-33	810
NLW-12	1,180	NLW-34	1,350
NLW-13	950	NLW-35	1,290
NLW-14	990	NLW-36	730
NLW-15	1,040	NLW-37	1,290
NLW-16	1,570	NLW-38	1,440
NLW-17	1,370	NLW-39	1,130
NLW-18	740	NLW-40	570
NLW-19	1,270	NLW-41	1,400
NLW-20	890	NLW-42	760
NLW-21	1,410	NLW-43	580
NLW-22	820	NLW-44	980
Total			48,140

See Fig. 14-4 for location.

McAleer and Thornton. All sewage can thus flow by gravity to the shores of the lake and thence can be transported along the lake to a central collection point at Thornton Creek. From there, pumping into the Lake Union sewerage area would be required.

For sewerage planning, the Northwest Lake Washington sewerage area is subdivided into 18 local service areas (Fig. 14-4 and Table 14-6).

South Lake Washington Sewerage Area

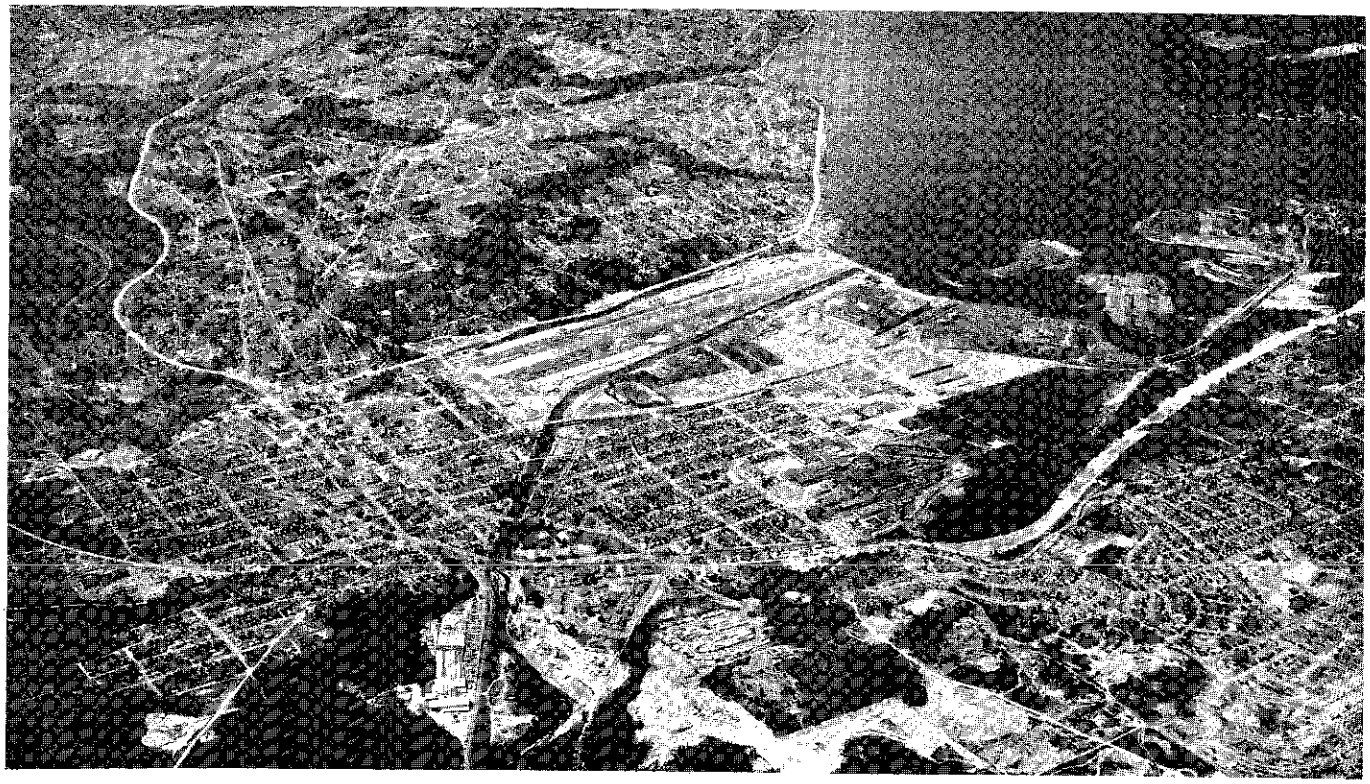
This area contains 34,230 acres and includes all of the Cedar River watershed within the metropolitan

Table 14-6. Local Service Areas within the Northwest Lake Washington Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
NWW-1	1,410	NWW-10	1,070
NWW-2	950	NWW-11	840
NWW-3	1,070	NWW-12	1,170
NWW-4	490	NWW-13	1,690
NWW-5	1,070	NWW-14	1,150
NWW-6	1,520	NWW-15	1,250
NWW-7	1,410	NWW-16	600
NWW-8	1,100	NWW-17	720
NWW-9	770	NWW-18	990
Total			19,270

See Fig. 14-4 for location.

THE CITY OF BOTHELL and the entire North Lake Washington sewerage area are expected to develop rapidly with expansion of growth northward from the city of Seattle. Sammamish River and its broad valley are shown clearly in the foreground.



RENTON lies at the southern end of Lake Washington and is the center of a growing industrial and residential area. Cedar River, which drains most of the South Lake Washington sewerage area, flows through the city.

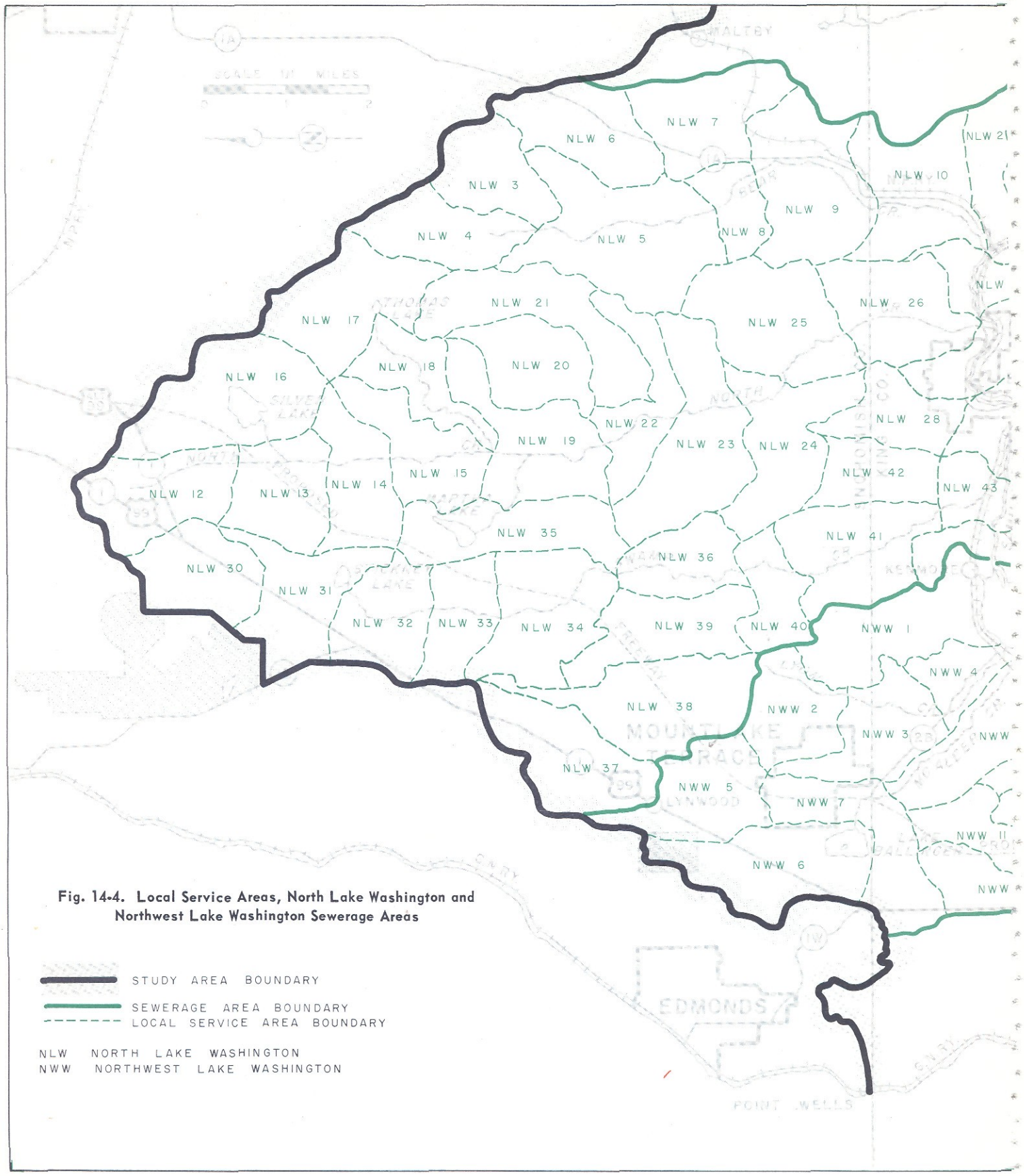


Fig. 14-4. Local Service Areas, North Lake Washington and Northwest Lake Washington Sewerage Areas

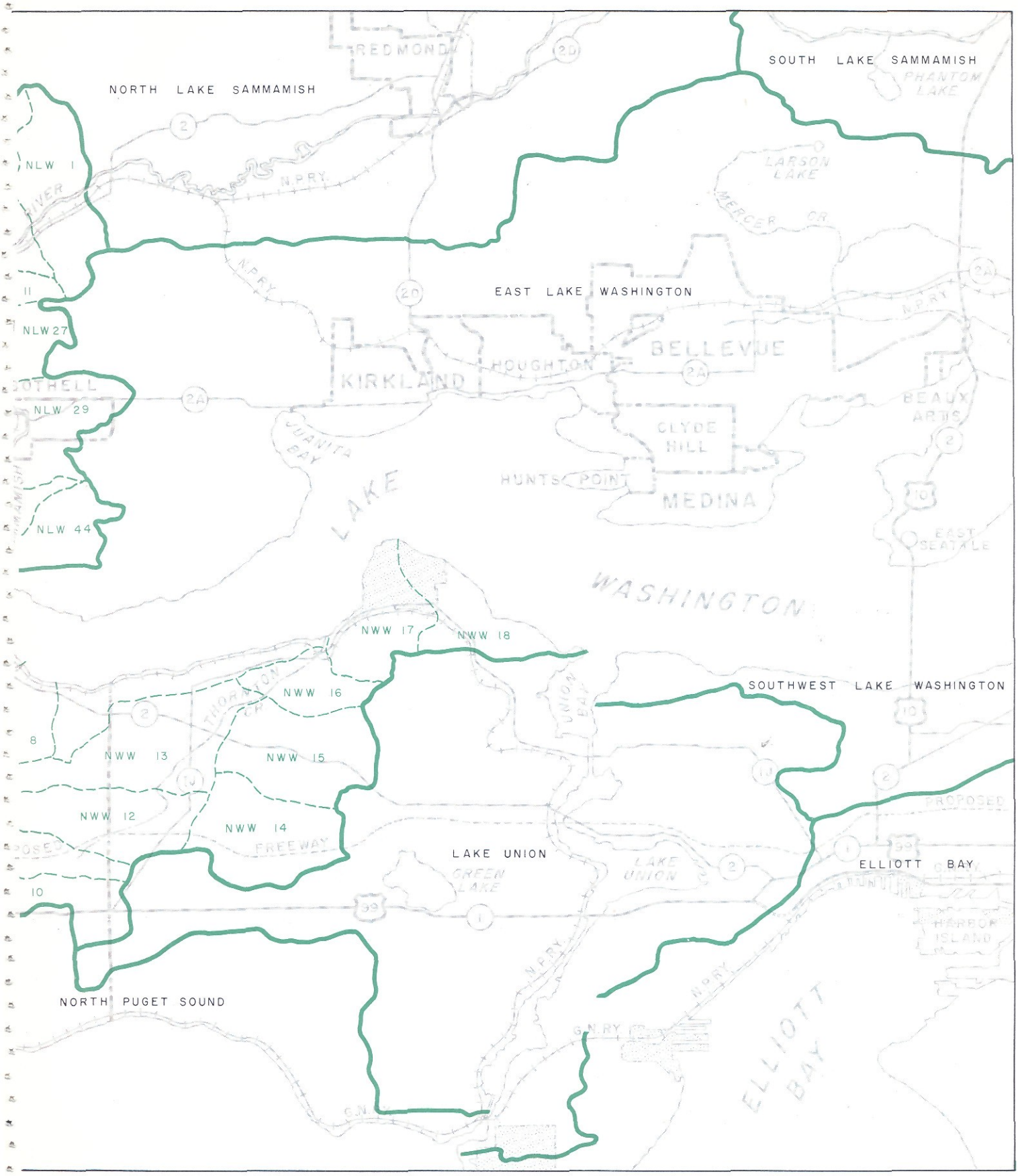
STUDY AREA BOUNDARY

SEWERAGE AREA BOUNDARY

LOCAL SERVICE AREA BOUNDARY

NLW NORTH LAKE WASHINGTON

NWW NORTHWEST LAKE WASHINGTON


NORTH LAKE SAMMAMISH
SOUTH LAKE SAMMAMISH
PHANTOM LAKE
REDMOND
LARSON LAKE
MERCER CR.
NLW 1
N.P.RY.
EAST LAKE WASHINGTON
NLW 27
KIRKLAND
HOUGHTON
BELLEVUE
BOTHELL
NLW 29
BEAUX ARTS
JUANITA BAY
CLYDE HILL
HUNTS POINT
MEDINA
LAKE
NLW 44
EAST SEATTLE
WASHINGTON
NWW 17
NWW 18
SOUTHWEST LAKE WASHINGTON
NWW 16
THORNTON CR.
UNION BAY
NWW 13
NWW 15
NWW 12
NWW 14
FREEWAY
PROPOSED
LAKE UNION
GREEN LAKE
ELLIOTT BAY
HARBOR ISLAND
NORTH PUGET SOUND
G.N.RY.
ELLIOTT BAY

area, as well as areas tributary to Lake Washington, principally through May Creek. It also includes a portion of the incorporated city of Renton, which provides the only sewerage service in the area.

Residential and industrial development is occurring in and around the city of Renton, but the area otherwise is mostly undeveloped. A gradual expansion is expected in the vicinity of Renton, with residential developments radiating outward from the city and along Highway 5 toward Maple Valley.

Although most of the area drains into Cedar River, a portion drains directly to Lake Washington. Sewage flows can be delivered to a central point at Renton and conveyed from there by gravity into the Green River sewerage area.

For sewerage planning, the South Lake Washington sewerage area is divided into 31 local service areas (Fig. 14-5 and Table 14-7).

Green River Sewerage Area

Excluding 2,560 acres which comprise the Lake Youngs watershed of the city of Seattle Water Department, the Green River area contains 71,580 acres. It comprises all of the Green River watershed lying in the study area and includes the cities of Algona, Auburn, and Kent, and part of both Renton and Tukwila. Sewerage service is provided by the cities of Auburn, Kent and Renton, and a small area in the northwest section is served by the Val-Vue Sewer District.

Industrial development, with resulting population growth, is occurring over much of the Green River valley, which is now predominantly agricultural. This development is expected to be accelerated upon completion of the Duwamish Waterway extension to Orillia.

Table 14-7. Local Service Areas within the South Lake Washington Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
SLW-1	1,000	SLW-17	1,160
SLW-2	630	SLW-18	1,430
SLW-3	1,460	SLW-19	670
SLW-4	1,050	SLW-20	860
SLW-5	1,000	SLW-21	1,040
SLW-6	1,070	SLW-22	1,290
SLW-7	1,180	SLW-23	1,230
SLW-8	700	SLW-24	1,040
SLW-9	1,880	SLW-25	1,080
SLW-10	1,150	SLW-26	750
SLW-11	1,330	SLW-27	1,160
SLW-12	770	SLW-28	940
SLW-13	1,040	SLW-29	1,160
SLW-14	1,240	SLW-30	1,090
SLW-15	1,210	SLW-31	1,450
SLW-16	1,170	Total	34,230

See Fig. 14-5 for location.

Table 14-8. Local Service Areas within the Green River Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
GR-1	1,020	GR-37	920
GR-2	1,900	GR-38	1,520
GR-3	1,460	GR-39	1,100
GR-4	2,760	GR-40	760
GR-5	1,310	GR-41	910
GR-6	1,500	GR-42	1,000
GR-7	390	GR-43	640
GR-8	980	GR-44	560
GR-9	1,060	GR-45	1,130
GR-10	980	GR-46	720
GR-11	870	GR-47	1,080
GR-12	1,380	GR-48	650
GR-13	1,310	GR-49	450
GR-14	1,550	GR-50	1,210
GR-15	1,140	GR-51	870
GR-16	1,320	GR-52	780
GR-17	550	GR-53	860
GR-18	510	GR-54	670
GR-19	690	GR-55	660
GR-20	930	GR-56	1,710
GR-21	870	GR-57	630
GR-22	530	GR-58	690
GR-23	1,090	GR-59	910
GR-24	800	GR-60	1,470
GR-25	820	GR-61	1,300
GR-26	1,060	GR-62	1,250
GR-27	1,090	GR-63	1,010
GR-28	420	GR-64	1,010
GR-29	380	GR-65	1,020
GR-30	1,050	GR-66	850
GR-31	1,190	GR-67	1,190
GR-32	530	GR-68	1,530
GR-33	1,090	GR-69	1,380
GR-34	580	GR-70	1,050
GR-35	1,080	GR-71	780
GR-36	930	GR-72	190
Total			71,580[a]

See Fig. 14-5 for location.

[a]*Excluding Lake Youngs reservoir and water department property, 2,560 acres.*

In all probability, the eastern portion of the Green River area will be the last to develop in the metropolitan Seattle area. It is expected, nevertheless, that large residential and industrial developments will take place during the period under consideration.

Drainage in the eastern portion is toward Big Soos Creek, a principal tributary of Green River. Sewage from this area can be conveyed by gravity toward the creek and then southward and westward to the Green River valley at Auburn. Since the valley consists of a broad, flat expanse sloping gently to the north, flows entering from the Big Soos area, together with those generated in the valley itself, can be conveyed northward by gravity over one or more routes.

THE CITY OF KENT in the Green-Duwamish River valley will share in the industrial growth which is expected to follow completion of the Duwamish Waterway extension.

For sewerage planning purposes, the Green River sewerage area is subdivided into 72 local service areas (Fig. 14-5 and Table 14-8).

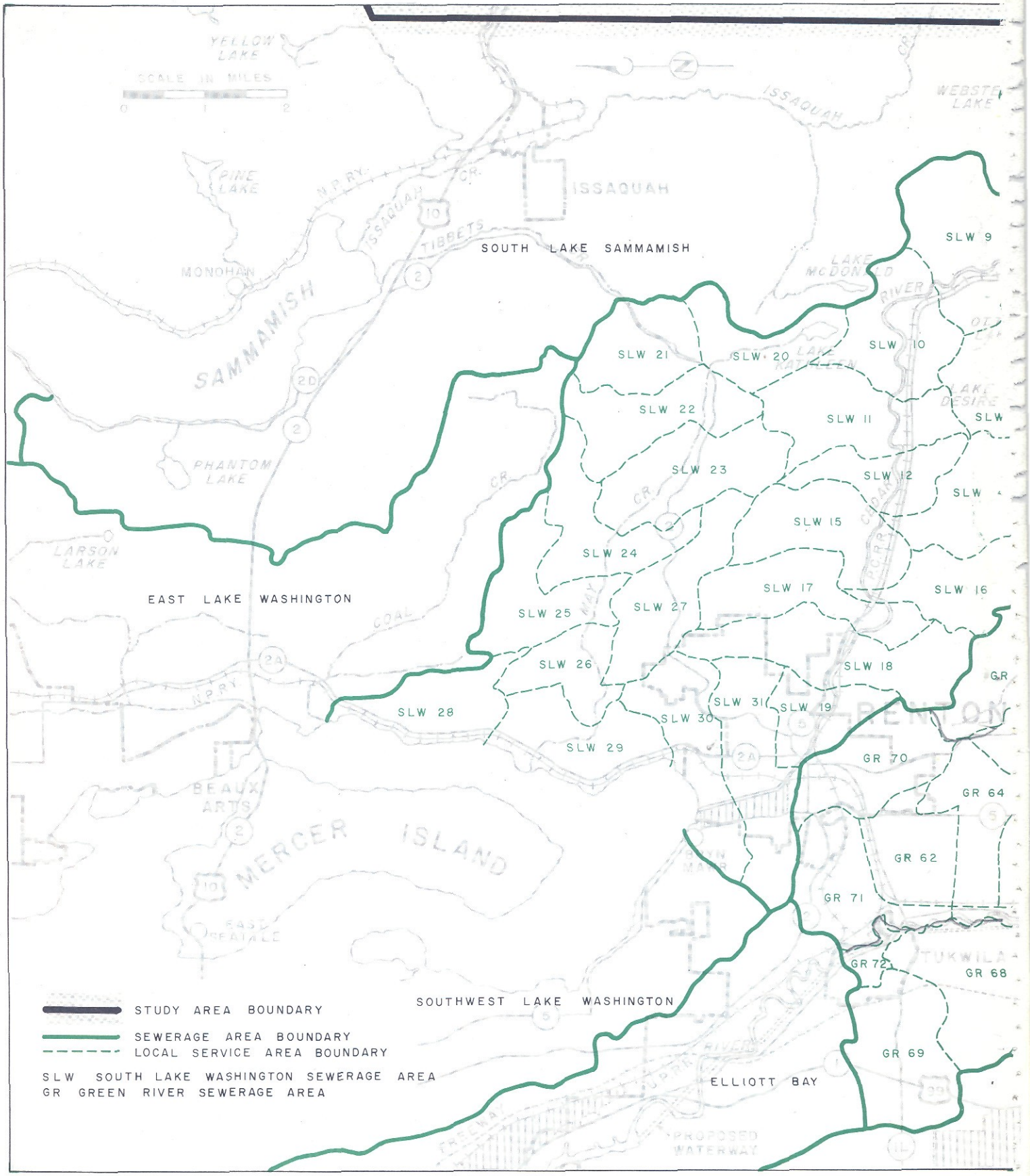
Southwest Lake Washington Sewerage Area

This area contains 8,890 acres and drains directly to Lake Washington through a number of small creeks and ravines. Consisting largely of the city of Seattle, it is already highly developed residentially. Further growth is expected to be predominantly residential and to approach saturation within the next two decades.

Seattle provides sewer service to that part of the area which lies within the city limit. Existing lake

Table 14-9. Local Service Areas within the Southwest Lake Washington Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
SWW-1	490	SWW-9	590
SWW-2	930	SWW-10	520
SWW-3	200	SWW-11	1,050
SWW-4	670	SWW-12	520
SWW-5	900	SWW-13	820
SWW-6	1,220	SWW-14	200
SWW-7	310	SWW-15	230
SWW-8	240	Total	8,890


SCALE IN MILES
0 1 2
YELLOW LAKE
PINE LAKE
MONOHAN
SAMMAMISH
PHANTOM LAKE
LARSON LAKE
ISSAQUAH
WEBSTER LAKE
SOUTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
SOUTHWEST LAKE WASHINGTON
MERCER ISLAND
BEAUX ARTS
EAST SEATTLE
ELLIOTT BAY
PROPOSED WATERWAY
TUKWILA
SLW 9
SLW 10
SLW 11
SLW 12
SLW 15
SLW 16
SLW 17
SLW 18
SLW 19
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SLW 30
SLW 31
GR 62
GR 64
GR 68
GR 69
GR 70
GR 71
GR 72
STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
SLW SOUTH LAKE WASHINGTON SEWERAGE AREA
GR GREEN RIVER SEWERAGE AREA

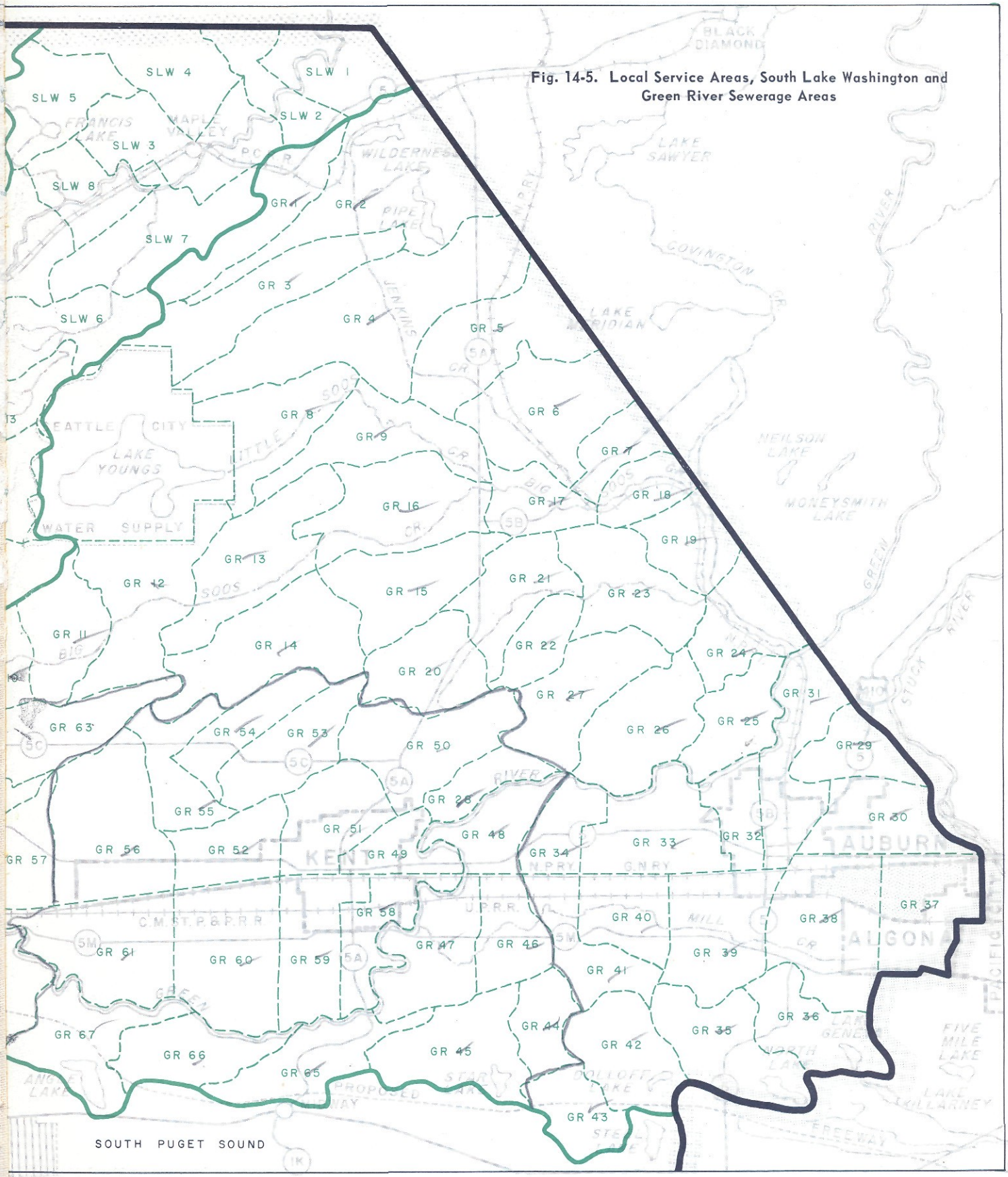


Fig. 14-5. Local Service Areas, South Lake Washington and Green River Sewerage Areas

PRESENT INDUSTRIAL DEVELOPMENT in the metropolitan area is concentrated in the Elliott Bay sewerage area. Industries lie along the Duwamish River and Elliott Bay waterfront. For the most part, sewage from the industrial area is discharged without treatment into the river or bay.

Table 14-10. Local Service Areas within the Elliott Bay Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
EB-1	560	EB-13	1,050
EB-2	1,410	EB-14	750
EB-3	1,500	EB-15	630
EB-4	1,420	EB-16	380
EB-5	1,280	EB-17	620
EB-6	1,360	EB-18	900
EB-7	860	EB-19	300
EB-8	1,110	EB-20	520
EB-9	850	EB-21	140
EB-10	80	EB-22	840
EB-11	1,290	EB-23	1,410
EB-12	1,190	Total	20,450

See Fig. 14-6 for location.

front interceptors convey sewage to three points. Sewage from the two most southerly of these points is transferred to the Elliott Bay sewerage area, while that from the northerly point is transferred to the Lake Union sewerage area.

The southern portion of the area is sewered by the Bryn Mawr-Lake Ridge Sewer District and by Sewerage and Drainage Improvement District No. 4. Sewage presently delivered to the treatment plant of the Bryn Mawr-Lake Ridge district can be conveyed by gravity to the South Lake Washington sewerage area.

For sewerage planning purposes, the Southwest Lake Washington sewerage area is subdivided into 15 local service areas (Fig. 14-6 and Table 14-9).

Elliott Bay Sewerage Area

The Elliott Bay sewerage area contains 20,450 acres and includes essentially all of the Duwamish

Table 14-11. Local Service Areas within the Lake Union Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
LU-1	1,580	LU-10	1,110
LU-2	470	LU-11	660
LU-3	970	LU-12	880
LU-4	560	LU-13	800
LU-5	900	LU-14	620
LU-6	1,660	LU-15	800
LU-7	580	LU-16	1,160
LU-8	1,060	LU-17	1,070
LU-9	680	LU-18	820
Total			16,300

See Fig. 14-6 for location.

River watershed, as well as a strip draining directly to Elliott Bay. In addition to the city of Seattle, which occupies most of it, this area contains a portion of the city of Tukwila and the communities of Allentown, Foster, Riverton, Southern Heights and West Duwamish.

Present industrial development is largely concentrated in the Elliott Bay area, as is most of the downtown commercial district of Seattle. Continued industrial and commercial growth is expected, along with some fringe residential development.

Sewage collection is provided in much of the area by the city of Seattle. Part of this flow is delivered to the Diagonal Avenue plant for treatment and disposal but most of it is discharged without treatment to either Duwamish River or Elliott Bay. A small area at the south boundary is served by the Val-Vue Sewer District. The remainder of the area, much of it heavily populated, depends on private septic tanks.

A major portion of the industrial section is served by independent systems, some of which are operated by the city and others by the industries themselves. Each of these discharges independently into Duwamish River and Elliott Bay without treatment.

Sewage and industrial wastes being generated in the Duwamish River watershed can be picked up at points along the river and conveyed northward by gravity. On the other hand, those being discharged into Elliott Bay through independent outfalls will necessitate construction of a waterfront interceptor. This line will convey the flow either to a central point near the mouth of the Duwamish or to the North Trunk sewer in the Lake Union sewerage area.

To enable sewerage planning on a metropolitan basis, the Elliott Bay sewerage area is subdivided into 23 local service areas (Fig. 14-6 and Table 14-10).

Lake Union Sewerage Area

This area contains 16,300 acres and, with the exception of a small portion at the north, lies entirely within the city of Seattle. With present development at or near saturation, it is the most highly concentrated commercial and residential section of the metropolitan area. Lack of additional building sites and conversion of existing residential areas to commercial use will limit future residential construction to multiple housing units. As indicated in Table 14-1, this trend is expected to result in a small but gradual increase in population.

Except for the northerly part, the Lake Union area is sewered by the city of Seattle. All sewage therefrom is conveyed to the North Trunk sewer, which discharges into Puget Sound offshore from Fort Lawton.

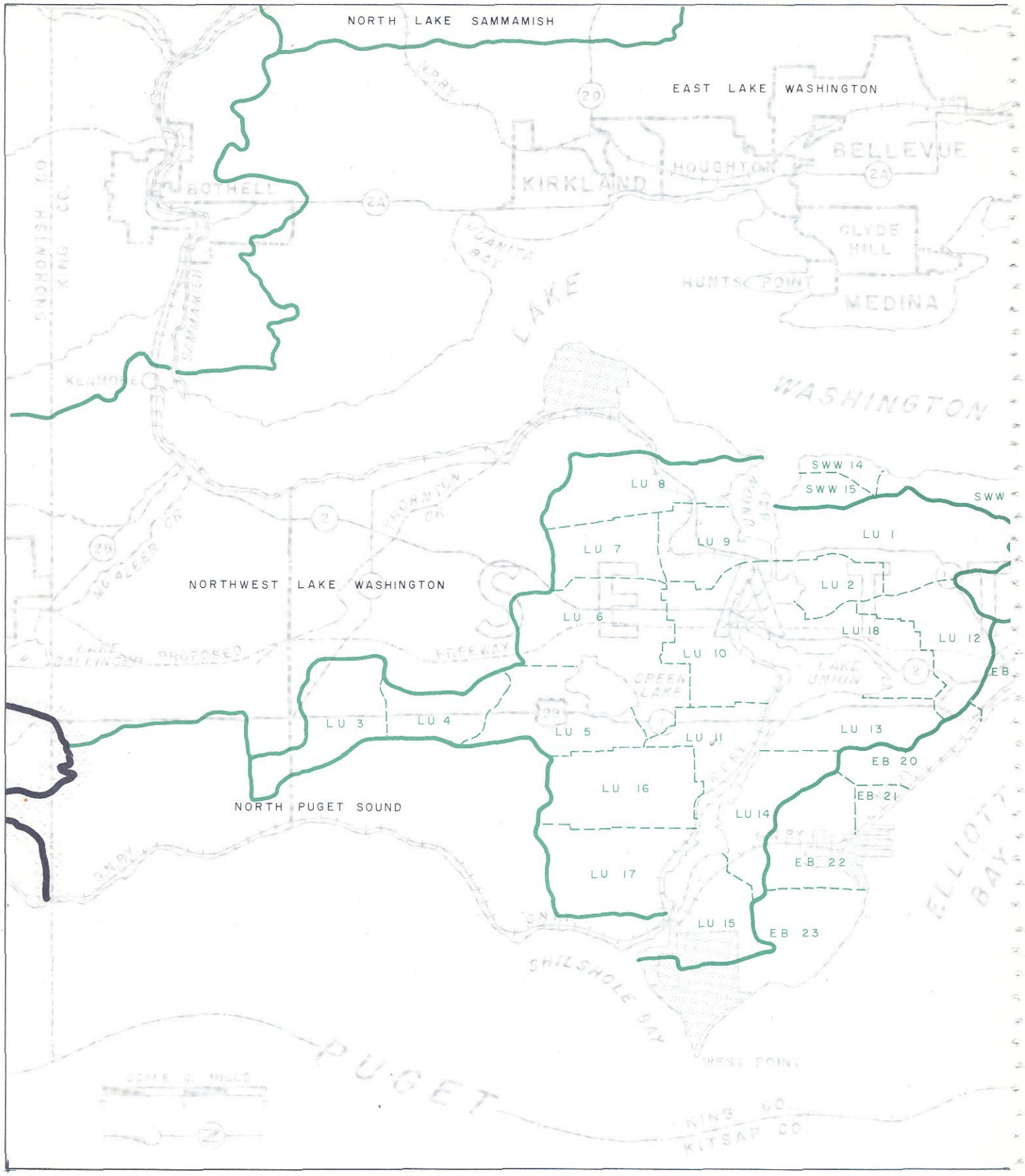
Drainage from the area is generally northward and southward to Union Bay, Lake Washington Ship Canal, Portage Bay, Lake Union and Salmon Bay Waterway. Lakefront interceptors will be necessary at Green Lake, a fresh water body of some 225 acres, lying in the northern portion of the area.

Local service areas, of which there are 18, are laid out basically to conform to the drainage pattern es-

Table 14-12. Local Service Areas within the South Puget Sound Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
Redondo Beach Subarea		SPS-24	770
SPS-1	650	SPS-25	820
SPS-2	830	SPS-26	800
SPS-3	1,000	SPS-27	370
SPS-4	1,010	SPS-28	710
SPS-5	330	SPS-29	910
SPS-6	1,150	SPS-30	820
SPS-7	350	SPS-31	910
SPS-8	410		7,420
SPS-9	280		
SPS-10	620	**Southwest Suburban Subarea**	
SPS-11	940	SPS-32	1,170
SPS-12	480	SPS-33	490
	8,050	SPS-34	360
		SPS-35	430
Des Moines Subarea		SPS-36	870
SPS-13	950		3,320
SPS-14	800		
SPS-15	280	**West Seattle Subarea**	
SPS-16	180	SPS-37	900
SPS-17	1,130	SPS-38	1,060
SPS-18	340	SPS-39	620
SPS-19	1,010	SPS-40	80
SPS-20	1,340	SPS-41	650
SPS-21	1,490	SPS-42	260
	7,520	SPS-43	140
			3,710
Miller Creek Subarea			
SPS-22	860	Total	30,020
SPS-23	450		

See Fig. 14-7 for location.


NORTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
BELLEVUE
HOUGHTON
KIRKLAND
BOTHELL
CLYDE HILL
MEDINA
HUNTS POINT
JUANITA BAY
LAKE WASHINGTON
KENMORE
SNOHOMISH CO.
KING CO.
SAMMAMISH
NORTHWEST LAKE WASHINGTON
NORTH PUGET SOUND
LAKE DALLINCOURT
PROPOSED FREEWAY
THORNTON CR.
McALEER CR.
GREEN LAKE
LAKE UNION
UNION BAY
SHILSHOLE BAY
WEST POINT
ELLIOTT BAY
PUGET
KING CO.
KITSAP CO.
SCALE IN MILES
SWW 14
SWW 15
SWW
LU 1
LU 2
LU 3
LU 4
LU 5
LU 6
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EB 20
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EB 23

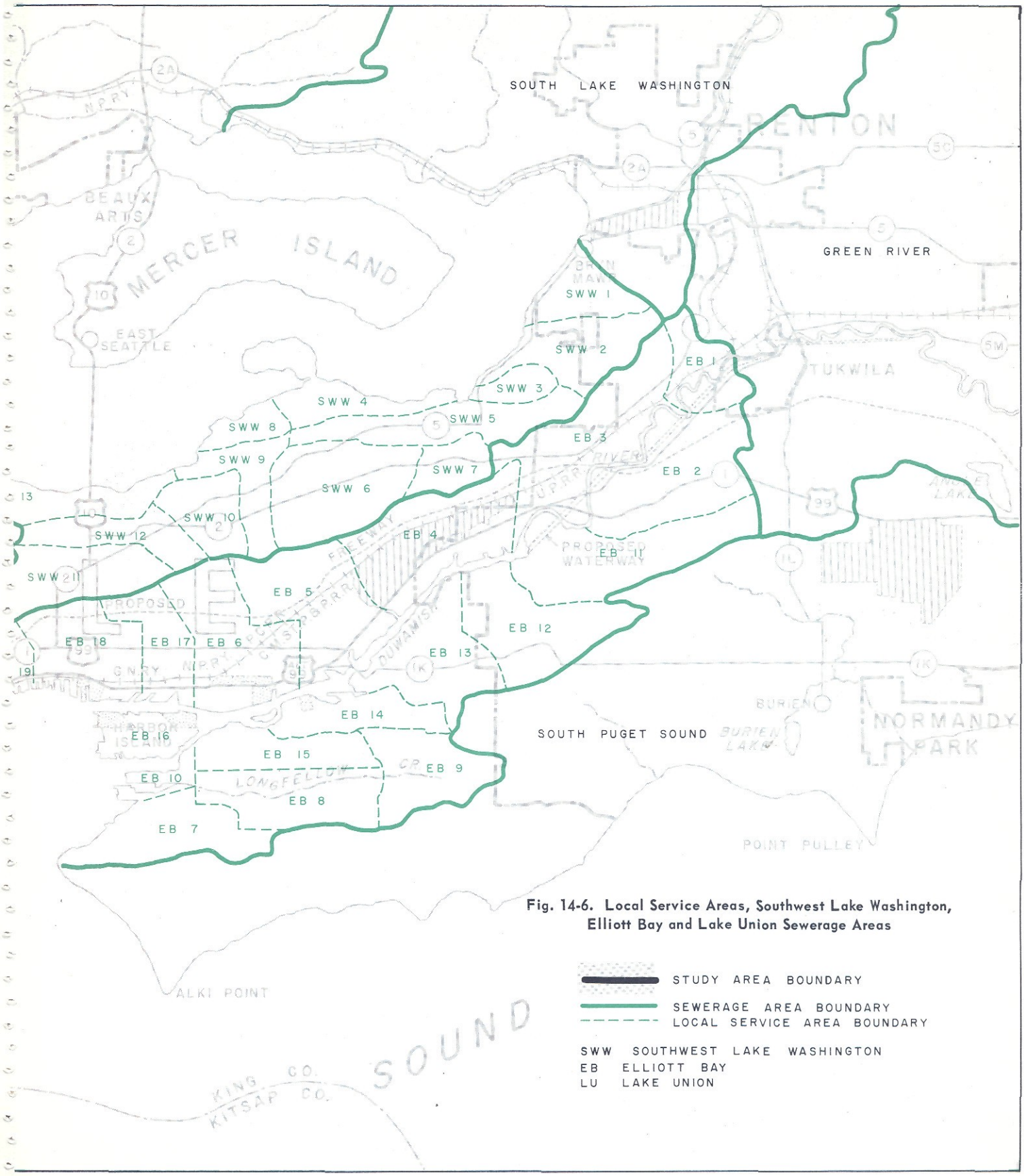


Fig. 14-6. Local Service Areas, Southwest Lake Washington, Elliott Bay and Lake Union Sewerage Areas

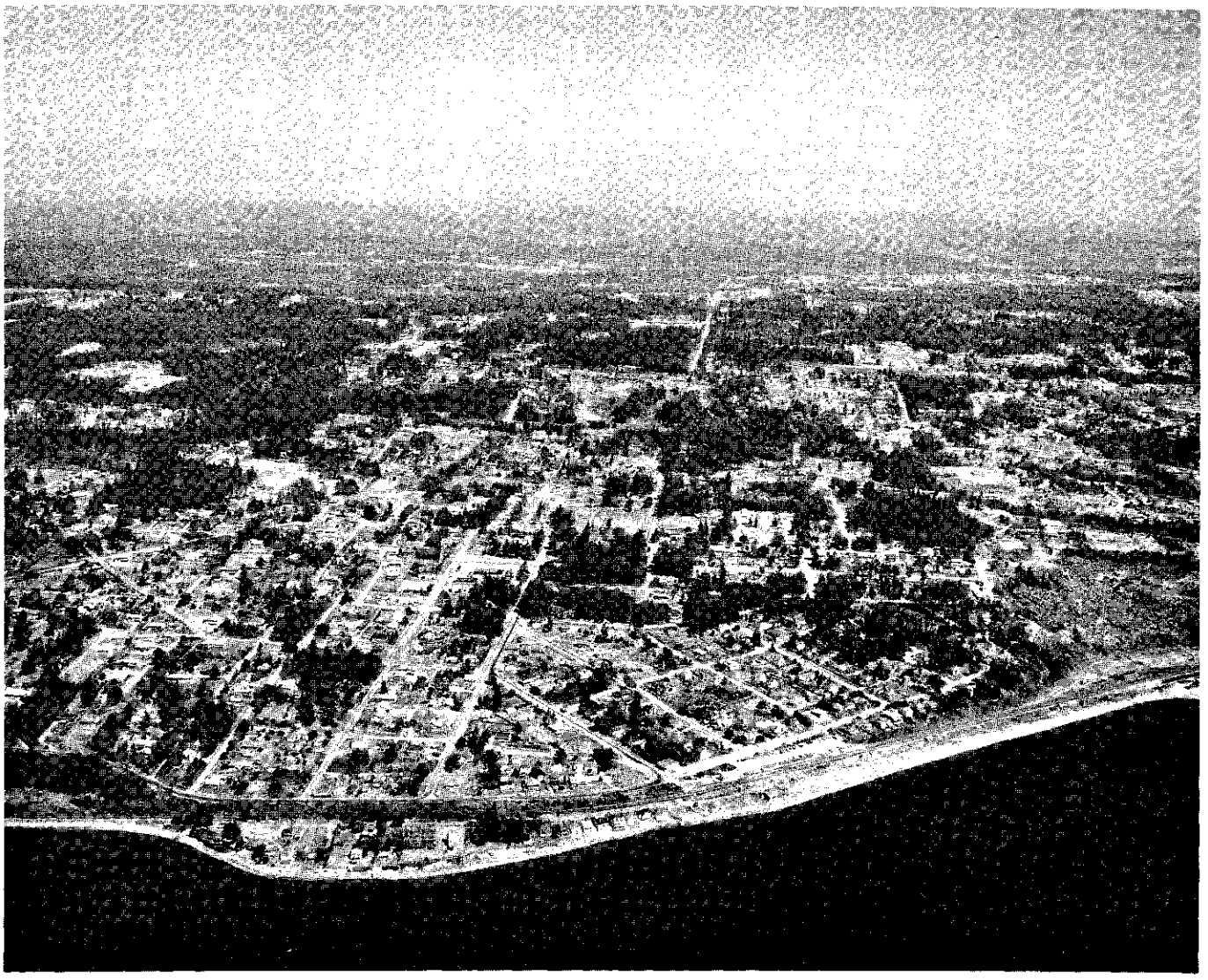
CONTINUED RESIDENTIAL GROWTH is expected in the South Puget Sound drainage area. The photo shows the city of Normandy Park and the development to the north and east of that city.

tablished by the existing sewerage system (Fig. 14-6 and Table 14-11).

South Puget Sound Sewerage Area

Situated entirely within watersheds directly tributary to Puget Sound, the South Puget Sound area comprises 30,020 acres. It is separated from the Green River and Elliott Bay areas by a ridge which runs in a general north and south direction and reaches elevations in excess of 500 feet. Drainage is westward to the sound through a number of small creeks and ravines, each of which is separated from the other by ridges exceeding 300 feet in elevation. For that reason, the sewerage area has been subdivided into five major subareas, each of which can be sewered to its own central point (Fig. 14-7 and Table 14-12). At these points, the collected flows either can be disposed of through a local treatment plant or can be transferred to another subarea for treatment and disposal. In most instances, transfer from one subarea to another would require pumping over the intervening ridge.

Redondo Beach Subarea. This subarea comprises the southernmost part of the South Puget Sound area and includes a small portion of Pierce County. Principal developments have been confined largely to shoreline areas fronting on the sound and on various lakes. These include such communities as Redondo Beach, Lakota, Woodmont Beach, Mirror Lake and Steel Lake. An increased rate of growth is anticipated following completion of the freeway and subsequent expansion of both the Seattle and Tacoma metropolitan areas. No public sewerage facilities are presently available.



THE NORTH PUGET SOUND SEWERAGE AREA will probably approach population saturation within the next two decades. Developments such as that shown at Richmond Beach are occurring throughout the area.

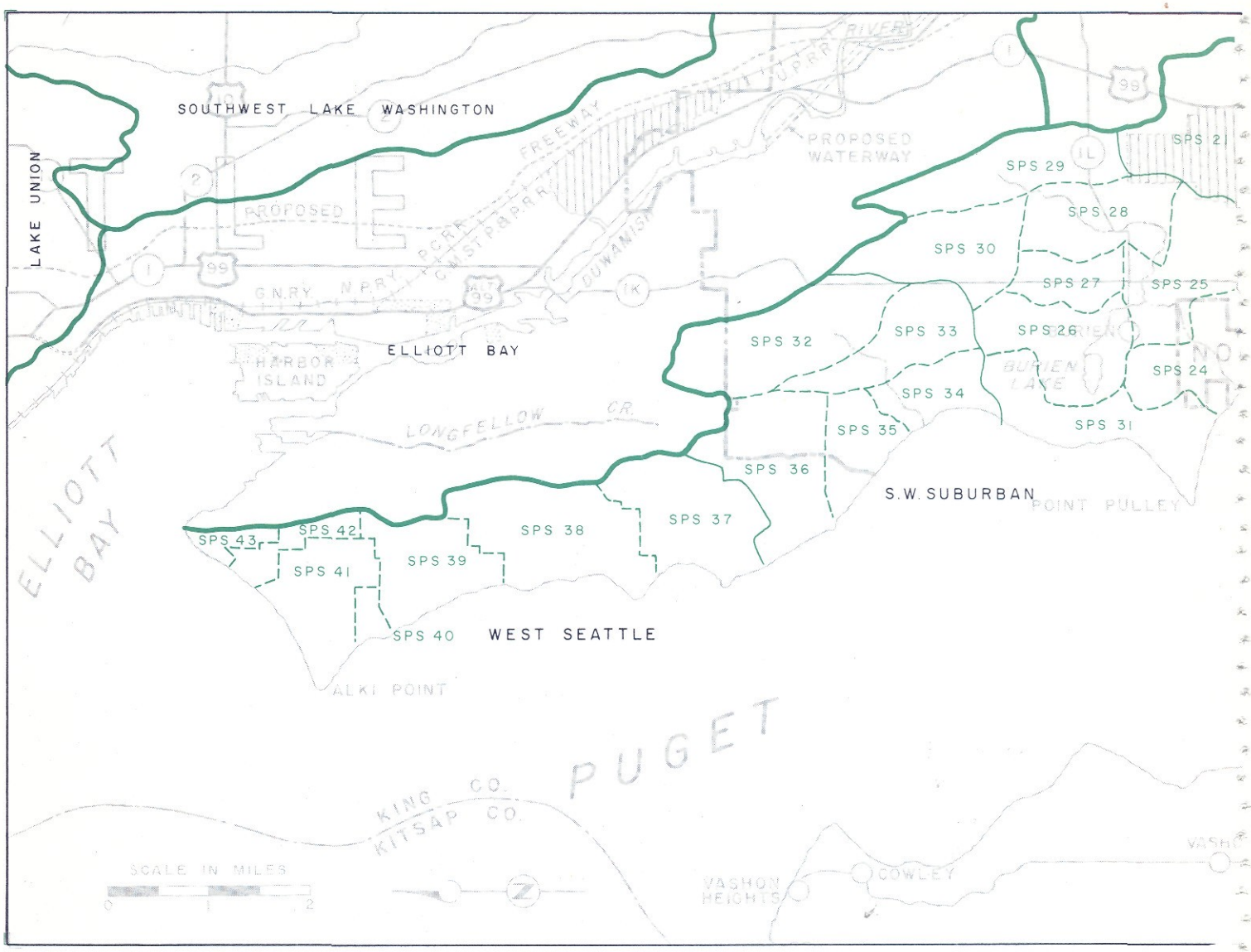
Topographically, the subarea is traversed by ravines and by many small creeks which discharge to Puget Sound. Much of the shoreline along the sound consists of steep bluffs and sheer cliffs, making it difficult to provide for waterfront interception of sewage. Collection of the sewage at one or more points can be achieved, however, by local pumping to an interceptor at the top of the cliffs. Conveyance of sewage out of the subarea would require pumping against a high head.

Des Moines Subarea. Development in this subarea has centered largely around the communities of Des Moines and Zenith, and in the area immediately adjacent to the Seattle-Tacoma Airport. As in the case of the Redondo Beach subarea, accelerated growth can be expected upon completion of the freeway and subsequent expansion of the metropolitan areas of Seattle and Tacoma.

At the airport, which is operated by the Port of Seattle, facilities are available for sewage collection, treatment and disposal. Elsewhere, bonds were voted recently for construction of a sewerage system by the Des Moines Sewer District.

The northern portion of the subarea drains southward from the airport to Des Moines, while the remainder drains westward to the sound. Due to the relatively flat topography in the vicinity of Des Moines, all sewage of the area can be delivered to a central point in that locality. Removal from the area would require high head pumping.

Miller Creek Subarea. As the name implies, this subarea is drained by Miller Creek, which enters



the sound just south of Pulley Point. It includes the city of Normandy Park and the highly developed properties west of the airport and around Burien Lake. The only sewerage service now available is that provided in the Burien Lake area by the Southwest Suburban Sewer District.

Sewage from most of the subarea can flow by gravity to a central collection point near the mouth of Miller Creek. Local drainage on either side of Pulley Point, however, is directly to the sound with the result that local pumping will be required. Sewage generated in the Miller Creek subarea could be conveyed to the Des Moines subarea but high head pumping would be required.

Southwest Suburban Subarea. The Southwest Suburban subarea includes the highly developed residential area lying to the south of the city of Seattle, as well as a portion of the city itself. Sewage collection, treatment and disposal facilities are provided for most of the area by the Southwest Suburban Sewer District. In addition, sewerage service is furnished by the city of Seattle, which has operated the Roxbury Heights system since the area was annexed to the city in 1956.

Drainage of the subarea is generally southward and westward to the sound. Most of the sewage is presently delivered to the Southwest Suburban Sewer District treatment plant near the mouth of Salmon Creek. Conveyance from this point out of the subarea and into the Miller Creek subarea would require high head pumping.

Sewage from part of the Roxbury Heights area is now delivered to a small treatment plant north of Seola Beach. Flow from the Roxbury area could, however, be conveyed to the Southwest Suburban plant by a gravity interceptor along the top of the bluff.

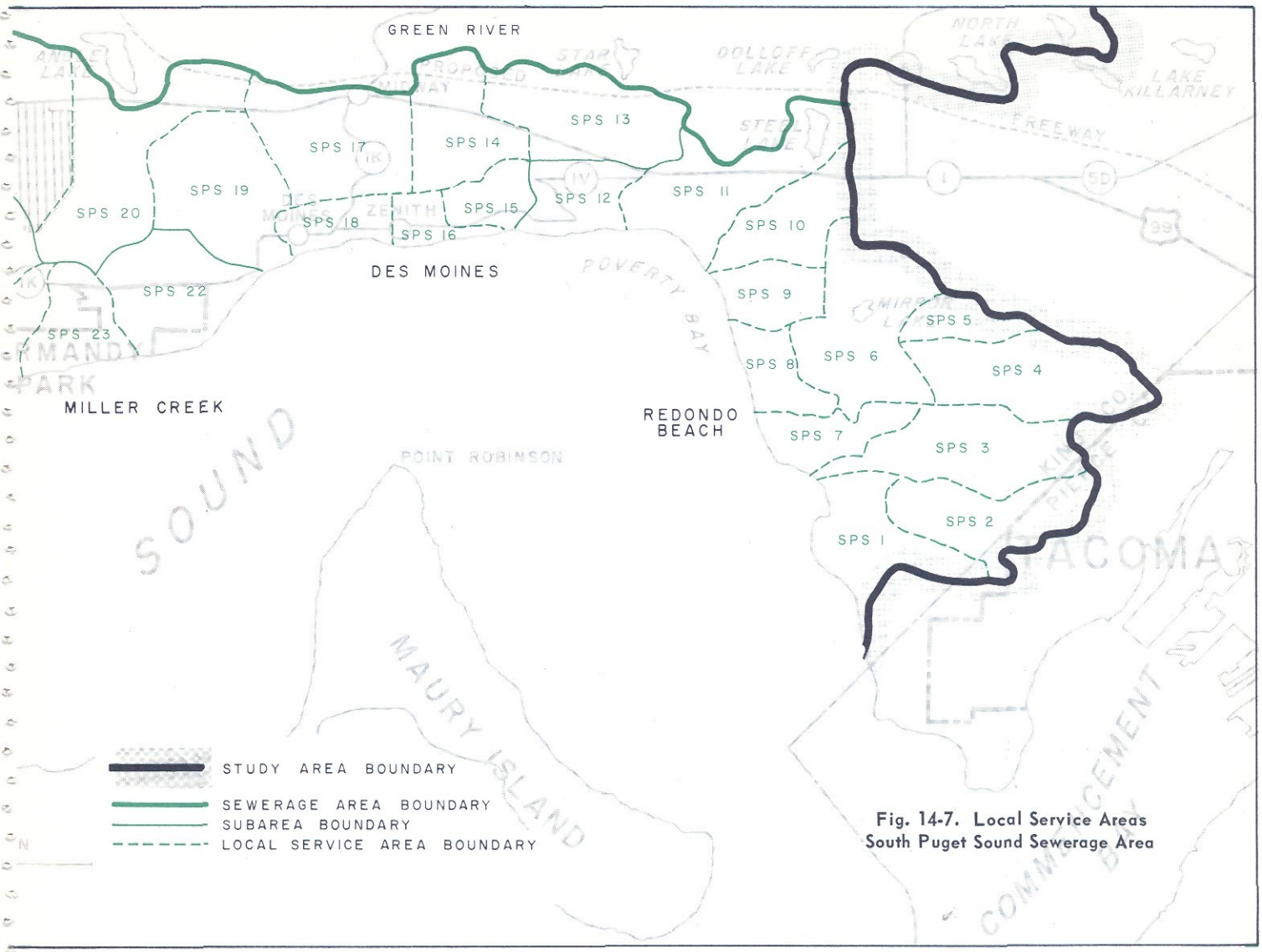


Fig. 14-7. Local Service Areas South Puget Sound Sewerage Area

West Seattle Subarea. This subarea lies wholly within the city of Seattle and is highly developed residentially. Sewerage service is furnished by a number of independent systems, each with a separate outfall discharging directly to the sound. This practice will be terminated in the near future upon completion of a waterfront interceptor whereby sewage from the area will be conveyed to the new Alki Point plant for treatment and disposal. As an alternative to treatment at this point, sewage could be conveyed around Duwamish Head to the Elliott Bay sewerage area.

North Puget Sound Sewerage Area

Situated at the northwesterly corner of the study area, the North Puget Sound sewerage area comprises 6,350 acres, 260 of which lie in Snohomish County. It drains westward to Puget Sound and is separated from sewerage areas lying to the east by a north-south ridge which reaches elevations in excess of 400 feet. Drainage to the sound is through a number of small ravines and creeks, the principal of which are Piper and Boeing. Residential development in the area is expanding rapidly and probably will approach saturation within the next two decades.

Because of its topography, the North Puget Sound sewerage area is divided into three major subareas (Fig. 14-8 and Table 14-13). Each of these is described briefly in the following sections.

Seaview Subarea. The Seaview subarea lies entirely within the city of Seattle and is presently served by a number of small systems, each discharging directly to the sound. Improvements consisting of a waterfront interceptor, a pumping station, and a force main across the Salmon Bay Waterway are now under construction and will enable delivery instead to the North Trunk sewer of the city.

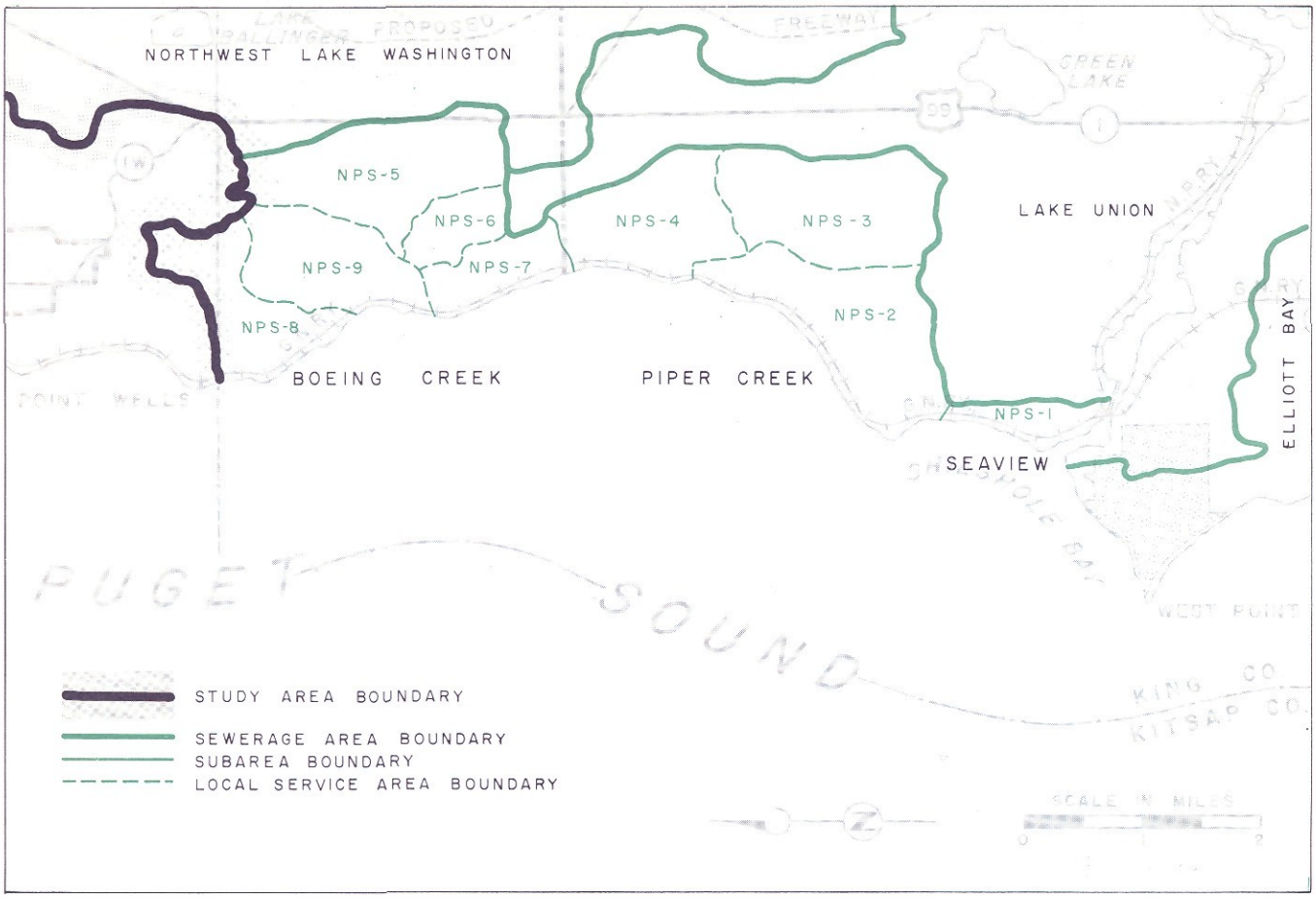


Fig. 14-8. Local Service Areas, North Puget Sound Sewerage Area

Piper Creek Subarea. This subarea, which lies entirely within the city of Seattle, is drained through a number of ravines and creeks, the principal of which is Piper Creek. Most of it is served by separate sanitary sewers which terminate at a treatment plant near the mouth of Piper Creek. Sewage from the remainder of the area also can be delivered to this point, although local pumping may be required for waterfront areas to the north and south. High head pumping would be required for transfer to another subarea.

Boeing Creek Subarea. The Boeing Creek subarea includes the Highlands, Richmond Beach and Ronald. Public sewerage service is provided in Richmond Beach by Sewerage and Drainage District No. 3 and in the Highlands area by a privately owned system. The Ronald Sewer District is currently planning construction of facilities.

Drainage is generally westward to the sound through Boeing Creek and other small channels. Sewage can be conveyed to a point at the mouth of Boeing Creek by means of a waterfront interceptor extending south from Richmond Beach and by an interceptor extending north from the Highlands along the top of the bluff. High head pumping would be necessary for transfer out of the subarea.

DRAINAGE AREAS

Except for those areas which were selected specifically for the drainage studies reported in Chapter 17, no consideration is given herein to a division of metropolitan Seattle into the many drainage areas

Table 14-13. Local Service Areas within the North Puget Sound Sewerage Area

Area designation	Area in acres	Area designation	Area in acres
Seaview Subarea		**Boeing Creek Subarea**	
NPS-1	230	NPS-5	1,060
	230	NPS-6	270
		NPS-7	350
Piper Creek Subarea		NPS-8	770
NPS-2	1,070	NPS-9	670
NPS-3	1,090		3,120
NPS-4	840		
	3,000	Total	6,350

See Fig. 14-8 for location.

it naturally contains. Such a division can be made only when the type of development is established and when the drainage pattern, as modified by street layouts and other factors, can be definitely ascertained. It should be emphasized, however, that storm drainage planning for a specific locality must take into account the total runoff from the upstream area tributary to it. This principle was adhered to in the development of the model drainage plans presented in Chapter 17.

PART V

SEWERAGE AND DRAINAGE PLANS

Chapter 15. Development of Sewerage Plans
Chapter 16. Stage Construction of Sewerage Facilities
Chapter 17. Development of Storm Drainage Plans
Chapter 18. Separation of Combined Sewers in the City of Seattle
Chapter 19. Financing of Recommended Facilities

Chapter 15
DEVELOPMENT OF SEWERAGE PLANS

In general, the most satisfactory and economic solution of the sewerage problem of a metropolitan area is achieved when sewage from the entire area is delivered either to a single point or to a relatively few points for treatment and disposal. To determine the feasibility of providing central sewerage facilities, it is necessary (1) to outline and analyze all reasonably possible projects, (2) to study the characteristics of each contributing area, and (3) to compare the costs of separate disposal facilities with those of central disposal.

Every project suggested for detailed analysis and comparison must satisfy certain fundamental controlling conditions and requirements. As set forth and discussed in preceding chapters of this report, some of the controlling factors are: geography, topography, geology and climate; recreational and other uses of beaches and waters; population numbers and distribution; value of existing sewerage facilities; characteristics of sewage; and disposal requirements for treated effluents.

METHOD OF ANALYSIS

In determining the best plan for sewerage of the metropolitan area, facilities were first laid out for four basic projects involving delivery of sewage to central locations for treatment and disposal. Each of these projects was analyzed in detail and all of them were compared on the basis of construction cost and total annual cost. In addition, where differences in cost were relatively minor, consideration was given to other aspects which then have to be taken into account in determining the over-all suitability of a sewerage project. As a final step, independent sewerage projects were developed for individual tributary areas and the costs thereof were compared with the costs of participation in the selected central project.

Central Sewerage Projects

In developing central sewerage projects, the first step is to determine what facilities are required in each sewerage area to convey sewage to logical points of concentration. These facilities are required in common regardless of whether the individual area is to have its own treatment and disposal system or is to be served by a central sewerage project. For convenience in reference, the facilities common to both individual and central projects are designated herein as service sewers.

As a second step, it is necessary to determine what facilities are required to convey the sewage from the points of concentration in each sewerage area to a point beyond which alternative plans can be developed for conveyance to appropriate locations for treatment and disposal. These facilities, likewise, are common to all alternative central sewerage projects and are referred to as feeder sewers.

The third step is to develop alternative plans for conveyance of the sewage from the terminus of the feeder sewer system to the final point of treatment and disposal. Facilities thus required, which include main intercepting and main trunk sewers, main pumping stations, treatment works, and outfall sewers, comprise what is referred to hereafter as a core plan.

In determining which of the several possible central sewerage projects would be the most suitable, comparisons need be based only on core plan facilities. This is because both the feeder and service sewer systems are common to each core plan.

Separate Projects for Independent Sewerage Areas

For comparison purposes, it is necessary in the case of each sewerage area (Chapter 14) to determine the cost of its share in the core plan and feeder sewer systems. This is accomplished by using the ratio of the flow from each individual area to the total flow for which the core and feeder facilities were designed.

To determine whether it would be economically feasible for each sewerage area to participate in the central sewerage project, studies were made of all independent projects which reasonably could be expected to provide adequate service either to the individual area or, in some cases, to combinations of such areas. In some areas, the choice between alternatives with respect to independent facilities was relatively simple and decisions could be made accordingly. In others, however, partial or complete cost comparisons were required.

Independent sewerage projects which were laid out for individual sewerage areas were analyzed in terms of construction cost and total annual cost, and figures thus obtained were compared with the corresponding costs of the core plan project. In general, the project recommended for adoption by each sewerage area is the one shown to represent the greatest economy to that area.

POSSIBLE TREATMENT PLANT AND DISPOSAL SITES

Selection of a site for sewage treatment and disposal operations is governed largely by two factors. These are (1) the ease with which sewage from a given area can be conveyed to a particular site, and (2) the requirements with respect to receiving water conditions. Less restrictive receiving water requirements at any one location may well justify the conveyance of sewage over a considerable distance for treatment and disposal.

Disposal Sites

Because of the requirement that all sewage and sewage effluents be removed from the Lake Washington watershed, the choice with respect to final disposal of the sewage of the metropolitan area is limited to Puget Sound and Green-Duwamish River. Of these alternatives, disposal to the salt waters of the sound presents fewer complications.

Puget Sound. For the purpose of the survey, investigations were made of 12 possible disposal sites in Puget Sound. The sites were selected primarily on the basis of sewage delivery, taking into account local disposal conditions and water use criteria. At 8 of the sites, it was found that satisfactory conditions could be maintained under ultimate peak flow rates by a combination of primary treatment followed by effluent disposal through a diffuser-equipped outfall. Secondary treatment will be required, however, at the Des Moines, Southwest Suburban, Meadow Point and Richmond Beach sites to obtain the necessary reduction in coliform organisms (Chapter 11).

It should be recognized, of course, that all of the decisions made herein with respect to treatment and disposal requirements are based on presently foreseeable uses of the waters of Puget Sound. In the event of unforeseeable developments, it is possible that a higher degree of treatment may be required at any one or all of the locations presently regarded as suitable for primary treatment. For that reason, and because future disposal requirements may become more stringent, primary sewage treatment plants discharging effluent to Puget Sound should be designed and planned, including purchase of necessary land, in such a manner that secondary units may be added later if and when the need arises.

Green-Duwamish River. Four possible sewage disposal sites were investigated along Green-Duwamish River (Chapter 12). Satisfactory performance at these sites is governed by the requirement that a minimum dissolved oxygen level be maintained in the river for the preservation of fish life. To meet that requirement, only effluents from plants providing complete treatment will be acceptable.

Treatment Plant Sites

To be fully satisfactory, a sewage treatment plant site should:

1. Be as close as possible to a body of water or a watercourse suitable for final disposal of treated effluent.
2. Be well isolated from residential or commercial developments, both present and future.
3. Be economically accessible to trunk and intercepting sewers and service roadways.
4. Have reasonably good soil characteristics to reduce the cost of special foundations.

In most areas, sites which meet all of the foregoing criteria are usually difficult, if not impossible, to find. Obviously, therefore, the problem is one of selecting sites which most nearly fulfill these requirements.

Core Plan Sites. Four possible treatment plant sites were selected as the most suitable for central sewerage projects. These are:

1. At West Point at the western extremity of Fort Lawton, hereinafter designated the West Point site. Effluent would be disposed of in Puget Sound.
2. In the industrial zone above the Government Locks on the Lake Washington Ship Canal in an area bordered by Commodore Way and the Great Northern Railroad tracks and 20th and 27th Avenues West, hereinafter designated the Government Locks site. Effluent would be disposed of in Puget Sound off West Point or in Lake Washington Ship Canal above the Government Locks.
3. In the industrial zone southeast of Elliott Bay, hereinafter designated the Elliott Bay site. In this area there are three possible sites, each of which is discussed in following sections of this chapter. Effluent would be disposed of in Elliott Bay or in Duwamish River.
4. At Black River Junction west of Renton, hereinafter designated the Renton site. Effluent would be disposed of in Duwamish River.

Of the four sites, the one which most nearly fulfills the requirements previously set forth is that at West Point. This site is (1) immediately adjacent to the final point of disposal, (2) well isolated from all residential and commercial areas, (3) near the present discharge point of the North Trunk sewer of the city of Seattle, and (4) in an area where soil borings indicate that no special foundation provisions need be made. There are, however, two major disadvantages. First, to obtain delivery of all sewage generated in the metropolitan area, a second tunnel will be required under Fort Lawton. This is because the exist-

ing North Trunk sewer has insufficient capacity, under gravity conditions, to accommodate the predicted ultimate flow. Second, sufficient land for the treatment plant would probably have to be developed by filling low-lying tideland areas to the north of the site.

In the case of the Government Locks site, the primary advantage would be that all sewage of the area could be readily concentrated at this point, utilizing to the fullest possible extent the existing system of the city of Seattle. For disposal to Puget Sound at West Point, treated effluent would be pumped through the existing Fort Lawton tunnel to a suitable outfall. Disposal to the Lake Washington Ship Canal, adjacent to the site, would require complete treatment. The obvious disadvantage to this site is its lack of isolation, although this could be overcome by proper architectural treatment and landscaping. Other disadvantages are high land values and the need for considerable site development, particularly excavation and leveling.

Available sites on Elliott Bay, while relatively good as far as the delivery of sewage is concerned, are not ideal because of remoteness from the point of effluent disposal, poor foundation conditions, and high land values. Although none of these sites is well isolated, the fact that they are all located in a heavy industrial zone makes isolation a matter of less importance.

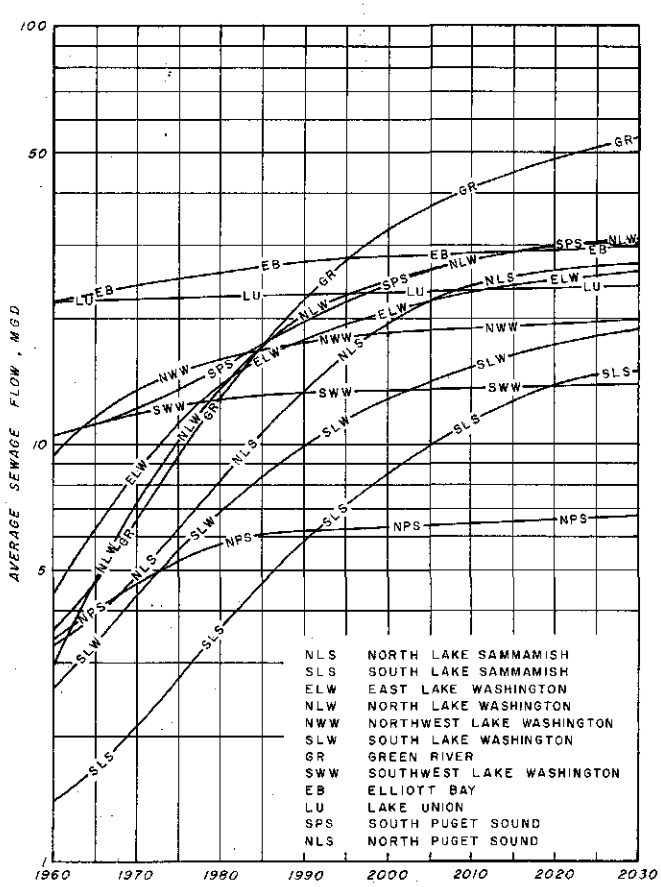


Fig. 15-1. Average Sewage Flows from Sewerage Areas

Disposal requirements in the Duwamish River are such that a plant at the Renton site would have to provide complete treatment. This site, however, offers the advantages of (1) relative ease of sewage delivery from a large part of the metropolitan area, (2) favorable soil and foundation conditions, (3) proximity to the effluent disposal point, and (4) location in an industrial area and consequent lesser need for isolation.

Sites for Plants for Separate Sewerage Areas. In general, the selection of treatment plant sites for individual sewerage areas was made on the basis of logical concentration points for sewage from the tributary area. As such, the locations selected for study purposes were broad rather than specific. Insofar as the present study is concerned, however, the difference costwise between specific sites within these broad locations is not significant in relation to the over-all costs of the individual systems.

PRELIMINARY DESIGN OF SEWERAGE FACILITIES

All plans set forth in this chapter are laid out to serve ultimate development of the tributary area. While it is evident that some of the required facilities, such as treatment plants, can be constructed in stages or increments, the relative economy of the various projects here considered can best be demonstrated by comparing their ultimate costs.

Core Plan Service Area

Because of the pumping required to convey sewage out of the South Puget Sound and North Puget Sound sewerage areas, it became apparent almost at the outset of the study that it probably would be uneconomical for these areas to join in any central sewerage project. Their flows, therefore, were excluded from all core plan facilities as initially conceived. Studies to determine the economic feasibility of their participation in the core plan were made, however, after determining the costs involved in providing separate treatment and disposal facilities.

With the two areas excluded, core plan facilities were laid out for the balance of the metropolitan area. As described in Chapter 14, this service area consists of ten individual sewerage areas.

Sewage Flows

Sewage flows in the facilities herein considered were estimated on the basis of design criteria presented in Chapter 13 and on ultimate population and industrial development of the various sewerage areas (Fig. 15-1 and Table 15-1). These flows are used throughout the report to determine the annual operating costs of treatment plants and pumping stations. They are used also to determine the dates on which facilities

Table 15-1. Average Sewage Flows from Sewerage Areas

Sewerage area	Average flow in mgd during period							Average flow in mgd during design period
	1960-1970	1970-1980	1980-1990	1990-2000	2000-2010	2010-2020	2020-2030	
North Lake Sammamish	4.1	6.5	11.0	16.9	21.0	24.0	26.5	15.7
South Lake Sammamish	1.8	2.9	4.8	7.3	9.7	12.0	14.1	7.5
East Lake Washington	6.7	11.5	15.5	18.8	21.1	23.6	25.8	17.6
North Lake Washington	5.4	11.2	16.9	22.3	26.2	29.0	30.6	20.2
Northwest Lake Washington	11.3	14.8	16.8	18.0	18.6	19.2	19.7	16.9
South Lake Washington	3.8	5.6	8.5	11.5	14.1	16.3	18.2	11.1
Green River	5.5	10.2	17.5	27.8	37.5	45.2	52.0	28.0
Southwest Lake Washington	11.3	12.3	13.0	13.4	13.6	13.8	14.0	13.1
Elliott Bay	23.1	25.0	26.5	27.6	28.7	29.4	29.9	27.2
Lake Union	22.3	22.5	22.8	23.3	23.5	23.7	23.9	23.1
South Puget Sound								
Redondo Beach Subarea	0.5	1.2	2.0	2.7	3.3	4.0	4.7	2.6
Des Moines Subarea	1.6	3.0	4.2	4.9	5.4	5.8	6.3	4.4
Miller Creek Subarea	2.0	3.4	4.8	5.8	6.4	6.8	7.2	5.2
Southwest Suburban Subarea	1.7	2.2	2.7	3.2	3.6	3.8	4.0	3.0
West Seattle Subarea	6.7	6.9	7.0	7.0	7.1	7.1	7.2	7.0
North Puget Sound								
Seaview Subarea	0.2	0.3	0.3	0.4	0.4	0.4	0.4	0.3
Piper Creek Subarea	2.3	3.0	3.3	3.6	3.6	3.7	3.7	3.3
Boeing Creek Subarea	1.6	2.1	2.4	2.5	2.6	2.6	2.6	2.3

should be constructed or enlarged under a stage construction program (Chapter 16).

Use of Existing Facilities

In general, the proposed system of trunk sewers is designed to utilize all local sewerage systems as they now exist. Some of the larger sewers within the more extensive systems, such as those of Southwest Suburban Sewer District and the city of Seattle, including the Lake City Sewer District, are utilized to their full capacity. Other smaller sewers, presently designated as trunk sewers within several of the sewerage agencies, were found to be of such size or in such location that they could not economically be included in any plan of trunk sewerage.

Since many of the existing sewers incorporated in the proposed plans were constructed 50 or more years ago and thus may be structurally weak at some points, their actual utilization will have to be preceded by a thorough inspection. Such an inspection is beyond the scope of this survey, but should be undertaken and completed as soon as possible. Sewers, or sections of sewers, found to be structurally unsound or otherwise damaged should be repaired or replaced. This program, which relates only to structural conditions, would not interfere with design and construction of core plan or other sewerage facilities.

None of the sewage treatment plants presently in use was found to be of the type or size or to be so situated that its inclusion in any long-term comprehensive program for central sewage treatment and disposal could be justified. Certain of the plants, however, particularly those of the city of Seattle at Alki Point and Lake City and that of the Southwest Suburban Sewer District, were found to be of ample capacity and suitable for inclusion in projects designed to serve individual sewerage areas.

INTERCEPTION OF COMBINED SEWERS

Most of the city of Seattle is presently sewered on a combined basis. As such, it presents a difficult problem with regard to the amount of storm water to be allowed for in an interceptor system. Obviously, the provision of interceptor capacity sufficient to accommodate flows from storms occurring at a frequency of once in 10 years would be an economic impossibility. That being the case, the only alternatives are either to provide a capacity which allows overflows from the system at certain specified frequencies, or to provide for complete separation of sanitary sewage from storm water. While the latter, of course, is the more attractive of the two, economic considerations may well preclude its general adoption.

Overflow Frequencies

The frequency at which overflows from a combined system should be allowed is governed by the use of the water into which the overflow occurs. Where the receiving waters are used extensively for recreation or shell fishing, the number of overflows should be limited to the minimum number commensurate with

economic feasibility. On the other hand, where water use is predominantly commercial, overflows can be tolerated at relatively frequent intervals.

Based on an analysis of water use in the area surrounding the city of Seattle and on economic considerations, the following overflow frequencies appear to be justifiable:

1. In all waters of Lake Washington and contiguous waters east of Montlake Bridge -- an average of once per summer.
2. In the Lake Washington Ship Canal and contiguous waters west of Montlake Bridge -- unlimited overflows under storm conditions.
3. In Duwamish River and Elliott Bay -- unlimited overflows under storm conditions.
4. In recreational waters of Puget Sound -- the present interceptor system at West Seattle is designed on the basis of 12 overflows per summer in accordance with specific requirements of the State Pollution Control Commission and State Health Department. Experience will demonstrate whether this frequency is satisfactory. If not, additional interceptor capacity, or its equivalent, will have to be provided to reduce the number of overflows.

Interceptor Capacity

Where the number of overflows is limited because of water use considerations, sufficient capacity must be provided in an interceptor to carry the runoff from a storm having a recurrence interval equal to the overflow frequency. Where storm water overflows are not objectionable, interception of the peak dry weather flow is sufficient.

The capacity, or equivalent capacity, of interceptors from which only a limited number of overflows is permissible can be provided by a number of means including:

1. Construction of an interceptor having a capacity sufficient for the flow.
2. Partial or complete separation of a part or all of the tributary area.
3. Construction of holding tanks at overflow points to store excess flow during periods of rain.

Combined Interceptor. As shown in Fig. 13-3, Chapter 13, a combined interceptor for flows occurring from storms with a recurrence interval of once per summer needs to have a capacity, depending on the time of concentration, of some 30 to 60 times the average dry weather flow. As a matter of comparison, the peak flows for which interceptors for a separate system are designed are usually two to four times average dry weather flow.

Separation. By complete or partial separation of all or part of an area tributary to an interceptor system, the amount of storm water entering the system is appreciably reduced. Complete separation over the entire area would, of course, mean that the system was separate and interceptor design would be as for separate systems. Partial separation, used in conjunction either with large interceptors or with holding tanks, would reduce the size of facilities required in both cases.

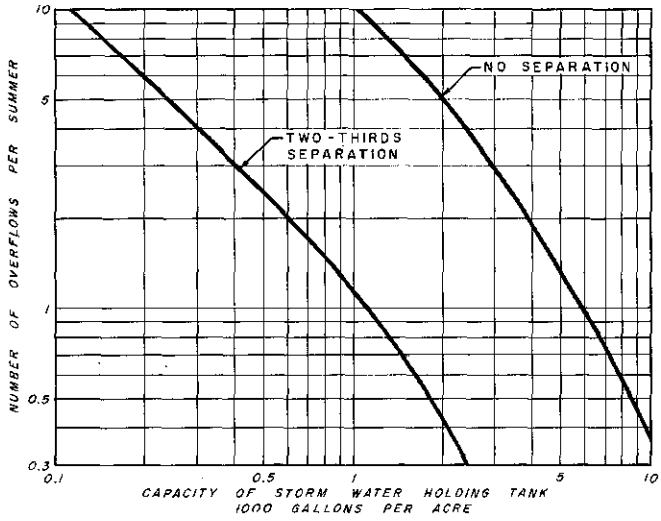


Fig. 15-2. Capacities of Storm Water Holding Tanks for Combined and Partially Separated Systems

The curves are based on an interceptor having a capacity of 2.5 times the average dry weather flow. See Fig. 13-7 for basis of calculations.

Holding Tanks. Provision of holding tanks at points of overflow for the purpose of storing excess water during periods of rain is a feasible alternative in any system where interceptors have inadequate storm flow capacity. In such installations, storm flows in excess of interceptor capacity can be diverted to the holding tanks and then pumped back into the interceptor upon the cessation of rain when capacity is available.

Studies involved in determining the required sizes of holding tanks both for various interceptor capacities and for various overflow frequencies are discussed in Chapter 13. Since partial separation of a tributary area will reduce the size of the holding tank needed for any given interceptor capacity, tank capacities were determined first for areas in which no separation is to be undertaken and second for areas in which two-thirds of the storm water, or roughly that contributed by street drainage only, is to be removed from the combined sewers (Fig. 15-2). These analyses were made on the basis of an interceptor capacity of 2.5 times the average dry weather flow, or about the design capacity of the existing waterfront interceptors in the Lake Washington drainage basin.

Analysis of Interception Methods. To determine which of the three methods outlined above should be adopted for interception of sewage and storm water from combined systems draining to Lake Washington, a study was made of a system in the Southwest Lake Washington sewerage area. This system, which is tributary to a lake front interceptor having a capacity of about 2.5 times average dry weather flow, covers an area of 4,190 acres in lower Rainier Valley from Seward Park on the north to the city limit of Seattle on the south. Consideration was given to the following alternatives:

1. System to remain on a combined basis with intercepting sewers designed for peak wet weather sanitary flow and trunk sewers for a 10-year storm. Storm water holding tanks of a size to allow an average of one overflow per summer would be provided at all overflow points. In addition, since some trunks serving the area do not have enough capacity to handle the flow from a 10-year storm, this alternative would include partial separation of local service areas (Fig. 14-6).
2. System to remain on a combined basis with intercepting sewers designed for storm flows resulting from a rainfall having a recurrence interval of once per summer, and trunk sewers designed for a 10-year storm. As under Alternative 1, this project would include partial separation of local service areas SWW-6 and SWW-7 and a part of SWW-5.
3. System to be partially separated with intercepting sewers designed for peak wet weather sanitary flow and trunk sewers for a 10-year storm. Storm water holding tanks of a size to allow an average of one overflow per summer would be provided at all overflow points. Because the storm water flow would be reduced by partial separation of the entire tributary area, the sizes of holding tanks would be reduced from those proposed under Alternative 1.
4. System to be completely separated with intercepting and trunk sewers designed for peak wet weather sanitary flow.

Estimated construction costs for the four alternatives are given in Table 15-2. As there indicated, the cost of $5,940,000 for Alternative 1, which calls for retention of the existing combined system and provision of storm water holding tanks, is $1,532,000 less than that of the next cheapest alternative. All intercepting sewers for combined systems fronting Lake Washington have, therefore, been designed on the basis (1) of retaining the present systems on a combined basis, (2) of providing holding tanks designed for one overflow per summer, and (3) of separating within the system only to the extent necessary to relieve trunk sewers which now have insufficient capacity.

Table 15-2. Comparison of Construction Costs for Alternative Designs of Trunk and Interceptor Sewers, Southwest Lake Washington Sewerage Area

Alternative designs	Construction cost,[a] dollars
Alternative 1	5,940,000
Alternative 2	7,472,000
Alternative 3	8,873,000
Alternative 4	17,700,000

Partial separation cost, $1,860 per acre.
Complete separation cost, $3,890 per acre.

[a]*Includes engineering and contingencies.*

DESCRIPTION OF CORE PLANS

As previously defined, core plans include only those facilities which are not common to each of the central sewerage plans herein considered. Four basic plans were investigated as follows:

Core Plan A - delivery of all sewage from the metropolitan area to a single primary type treatment plant at the Government Locks site, with effluent disposal to Puget Sound off West Point.

Core Plan B - delivery of sewage to two treatment plants, the first a primary type plant at the West Point site with effluent disposal to Puget Sound, and the second a complete type plant at the Renton site with effluent disposal to Duwamish River.

Core Plan C - delivery of sewage to two treatment plants, both of the primary type. Of these, the first would be at the West Point site with effluent disposal to Puget Sound, and the second at the Elliott Bay site with effluent disposal to Elliott Bay. Four alternatives, differing with respect to plant location, degree of treatment, and effluent disposal, were considered for the Elliott Bay site.

Core Plan D - delivery of sewage to three treatment plants, one a primary type at the West Point site with effluent disposal to Puget Sound, the second a primary type at the Elliott Bay site with effluent disposal to Elliott Bay, and the third a complete type at the Renton site with effluent disposal to the Duwamish River. Three alternatives, differing only in plant location, were considered for the Elliott Bay site.

In all studies relating to selection of the most appropriate core plan, it was assumed that sewage from the east side of Lake Washington, including that from North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, would be conveyed southward and combined with that from the South Lake Washington and Green River sewerage areas. A study of the possibility of conveying sewage from the east to the west side of Lake Washington across the lake was deferred until the most suitable core plan had been determined. Similarly, other feasible modifications of the selected core plan were studied and evaluated.

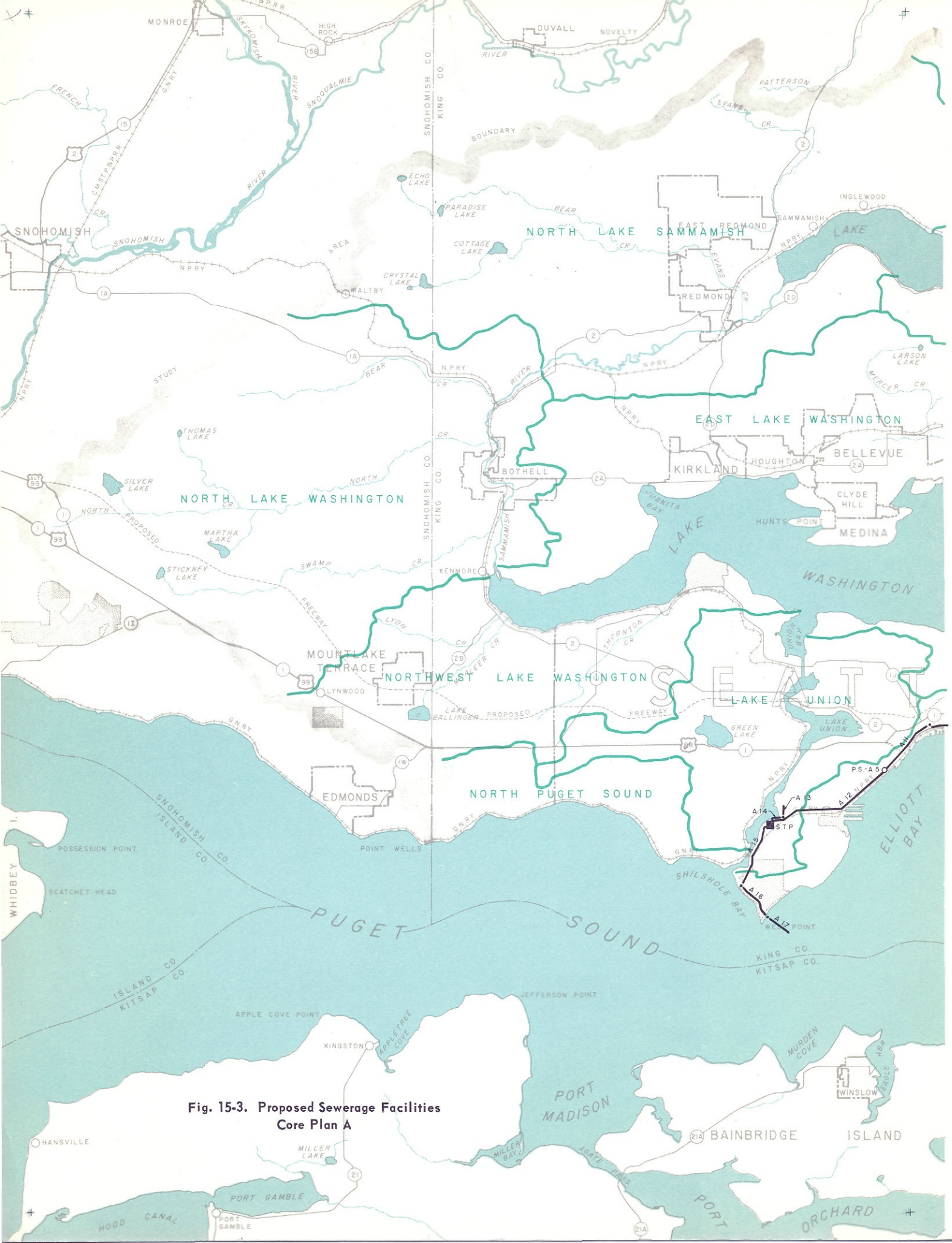
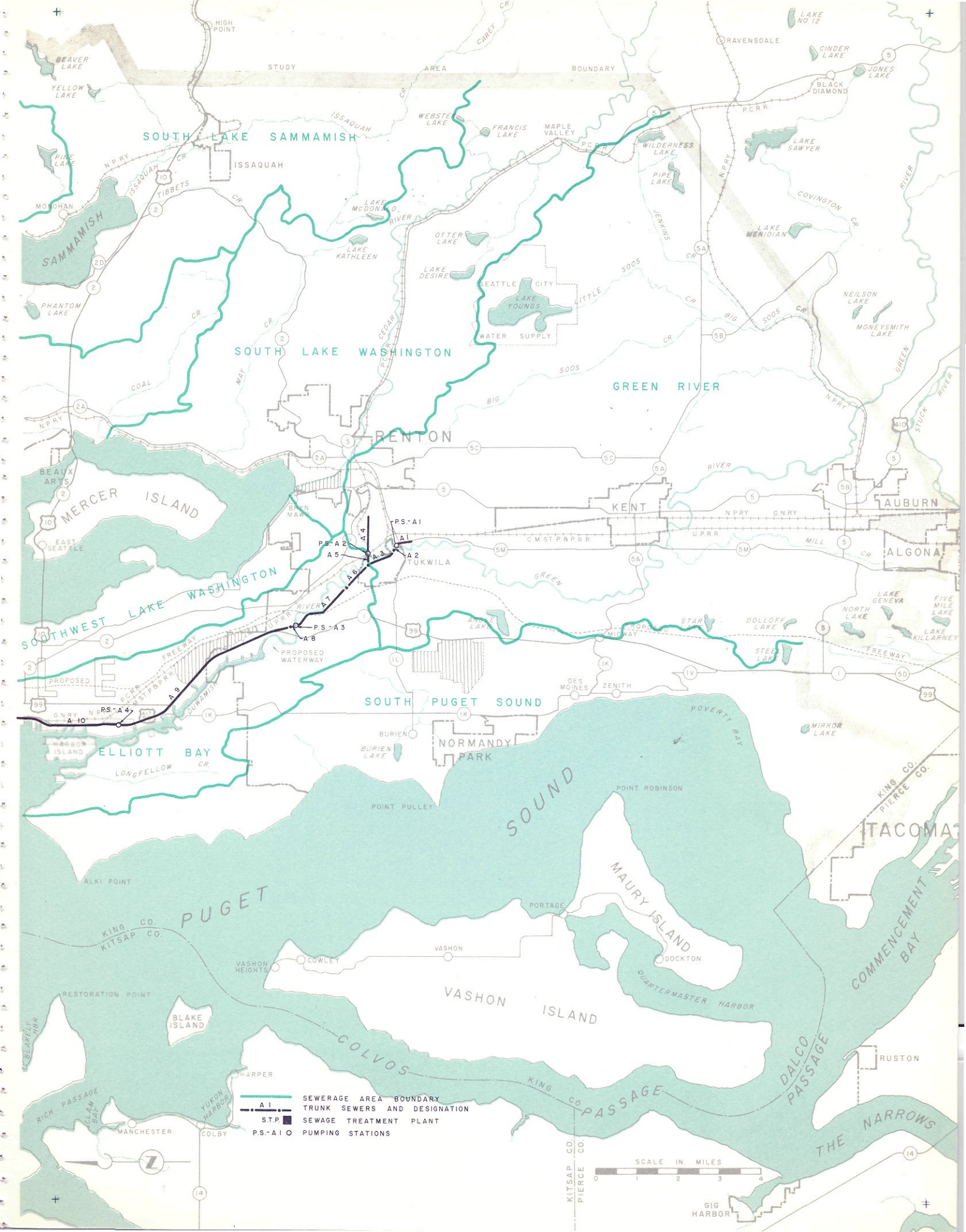


Fig. 15-3. Proposed Sewerage Facilities Core Plan A


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
VASHON ISLAND
MAURY ISLAND
MERCER ISLAND
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
SEATTLE CITY WATER SUPPLY
NORMANDY PARK
BURIEN
DES MOINES
ZENITH
MIDWAY
PROPOSED WATERWAY
PROPOSED FREEWAY
POINT ROBINSON
POINT PULLEY
ALKI POINT
PORTAGE
DOCKTON
VASHON
COWLEY
VASHON HEIGHTS
RESTORATION POINT
BLAKE ISLAND
HARPER
MANCHESTER
COLBY
RUSTON
GIG HARBOR
TACOMA
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
POVERTY BAY
QUARTERMASTER HARBOR
KING CO.
KITSAP CO.
PIERCE CO.
P.S.-A1
P.S.-A2
P.S.-A3
P.S.-A4
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A9
A10
SEWERAGE AREA BOUNDARY
TRUNK SEWERS AND DESIGNATION
S.T.P. SEWAGE TREATMENT PLANT
P.S.-A1 PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 15-3. Description and Estimated Construction Cost, Core Plan A

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
A-1	52	130	2,300 ft of 96-in. RC at 0.05%, average cut 24 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	395,000
A-2	52	130	600 ft of twin 42-in. force mains across Duwamish River	70,000
A-3	52	130	3,300 ft of 96-in. RC at 0.05%, average cut 15 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	401,000
A-4	90	238	5,400 ft of 114-in. RC at 0.07%, average cut 27 - 31 ft, difficult wet, includes sheeting, dewatering and railroad crossing	1,333,000
A-5	90	238	600 ft of parallel 36-in., 42-in. and 48-in. force mains across Duwamish River	110,000
A-6	143	368	2,400 ft of 114-in. RC at 0.16%, average cut 20 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	446,000
A-7	143-146	370-376	10,900 ft of 120-in. RC at 0.13%, average cut 21 - 32 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	2,948,000
A-8	146	376	500 ft of parallel 48-in. and twin 54-in. force mains across Duwamish River	113,000
A-9	146-160	376-411	26,200 ft of 132-in. RC at 0.075 - 0.086%, average cut 16 - 30 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and overflow structure on tributary sewer	7,550,000
A-10	162-181	417-468	16,800 ft of 144-in. RC at 0.058 - 0.068%, average cut 17 - 32 ft, difficult wet, through congested industrial area and through debris fill containing wood and cinders at site of old Yessler Mill, includes imported backfill, repaving, sheeting, dewatering, and underpinning of structures adjacent to 1st Avenue	5,015,000
A-11	181	468	8,400 ft of 144-in. RC tunnel at 0.068%, includes allowance of 20% for uncertainties	6,451,000
A-12	182-189	471-489	15,500 ft of 144-in. RC at 0.068 - 0.075%, average cut 18 - 31 ft, difficult wet, includes 2,000 ft on piles, imported backfill, repaving, sheeting, dewatering, railroad and highway crossings, and overflow structure on tributary sewer	4,841,000
A-13	70	175	1,600 ft of existing 138-in. at 0.035%	Existing
A-14	70	175	2,900 ft of existing 144-in. at 0.032%	Existing
Subtotal, sewers				29,673,000
PS-A-1	52	130	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 28 ft, structure about 30 ft below ground, difficult wet, includes sheeting and dewatering	588,000
PS-A-2	90	238	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 34 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	863,000
PS-A-3	146	376	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 37 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	1,113,000
PS-A-4	160	411	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 31 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	1,173,000
PS-A-5	181	468	Pumping station, single stage, motor and diesel engine driven, static lift 30 ft, total head at peak flow 34 ft, structure about 40 ft below ground, difficult wet, includes sheeting and dewatering	1,273,000
Subtotal, pumping stations				5,010,000

Continued on next page

Table 15-3. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[a] dollars
	Average DWF	Maximum WWF		
STP	265	660	Sewage treatment plant, primary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes purchase of 50 acres of land and site development	19,950,000
A-15	265	660	8,000 ft of existing 144-in. at 0.19%, to be converted to effluent outfall	Existing
A-16	265	660	4,700 ft of 144-in. RC effluent outfall, includes construction of 2,500 ft under tidal conditions, and dewatering	1,259,000
A-17	265	660	3,900 ft of twin 84-in. RC submarine outfalls to a water depth of 210 ft, includes diffuser sections over last 475 ft	3,254,000
Subtotal, outfall				4,513,000
Total contract cost, Core Plan A				59,146,000
Engineering and contingencies, 25 per cent				14,786,000
Total construction cost, Core Plan A				73,932,000

See Fig. 15-3 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

Core Plan A

Sewers, pumping stations, treatment works and outfalls called for under Core Plan A are designated as to location in Fig. 15-3 and are described in Table 15-3. Under this plan, effluent from the treatment plant would be pumped through the existing North Trunk sewer of the city of Seattle for final disposal in Puget Sound off West Point.

Intercepting Sewers. Intercepting sewers under Core Plan A include two branches, a south and a north. The south branch would intercept all sewage from the North Lake Sammamish, South Lake Sammamish, East Lake Washington, South Lake Washington and Green River sewerage areas at a point east of Renton and would convey this flow northward, following generally the route of State Highway 5M and East Marginal Way to the Elliott Bay waterfront. Along the way, additional sewage would be picked up from the Southwest Lake Washington and Elliott Bay sewerage areas and all existing outfalls would be intercepted. On the Elliott Bay waterfront, the sewer would be laid in Alaskan Way and would intercept all industrial and sanitary sewage outfalls.

North of Madison Street, Alaskan Way is constructed on a pile-supported platform, thus precluding the possibility of continuing the south branch in open cut. It would be necessary, therefore, to route the sewer east at Columbia Street to First Avenue, from which point a tunnel would be constructed northwestward along First and Western Avenue alignments to terminate at the intersection of Western and Elliott Avenues. At its deepest point, this tunnel would be about 105 feet below ground surface. From the tunnel exit, the interceptor would be constructed by open cut along Elliott Avenue to north of Pier 91 where it would turn north along the Great Northern Railroad tracks to the treatment plant. To avoid excessive cuts, a total of five pumping stations would be required.

The north branch would consist of a short leg extending eastward from the treatment plant and would convey sewage from the North Lake Washington, Northwest Lake Washington and Lake Union sewerage areas to the plant. This section would utilize a portion of the existing North Trunk sewer of the city of Seattle.

Sewage Treatment Plant. Designed for an average dry weather flow of 265 mgd and a peak storm flow of 660 mgd, the sewage treatment plant would be of the primary type and would require influent pumping from both interceptors. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, as about 30 minutes detention time would be available in the outfall sewer even at the peak flow of 660 mgd. Digested sludge would be hauled away in tank trucks for disposal elsewhere. Sludge gas

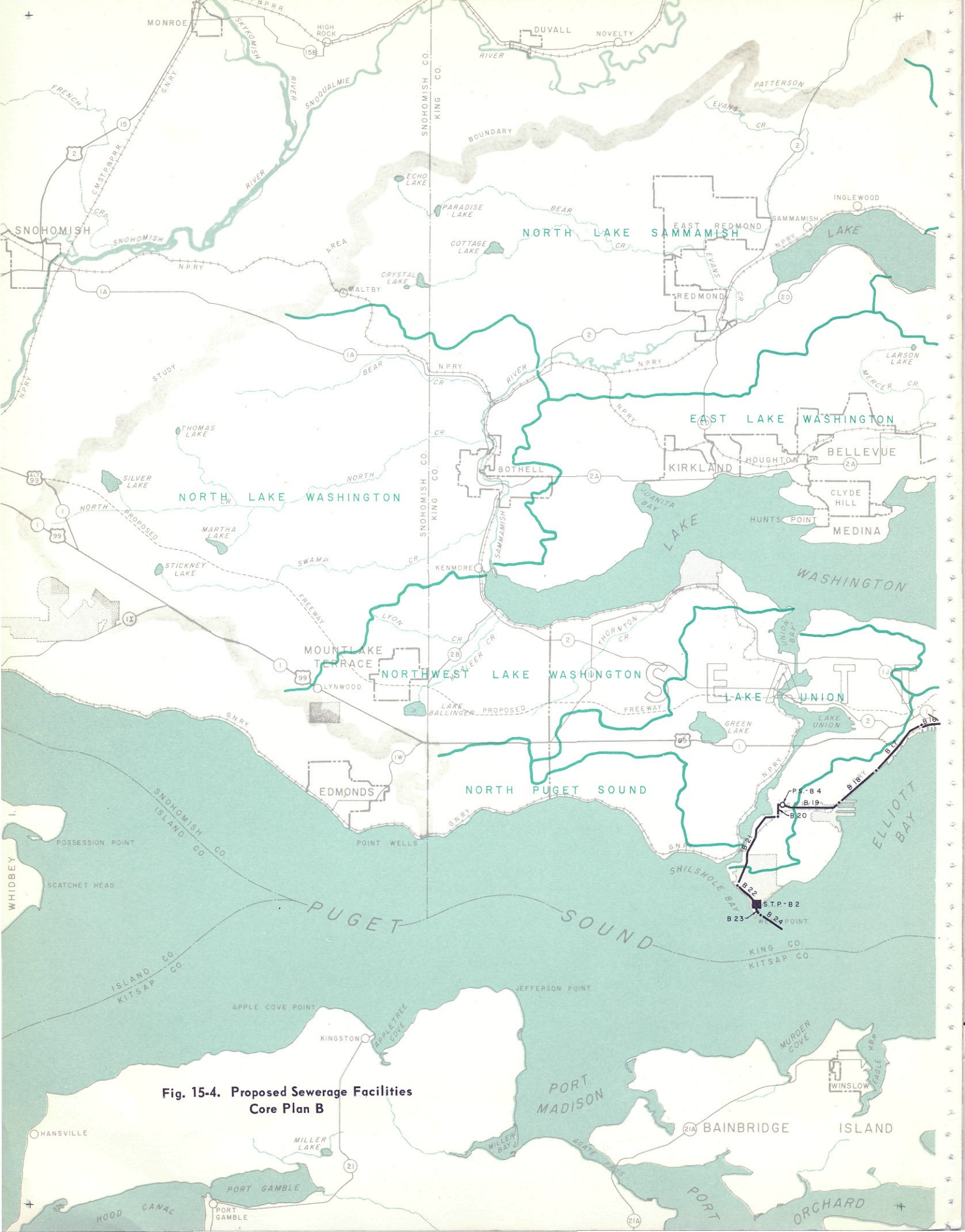
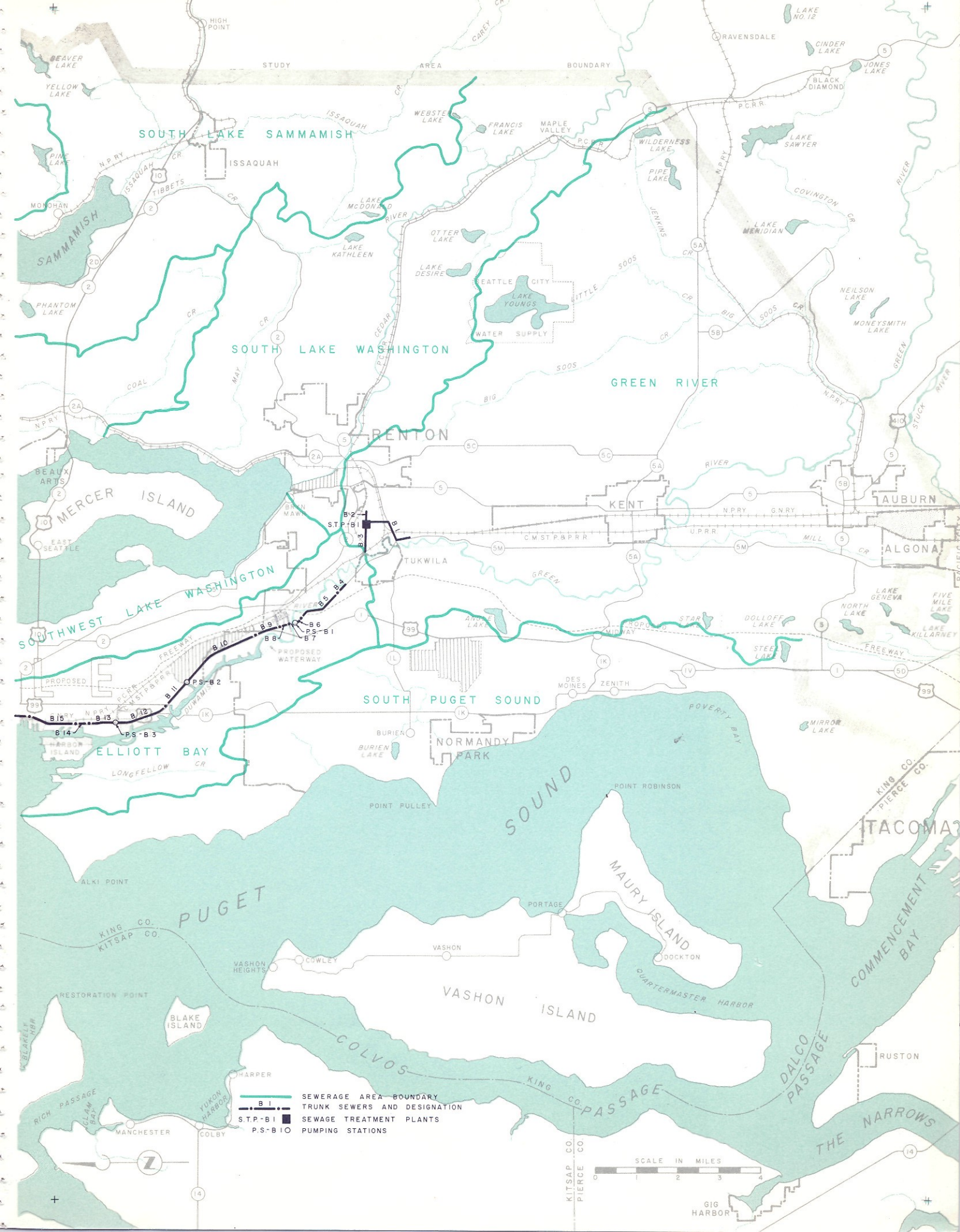


Fig. 15-4. Proposed Sewerage Facilities Core Plan B


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
MERCER ISLAND
VASHON ISLAND
MAURY ISLAND
BLAKE ISLAND
RENTON
KENT
AUBURN
ALGONA
TUKWILA
NORMANDY PARK
TACOMA
SEATTLE CITY WATER SUPPLY
SAMMAMISH
ISSAQUAH
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
HIGH POINT
MONOHAN
EAST SEATTLE
BURIEN
DES MOINES
ZENITH
VASHON
COLBY
MANCHESTER
HARPER
RUSTON
GIG HARBOR
DOCKTON
PORTAGE
POINT ROBINSON
POINT PULLEY
ALKI POINT
RESTORATION POINT
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
QUARTERMASTER HARBOR
POVERTY BAY
KING CO.
KITSAP CO.
PIERCE CO.
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B-11
B-12
B-13
B-14
B-15
PS-B1
PS-B2
PS-B3
S.T.P.-B1
SEWERAGE AREA BOUNDARY
TRUNK SEWERS AND DESIGNATION
SEWAGE TREATMENT PLANTS
PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

would be used for the generation of all power required in the plant and as a source of heat for sludge heating purposes. Because of the proximity to residential and commercial areas, particular emphasis would be placed on architectural treatment of plant structures and on landscaping of the grounds.

Effluent Disposal. Treated effluent would be chlorinated during the recreational season, May to September, and pumped to the western extremity of West Point, where it would be discharged to Puget Sound approximately 3,900 feet offshore at a water depth of about 210 feet. The outfall would consist of a portion of the existing North Trunk sewer, a land section from the terminus of the North Trunk along the north shoreline of Fort Lawton, a land section across West Point, and a twin 84-inch submarine section. Since the land section along the shoreline of Fort Lawton would be constructed under tidal conditions, cost estimates are adjusted accordingly.

Construction Cost. Estimated construction costs of Core Plan A, including engineering and contingencies, total $73,932,000 (Table 15-3). Of this total, approximately 50 per cent is for intercepting sewers, 8 per cent for pumping stations, 34 per cent for the treatment plant, and 8 per cent for the outfall.

Core Plan B

Locations of sewers, pumping stations, treatment works and outfalls are designated in Fig. 15-4. Descriptions of these facilities are given in Table 15-4.

Intercepting Sewers - Renton System. Intercepting sewers for the Renton system include 2 short branches which join at the treatment plant. One would extend southward 6,700 feet from the plant and would serve the Green River sewerage area. The second branch, serving the North Lake Sammamish, South Lake Sammamish, East Lake Washington and South Lake Washington sewerage areas, would extend 1,300 feet east from the plant.

Sewage Treatment Plant - Renton System. As explained in Chapter 12, the waste receiving capacity of Duwamish River is such that a daily organic load equivalent to 15,000 pounds of 5-day BOD could be discharged at the Renton site while still maintaining satisfactory conditions with respect to dissolved oxygen. Compared to this, the ultimate BOD load which would be delivered to the treatment plant amounts to 332,000 pounds per day, based on an estimated equivalent population of 1,660,000 and a BOD contribution of 0.2 pounds per capita per day. To produce an effluent satisfactory for discharge to the river, the plant would thus have to remove about 95 per cent of the incoming BOD. This high degree of purification would necessitate secondary treatment by the activated sludge process, using a design loading of 35 pounds of BOD per 1,000 cubic feet of aeration tank capacity.

Secondary treatment would be provided at the Renton plant for an ultimate average dry weather flow of 143 mgd and a peak storm flow of 360 mgd. Influent pumping would be required to lift the sewage from both interceptors into the plant. Plant units would consist of preaeration and primary sedimentation tanks, aeration tanks, secondary sedimentation tanks, chlorine contact tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons having an area of 50 acres. When the lagoons are full, the dried sludge would be removed and sold as a soil conditioner or disposed of otherwise.

Effluent Disposal - Renton System. Chlorinated effluent would be discharged through twin 78-inch outfall lines to Duwamish River. Each outfall would be provided with diffusers to obtain effective dilution and dispersion in the river.

Intercepting Sewer - West Point System. The intercepting sewer for the West Point system under Core Plan B would begin north of the city of Tukwila in the Elliott Bay sewerage area and run northward, generally following the route of State Highway 5M and East Marginal Way to the Elliott Bay waterfront. From there, the route would be along Alaskan Way as far as Columbia Street, at which point, as under Core Plan A, a tunnel would be constructed to the intersection of Western and Elliott avenues. From the tunnel exit, the interceptor would be laid along Elliott Avenue to north of Pier 91 and then northward along the Great Northern railroad tracks to a junction with the North Trunk sewer.

Pumping would be required at four locations, one at the North Trunk junction and three others along the route of the interceptor. From the junction, sewage would then be conveyed in the North Trunk along Salmon Bay Waterway and through Fort Lawton to the terminus at Shilshole Bay. From this point, a new sewer would be constructed along the north shore of Fort Lawton to the treatment plant at West Point.

Sewage Treatment Plant - West Point System. Under Core Plan B, the treatment plant at this site would provide primary treatment for an average dry weather flow of 118 mgd, with a peak hydraulic capacity of 302 mgd. Influent pumping would be required to lift the sewage from the interceptor into the plant. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks and other

Table 15-4. Description and Estimated Construction Cost, Core Plan B

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
Renton System				
B-1	52	130	6,700 ft of 96-in. RC at 0.05%, average cut 23 - 24 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossings	1,123,000
B-2	89	237	1,300 ft of 108-in. RC at 0.095%, average cut 24 ft, difficult wet, includes sheeting and dewatering	271,000
Subtotal, sewers, Renton system				1,394,000
STP-B-1	143	360	Sewage treatment plant, secondary type, includes influent pumping and facilities for screening and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes purchase of 100 acres of land	16,780,000
B-3	143	360	3,100 ft of twin 78-in. RC outfall sewers, average cut 15 ft, difficult wet, includes sheeting, dewatering and railroad crossing	508,000
Total contract cost, Renton system				18,682,000
Engineering and contingencies, 25 per cent				4,670,000
Total construction cost, Renton system				23,352,000
West Point System				
B-4	1.2	2.8	1,500 ft of 18-in. RC at 0.17%, average cut 14 ft, difficult wet, includes imported backfill, repaving and dewatering	47,000
B-5	2.7	6.9	6,100 ft of 27-in. RC at 0.12%, average cut 22 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	373,000
B-6	4.1	10	900 ft of 33-in. RC at 0.09%, average cut 21 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	70,000
B-7	4.1	10	500 ft of twin 14-in. force mains across Duwamish River	22,000
B-8	4.1	10	1,100 ft of 27-in. RC at 0.25%, average cut 10 ft, difficult wet, includes imported backfill, repaving and dewatering	34,000
B-9	6.7	16	3,700 ft of 36-in. RC at 0.18%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	134,000
B-10	13-14	35-37	10,300 ft of 42-in. RC at 0.1% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	654,000
B-11	15	38-39	4,300 ft of 42-in. RC at 0.12% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	191,000
B-12	17	44	7,000 ft of existing 60-in. at 0.05 - 0.055%. Cost is for reconstruction of existing overflow and regulator	18,000
B-13	19	50	3,500 ft of 72-in. RC at 0.035%, average cut 12 ft, difficult wet, includes imported backfill, repaving and dewatering	287,000
B-14	30	80	1,400 ft of 84-in. RC at 0.04%, average cut 12 ft, difficult wet through congested industrial area, includes imported backfill, repaving, sheeting and dewatering	155,000
B-15	34-36	93-97	6,300 ft of 90-in. RC at 0.04%, average cut 14 ft, difficult wet through congested industrial area, includes imported backfill, repaving, sheeting and dewatering	695,000
B-16	37-38	101-102	4,300 ft of 96-in. RC at 0.032%, average cut 16 - 28 ft, difficult wet through congested industrial area and through debris fill containing wood and cinders at site of old Yesler Mill, includes imported backfill, repaving, sheeting, dewatering and underpinning of structures adjacent to 1st Avenue	679,000

Continued on next page

Table 15-4. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
B-17	38	102	10,100 ft of 102-in. RC tunnel at 0.03%, includes allowance of 20% for uncertainties	4,437,000
B-18	39-40	106	5,100 ft of 102-in. RC at 0.025%, average cut 23 - 24 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering, highway crossing and overflow structure on tributary sewer	989,000
B-19	42-43	113-117	7,900 ft of 108-in. RC at 0.021%, average cut 24 - 31 ft, difficult wet, includes 2,000 ft on piles, sheeting, and dewatering	1,495,000
B-20	113	287	1,600 ft of existing 138-in. at 0.035%. Cost is for overflow structure on tributary sewers	10,000
B-21	113-118	288-302	12,000 ft of existing 144-in. at 0.032 - 0.033%. Cost is for overflow structures on tributary sewers	79,000
B-22	118	302	2,600 ft of 144-in. RC at 0.033%, includes construction of 2,500 ft under tidal conditions and dewatering	760,000
Subtotal, sewers, West Point system				11,129,000
PS-B-1	4.1	10	Pumping station, single stage, motor driven, static lift 19 ft, total head at peak flow 30 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering	133,000
PS-B-2	15	38	Pumping station, single stage, motor driven, static lift 16 ft, total head at peak flow 20 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering. To replace existing city of Seattle pumping station having a total installed capacity of 5.0 mgd, includes connections to existing sewers and diversion of present raw sewage outfall to station	298,000
PS-B-3	19	50	Pumping station, single stage, motor driven, static lift 14 ft, total head at peak flow 20 ft, difficult wet, includes sheeting, dewatering and special foundations. Station at site of existing Diagonal Avenue sewage treatment plant, which is to be abandoned, includes connections to existing sewers and diversion of present raw sewage outfall to station	349,000
PS-B-4	43	117	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 32 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	590,000
Subtotal, pumping stations, West Point system				1,370,000
STP-B-2	118	302	Sewage treatment plant, primary type, includes influent pumping and facilities for screening and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes site preparation and 4,000 ft of 12-in. outfall sludge line to a water depth of 400 ft	9,219,000
B-23	118	302	1,900 ft of 120-in. RC effluent outfall, minimum depth, difficult wet, includes dewatering	288,000
B-24	118	302	3,700 ft of twin 78-in. RC submarine outfalls to a water depth of 150 ft, includes diffuser sections over last 440 ft	2,789,000
Subtotal, outfall, West Point system				3,077,000
Total contract cost, West Point system				24,795,000
Engineering and contingencies, 25 per cent				6,199,000
Total construction cost, West Point system				30,994,000
Total construction cost, Core Plan B				54,346,000

See page 357 for footnotes

Table 15-5. Construction Costs of Alternatives for Elliott Bay Site, Core Plan C

Facility	Construction cost,[a] dollars			
	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Sewers	16,478,000	16,478,000	18,954,000	18,954,000
Pumping stations	3,205,000	3,205,000	3,205,000	3,205,000
Sewage treatment plant	23,205,000	33,772,000	21,224,000	31,804,000
Outfall	8,047,000	409,000	7,065,000	920,000
Total	50,935,000	53,864,000	50,448,000	54,883,000

[a]*Includes engineering and contingencies.*

necessary structures and appurtenances. Chlorine contact tanks would not be required since detention times of about 15 minutes would be available in the outfall sewer at the ultimate peak flow of 302 mgd and of about 30 minutes at the ultimate average flow of 118 mgd. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 4,000 feet offshore to a water depth of 400 feet.

Effluent Disposal - West Point System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged through twin 78-inch outfalls approximately 3,700 feet offshore in water at a depth of about 150 feet.

Construction Cost. Estimated construction costs of Core Plan B, including engineering and contingencies, total $54,346,000 (Table 15-4). Of this total, approximately 29 per cent is for intercepting sewers, 3 per cent for pumping stations, 60 per cent for treatment plants and 8 per cent for outfalls.

Core Plan C

Under Core Plan C, sewage of the metropolitan Seattle area would be treated at two plants, one at the Elliott Bay site and the other at West Point. Four alternatives were considered for the Elliott Bay plant, as follows:

Alternative 1 - A primary type treatment plant situated between First and Fourth Avenues South and Brandon and Front Streets, with effluent disposal to Elliott Bay.

Alternative 2 - A complete type treatment plant at the same location as Alternative 1, with effluent disposal to Duwamish River.

Alternative 3 - A primary type treatment plant situated between Airport Way and Fifth Avenue South and Spokane and Dakota Streets, with effluent disposal to Elliott Bay.

Alternative 4 - A complete type treatment plant at the same location as Alternative 3, with effluent disposal to Duwamish River.

Estimated construction costs for the four alternatives (Table 15-5) show little choice between Alternatives 1 and 3. In view, however, of its somewhat lower cost, Alternative 3 was selected for comparison purposes.

Locations of sewers, pumping stations, treatment works and outfalls included under Core Plan C are shown in Fig. 15-5. Descriptions of these facilities are given in Table 15-6.

Intercepting Sewers - Elliott Bay System. South of Fourth Avenue South, the intercepting sewer would be identical to that for Core Plan A. At the intersection of Fourth Avenue South and East Marginal Way, the south interceptor for Core Plan C would turn north along Fourth Avenue South to the treatment plant where it would join the north interceptor. The north interceptor would begin at the intersection of Alaskan Way and Connecticut Street, and would be routed along Alaskan Way, East Marginal Way, Spokane Street and Fourth Avenue South to the treatment plant.

Sewage Treatment Plant - Elliott Bay System. Primary treatment would be provided at the Elliott Bay plant for an ultimate average dry weather flow of 181 mgd, with a peak hydraulic capacity of 457 mgd. Plan units would consist of influent pumps, preaeration and primary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since a detention time in excess of 20 minutes would be available in the outfall sewer, even at the peak flow of 457 mgd. Because of soil conditions at the site, foundation piles would be required. Digested sludge would be hauled away in tank trucks for disposal elsewhere.

Table 15-4 footnotes

See Fig. 15-4 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

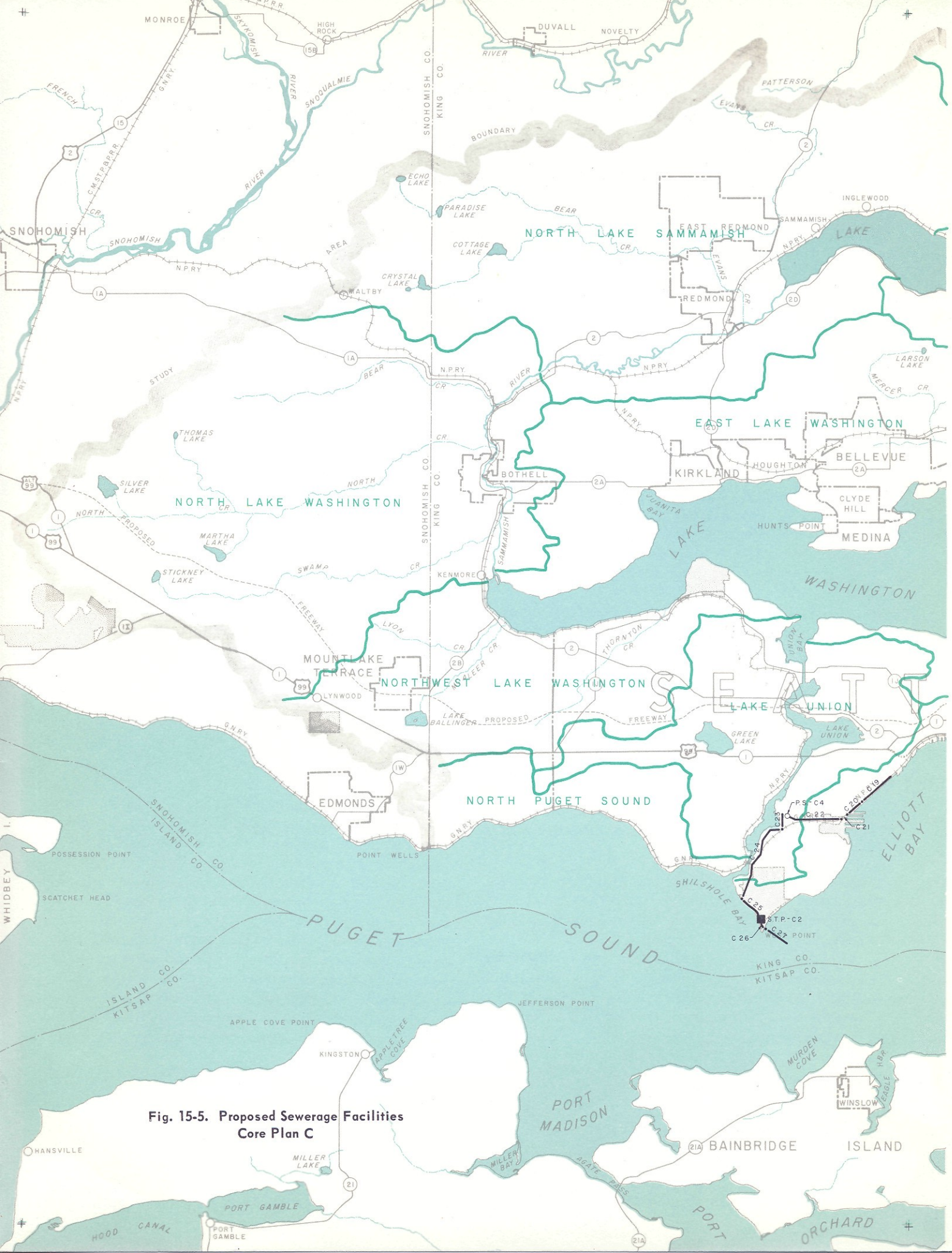
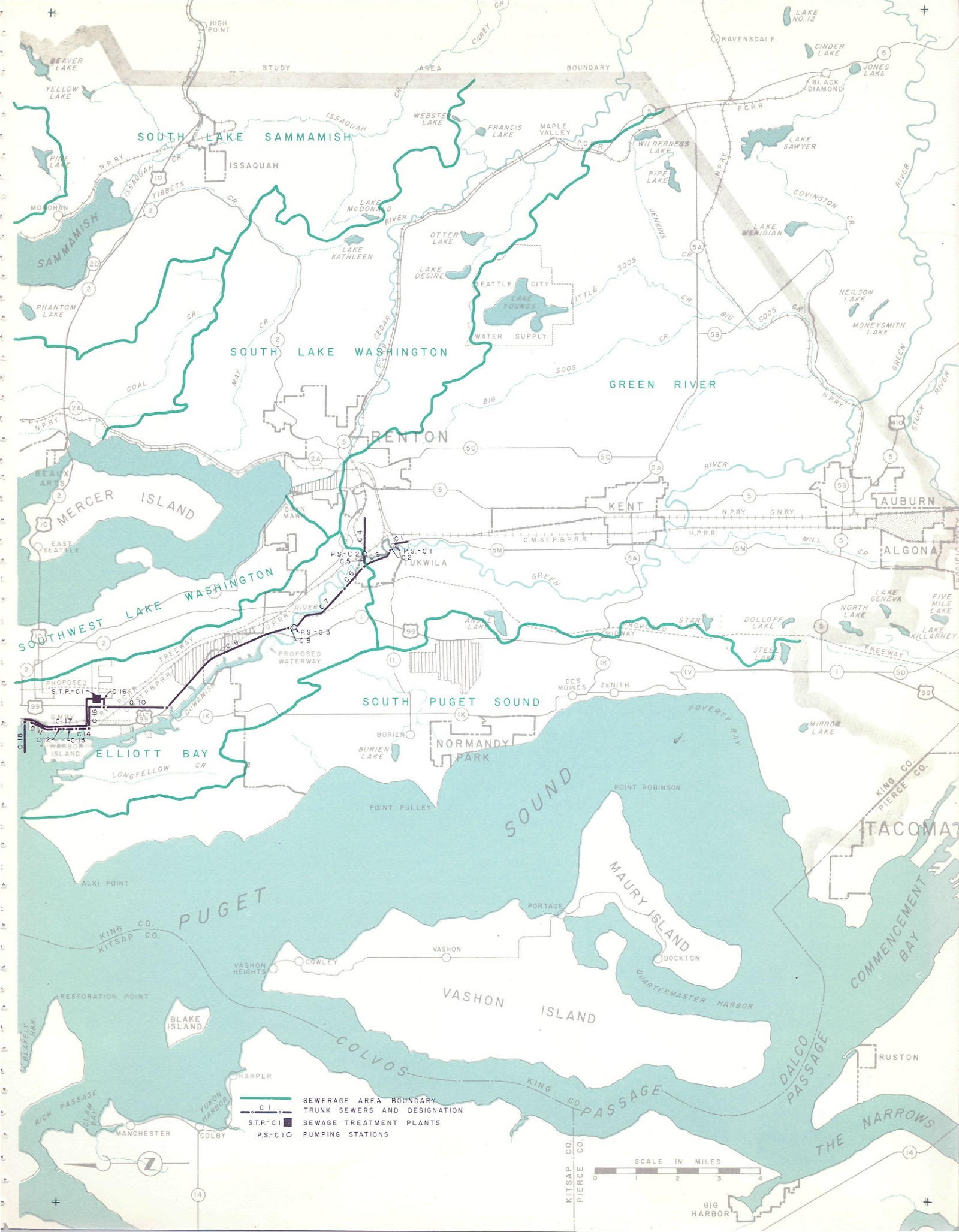


Fig. 15-5. Proposed Sewerage Facilities Core Plan C


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
PUGET SOUND
ELLIOTT BAY
MERCER ISLAND
VASHON ISLAND
MAURY ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
RENTON
KENT
AUBURN
ALGONA
TUKWILA
SEATTLE
TACOMA
ISSAQUAH
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
HIGH POINT
MOUNTAIN VIEW
SEAGOVE
BURIEN
NORMANDY PARK
DES MOINES
ZENITH
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DOCKTON
PORTAGE
POINT ROBINSON
POINT PULLEY
ALKI POINT
RESTORATION POINT
BLAKE ISLAND
HARPER
MANCHESTER
COLBY
RUSTON
GIG HARBOR
KING CO.
KITSAP CO.
PIERCE CO.
PROPOSED WATERWAY
PROPOSED
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P.S.-C2
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SEWERAGE AREA BOUNDARY
TRUNK SEWERS AND DESIGNATION
SEWAGE TREATMENT PLANTS
PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 15-6. Description and Estimated Construction Costs, Core Plan C

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Elliott Bay System				
C-1	52	130	2,300 ft of 96-in. RC at 0.05%, average cut 24 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	395,000
C-2	52	130	600 ft of twin 42-in. force mains across Duwamish River	70,000
C-3	52	130	3,300 ft of 96-in. RC at 0.05%, average cut 15 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	401,000
C-4	90	238	5,400 ft of 114-in. RC at 0.07%, average cut 27 - 31 ft, difficult wet, includes sheeting, dewatering and railroad crossing	1,333,000
C-5	90	238	600 ft of parallel 36-in., 42-in. and 48-in. force mains across Duwamish River	110,000
C-6	143	368	2,400 ft of 114-in. RC at 0.16%, average cut 20 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	446,000
C-7	143-146	370-376	10,900 ft of 120-in. RC at 0.13%, average cut 21 - 32 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	2,948,000
C-8	146	376	500 ft of parallel 48-in. and twin 54-in. force mains across Duwamish River	113,000
C-9	146-160	376-411	18,600 ft of 132-in. RC at 0.075 - 0.086%, average cut 16 - 25 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	5,000,000
C-10	160	411	7,900 ft of 144-in. RC at 0.058%, average cut 27 - 31 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	2,988,000
C-11	2.3	5.9	3,900 ft of 24-in. RC at 0.17%, average cut 15 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	170,000
C-12	2.8	7.2	900 ft of 27-in. RC at 0.14%, average cut 19 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	52,000
C-13	4.4	11	1,400 ft of 33-in. RC at 0.10%, average cut 20 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	91,000
C-14	8.7	24	1,800 ft of 48-in. RC at 0.068%, average cut 21 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	160,000
C-15	19	53	4,700 ft of 54-in. RC at 0.19%, average cut 25 - 31 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	586,000
C-16	178	457	900 ft of 144-in. RC at 0.068%, average cut 34 ft, difficult wet, includes sheeting and dewatering	300,000
Subtotal, sewers, Elliott Bay system				15,163,000
PS-C-1	52	130	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 28 ft, structure about 30 ft below ground, difficult wet, includes sheeting and dewatering	588,000
PS-C-2	90	238	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 34 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	863,000
PS-C-3	146	376	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 37 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	1,113,000

Continued on next page

Table 15-6. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
Subtotal, pumping stations, Elliott Bay system				2,564,000
STP-C-1	181	457	Sewage treatment plant, primary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes purchase of 45 acres of land and special foundations	16,979,000
C-17	181	457	11,000 ft of 120-in. RC effluent outfall, minimum depth, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	1,993,000
C-18	181	457	3,300 ft of twin 78-in. RC submarine outfall to a water depth of 200 ft, includes diffuser sections over last 350 ft	3,659,000
Subtotal, outfall, Elliott Bay system				5,652,000
Total contract cost, Elliott Bay system				40,358,000
Engineering and contingencies, 25 per cent				10,090,000
Total construction cost, Elliott Bay system				50,448,000
West Point System				
C-19	4.3	11	5,000 ft of 33-in. RC at 0.11%, average cut 10 - 13 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and overflow structures on tributary sewer	268,000
C-20	4.5	12	2,400 ft of 36-in. RC at 0.077%, average cut 16 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering, and highway crossing	152,000
C-21	6.8	19	1,000 ft of 42-in. RC at 0.085%, average cut 18 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	56,000
C-22	7.2-8.0	20-22	6,800 ft of 48-in. RC at 0.047 - 0.055%, average cut 22 - 27 ft, difficult wet, includes 2,000 ft on piles, sheeting and dewatering	538,000
C-23	78	197	1,600 ft of existing 138-in. at 0.035%. Cost is for overflow structure on tributary sewer	10,000
C-24	78-85	197-212	12,000 ft of existing 144-in. at 0.032 - 0.033%. Cost is for overflow structures on tributary sewers	79,000
C-25	85	212	3,000 ft of 120-in. RC at 0.045%, includes construction of 2,500 ft under tidal conditions, and dewatering	670,000
Subtotal, sewers, West Point system				1,773,000
PS-C-4	8.0	22	Pumping station, single stage, motor driven, static lift 23 ft, total head at peak flow 30 ft, structure about 32 ft below ground, difficult wet, includes sheeting and dewatering	208,000
STP-C-2	85	212	Sewage treatment plant, primary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration, and laboratory facilities, includes site preparation and 4,000 ft of 10-in. outfall sludge line to a water depth of 400 ft	6,958,000
C-26	85	212	1,900 ft of 108-in. RC effluent outfall, minimum depth, difficult wet, includes dewatering	247,000
C-27	85	212	3,500 ft of twin 72-in. RC submarine outfalls to a water depth of 120 ft, includes diffuser sections over last 210 ft	2,392,000

Continued on next page

Table 15-6. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Subtotal, outfall, West Point system				2,639,000
Total contract cost, West Point system				11,578,000
Engineering and contingencies, 25 per cent				2,894,000
Total construction cost, West Point system				14,472,000
Total construction cost, Core Plan C				64,920,000

See Fig. 15-5 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

Effluent Disposal - Elliott Bay System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged to Elliott Bay north of Harbor Island approximately 3,300 feet offshore in water at a depth of about 200 feet. The outfall would consist of a land section 11,000 feet in length, extending from the treatment plant along Spokane Street, East Marginal Way and Alaskan Way to Pier 39, and a twin 78-inch submarine section extending 1,400 feet beyond the pier line.

Intercepting Sewers - West Point System. The intercepting sewer for the West Point system would begin at the intersection of Denny Way and Elliott Avenue. From that point, it would be routed along Elliott Avenue to north of Pier 91 and then northward along the Great Northern railroad tracks to a junction with the existing North Trunk sewer of Seattle. Sewage from the new interceptor would be pumped into the North Trunk, and would be conveyed therein, along with sewage from the North Lake Washington, Northwest Lake Washington and Lake Union sewerage areas, to the terminus at Shilshole Bay. From there, a new sewer would be constructed along the north shore of Fort Lawton to the treatment plant at West Point.

Sewage Treatment Plant - West Point System. Except for capacity, the sewage treatment plant at West Point would be identical to that proposed for Core Plan B. Under Core Plan C, the treatment plant would provide primary treatment for an average dry weather flow of 85 mgd, with a peak hydraulic capacity of 212 mgd.

Effluent Disposal - West Point System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged through twin 72-inch outfalls approximately 3,500 feet offshore in water at a depth of about 120 feet.

Construction Cost. As given in Table 15-6, the estimated total construction cost of Core Plan C, including engineering and contingencies, is $64,920,000. Of this total, approximately 33 per cent is for intercepting sewers, 5 per cent for pumping stations, 46 per cent for treatment plants, and 16 per cent for outfalls.

Core Plan D

Of the three treatment plants called for under Core Plan D, one would be at the Renton site, the second at the Elliott Bay site, and the third at the West Point site. Three alternatives were considered for the Elliott Bay site, as follows:

Alternative 1 - A primary type treatment plant situated between First and Fourth Avenues South and Brandon and Front Streets, with effluent disposal to Elliott Bay.

Alternative 2 - A primary type treatment plant situated between Airport Way and Fifth Avenue South and Spokane and Dakota streets, with effluent disposal to Elliott Bay.

Alternative 3 - A primary type treatment plant situated at the site of the existing Diagonal Avenue treatment plant of the city of Seattle, with effluent disposal to Elliott Bay.

Table 15-7. Construction Costs of Alternatives for Elliott Bay Site, Core Plan D

Facility	Construction cost[a], dollars		
	Alternative 1	Alternative 2	Alternative 3
Sewers	4,638,000	4,919,000	3,321,000
Pumping station	166,000	166,000	539,000
Sewage treatment plant	8,706,000	8,081,000	7,259,000
Outfall	2,603,000	1,995,000	2,086,000
Total	16,113,000	15,161,000	13,205,000

[a]*Includes engineering and contingencies.*

Table 15-8. Description and Estimated Construction Costs, Core Plan D

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
Renton System				
D-1 – D-2	–	–	Identical to B-1 – B-2, Core Plan B, see Table 15-4	1,394,000
STP-D-1	143	360	Identical to STP-B-1, Core Plan B, see Table 15-4	16,780,000
D-3	143	360	Identical to B-3, Core Plan B, see Table 15-4	508,000
Total contract cost, Renton system				18,682,000
Engineering and contingencies, 25 per cent				4,670,000
Total construction cost, Renton system				23,352,000
Elliott Bay System				
D-4	1.2	2.8	1,500 ft of 18-in. RC at 0.17%, average cut 14 ft, difficult wet, includes imported backfill, repaving and dewatering	47,000
D-5	2.7	6.9	6,100 ft of 27-in. RC at 0.12%, average cut 22 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	373,000
D-6	4.1	10	900 ft of 33-in. RC at 0.09%, average cut 21 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	70,000
D-7	4.1	10	500 ft of twin 14-in. force mains across Duwamish River	22,000
D-8	4.1	10	1,100 ft of 27-in. RC at 0.25%, average cut 10 ft, difficult wet, includes imported backfill, repaving and dewatering	34,000
D-9	6.7	16	3,700 ft of 36-in. RC at 0.18%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	134,000
D-10	13-14	35-37	10,300 ft of 42-in. RC at 0.1% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	654,000
D-11	15	38-39	4,300 ft of 42-in. RC at 0.12% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	191,000
D-12	17	44	7,000 ft of existing 60-in. RC at 0.05 - 0.055%. Cost is for reconstruction of existing overflow and regulator	18,000
D-13	2.3	5.9	3,900 ft of 24-in. RC at 0.17%, average cut 15 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	170,000
D-14	2.8	7.2	900 ft of 27-in. RC at 0.14%, average cut 19 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	52,000
D-15	4.4	11	1,400 ft of 33-in. RC at 0.10%, average cut 20 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	91,000
D-16	8.7	24	1,800 ft of 48-in. RC at 0.068%, average cut 21 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	160,000
D-17	19	53	3,900 ft of 72-in. RC at 0.04%, average cut 28 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	641,000
Subtotal, sewers, Elliott Bay system				2,657,000
PS-D-1	4.1	10	Pumping station, single stage, motor driven, static lift 19 ft, total head at peak flow 30 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering	133,000
PS-D-2	15	38	Pumping station, single stage, motor driven, static lift 16 ft, total head at peak flow 20 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering. To replace existing city of Seattle pumping station having a total installed capacity of 5.0 mgd, includes connection to existing sewers and diversion of present raw sewage outfall to station	298,000

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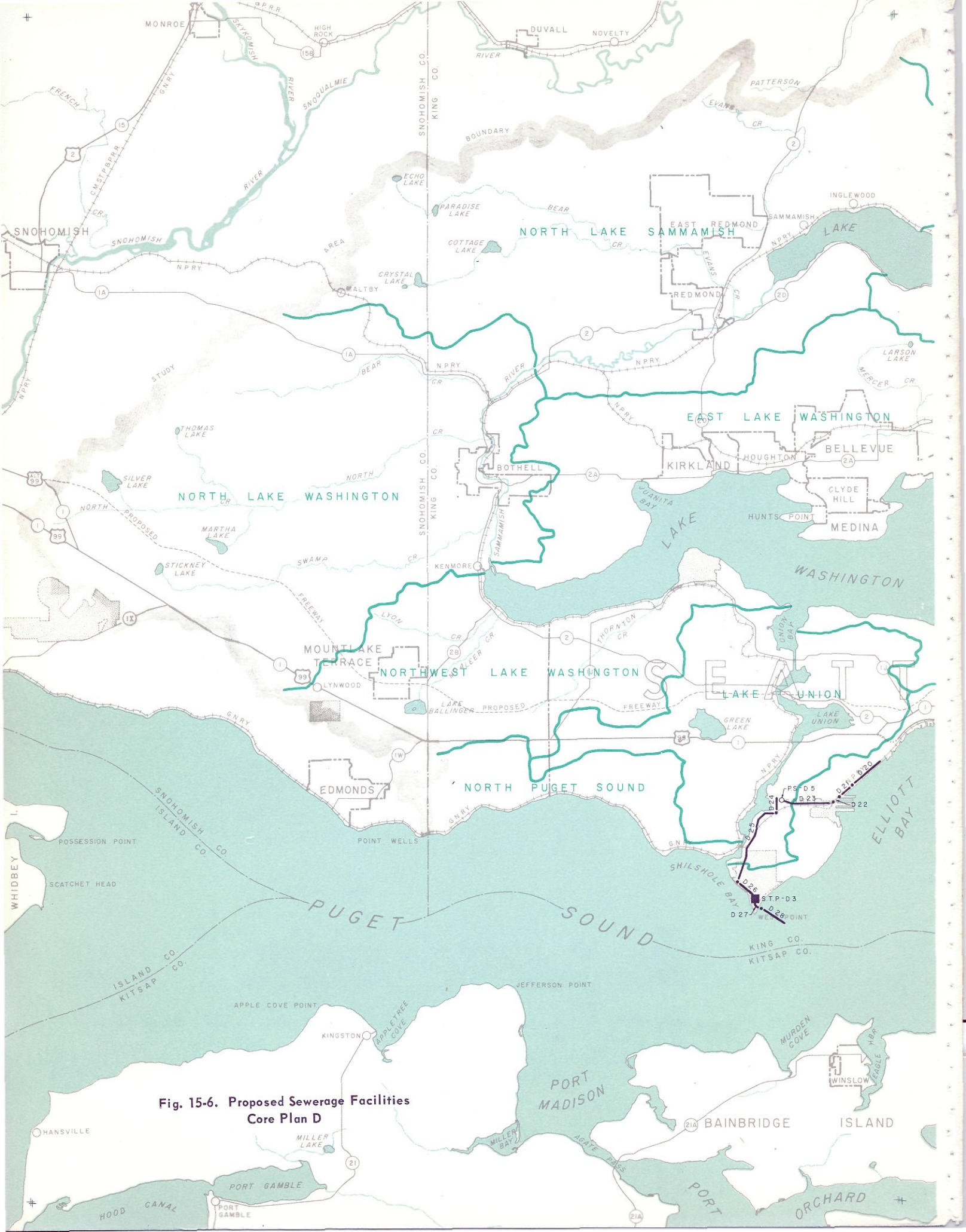
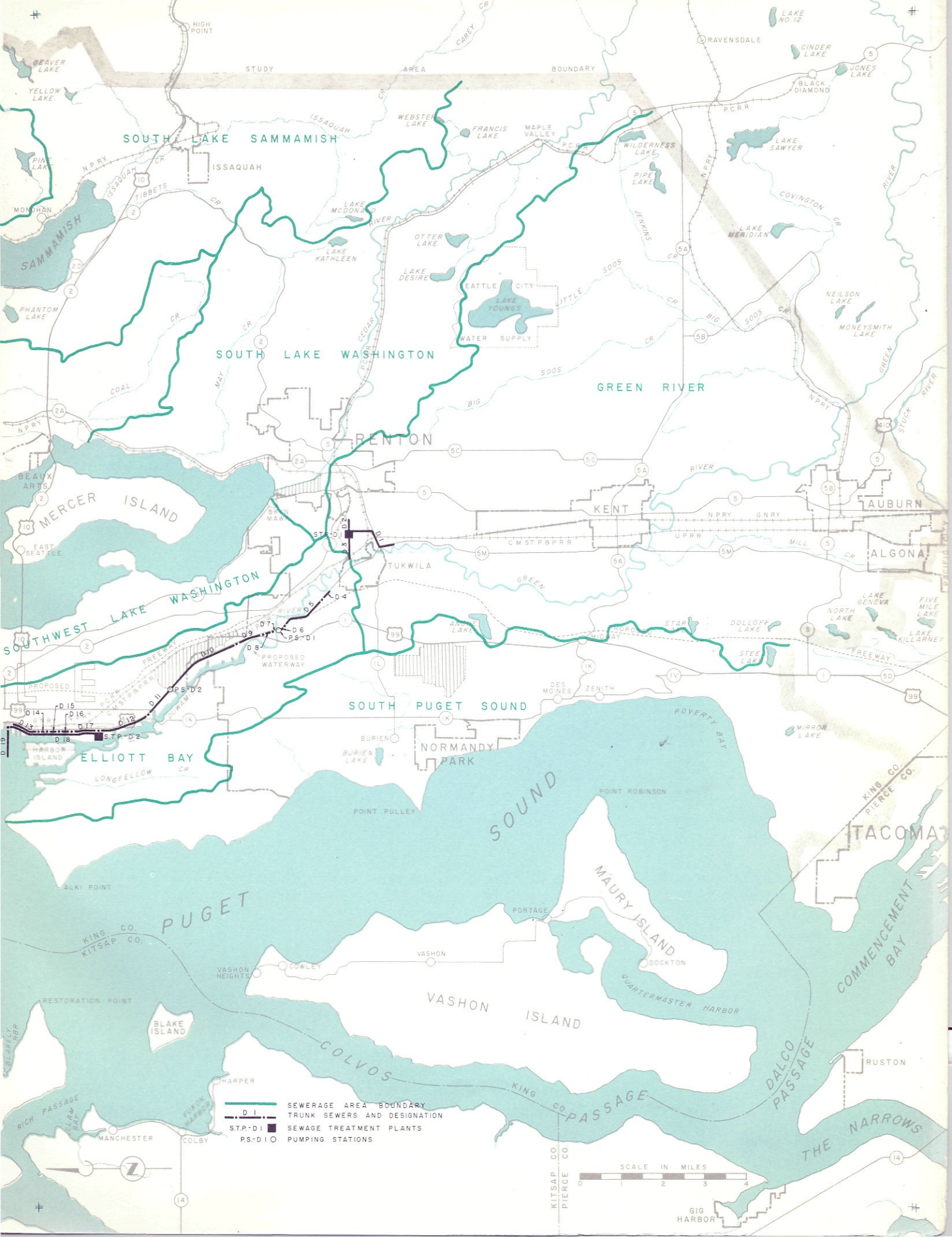


Fig. 15-6. Proposed Sewerage Facilities
Core Plan D


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
VASHON ISLAND
MAURY ISLAND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
RENTON
KENT
AUBURN
ALGONA
TUKWILA
MERCER ISLAND
TACOMA
RUSTON
GIG HARBOR
NORMANDY PARK
SAMMAMISH
ISSAQUAH
RAVENSDALE
BLACK DIAMOND
MAPLE VALLEY
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
PROPOSED WATERWAY
S.T.P.-D1
S.T.P.-D2
P.S.-D1
P.S.-D2
D1
D2
D4
D5
D6
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D15
D16
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D18
D19
SEWERAGE AREA BOUNDARY
TRUNK SEWERS AND DESIGNATION
SEWAGE TREATMENT PLANTS
PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4
KING CO.
KITSAP CO.
PIERCE CO.

Table 15-8. Continued from Page 363

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
Subtotal, pumping stations, Elliott Bay system				431,000
STP-D-2	38	102	Sewage treatment plant, primary type, includes influent and effluent pumping and facilities for screening and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities. Plant to be located at site of existing Diagonal Avenue treatment plant, includes purchase of additional land to make total of 25 acres, site preparation, and special foundations	5,807,000
D-18	38	102	11,900 ft of 72-in. RC effluent outfall, minimum depth, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	966,000
D-19	38	102	2,200 ft of 60-in. RC submarine outfall to a water depth of 175 ft, includes diffuser section over last 225 ft	703,000
Subtotal, outfall, Elliott Bay system				1,669,000
Total contract cost, Elliott Bay system				10,564,000
Engineering and contingencies, 25 per cent				2,641,000
Total construction cost, Elliott Bay system				13,205,000
West Point System				
D-20 – D-26	–	–	Identical to C-19 – C-25, Core Plan C, see Table 15-6	1,773,000
PS-D-3	8.0	22	Identical to PS-C-4, Core Plan C, see Table 15-6	208,000
STP-D-3	85	212	Identical to STP-C-2, Core Plan C, see Table 15-6	6,958,000
D-27 – D-28	85	212	Identical to C-26 – C-27, Core Plan C, see Table 15-6	2,639,000
Total contract cost, West Point system				11,578,000
Engineering and contingencies, 25 per cent				2,894,000
Total construction cost, West Point system				14,472,000
Total construction cost, Core Plan D				51,029,000

See Fig. 16-6 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

Estimated construction costs for the three alternatives are presented in Table 15-7. As therein shown, the estimated cost of $13,205,000 for Alternative 3 is $1,956,000 less than the cost for Alternative 2, the next lowest. Alternative 3, therefore, is used in all subsequent discussions relating to Core Plan D.

Locations of sewers, pumping stations, treatment works and outfalls included under Core Plan D are designated in Fig. 15-6. Descriptions of these facilities are given in Table 15-8.

Renton System. All facilities proposed for the Renton system under Core Plan D would be identical to those proposed under Core Plan B for that system.

Intercepting Sewers - Elliott Bay System. Intercepting sewers for the Elliott Bay system would consist of a south branch and a north branch, of which the south branch would be identical to that called for under Core Plan B. The north branch would begin at the intersection of Alaskan Way and Connecticut Street and would be routed south along Alaskan Way and East Marginal Way to the treatment plant.

Sewage Treatment Plant - Elliott Bay System. Except for capacity and location, the sewage treatment plant at Elliott Bay would be identical to that proposed under Core Plan C. Under Core Plan D, the treatment plant would be situated at the site of the existing Diagonal Avenue plant, and would provide primary treatment

for an average dry weather flow of 38 mgd, with a peak hydraulic capacity of 102 mgd.

Effluent Disposal - Elliott Bay System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged to Elliott Bay north of Harbor Island approximately 2,200 feet offshore in water at a depth of about 175 feet. The outfall would consist of a land section and a submarine section, of which the former would be 11,900 feet in length and would extend from the treatment plant along East Marginal Way and Alaskan Way to Pier 39. The latter would extend 1,200 feet beyond the pier line.

West Point System. All facilities proposed for the West Point system would be identical to those proposed for Core Plan C.

Construction Cost. Total construction costs of Core Plan D, including engineering and contingencies, are estimated to be $51,029,000 (Table 15-8). Of this total, approximately 14 per cent is for intercepting sewers, 2 per cent for pumping stations, 72 per cent for treatment plants, and 12 per cent for outfalls.

COMPARISON OF CORE PLANS

In general, the principal factor to be taken into account when comparing two or more projects to perform a given function is that of cost, both construction and total annual. In some cases, however, particularly where differences in annual costs are not significant, factors other than cost must be considered and evaluated.

Construction Costs

Estimated construction costs for each of the four core plans are summarized in Table 15-9. These estimates, which are based on preliminary layouts of the required facilities, range from $51,029,000 for Core Plan D to $73,932,000 for Core Plan A.

Annual Costs

Assuming that the capital cost can be financed and that all other requirements are fulfilled, the economic merit of any given project is established by determining its total annual cost. This cost is predicated on the anticipated useful life of the required facilities and includes, as set forth in Chapter 13, depreciation, interest on invested capital, and the costs of administration, operation and maintenance.

Depreciation and interest charges used herein are based on the estimated construction costs, as are maintenance and operation costs for sewers and outfalls. In the case of pumping stations and treatment plants, maintenance and operation costs, including those of chlorination, are determined on the basis of estimated average flows during the 70-year design period, 1960 to 2030.

Estimated average annual costs for each of the four core plans (Table 15-10) range from a total of $4,203,000 for Core Plan D to a total of $5,708,000 for Core Plan A. Fixed charges for interest and depreciation vary, of course, in direct proportion to invested capital and range from $3,183,000 per year for Core Plan D to $4,371,000 per year for Core Plan A. From the standpoint of operating costs, Core Plan B is the lowest at $905,000 per year (Table 15-10).

Table 15-9. Comparison of Construction Costs of Core Plans

Facility	Construction cost[a], dollars			
	Core Plan A	Core Plan B	Core Plan C	Core Plan D
Sewers	37,091,000	15,654,000	21,170,000	7,280,000
Pumping stations	6,262,000	1,712,000	3,465,000	798,000
Sewage treatment plants	24,938,000	32,499,000	29,921,000	36,931,000
Outfalls	5,641,000	4,481,000	10,364,000	6,020,000
Total	73,932,000	54,346,000	64,920,000	51,029,000

Plan A proposes one sewage treatment plant of the primary type located at the Government Locks site with discharge of effluent to Puget Sound.

Plan B proposes two sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan C proposes two sewage treatment plants: one of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan D proposes three sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; the second of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the third located at the West Point site with discharge of effluent to Puget Sound.

[a]*From Tables 15-3, 15-4, 15-6, and 15-8; includes allowances for engineering and contingencies.*

Table 15-10. Comparison of Annual Costs of Core Plans

Cost Item	Annual cost, dollars			
	Core Plan A	Core Plan B	Core Plan C	Core Plan D
Fixed costs[a]				
Sewers and outfalls	2,341,000	1,103,000	1,727,000	729,000
Pumping stations and sewage treatment plants	2,030,000	2,225,000	2,172,000	2,454,000
	4,371,000	3,328,000	3,899,000	3,183,000
Maintenance and operation				
Sewers and outfalls[b]	107,000	50,000	79,000	35,000
Pumping stations[c]	314,000	66,000	150,000	14,000
Sewage treatment plants[c]	769,000[d]	634,000[e]	899,000[f]	811,000[f]
Effluent chlorination[c]	147,000[g]	155,000[g]	147,000[g]	160,000[g]
	1,337,000	905,000	1,275,000	1,020,000
Average annual cost[h]	5,708,000	4,233,000	5,174,000	4,203,000

Plan A proposes one sewage treatment plant of the primary type located at the Government Locks site with discharge of effluent to Puget Sound.

Plan B proposes two sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan C proposes two sewage treatment plants: one of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan D proposes three sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; the second of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the third located at the West Point site with discharge of effluent to Puget Sound.

[a]*Includes interest and depreciation calculated by capital recovery method based on five per cent interest and depreciation life of 50 years for sewers and outfalls and of 30 years for pumping stations and sewage treatment plants.*

[b]*0.25 per cent of construction cost.*

[c]*Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[d]*Includes allowance of $10.00 per dry ton for hauling of digested sludge for disposal.*

[e]*Costs reduced for sludge disposal; $2.25 per dry ton at Renton plant (lagooning) and $3.00 per dry ton at West Point plant (disposal to sound).*

[f]*Allowances for sludge disposal as follows: reduction of $2.25 per dry ton at Renton plant (lagooning); increase of $10.00 per dry ton at Elliott Bay plant (hauling); and reduction of $3.00 per dry ton at West Point plant (disposal to sound).*

[g]*Effluent chlorination at plants discharging to Puget Sound or Elliott Bay during period May - September only.*

[h]*During design period, 1960 - 2030.*

Factors Other Than Cost

Assuming the validity of the cost estimates, it appears that the most acceptable plan from an economic standpoint is Core Plan D, under which provision is made for sewage treatment and disposal at three sites, namely, Renton, Elliott Bay and West Point. It will be seen, however, that the annual cost of this plan is but slightly less than that of Core Plan B, which provides for sewage treatment and disposal at two sites, Renton and West Point.

The difference in total annual cost, amounting to $30,000 per year in favor of Core Plan D, is less than one per cent of the average annual cost of that project and undoubtedly is within the accuracy of the cost estimates. In making comparisons, therefore, it is necessary to consider other pertinent factors and to make decisions accordingly. At Seattle, these factors include such items as duplication of operation, interference with business activity during construction, possible future upgrading of disposal requirements, ability to expand facilities in the event that the estimated growth of the tributary area is exceeded, and the simplicity and flexibility of the treatment process. In addition, consideration should be given to possible esthetic objections.

Since both Core Plan B and Core Plan D provide for a sewage treatment plant at the Renton site to serve five of the ten sewerage areas, comparison of the two plans rests on whether one plant (West Point) or two plants (West Point and Elliott Bay) should be provided to serve the remaining five.

Duplication of Operation. In common with many industrial operations, experience with sewage treatment

operations has demonstrated that, for a given capacity, more satisfactory over-all performance can be obtained in a single plant rather than in two or more plants. This advantage, though not always evidenced by cost, leads to better quality control, which in sewage treatment plants means a consistently superior effluent. It also means a lesser possibility of plant upsets requiring the bypassing of raw sewage.

Another factor to be considered is the effect of increases in plant maintenance and operation costs. During the past decade, these costs have increased in the order of 25 per cent, primarily due to rising costs of labor and materials. A similar change in the future would increase the indicated maintenance and operation differential from $115,000 per year (Table 15-10) to $143,000 per year in favor of Core Plan B. This change is enough to make the total annual cost of Core Plan D about equal to that of Core Plan B. Further increases would serve to magnify the advantage of Core Plan B.

Interference with Business Activity. The magnitude and complexity of the construction work in the downtown area of Seattle would be less under Core Plan D than Core Plan B. For that reason, interference with existing utilities, such as water mains and underground telephone and electric cables, would be less of a problem. In addition, street excavations would be smaller, construction time would be shorter, and business activity would be less affected.

Quality of Effluent. A possible future upgrading of water quality requirements, though presently unforeseeable, may possibly necessitate a higher degree of treatment at both disposal sites. In such an event, the estimated additional construction cost of providing secondary treatment facilities would be $6,425,000 under Core Plan B. For Core Plan D, the added cost would be $3,045,000 at the Elliott Bay site and $4,925,000 at the West Point site, or a total of $7,970,000. The difference in additional construction costs ($1,545,000) would result in a fixed annual charge of $100,000 more for Core Plan D than for Core Plan B. Similarly, operating costs would increase by $100,000 per year for Core Plan B and $129,000 per year for Core Plan D. If secondary treatment became necessary, the difference in total annual cost would amount to $129,000, thus making Core Plan B superior to Core Plan D from an economic standpoint.

Expansion of Treatment Facilities. Since predictions of future population and industrial development, particularly with respect to their distribution, cannot be regarded as precise, the possibility always exists that presently planned facilities will have to be expanded at some future date to allow for growth in excess of that now estimated.

Future acquisition of additional land at the Elliott Bay site, which is in a densely built-up industrial area, would be both difficult and costly. On the other hand, additional land required at West Point could be obtained by the less costly expedient of filling low-lying tidelands to the north of that site.

Simplicity of Treatment Processes. The simplicity and ease of operation of any sewage treatment process is related directly to the degree of treatment provided. The plants proposed at both the Elliott Bay and West Point sites under Core Plan D, as well as the West Point plant under Core Plan B, would each provide primary treatment. As such, there is no basic difference in the treatment processes. There is, however, a difference in sludge handling requirements between the Elliott Bay and West Point sites.

At the West Point site, digested sludge could be discharged to Puget Sound without producing deleterious effects. This arrangement, in addition to being the least subject to operating complications, would avoid the necessity and cost of truck hauling and would require the least effort on the part of plant attendants.

At the Elliott Bay site, it would not be possible to discharge digested sludge to the bay. This is because current conditions indicate that the sludge would not be adequately dispersed and that, as a consequence, sludge banks would be formed. Accordingly, wet sludge would have to be hauled to remotely located and more suitable disposal points.

Esthetic Considerations. Although conditions at both sites are satisfactory from the standpoint of avoiding nuisance, the isolation provided by the West Point site makes it eminently suitable as a sewage treatment plant location. In contrast, the Elliott Bay site is situated near the center of industrial and commercial activity and is adjacent to a major highway. A treatment plant located at such a site, coupled with the necessity of effluent discharge to Elliott Bay, would possibly meet with serious objection from an esthetic point of view.

SELECTION OF MOST ACCEPTABLE CORE PLAN

As evidenced by the foregoing discussion, it is all too clear that the choice of the most acceptable core plan for central sewerage of the metropolitan area must be more or less arbitrary. With all factors taken into account and evaluated objectively, we are convinced that the most satisfactory plan for central sewerage of the metropolitan Seattle area is that designated herein as Core Plan B. Under this plan, the sewage of the metropolitan area would be conveyed to two treatment plants, one at Renton and the other at

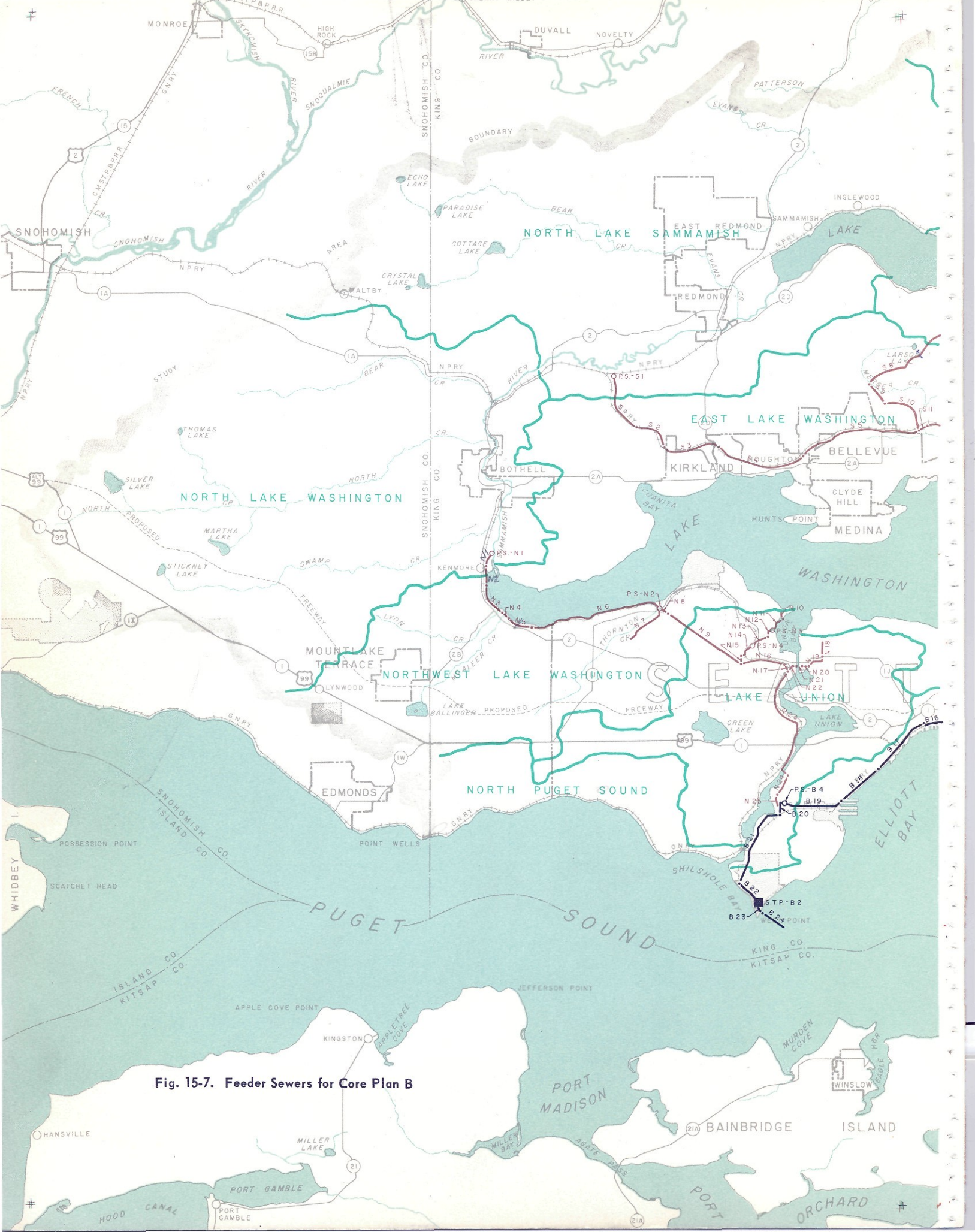
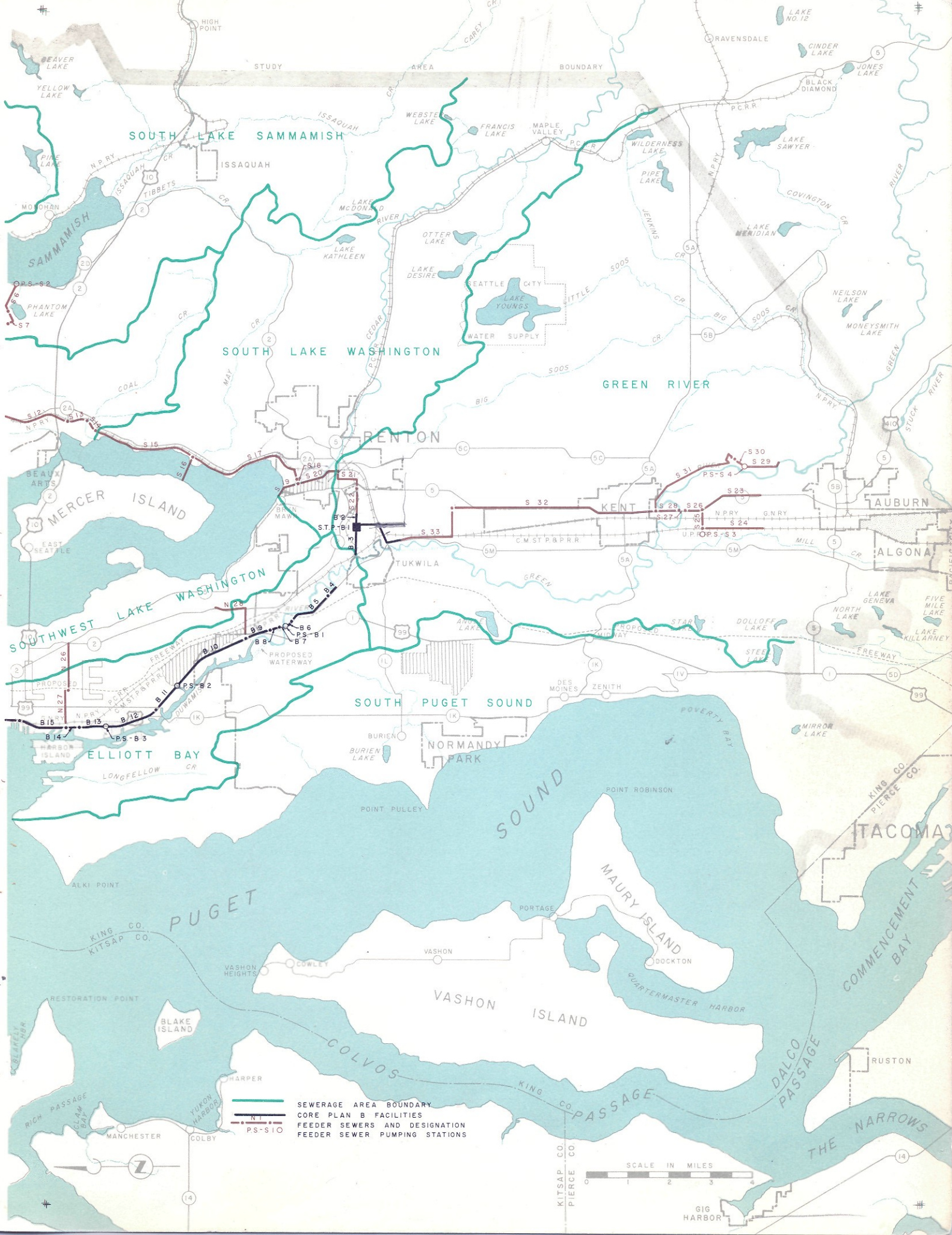


Fig. 15-7. Feeder Sewers for Core Plan B


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
MERCER ISLAND
VASHON ISLAND
MAURY ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
RENTON
KENT
AUBURN
ALGONA
TACOMA
ISSAQUAH
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
SEATTLE CITY WATER SUPPLY
TUKWILA
NORMANDY PARK
PROPOSED WATERWAY
SEWERAGE AREA BOUNDARY
CORE PLAN B FACILITIES
FEEDER SEWERS AND DESIGNATION
FEEDER SEWER PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

West Point. At the Renton site, complete treatment would be provided and effluent would be disposed of in Duwamish River. At the West Point site, primary treatment would be provided and effluent would be disposed of in Puget Sound.

FEEDER SEWERS FOR CORE PLAN B

Feeder sewers, as defined for purposes of this report, include major trunk sewers and appurtenant pumping stations required to bring sewage from outlying sewerage areas into the core plan system. Since two treatment plants, and hence two points of concentration, are proposed under this plan, feeder sewers are laid out accordingly.

Locations of trunk sewers and pumping stations included in the feeder system for Core Plan B are shown in Fig. 15-7. Descriptions of these facilities appear in Table 15-11.

Renton System

Feeder sewers for the Renton system consist of a south branch and a north branch. The south branch would extend southward through the city of Kent, beyond which it would split into three branches, one to serve the western part of Green River valley and higher areas to the west, the second to serve the central portion of the valley, including the city of Auburn, and the third to serve the eastern portion of the Green River sewerage area. To avoid excessive cuts, two pumping stations would be required on the south branch.

The north branch, or East Side sewer, would extend eastward through the city of Renton and then northward along the Northern Pacific railroad right-of-way through the East Lake Washington sewerage area to the North Lake Sammamish sewerage area. A pumping station would be necessary to pick up the sewage from the latter area.

Sewage collected within the East Lake Washington and South Lake Washington sewerage areas would enter the north branch at various points along its route. In addition, this branch would include a side branch extending generally eastward past Phantom Lake to the South Lake Sammamish sewerage area. A pumping station would be required to serve this area.

Along the railroad right-of-way, the north branch would pass through sections of restricted clearance where the railroad has been constructed in cuts or on fills. In such sections, the sewer would have to be constructed in tunnel or on fill. Additionally, trench support for a Cooper E-60 train load would probably be required where the trench could not be dug farther away from the tracks than about one and one-half times its depth. Because of the possibility of better soil conditions, realignment of a portion of sewer S-20 (Fig. 15-7) to the east of Duwamish River should be investigated during final design.

West Point System

Feeder sewers for the West Point system consist principally of a single main branch which utilizes the existing North Trunk of Seattle to Ravenna Way and East 54th Street. A tunnel would be constructed from this point to a pumping station situated at the mouth of Thornton Creek. Between the pumping station and Sheridan Beach, the route would be offshore in Lake Washington at water depths of 15 to 20 feet. In this section, the sewer would be laid in a trench excavated in the bottom of the lake. The trench would vary from 10 to 19 feet in depth and would be backfilled with rock. From Sheridan Beach, the route would be along the lake front to the North Lake Washington sewerage area, where a pumping station would be necessary to pick up the sewage from that area.

At various points along its route, the main West Point feeder sewer would intercept sewage flows from the Northwest Lake Washington, Lake Union and Southwest Lake Washington sewerage areas. In addition to the main branch, feeder sewers to serve additional areas in the Southwest Lake Washington sewerage area would be provided.

POSSIBLE MODIFICATIONS OF CORE PLAN B

Three basic modifications of Core Plan B were investigated. Of these, two involved possible economies in first cost, while the third involved a determination of the additional cost of providing complete rather than primary treatment should water reclamation be desirable. These modifications are:

1. Conveyance of the sewage, including that from the North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, from the east to the west side of Lake Washington across the lake rather than to the south as proposed under Core Plan B. Three alternatives were considered for this modification, each necessitating a change in the size of the treatment plants proposed at the various sites.
2. Construction of a primary type treatment plant at the Renton site, with an effluent outfall to Puget Sound.
3. Construction of complete type treatment plant at the Government Locks site, with effluent disposal to the Lake Washington Ship Canal above the locks. This would be in lieu of a primary treatment plant at West Point.

Modification of East Side Sewer

By transporting sewage from the east to the west side of Lake Washington, the North Lake Sammamish,

Table 15-11. Description and Estimated Construction Costs, Feeder Sewers for Core Plan B

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Renton System				
S-1	27	76	8,700 ft of twin 36-in. force mains	389,000
S-2	28	79	4,600 ft of 63-in. RC at 0.18%, average cut 9 ft, dry to moderately wet, includes sheeting for train loads where close to tracks	239,000
S-3	34	95	7,500 ft of 78-in. RC at 0.07%, average cut 10 - 15 ft, dry to wet, includes sheeting for train loads where close to tracks	512,000
S-4	36	100	12,300 ft of 78-in. RC at 0.09%, average cut 12 - 16 ft, dry to moderately wet, includes sheeting for train loads where close to tracks and tunneling in areas of restricted clearance	852,000
S-5	38	107	16,500 ft of 78-in. RC at 0.10 - 0.20%, average cut 10 - 19 ft, dry to difficult wet, includes sheeting and dewatering in difficult wet section, sheeting for train loads where close to tracks and tunneling or filling in areas of restricted clearance	1,240,000
S-6	14	42	4,500 ft of twin 30-in. force mains	158,000
S-7	15	44	1,900 ft of 48-in. RC at 0.23%, average cut 10 ft, dry to difficult wet, includes sheeting and dewatering	86,000
S-8	16	45	11,500 ft of 48-in. RC at 0.23 - 1.8%, average cut 12 - 15 ft, dry to wet	459,000
S-9	18	51	3,200 ft of 48-in. RC at 0.32 - 0.45%, average cut 8 - 9 ft, dry to moderately wet, includes imported backfill and repaving	114,000
S-10	19	55	7,200 ft of 57-in. RC at 0.18%, average cut 10 ft, dry to moderately wet, includes railroad crossing	259,000
S-11	58	156	1,600 ft of parallel 36-in, 48-in. and 54-in. inverted siphons, includes inlet and outlet structures	307,000
S-12	60	162	7,600 ft of 78-in. RC at 0.28%, average cut 12 ft, dry to wet, includes sheeting for train loads where close to tracks	549,000
S-13	65	173	5,000 ft of 78-in. RC tunnel at 0.45%, includes allowance of 20% for uncertainties	1,641,000
S-14	67	179	2,400 ft of 78-in. RC at 0.29%, average cut 20 ft, wet, includes tunneling in areas of restricted clearance	378,000
S-15	68	182	13,000 ft of 84-in. RC at 0.27%, average cut 12 ft, wet to difficult wet, includes sheeting and dewatering, sheeting for train loads where close to tracks, and tunneling and filling in areas of restricted clearance	1,735,000
S-16	1.1	2.8	3,500 ft of 12-in. force main across Lake Washington	94,000
S-17	73	195	17,500 ft of 102-in. RC at 0.08%, average cut 15 ft, difficult wet, includes sheeting and dewatering, sheeting for train loads where close to tracks, tunneling and filling in areas of restricted clearance, and piling	2,817,000
S-18	73	195	400 ft of parallel 36-in., 48-in. and 54-in. inverted siphons, includes inlet and outlet structures	77,000
S-19	0.5	1.7	3,800 ft of 15-in. RC at 0.34%, average cut 6 - 8 ft, difficult wet, includes connections to existing sewers at Bryn Mawr - Lake Ridge sewage treatment plant which is to be abandoned	67,000
S-20	73	195	4,900 ft of 102-in. RC at 0.08%, average cut 25 ft, difficult wet, includes sheeting, dewatering and piling	1,020,000
S-21	87	232	4,300 ft of 108-in. RC at 0.085%, average cut 29 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	909,000

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Table 15-11. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
S-22	89	237	3,200 ft of 108-in. RC at 0.095%, average cut 27 ft, difficult wet, includes sheeting, dewatering, and piling	763,000
S-23	5.1	12	7,600 ft of 30-in. RC at 0.19 - 0.5%, average cut 7 - 8 ft, difficult wet, includes imported backfill and repaving	233,000
S-24	7.8	18	7,700 ft of 36-in. RC at 0.16 - 0.22%, average cut 7 - 8 ft, difficult wet	222,000
S-25	11	27	2,500 ft of 42-in. RC at 0.11%, average cut 11 ft, difficult wet, includes imported backfill, repaving, dewatering and railroad crossings	126,000
S-26	14	35	2,500 ft of 42-in. RC at 0.48%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	116,000
S-27	14	35	400 ft of twin 30-in. inverted siphons, includes inlet and outlet structures	35,000
S-28	14	35	3,100 ft of 60-in. RC at 0.045%, average cut 12 - 16 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	249,000
S-29	22	55	5,000 ft of 51-in. RC at 0.26%, average cut 17 - 24 ft, difficult wet, includes sheeting and dewatering	331,000
S-30	22	55	1,200 ft of 36-in. force main	27,000
S-31	22-23	55-58	15,500 ft of 48-in. RC at 0.40%, average cut 11 - 18 ft, dry to difficult wet, includes imported backfill, repaving, sheeting and dewatering	682,000
S-32	37-42	92-105	26,100 ft of 90-in. RC at 0.036 - 0.05%, average cut 12 - 27 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	3,799,000
S-33	45-47	112-118	10,500 ft of 90-in. RC at 0.063 - 0.15%, average cut 15 - 22 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	1,498,000
Subtotal, sewers, Renton system				21,983,000
PS-S-1	27	76	Pumping station, two stage, motor and diesel engine driven, static lift 110 ft, total head at peak flow 140 ft, structure about 25 ft below ground, includes sheeting and dewatering	524,000
PS-S-2	14	42	Pumping station, two stage, motor and diesel engine driven, static lift 200 ft, total head at peak flow 230 ft, structure about 16 ft below ground	342,000
PS-S-3	3.4	8.7	Pumping station, single stage, motor driven, static lift 14 ft, total head at peak flow 20 ft, structure about 20 ft below ground, includes sheeting and dewatering	112,000
PS-S-4	22	55	Pumping station, single stage, motor driven, static lift 56 ft, total head at peak flow 80 ft, structure about 30 ft below ground, includes sheeting and dewatering	357,000
Subtotal, pumping stations, Renton system				1,335,000
Total contract cost, feeder sewers for Renton system				23,318,000
Engineering and contingencies, 25 per cent				5,830,000
Total construction cost, feeder sewers for Renton system				29,148,000
West Point System				
N-1	31	84	2,100 ft of twin 36-in. force mains	94,000
N-2	31	84	2,300 ft of 60-in. RC at 0.26%, average cut 13 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	166,000

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Table 15-11. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
N-3	32	87	4,800 ft of 60-in. RC at 0.28%, average cut 18 - 23 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	496,000
N-4	34	92	1,000 ft of 60-in. RC at 0.31%, average cut 28 ft, difficult wet, includes sheeting and dewatering	104,000
N-5	38	101	3,700 ft of 66-in. RC at 0.24%, average cut 25 ft, difficult wet, includes sheeting and dewatering	375,000
N-6	39	103	16,800 ft of 84-in. RC at 0.067% laid in Lake Washington at water depth of 15-20 ft. Pipe laid in trench 10 - 19 ft deep excavated in lake bottom, includes rock backfill	2,961,000
N-7	10	27	6,100 ft of 42-in. RC at 0.14 - 0.45%, average cut 8 - 10 ft, difficult wet, includes special bedding, railroad crossing and connections to existing sewers at Lake City sewege treatment plant which is to be abandoned	240,000
N-8	49	126	1,100 ft of twin 42-in. force mains, includes railroad crossing	76,000
N-9	49	126	11,600 ft of 90-in. RC tunnel at 0.09%, includes allowance of 20 per cent for uncertainties	4,651,000
N-10	1.5	4.4	1,700 ft of existing 42-in. at 0.06 - 0.07%	Existing
N-11	1.5-2.0	4.4-5.9	2,000 ft of existing 48-in. at 0.045 - 0.07%. Cost is for 1,740,000 gal. holding tank on tributary sewer serving portion of local service area LU-8	160,000
N-12	3.4	9.7	1,300 ft of existing 30-in. at 0.24 - 0.4%	Existing
N-13	3.4	9.7	2,400 ft of existing 42-in. at 0.08%	Existing
N-14	4.3	12	1,700 ft of existing 42-in. at 0.08%	Existing
N-15	5.1	14, 101[c]	900 ft of existing 96-in. at 0.10%	Existing
N-16	57	140, 298[c]	6,900 ft of existing 138-in. at 0.16%	Existing
N-17	57	140.	100 ft of existing 96-in. at 0.074%	Existing
N-18	2.0	5.8	1,200 ft of existing 66-in. at 0.16%	Existing
N-19	5.4	15, 105[c]	2,800 ft of existing 90-in. at 0.17%. Requires partial separation of local service area LU-1	Existing
N-20	6.4	17, 175[c]	800 ft of existing 114-in. at 0.12%	Existing
N-21	6.4	17	500 ft of existing 48-in. inverted siphon	Existing
N-22	6.4	17	1,100 ft of existing 48-in. at 0.15%	Existing
N-23	64-66	159-164	15,300 ft of existing 108-in. at 0.065 - 0.087%. Cost is for overflow from tributary sewers serving local service areas LU-10 and LU-11	126,000
N-24	67	167	400 ft of existing parallel 48-in. and 60-in. inverted siphons	Existing
N-25	70	174	4,600 ft of existing 138-in. at 0.033%	Existing
N-26	4.3	13, 288[c]	6,100 ft of existing 108-in. at 0.4%	Existing
N-27	4.3	13, 288[c]	5,400 ft of existing 100-in. by 150-in. horseshoe section at 0.075%	Existing
N-28	6.7	19	7,400 ft of existing 84-in. at 0.11%, 72-in. at 0.10%, and 60-in. at 0.10 - 0.21%	Existing
Subtotal, sewers, West Point system				9,449,000
PS-N-1	31	84	Pumping station, single stage, motor and diesel engine driven, static lift 33 ft, total head at peak flow 53 ft, structure about 30 ft below ground, includes sheeting, dewatering and special foundations	541,000
PS-N-2	49	126	Pumping station, single stage, motor and diesel engine driven, static lift 75 ft, total head at peak flow 90 ft, structure about 45 feet below ground, includes sheeting and dewatering.	590,000

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Table 15-11. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
PS-N-3	3.4	9.7	Pumping station, single stage, motor driven with gas engine standby, static lift 17 ft, total head at peak flow 22 ft. To replace existing city of Seattle pumping station having 2 pumps and a total installed capacity of 9.0 mgd, includes sheeting, dewatering, connections to existing sewers, and a 4,900,000 gal. holding tank on tributary sewer serving portion of local service area LU-8	528,000
PS-N-4	4.3	12	Pumping station, single stage, motor driven with gas engine standby, static lift 35 ft, total head at peak flow 39 ft. To replace existing city of Seattle pumping station having 2 pumps and a total installed capacity of 10.6 mgd, includes sheeting, dewatering, connections to existing sewers, and a 3,200,000 gal. holding tank on tributary sewer serving portion of local service area LU-9	426,000
Subtotal, pumping stations, West Point system				2,085,000
Total contract cost, feeder sewers for West Point system				11,534,000
Engineering and contingencies, 25 per cent				2,884,000
Total construction cost, feeder sewers for West Point System				14,418,000
Total construction cost, feeder sewers for Core Plan B				43,566,000

See Fig. 15-7 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

[c]*Flow resulting from 10-year storm over tributary area.*

South Lake Sammamish and East Lake Washington sewerage areas would be tributary to plants located either at the Elliott Bay or the Government Locks sites. Three alternatives were considered in determining the relative economy of this modification as compared with Core Plan B.

Alternative 1. Under this alternative, sewage from the east side of Lake Washington would be concentrated at a point south of Meydenbauer Bay. From that point, it would be conveyed across Lake Washington in parallel twin 42- and twin 54-inch sewers. These lines, each 20,800 feet in length, would be floated in the lake about 50 feet below the water surface and would be provided with necessary buoyancy chambers and concrete anchors. If feasible, the portion of the lines west of Mercer Island, about 7,200 feet in length, could be attached to the floating bridge rather than floated. In either case, the cost of these sewers would approximate the cost of shallow depth submarine outfalls. On the west side of Lake Washington, the sewage would be conveyed by sewer and tunnel to the existing Hanford Street tunnel of the city of Seattle, which has sufficient excess capacity to accommodate expected ultimate flows from the east side. Treatment would be obtained in a primary type plant at the Elliott Bay site, as proposed under Core Plan C, and effluent would be discharged to Elliott Bay. With the removal of east side sewage from the Renton system, this system would serve only the South Lake Washington and Green River sewerage areas.

Alternative 2. Under this alternative, sewage from the east side of Lake Washington would be conveyed to a point south of Meydenbauer Bay, from which an inverted siphon, consisting of parallel 42-, 48- and 60-inch pipes, would be laid across the East Channel of Lake Washington to Mercer Island. Conveyance across Mercer Island would be in tunnel and sewer sections to the west side of the island, from which point Lake Washington would be crossed in parallel twin 42- and twin 54-inch lines. As in Alternative 1, these sections either would be constructed as submerged floating lines or would be attached to the floating bridge. Sewers and treatment and disposal facilities on the west side of Lake Washington and in the Renton system would be identical to those under Alternative 1.

Alternative 3. Sewage concentration and conveyance across Lake Washington under this alternative are identical to Alternative 2. West of the lake, however, sewage from the east side would be conveyed along Elliott Bay waterfront to a treatment plant at the

Government Locks site. Treated effluent would be pumped to Puget Sound for disposal. As in Core Plan A, the treatment plant under this alternative would be situated at the Government Locks site rather than at the West Point site.

Construction Costs. In order to compare the three alternatives with the arrangement called for under Core Plan B, it is necessary to determine the cost of each alternative and to compare these costs with those of the core plan and feeder and service sewer facilities which would be modified or eliminated (Table 15-12). It will be seen that the least costly of the three alternatives, No. 2, would cost $4,496,000 more than the proposed arrangement. It is obvious, therefore, that sewage from the east side of Lake Washington should be conveyed south to the Renton plant rather than across Lake Washington for disposal at either the Elliott Bay site or the Government Locks site.

Modification of Renton Plant

Because both construction and operating costs are less for a primary type treatment plant than for a complete type plant, a modification of Core Plan B was considered whereby primary treatment would be provided at the Renton site and effluent would be disposed of in Puget Sound. Under this modification, the primary effluent, chlorinated during the recreational season, would be pumped through an outfall running westerly to Puget Sound. This outfall would consist of (1) 3,600 feet of parallel 48-, 54-, and 66-inch force mains, (2) 27,000 feet of 132-inch reinforced concrete tunnel, and (3) 3,200 feet of twin 72-inch submarine outfalls to a water depth of about 260 feet.

Estimated construction costs for a primary treatment plant and an effluent pumping station amount to $13,362,000, or $7,613,000 less than that of the secondary plant proposed under Core Plan B. On the other hand, the estimated cost of the effluent outfall for the modification is $18,601,000, or $17,966,000 more than the cost of the Core Plan B outfall, representing a net increase of $10,353,000 for the modification.

As for annual costs, fixed costs for the modified plan would increase by $489,000 per year over Core Plan B, while operating costs would decrease by $36,000 per year. It is apparent, therefore, that secondary treatment at the Renton plant, with effluent disposal to Duwamish River, is a much more satisfactory arrangement than primary treatment with effluent disposal to Puget Sound.

Modification of West Point Plant

As stated earlier (Chapter 10), a need may develop in the future for additional fresh water both to operate the locks on Lake Washington Ship Canal and to prevent salt water intrusion into Lake Washington. In view of that prospect, a study was made to determine the cost of providing secondary treatment at the Government Locks site with effluent disposal to the ship canal above the locks. Estimated construction cost of a secondary plant at the Government Locks site, including land, amounts to $21,631,000, as compared to $11,524,000 for the primary plant at West Point. Under the modification, the sewer along the north shore of Fort Lawton and the outfall to Puget Sound, which are required for the West Point plant, would not be necessary. This means that the cost of these facilities, amounting to $4,940,000, can be deducted from the total construction cost attributable to the Government Locks plant. On that basis, the net increase in construction costs of the Government Locks plant over the West Point plant amounts to $5,167,000.

Fixed costs would be $386,000 per year higher for the Government Locks plant, while operating costs would increase by $287,000 per year, including an allowance of $10 per dry ton for special handling of digested sludge. Hence, the total increase in annual

Table 15-12. Comparison of Construction Costs for Modified East Side Sewer and Core Plan B

Facility	Construction cost,[a] dollars			
	Core Plan B[b]	Modified east side sewer		
		Alternative 1	Alternative 2	Alternative 3
Sewers	28,084,000	29,549,000	26,024,000	38,318,000
Pumping stations	2,225,000	2,485,000	2,485,000	1,825,000
Sewage treatment plants	32,499,000	35,411,000	35,411,000	31,425,000
Outfalls	4,481,000	7,865,000	7,865,000	5,138,000
Total	67,289,000	75,310,000	71,785,000	76,706,000

[a]*Includes engineering and contingencies.*

[b]*Includes, in addition to Core Plan B facilities, the costs of all feeder and service sewers which change because of the possible modification.*

Table 15-13. Apportionment of Construction Costs to Sewerage Areas of Core Plan B and Feeder Sewer Facilities

Sewerage area	Construction cost,[a] dollars				
	Sewers	Pumping stations	Sewage treatment plants	Outfalls	Total
Core Plan B facilities					
North Lake Sammamish	106,000	–	3,964,000	131,000	4,201,000
South Lake Sammamish	60,000	–	2,202,000	74,000	2,336,000
East Lake Washington	92,000	–	3,818,000	115,000	4,025,000
North Lake Washington	321,000	–	3,019,000	1,019,000	4,359,000
Northwest Lake Washington	200,000	–	1,959,000	639,000	2,798,000
South Lake Washington	70,000	–	2,790,000	86,000	2,946,000
Green River	1,411,000	–	8,055,000	225,000	9,691,000
Southwest Lake Washington	4,020,000	554,000	1,414,000	473,000	6,461,000
Elliott Bay	9,024,000	1,133,000	2,927,000	992,000	14,076,000
Lake Union	350,000	25,000	2,351,000	727,000	3,453,000
Total, Core Plan B[b]	15,654,000	1,712,000	32,499,000	4,481,000	54,346,000
Feeder sewer facilities					
North Lake Sammamish	8,371,000	655,000	–	–	9,026,000
South Lake Sammamish	3,891,000	428,000	–	–	4,319,000
East Lake Washington	5,097,000	–	–	–	5,097,000
North Lake Washington	8,200,000	1,145,000	–	–	9,345,000
Northwest Lake Washington	3,254,000	405,000	–	–	3,659,000
South Lake Washington	835,000	–	–	–	835,000
Green River	9,169,000	586,000	–	–	9,755,000
Southwest Lake Washington	116,000	–	–	–	116,000
Elliott Bay	–	–	–	–	–
Lake Union	358,000	1,056,000	–	–	1,414,000
Total, feeder sewers[c]	39,291,000	4,275,000	–	–	43,566,000

[a]*Includes engineering and contingencies.*

[b]*From Table 15-4.*

[c]*From Table 15-11.*

costs amounts to $673,000. Based on an average flow of 99 mgd to the plant during the period 1960-2030, this means that the average cost of providing additional treatment would amount to $18.60 per million gallons, or about $6.20 per acre-foot.

Since the purpose of the present survey is to determine the most economical solution to the sewerage problems of the metropolitan area, it obviously is not within the province of the report to recommend any action in regard to the reclamation modification. Nonetheless, if the needs of the community are such that the additional cost could be justified, then adoption of the modification would be perfectly feasible. In the event of such a program, the additional cost of construction and operation should not be charged against the sewage collection, treatment and disposal function but rather against the agency requiring the reclaimed water.

SEPARATE PROJECTS FOR INDIVIDUAL SEWERAGE AREAS

To determine whether central sewerage would provide the most economical medium for the collection, treatment and disposal of the sewage of the metropolitan Seattle area, it was necessary to compare the cost thereof with the cost of separate or independent projects to serve individual sewerage areas. In each such area, therefore, all independent projects considered to be economical and otherwise feasible were investigated and were compared with participation in the central sewerage project. In some areas, the number of projects was limited by geographical or other conditions and the choice was relatively simple. In others, detailed analyses had to be made of each of several possibilities.

Apportionment of Core Plan B and Feeder Sewer Costs

To determine the relative economy of the central sewerage project as compared to independent projects for each sewerage area, it is necessary to apportion the costs, both construction and annual, of Core Plan B and its feeder sewers to each of the ten tributary areas.

Construction Costs. Apportionment to each sewerage area of the construction costs for central sewerage is based on ultimate flow or capacity re-

quirements. For example, assume the cost of a certain section of trunk sewer is $100,000 and that the design capacity of 10 mgd includes capacity for a flow of 5 mgd from one sewerage area, 3 mgd from a second area and 2 mgd from a third area. On that basis, the total construction cost would be apportioned as follows: $50,000 to the first area, $30,000 to the second and $20,000 to the third.

Apportionment to the tributary sewerage areas of estimated construction costs for Core Plan B facilities and feeder sewers is presented in Table 15-13.

Annual Costs. Apportionment to each sewerage area of fixed costs for the core plan system, including feeder sewers, is based on the apportioned construction costs given in Table 15-13, as are maintenance and operation costs of sewers and outfalls. Apportionment of maintenance and operation costs of pumping stations and treatment plants, including chlorination, is based on the ratio which the average flow from a sewerage area during the design period bears to the total average flow in the particular facility during that period. Average flows, both total and incremental, from each sewerage area were determined from Table 15-1.

Apportionment of estimated annual costs for Core Plan B facilities and feeder sewers is presented in Table 15-14.

North Lake Sammamish, South Lake Sammamish and East Lake Washington Sewerage Areas

Since the sewage generated in these three areas is tributary to a single trunk under the core plan system,

Table 15-14. Apportionment of Annual Costs to Sewerage Areas of Core Plan B and Feeder Sewer Facilities

Cost Item	Total annual cost, $1,000	Sewerage area apportionment, $1,000									
		NLS	SLS	ELW	NLW	NWW	SLW	GR	SWW	EB	LU
Core Plan B facilities											
Fixed costs[a]											
Sewers and outfalls	1,103	13	7	11	73	46	9	90	246	549	59
Pumping stations and treatment plants	2,225	258	143	248	196	127	182	524	128	264	155
	3,328	271	150	259	269	173	191	614	374	813	214
Maintenance and operation											
Sewers[b]	50	1	–	1	3	2	–	4	11	25	3
Pumping stations[c]	66	–	–	–	–	–	–	–	21	44	1
Sewage treatment plants[c]	634	65	31	73	61	51	46	116	40	81	70
Effluent chlorination[c]	155	15	7	17	16	13	11	26	11	21	18
	905	81	38	91	80	66	57	146	83	171	92
Average annual cost,[d] Core Plan B	4,233	352	188	350	349	239	248	760	457	984	306
Feeder sewer facilities											
Fixed costs[a]											
Sewers	2,152	459	213	279	449	178	46	502	6	–	20
Pumping stations	278	43	28	–	74	26	–	38	–	–	69
	2,430	502	241	279	523	204	46	540	6	–	89
Maintenance and operation											
Sewers[b]	98	21	10	13	21	8	2	23	–	–	–
Pumping stations[c]	195	42	38	–	54	28	–	27	–	–	6
	293	63	48	13	75	36	2	50	–	–	6
Average annual cost,[d] feeder sewers	2,723	565	289	292	598	240	48	590	6	–	95

NLS - North Lake Sammamish; SLS - South Lake Sammamish; ELW - East Lake Washington; NLW - North Lake Washington; NWW - Northwest Lake Washington; SLW - South Lake Washington; GR - Green River; SWW - Southwest Lake Washington; EB - Elliott Bay; LU - Lake Union.

[a]*Includes interest and depreciation calculated by the capital recovery method based on 5 per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[b]*0.25 per cent of construction costs.*

[c]*Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[d]*For design period, 1960 - 2030.*

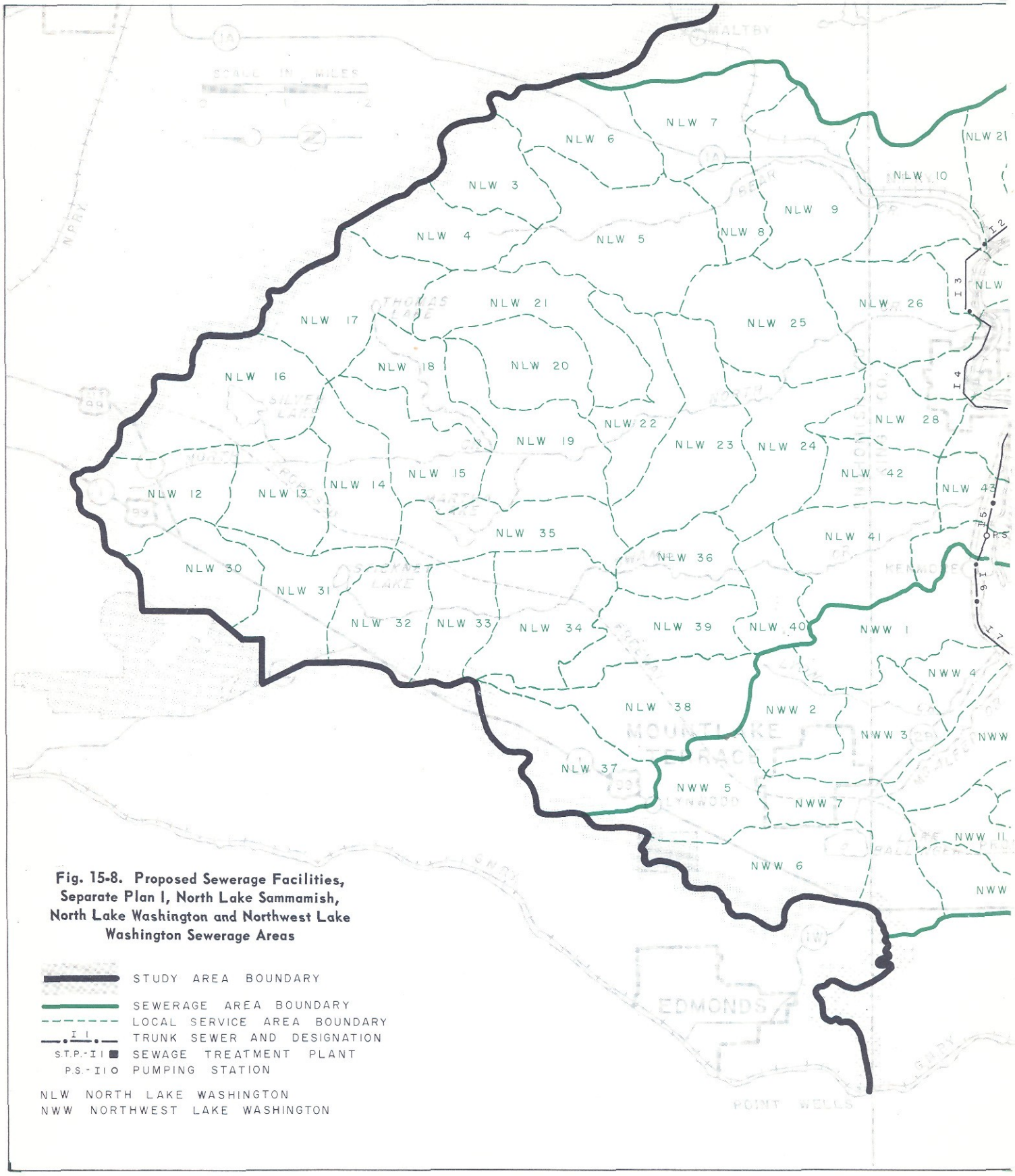

Fig. 15-8. Proposed Sewerage Facilities, Separate Plan I, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

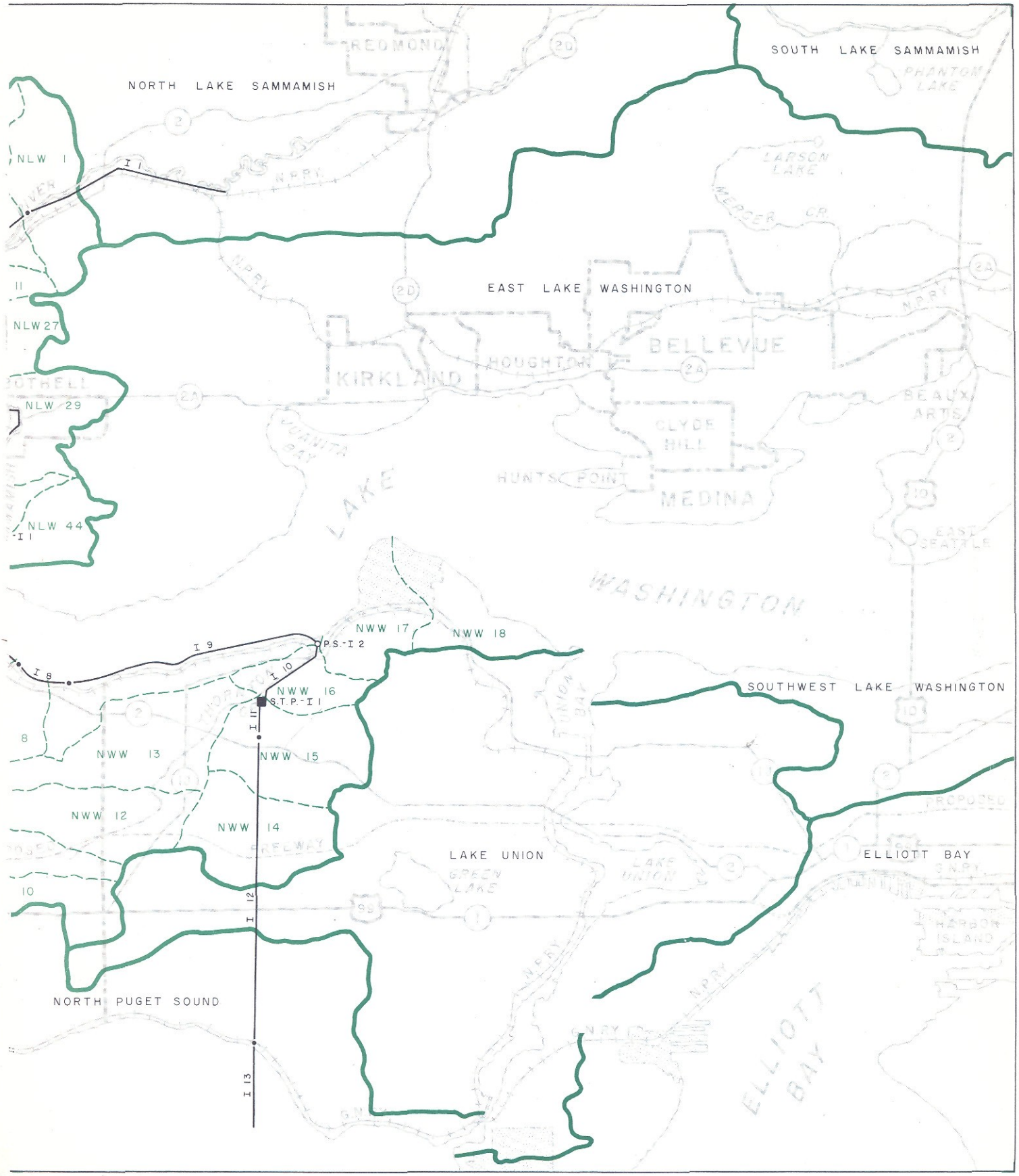
NORTH LAKE SAMMAMISH
SOUTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
SOUTHWEST LAKE WASHINGTON
LAKE UNION
ELLIOTT BAY
NORTH PUGET SOUND
LAKE WASHINGTON
REDMOND
KIRKLAND
BELLEVUE
HOUGHTON
CLYDE HILL
MEDINA
HUNTS POINT
BEAUX ARTS
EAST SEATTLE
PHANTOM LAKE
LARSON LAKE
MERCER CR.
QUANITA BAY
GREEN LAKE
HARBOR ISLAND
NLW 1
NLW 27
NLW 29
NLW 44
I 1
I 8
I 9
I 10
I 11
I 12
I 13
P.S.-I 2
S.T.P.-I 1
NWW 12
NWW 13
NWW 14
NWW 15
NWW 16
NWW 17
NWW 18

Table 15-15. Description and Estimated Construction Costs, Separate Plan I, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
I-1	27	75-76	13,000 ft of 84-in. RC at 0.035 - 0.036%, average cut 21 - 26 ft, difficult wet, includes sheeting, dewatering and river crossing	1,678,000
I-2	28	78-79	3,900 ft of 84-in. RC at 0.038 - 0.040%, average cut 25 - 27 ft, difficult wet, includes sheeting, dewatering and railroad crossing	524,000
I-3	35-36	96-97	6,900 ft of 84-in. RC at 0.06%, average cut 13 - 21 ft, difficult wet, includes sheeting and dewatering	669,000
I-4	47-49	125-130	18,100 ft of 102-in. RC at 0.028%, average cut 13 - 18 ft, difficult wet, includes sheeting, dewatering and railroad and highway crossings	2,267,000
I-5	58	152	2,100 ft of 102-in. RC at 0.052%, average cut 18 ft, difficult wet, includes sheeting and dewatering	274,000
I-6	58	154	2,100 ft of twin 42-in. force mains	112,000
I-7	58-59	154,157	7,100 ft of 78-in. RC at 0.23%, average cut 16 - 24 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	892,000
I-8	61-65	161-170	4,700 ft of 84-in. RC at 0.23%, average cut 25 - 27 ft, difficult wet, includes sheeting and dewatering	564,000
I-9	65-66	170-172	16,800 ft of 102-in. RC at 0.067%, laid in Lake Washington at water depth of 15 - 20 ft. Pipe laid in trench 10 - 19 feet deep excavated in lake bottom. Cost includes rock backfill	3,797,000
I-10	68	177	5,100 ft of parallel 48-in. and 54-in. force mains	342,000
Subtotal, sewers, Plan I				11,119,000
PS-I-1	58	154	Pumping station, single stage, motor and diesel engine driven, static lift 28 ft, total head at peak flow 55 ft, structure about 25 ft below ground, includes special foundations, sheeting and dewatering	778,000
PS-I-2	68	177	Pumping station, single stage, motor and diesel engine driven, static lift 70 ft, total head at peak flow 100 ft, structure about 45 ft below ground, includes sheeting and dewatering	722,000
Subtotal, pumping stations, Plan I				1,500,000
STP	76	193	Sewage treatment plant, secondary type, includes effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	9,539,000
I-11	76	193	2,400 ft of 72-in. force main, effluent outfall	118,000
I-12	76	193	20,500 ft of 102-in. RC tunnel, effluent outfall, includes allowance of 20% for uncertainties	9,840,000
I-13	76	193	2,800 ft of 96-in. RC submarine outfall to a water depth of 120 ft, includes diffuser sections over last 210 ft	1,948,000
Subtotal, outfall, Plan I				11,906,000
Total contract cost, Plan I				34,064,000
Engineering and contingencies, 25 per cent				8,516,000
Total construction cost, Plan I				42,580,000

See Fig. 15-8 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

it is appropriate to consider them as a unit in developing independent projects. For reasons set forth later, the North Lake Sammamish area is also included with the North Lake Washington and Northwest Lake Washington sewerage areas in another analysis.

Alternative projects available for providing independent sewage collection, treatment and disposal facilities for the three areas include: (1) conveyance of all the sewage to a treatment plant near Yarrow Bay, with effluent pumped across Lake Washington for disposal either in Lake Union or Shilshole Bay; (2) construction of separate plants in each sewerage area, with effluent therefrom pumped to a central point near Yarrow Bay for disposal as under (1); and (3) various combinations of these two alternatives.

Along the route of the Lake Washington crossing,

Table 15-16. Description and Estimated Construction Costs, Separate Plan II, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
North Lake Washington and Northwest Lake Washington Sewerage Areas[b]				
II-1 - II-6	–	–	Same as N-1 - N-6, Core Plan B feeder sewers, see Table 15-11	4,196,000
II-7	40	103	5,100 ft of parallel 36-in. and 48-in. force mains	268,000
Subtotal, sewers				4,464,000
PS-II-1	31	84	Same as PS-N-1, Core Plan B feeder sewers, see Table 15-11	541,000
PS-II-2	41	103	Pumping station, motor and diesel engine driven, single stage, static lift 70 ft, total head at peak flow 100 ft, structure about 45 ft below ground, includes sheeting and dewatering	526,000
Subtotal, pumping stations				1,067,000
STP	49	121	Sewage treatment plant, secondary type, includes effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	6,722,000
II-8	49	121	2,400 ft of 54-in. force main, effluent outfall	83,000
II-9	49	121	20,500 ft of 84-in. RC tunnel at 0.13%, effluent outfall, includes allowance of 20% for uncertainties	7,725,000
II-10	49	121	2,500 ft of 78-in. RC submarine outfall to water depth of 100 ft, includes diffuser section over last 125 ft	799,000
Subtotal, outfalls				8,607,000
Total contract cost, Plan II, North Lake Washington and Northwest Lake Washington Sewerage Areas				20,860,000
Engineering and contingencies, 25 per cent				5,215,000
Total construction cost, Plan II, North Lake Washington and Northwest Lake Washington Sewerage Areas				26,075,000
North Lake Sammamish Sewerage Area[c]				
–	–	–	Sewers, same as Core Plan B and feeder sewers, see Table 15-13	8,477,000[d]
–	–	–	Pumping stations, same as Core Plan B and feeder sewers, see Table 15-13	655,000[d]
–	–	–	Sewage treatment plant, same as Core Plan B, see Table 15-13	3,964,000[d]
–	–	–	Outfall, same as Core Plan B, see Table 15-13	131,000[d]
Total construction cost, Plan II, North Lake Sammamish Sewerage Area				13,227,000[d]
Total construction cost, Plan II				39,302,000

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*See Fig. 15-9 for location of facilities.*

[c]*See Figs. 15-4 and 15-7 for location of facilities.*

[d]*Apportioned cost to North Lake Sammamish Sewerage Area; includes engineering and contingencies.*

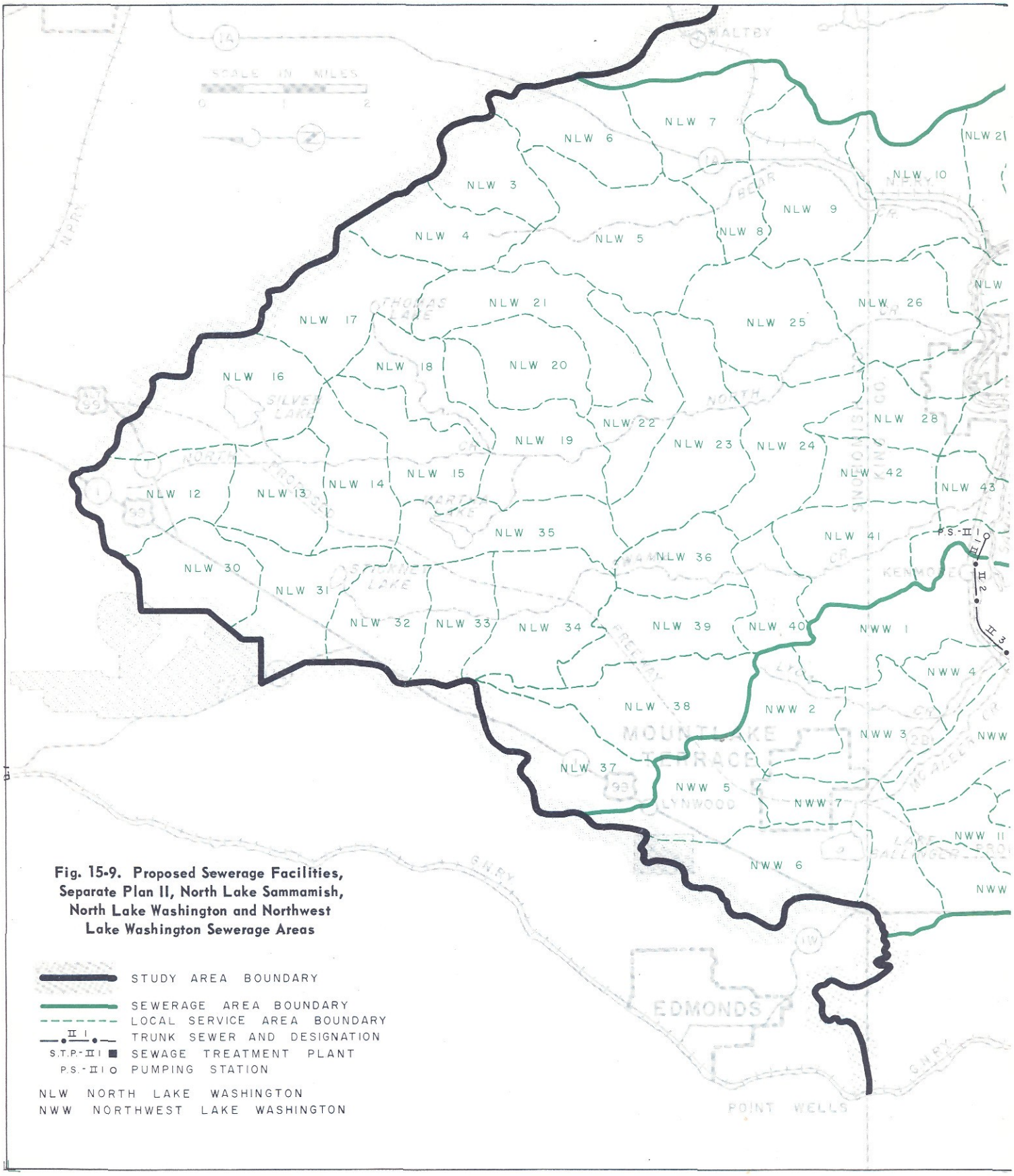

Fig. 15-9. Proposed Sewerage Facilities, Separate Plan II, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

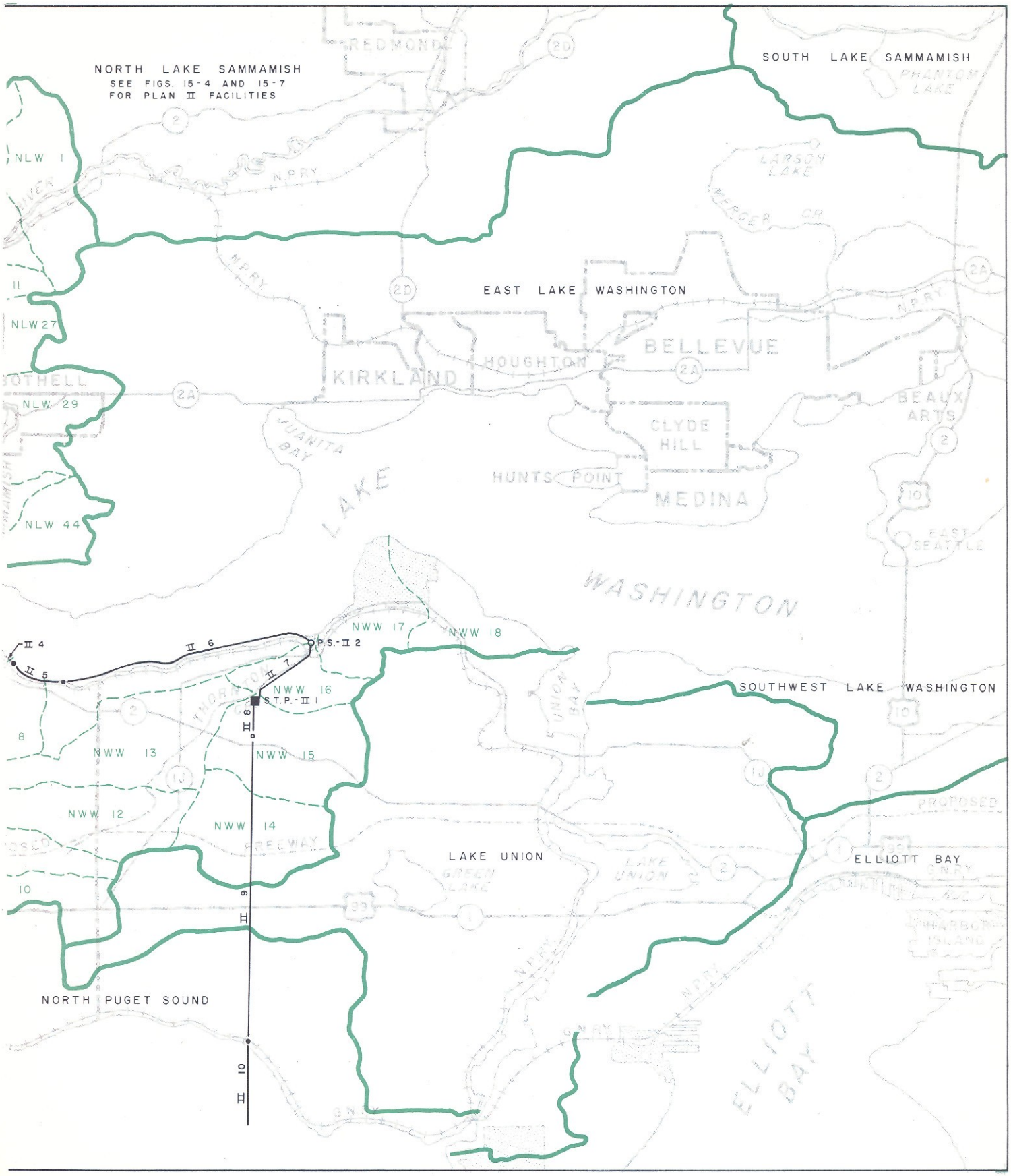
NORTH LAKE SAMMAMISH
SEE FIGS. 15-4 AND 15-7
FOR PLAN II FACILITIES
SOUTH LAKE SAMMAMISH
PHANTOM LAKE
REDMOND
LARSON LAKE
MERCER CR
EAST LAKE WASHINGTON
BELLEVUE
HOUGHTON
KIRKLAND
BOTHELL
BEAUX ARTS
CLYDE HILL
HUNTS POINT
MEDINA
JUANITA BAY
LAKE WASHINGTON
EAST SEATTLE
SOUTHWEST LAKE WASHINGTON
UNION BAY
THORNTON CR
LAKE UNION
GREEN LAKE
ELLIOTT BAY
HARBOR ISLAND
NORTH PUGET SOUND
PROPOSED FREEWAY
N.P.RY
G.N.RY
NLW 1
NLW 27
NLW 29
NLW 44
NWW 12
NWW 13
NWW 14
NWW 15
NWW 16
NWW 17
NWW 18
II 4
II 5
II 6
II 7
II 8
II 9
II 10
P.S.-II 2
S.T.P.-II 1

Table 15-17. Description and Estimated Construction Costs, Separate Plan III, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
Northwest Lake Washington Sewerage Area[b]				
III-1	1.1	2.7	4,800 ft of 18-in. RC at 0.16%, average cut 9 - 11 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	134,000
III-2	3.0	7.4	1,000 ft of 24-in. RC at 0.27%, average cut 13 ft, difficult wet, includes sheeting and dewatering	33,000
III-3	6.7	17	3,700 ft of 33-in. RC at 0.22%, average cut 12 ft, difficult wet, includes sheeting and dewatering	136,000
III-4	7.0-7.5	18-19	16,800 ft of 42-in. RC at 0.085%, laid in Lake Washington at water depth of 15-20 ft. Pipe laid in trench 11 - 23 ft deep excavated in lake bottom, includes rock backfill	1,518,000
III-5	9.1	24	5,100 ft of parallel 20-in. and 30-in. force mains	148,000
Subtotal, sewers				1,969,000
PS-III-1	9.1	24	Pumping station, single stage, motor driven with gas engine standby, static lift 75 ft, total head at peak flow 100 ft, structure about 45 ft below ground, includes sheeting and dewatering	221,000
STP-III-1	18	48	Sewage treatment plant, secondary type, includes effluent pumping and facilities for screenings and grit removal, aeration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination	1,988,000
III-6	18	48	2,400 ft of 36-in. force main, effluent outfall	54,000
III-7	18	48	20,500 ft of 72-in. RC tunnel at 0.045%, effluent outfall, includes allowance of 20% for uncertainties	6,150,000
III-8	18	48	1,900 ft of 60-in. RC submarine outfall to water depth of 50 ft, includes diffuser section over last 40 ft	307,000
Subtotal, outfall				6,511,000
Total contract cost, Plan III, Northwest Lake Washington Sewerage Area				10,689,000
Engineering and contingencies, 25 per cent				2,672,000
Total construction cost, Plan III, Northwest Lake Washington Sewerage Area				13,361,000
North Lake Washington Sewerage Area[b]				
STP-III-2	31	78	Sewage treatment plant, secondary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes special foundations	5,564,000
III-9	31	78	2,700 ft of 42-in. force main, effluent outfall, includes railroad and highway crossing	72,000
III-10	31	78	32,800 ft of 78-in. RC tunnel at 0.075%, effluent outfall, includes allowance of 20% for uncertainties	11,021,000
III-11	31	78	800 ft of 78-in. RC at 0.055%, effluent outfall, minimum depth, difficult wet, includes sheeting, dewatering and railroad crossing	75,000
III-12	31	78	1,000 ft of 66-in. RC submarine outfall to water depth of 75 ft, includes diffuser section over last 125 ft	257,000
Subtotal, outfall				11,425,000

Continued on next page

Table 15-17. Continued

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
Total contract cost, Plan III, North Lake Washington Sewerage Area				16,989,000
Engineering and contingencies, 25 per cent				4,247,000
Total construction cost, Plan III, North Lake Washington Sewerage Area				21,236,000
North Lake Sammamish Sewerage Area[c]				
–	–	–	Sewers, same as Core Plan B and feeder sewers, see Table 15-13	8,477,000[d]
–	–	–	Pumping stations, same as Core Plan B and feeder sewers, see Table 15-13	655,000[d]
–	–	–	Sewage treatment plant, same as Core Plan B, see Table 15-13	3,964,000[d]
–	–	–	Outfall, same as Core Plan B, see Table 15-13	131,000[d]
Total construction cost, Plan III, North Lake Sammamish Sewerage Area				13,227,000[d]
Total construction cost, Plan III				47,824,000

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*See Fig. 15-10 for location of facilities.*

[c]*See Figs. 15-4 and 15-7 for location of facilities.*

[d]*Apportioned cost to North Lake Sammamish Sewerage Area; includes engineering and contingencies.*

over 6,000 feet of the effluent outfall would be in water having a depth between 180 and 210 feet. Additionally, the lake bottom is composed of up to 40 feet of soft, almost fluid, organic peat-like sediments which overlie a stratum of compressible glacial blue clay having a depth in excess of 100 feet (Chapter 3). Because of these conditions, the lake crossing would be both extremely difficult and costly to construct. Moreover, the permanence of such a line could not be guaranteed.

Consideration was given also to the possibility of constructing a floating submerged pipeline. This idea was discarded, however, because it is unlikely that approval could be obtained for such an undertaking in this part of the lake. Similarly, the possibility of suspending the line on a proposed new bridge along this route was discarded, since it is highly improbable that such a bridge will be constructed by the time the line would be required.

In view of the problems just mentioned, detailed design and cost estimates were not prepared. A rough estimate, however, indicates that the lake crossing alone would cost in excess of $20 million, or almost the total cost to the three areas for participation in the core plan system. It is evident, therefore, that the interests of the South Lake Sammamish and East Lake Washington sewerage areas will best be served by sewering south to the Renton plant as proposed under Core Plan B. Additional alternatives were considered for the North Lake Sammamish sewerage area and are discussed in the following section.

North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Since the North Lake Sammamish sewerage area can sewer into and through the North Lake Washington sewerage area by gravity, these areas, together with Northwest Lake Washington, should be considered jointly with respect to possible separate sewerage projects. Basically, the alternatives available are the construction either of a single or of several treatment plants, all with effluent disposal to Puget Sound. As indicated in Chapter 11, disposal conditions along the shore of the sound in the northerly part of the study area require that secondary treatment be provided if large volumes of effluent are to be discharged. On that basis, five separate plans were developed as follows:

Plan I - Conveyance of the sewage from all three areas to a secondary type treatment plant at the site of the existing Lake City treatment plant, with effluent disposal to Puget Sound.

Plan II - Conveyance of the sewage from the North Lake Washington and Northwest Lake Washington sewerage areas to a secondary type treatment plant at the site of the Lake City treatment plant, with effluent disposal to Puget Sound. Under this alternative, the North Lake Sammamish sewerage area would be served by the core plan.

Plan III - Construction of separate secondary type treatment plants in the North Lake Washington and Northwest Lake Washington sewerage areas, with each plant discharging its effluent separately to Puget

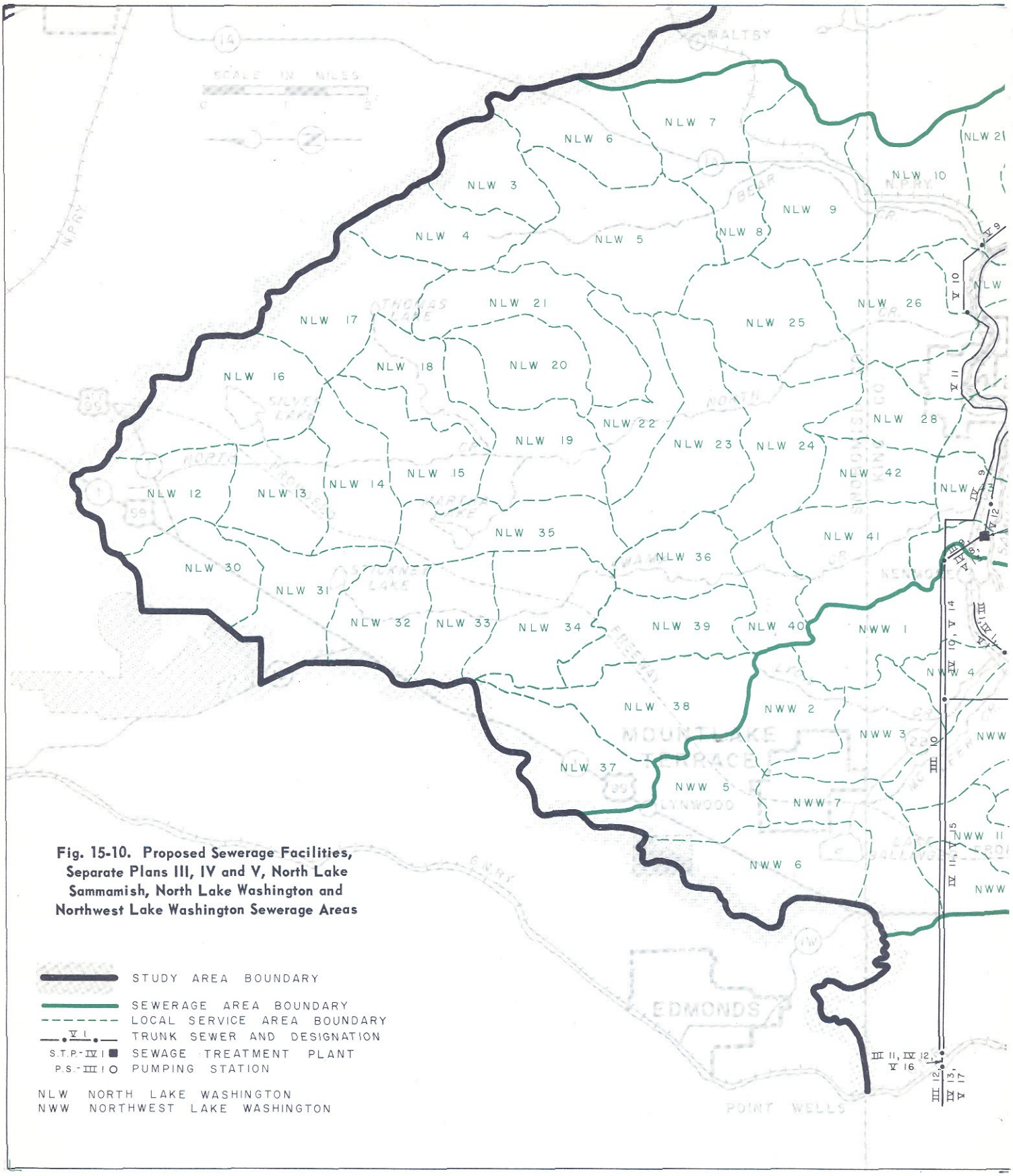

Fig. 15-10. Proposed Sewerage Facilities, Separate Plans III, IV and V, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

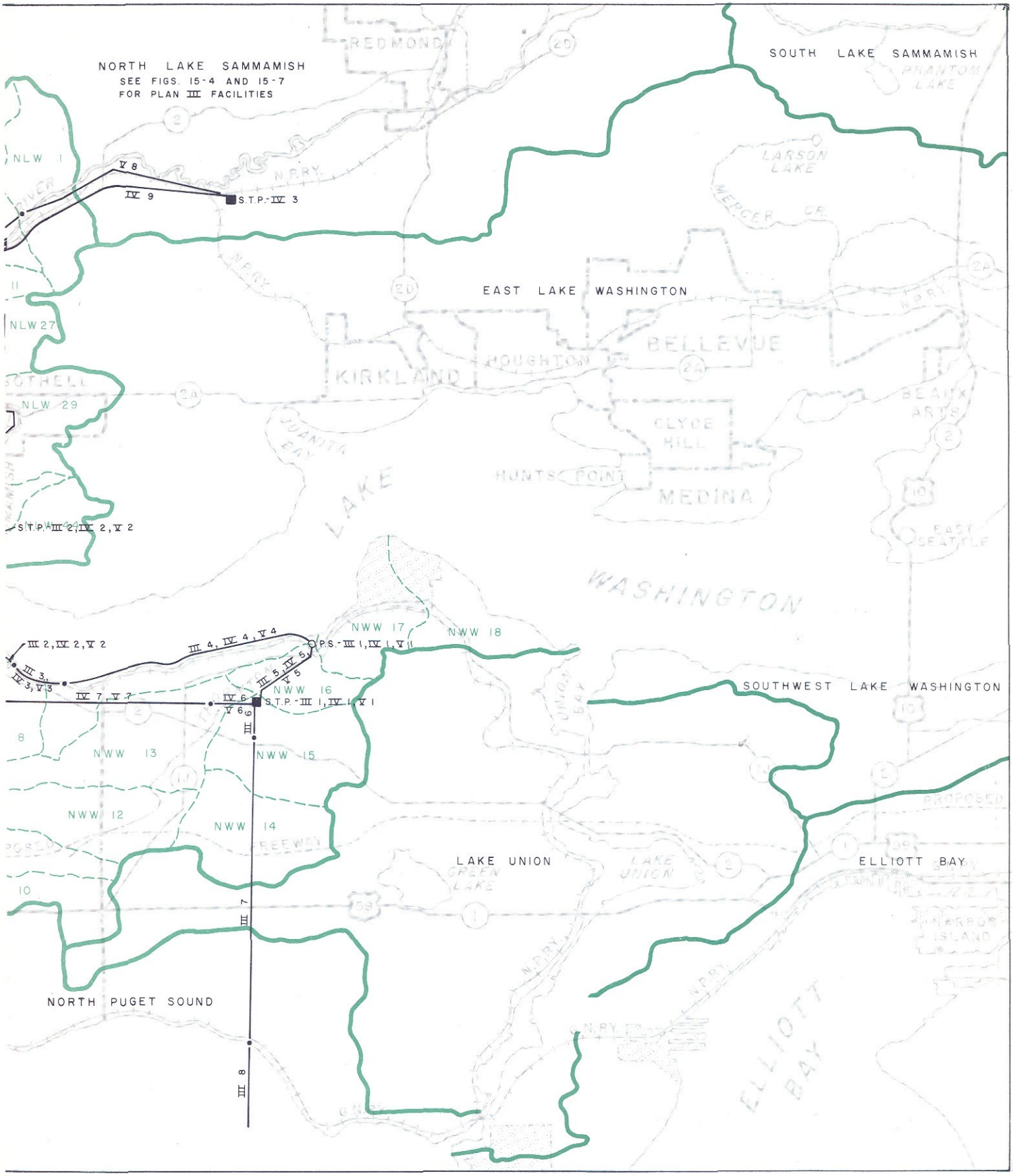
NORTH LAKE SAMMAMISH
SEE FIGS. 15-4 AND 15-7
FOR PLAN III FACILITIES
SOUTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
SOUTHWEST LAKE WASHINGTON
LAKE UNION
ELLIOTT BAY
NORTH PUGET SOUND
LAKE WASHINGTON
REDMOND
KIRKLAND
BELLEVUE
MEDINA
S.T.P.-IV 3
S.T.P. III 2, IV 2, V 2
P.S.-III 1, IV 1, V 1
S.T.P.-III 1, IV 1, V 1
III 2, IV 2, V 2
III 4, IV 4, V 4
III 3, IV 3, V 3
IV 7, V 7
IV 6, V 6
III 5, IV 5, V 5
III 6
III 7
III 8
V 8
IV 9
NLW 1
NLW 27
NLW 29
NWW 12
NWW 13
NWW 14
NWW 15
NWW 16
NWW 17
NWW 18

Table 15-18. Description and Estimated Construction Costs, Separate Plan IV, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
Northwest Lake Washington Sewerage Area				
IV-1 - IV-5	–	–	Same as III-1 - III-5, Plan III, see Table 15-17	1,969,000
PS-IV-1	–	–	Same as PS-III-1, Plan III, see Table 15-17	221,000
STP-IV-1	–	–	Same as STP-III-1, Plan III, see Table 15-17	1,988,000
IV-6	18	48	2,100 ft of 36-in. force main, effluent outfall	47,000
IV-7	18	48	18,300 ft of 72-in. RC tunnel at 0.045%, effluent outfall, includes allowance of 20% for uncertainties	5,490,000
Subtotal, outfall				5,537,000
Total contract cost, Plan IV, Northwest Lake Washington Sewerage Area				9,715,000
Engineering and contingencies, 25 per cent				2,429,000
Total construction cost, Plan IV, Northwest Lake Washington Sewerage Area				12,144,000
North Lake Washington Sewerage Area				
STP-IV-2	–	–	Same as STP-III-2, Plan III, see Table 15-17	5,564,000
IV-8	–	–	Same as III-9, Plan III, see Table 15-17	72,000
Total contract cost, Plan IV, North Lake Washington Sewerage Area				5,636,000
Engineering and contingencies, 25 per cent				1,409,000
Total construction cost, Plan IV, North Lake Washington Sewerage Area				7,045,000
North Lake Sammamish Sewerage Area				
STP-IV-3	27	72	Sewage treatment plant, secondary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, separate sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	4,495,000
IV-9	27	72	41,800 ft of 66-in. force main, effluent outfall, includes imported backfill, repaving and railroad and river crossings	2,038,000
Total contract cost, North Lake Sammamish Sewerage Area				6,533,000
Engineering and contingencies, 25 per cent				1,633,000
Total construction cost, North Lake Sammamish Sewerage Area				8,166,000
Joint Outfall				
IV-10	58	150	8,700 ft of 96-in. RC tunnel at 0.09%, effluent outfall, includes allowance of 20% for uncertainties	3,863,000
IV-11	76	193	24,100 ft of 102-in. RC tunnel at 0.11%, effluent outfall, includes allowance of 20% for uncertainties	11,568,000
IV-12	76	193	800 ft of 102-in. RC at 0.085%, effluent outfall, minimum depth, difficult wet, includes sheeting, dewatering and railroad crossing	115,000
IV-13	76	193	1,200 ft of 96-in. RC submarine outfall to water depth of 90 ft, includes diffuser section over last 160 ft	727,000
Total contract cost, Plan IV, joint outfall				16,273,000
Engineering and contingencies, 25 per cent				4,068,000
Total construction cost, Plan IV, joint outfall				20,341,000
Total construction cost, Plan IV				47,696,000

See Fig. 15-10 for location of facilities. [a]*Expressed as average dry weather flow and maximum wet weather flow.*

Sound. As in Plan II, the North Lake Sammamish sewerage area would be served by the core plan.

Plan IV - Construction of separate secondary type treatment plants in each of the three areas, with effluent disposal through a joint outfall to Puget Sound.

Plan V - Conveyance of the sewage from the North Lake Sammamish and North Lake Washington sewerage areas to a secondary type treatment plant west of Bothell, and treatment of the sewage from the Northwest Lake Washington sewerage area in a separate secondary type plant, with both effluents discharged through a joint outfall to Puget Sound.

Plan I. Locations of sewers, pumping stations, treatment works and outfall called for under Plan I are shown in Fig. 15-8. Descriptions of these facilities, together with their estimated costs, are given in Table 15-15. For this plan, the estimated construction cost totals $42,580,000.

Under Plan I, the intercepting sewer would begin north of Redmond in the North Lake Sammamish sewerage area and would follow the Sammamish River valley through the North Lake Washington sewerage area to a pumping station east of Bothell. At this station, sewage would be lifted to a sewer on the Northern Pacific Railroad right-of-way and would flow by gravity along the lake front to Sheridan Beach. From that point, the sewer would be laid parallel to the shore in a trench excavated in the bottom of Lake Washington and would extend to a pumping station at the mouth of Thornton Creek.

The treatment plant would be constructed at the site of the existing Lake City plant and would provide secondary treatment for an estimated ultimate average flow of 76 mgd, with a peak hydraulic capacity of 193 mgd. Since the present Lake City plant is not of a size that could be converted economically to handle the anticipated ultimate flows, a completely new plant would have to be constructed. Plant units would include preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks and all other necessary structures and appurtenances. Digested sludge would be hauled away in tank trucks and disposed of elsewhere. Treated effluent would be chlorinated and pumped to south of Piper Creek, at which location it would be discharged to Puget Sound approximately 2,800 feet offshore at a water depth of about 120 feet.

Plan II. Locations of sewers, pumping stations, treatment works and outfall for the North Lake Washington and Northwest Lake Washington sewerage areas are shown in Fig. 15-9, while Core Plan B and feeder sewer facilities for the North Lake Sammamish sewerage area are shown in Figs. 15-4 and 15-7. Descriptions and estimated construction costs of the facilities for Plan II are given in Table 15-16. Total construction costs are estimated to be $39,302,000.

Interception of sewage from the North Lake Washington and Northwest Lake Washington sewerage areas would be accomplished in the same way as interception under the feeder sewer system for Core Plan B as far as the pumping station at the mouth of Thornton Creek. From this point, the sewage would be pumped to a treatment plant at the site of the existing Lake City plant.

As in Plan I, the present Lake City plant, because of its size, is not suitable for incorporation into the plant proposed under Plan II. A new plant would therefore be constructed at this site and would provide secondary treatment for an ultimate average flow of 49 mgd, with a peak hydraulic capacity of 121 mgd. Plant units, except for size, and plant operation would be identical to Plan I. Treated effluent would be disposed of as under Plan I, but the size of the outfall, because of the smaller peak flow, would be reduced accordingly.

Under Plan II, the North Lake Sammamish sewerage area would be served by the central sewerage project. All facilities for this area would thus be identical to those required for participation in that project.

Plan III. Locations of sewers, pumping stations, treatment works and outfalls for the North Lake Washington and Northwest Lake Washington sewerage areas are shown in Fig. 15-10, while Core Plan B and feeder sewer facilities for the North Lake Sammamish sewerage area are shown in Figs. 15-4 and 15-7. These facilities are described in Table 15-17, which also gives their estimated construction costs. As indicated therein, facilities called for under Plan III are estimated to cost a total of $47,824,000.

The intercepting sewer for the Northwest Lake Washington sewerage area would start at the eastern boundary of the area and would be laid along the lake front to Sheridan Beach. From there, the sewer would be laid parallel to shore in a trench excavated in the bottom of Lake Washington and would extend to a pumping station at the mouth of Thornton Creek. The sewage would then be pumped to a treatment plant at the site of the existing Lake City plant.

Under Plan III, the present Lake City plant could be enlarged to provide secondary treatment for the expected ultimate average flow of 18 mgd. This phase of the project, including the provision of necessary new facilities and structures, is estimated to cost $1,988,000, not including engineering and contingencies. Digested sludge would be hauled away in tank trucks for disposal elsewhere. Treated effluent would be pumped through a force main and tunnel to a submarine outfall discharging to Puget Sound south of Piper Creek. Discharge would be approximately

1,900 feet offshore in water at a depth of about 50 feet.

No intercepting sewers would be required in the North Lake Washington area. This is because the sewage treatment plant serving that area would be situated at a site about two miles west of Bothell, the point to which all sewage would be conveyed by the service sewer system.

Treatment would be obtained in a secondary type plant capable of accommodating an ultimate average dry weather flow of 31 mgd, with a peak hydraulic capacity of 78 mgd. Plant units would include preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks and all other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons. Because of soil conditions at the site, foundation piles would be required. Treated effluent would be chlorinated and pumped to south of Richmond Beach, where it would be discharged to Puget Sound approximately 1,000 feet offshore in water at a depth of about 75 feet.

Under Plan III, the North Lake Sammamish sewerage area would be served by the central sewerage project. All facilities for this area would thus be identical to those required for participation in that project.

Plan IV. Sewers, pumping stations, treatment works and outfalls called for under Plan IV are shown in Fig. 15-10 and are described in Table 15-18. For these facilities, the estimated construction cost amounts to $47,696,000.

The intercepting sewer and the sewage treatment

Table 15-19. Description and Estimated Construction Costs, Separate Plan V, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
Northwest Lake Washington Sewerage Area				
V-1 - V-5	–	–	Same as III-1 - III-5, Plan III, see Table 15-17	1,969,000
PS-V-1	–	–	Same as PS-III-1, Plan III, see Table 15-17	221,000
STP-V-1	–	–	Same as STP-III-1, Plan III, see Table 15-17	1,988,000
V-6 - V-7	–	–	Same as IV-6 - IV-7, Plan IV, see Table 15-18	5,537,000
Total contract cost, Plan V, Northwest Lake Washington Sewerage Area				9,715,000
Engineering and contingencies, 25 per cent				2,429,000
Total construction cost, Plan V, Northwest Lake Washington Sewerage Area				12,144,000
North Lake Sammamish and North Lake Washington Sewerage Areas				
V-8 - V-12	–	–	Same as I-1 - I-5, Plan I, see Table 15-15	5,412,000
STP-V-2	58	150	Sewage treatment plant, secondary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes special foundations	9,577,000
V-13	58	150	2,700 ft of 60-in. force main, effluent outfall, includes railroad and highway crossing	109,000
Total contract cost, Plan V, North Lake Sammamish and North Lake Washington Sewerage Areas				15,098,000
Engineering and contingencies, 25 per cent				3,774,000
Total construction cost, Plan V, North Lake Sammamish and North Lake Washington Sewerage Areas				18,872,000
Joint Outfall				
V-14 - V-17	–	–	Same as IV-10 - IV-13, Plan IV, see Table 15-18	20,341,000[b]
Total construction cost, Plan V				51,357,000

See Fig. 15-10 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Includes allowance for engineering and contingencies.*

plant for the Northwest Lake Washington sewerage area are identical to those provided under Plan III. Plant effluent would be pumped through a force main and tunnel to the joint outfall serving the three areas. The treatment plant and effluent force main for the North Lake Washington sewerage area are also identical to those proposed under Plan III.

In the North Lake Sammamish sewerage area, the

Table 15-20. Comparison of Construction Costs, Core Plan B and Separate Plans, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Construction cost,[a] $1,000							
	Core Plan B[b]				Plan I			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Sewers	20,452	8,477	8,521	3,454	13,898	8,316	4,980	602
Pumping stations	2,205	655	1,145	405	1,875	844	914	117
Sewage treatment plants	8,942	3,964	3,019	1,959	11,924	4,236	4,864	2,824
Outfalls	1,789	131	1,019	639	14,883	5,552	6,015	3,316
Total	33,388	13,227	13,704	6,457	42,580	18,948	16,773	6,859

Facility	Construction cost,[a] $1,000							
	Plan II				Plan III			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Sewers	14,057	8,477	4,709	871	10,938	8,477	–	2,461
Pumping stations	1,989	655	1,186	148	931	655	–	276
Sewage treatment plants	12,366	3,964	5,316	3,086	13,404	3,964	6,955	2,485
Outfalls	10,890	131	6,936	3,823	22,551	131	14,281	8,139
Total	39,302	13,227	18,147	7,928	47,824	13,227	21,236	13,361

Facility	Construction cost,[a] $1,000							
	Plan IV				Plan V			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Sewers	2,461	–	–	2,461	9,226	5,178	1,587	2,461
Pumping stations	276	–	–	276	276	–	–	276
Sewage treatment plants	15,059	5,619	6,955	2,485	14,456	5,573	6,398	2,485
Outfalls	29,900	10,653[c]	8,870[c]	10,377[c]	27,399	8,171[c]	8,851[c]	10,377[c]
Total	47,696	16,272	15,825	15,599	51,357	18,922	16,836	15,599

Plan I proposes concentration of the sewage from all three areas in one secondary type treatment plant with effluent disposal to Puget Sound.

Plan II proposes concentration of the sewage from the North Lake Washington and Northwest Lake Washington Sewerage Area in one secondary type treatment plant with effluent disposal to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan III proposes construction of separate secondary type treatment plants in the North Lake Washington and Northwest Lake Washington Sewerage Areas with each plant discharging its effluent separately to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan IV proposes construction of separate secondary type treatment plants in each of the three areas with effluent disposal through a joint outfall to Puget Sound.

Plan V proposes concentration of the sewage from the North Lake Sammamish and North Lake Washington Sewerage Areas in one secondary type treatment plant and concentration of the sewage from the Northwest Lake Washington Sewerage Area in a separate secondary type treatment plant with effluent disposal through a joint outfall to Puget Sound.

[a]*Includes engineering and contingencies.*

[b]*Apportioned cost to the three areas, see Table 15-13; includes feeder sewers.*

[c]*Includes apportioned cost of joint outfall.*

Table 15-21. Comparison of Annual Costs, Core Plan B and Separate Plans, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Cost Item	Average annual cost, $1,000							
	Core Plan B[a]				Plan I			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Fixed costs[b]								
Sewers and outfalls	1,218	472	522	224	1,577	760	602	215
Pumping stations and treatment plants	724	301	270	153	897	330	376	191
	1,942	773	792	377	2,474	1,090	978	406
Maintenance and operation								
Sewers[c]	56	22	24	10	72	35	27	10
Pumping stations[d]	124	42	54	28	108	41	53	14
Sewage treatment plants[d]	177	65	61	51	342	105[e]	135[e]	102[e]
Effluent chlorination[d]	44	15	16	13	50	15	20	15
	401	144	155	102	572	196	235	141
Total annual cost	2,343	917	947	479	3,046	1,286	1,213	547

Cost Item	Average annual cost, $1,000							
	Plan II				Plan III			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Fixed costs[b]								
Sewers and outfalls	1,367	472	638	257	1,835	472	782	581
Pumping stations and treatment plants	934	301	423	210	933	301	452	180
	2,301	773	1,061	467	2,768	773	1,234	761
Maintenance and operation								
Sewers[c]	63	22	29	12	85	22	36	27
Pumping stations[d]	114	42	58	14	59	42	–	17
Sewage treatment plants[d]	329	65	151[e]	113[e]	358	65	146[f]	147[e]
Effluent chlorination[d]	50	15	20	15	50	15	20	15
	556	144	258	154	552	144	202	206
Total annual cost	2,857	917	1,319	621	3,320	917	1,436	967

Cost Item	Average annual cost, $1,000							
	Plan IV				Plan V			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Fixed costs[b]								
Sewers and outfalls	1,773	584	486	703	2,006	731	572	703
Pumping stations and treatment plants	998	366	452	180	959	363	416	180
	2,771	950	938	883	2,965	1,094	988	883
Maintenance and operation								
Sewers[c]	81	27	22	32	91	33	26	32
Pumping stations[d]	17	–	–	17	17	–	–	17
Sewage treatment plants[d]	419	126[f]	146[f]	147[e]	355	92[f]	116[f]	147[e]
Effluent chlorination[d]	50	15	20	15	50	15	20	15
	567	168	188	211	513	140	162	211
Total annual cost	3,338	1,118	1,126	1,094	3,478	1,234	1,150	1,094

Continued on next page

treatment plant would be constructed at a site about two miles north of Redmond. This plant would be of the secondary type and would be capable of treating an ultimate average flow of 27 mgd, with a peak hydraulic capacity of 72 mgd. Both influent and effluent pumping would be required. Plant units would consist of preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks, and all other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons. Chlorinated effluent would be pumped to the joint outfall. This outfall would consist of tunnels and pipe to the shoreline south of Richmond Beach and of a 96-inch submarine section. Discharge to Puget Sound would be approximately 1,200 feet offshore in water at a depth of about 90 feet.

Plan V. Locations of sewers, pumping stations, treatment works and outfalls proposed under Plan V are shown in Fig. 15-10. These facilities are described in Table 15-19 and are estimated to cost $51,357,000.

The intercepting sewer and the sewage treatment plant for the Northwest Lake Washington sewerage area are identical to those proposed under Plan III. Effluent disposal would be achieved in the same manner as under Plan IV.

The intercepting sewer for the North Lake Sammamish and North Lake Washington sewerage areas would consist of a gravity sewer in the Sammamish River valley which would extend from the point of concentration north of Redmond through the North Lake Washington sewerage area to a treatment plant approximately two miles west of Bothell. Except for size, the proposed treatment plant would be the same as that proposed under Plan IV. Treated and chlorinated effluent would be pumped to the joint outfall, which would be identical to the outfall called for in Plan IV.

Comparison of Plans. Apportioned costs to the three sewerage areas of the estimated construction cost of Core Plan B and feeder sewer facilities are given in Table 15-20, as are the estimated construction costs of the five separate plans just considered. Total costs to the three sewerage areas are estimated to range from $33,388,000 for participation in the central project to $51,357,000 for Plan V. It is shown also that the cost to each individual area is lower for participation in the central project than it would be under any of the five plans.

Estimated average annual costs for the central project and for each of the five separate plans are given in Table 15-21. As there indicated, the lowest total annual cost to the three areas, amounting to $2,343,000, is for participation in the central sewerage project. It will be seen also that the annual cost to the individual areas is the lowest for participation in the central project.

Selection of Most Acceptable Plan. From the foregoing discussion of costs, both construction and annual, it is apparent that the most economical means of sewage collection, treatment and disposal for the North Lake Sammamish, North Lake Washington and Northwest Lake Washington sewerage areas would be obtained by

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Table 15-21 footnotes continued from page 394

Plan I proposes concentration of the sewage from all three areas in one secondary type treatment plant with effluent disposal to Puget Sound.

Plan II proposes concentration of the sewage from the North Lake Washington and Northwest Lake Washington Sewerage Area in one secondary type treatment plant with effluent disposal to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan III proposes construction of separate secondary type treatment plants in the North Lake Washington and Northwest Lake Washington Sewerage Areas with each plant discharging its effluent separately to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan IV proposes construction of separate secondary type treatment plants in each of the three areas with effluent disposal through a joint outfall to Puget Sound.

Plan V proposes concentration of the sewage from the North Lake Sammamish and North Lake Washington Sewerage Areas in one secondary type treatment plant and concentration of the sewage from the Northwest Lake Washington Sewerage Area in a separate secondary type treatment plant with effluent disposal through a joint outfall to Puget Sound.

[a] *Apportioned cost to the three areas, see Table 15-14; includes feeder sewers.*

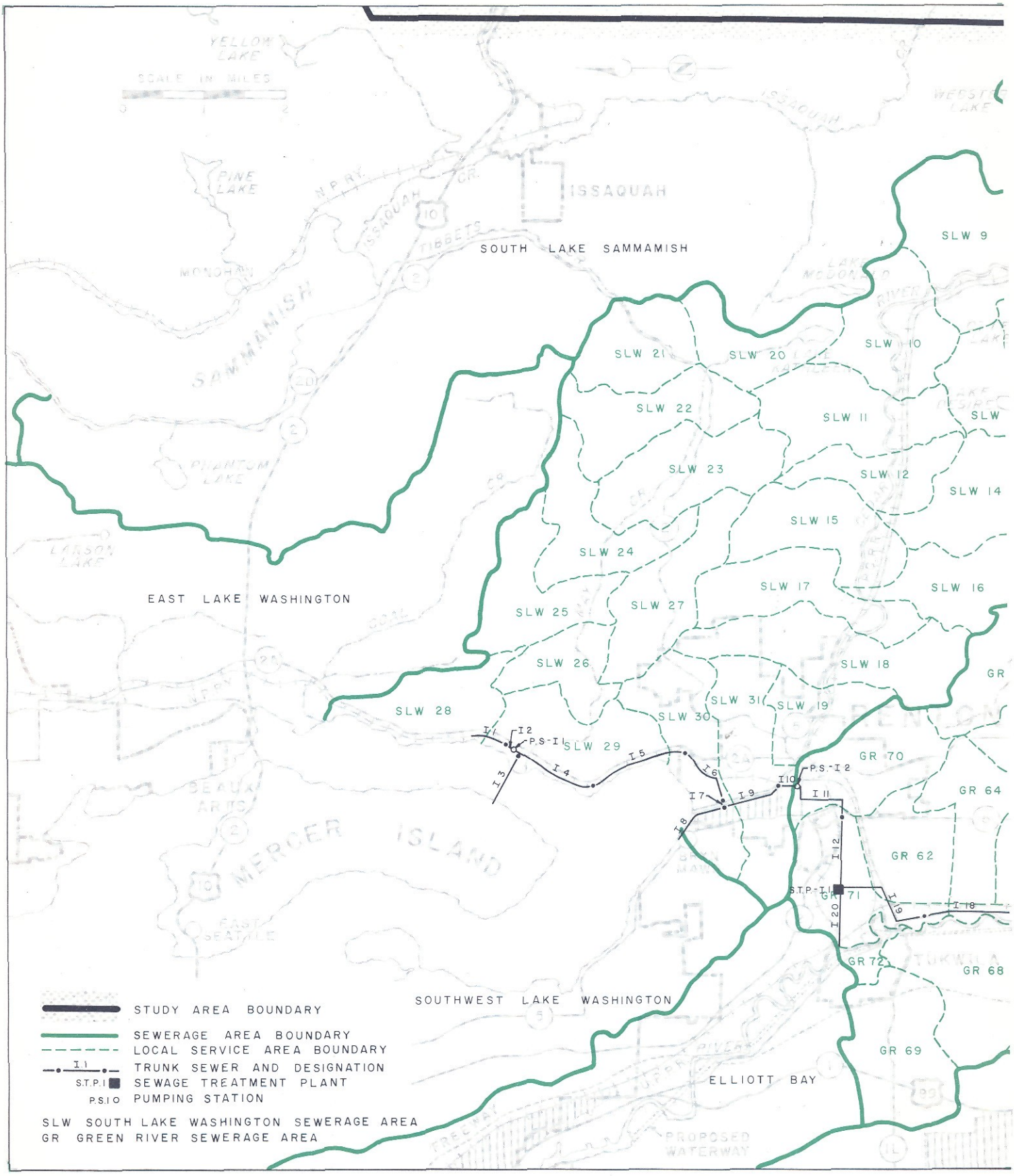
[b] *Includes interest and depreciation calculated by the capital recovery method based on five per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[c] *0.25 per cent of construction costs.*

[d] *Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[e] *Includes allowance of $10.00 per dry ton for hauling of digested sludge for disposal.*

[f] *Includes reduction of $2.25 per dry ton for lagooning of digested sludge.*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SAMMAMISH
PHANTOM LAKE
LARSON LAKE
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MERCER ISLAND
EAST SEATTLE
SOUTHWEST LAKE WASHINGTON
ELLIOTT BAY
PROPOSED WATERWAY
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STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
I.1 TRUNK SEWER AND DESIGNATION
S.T.P.I SEWAGE TREATMENT PLANT
P.S.I O PUMPING STATION
SLW SOUTH LAKE WASHINGTON SEWERAGE AREA
GR GREEN RIVER SEWERAGE AREA

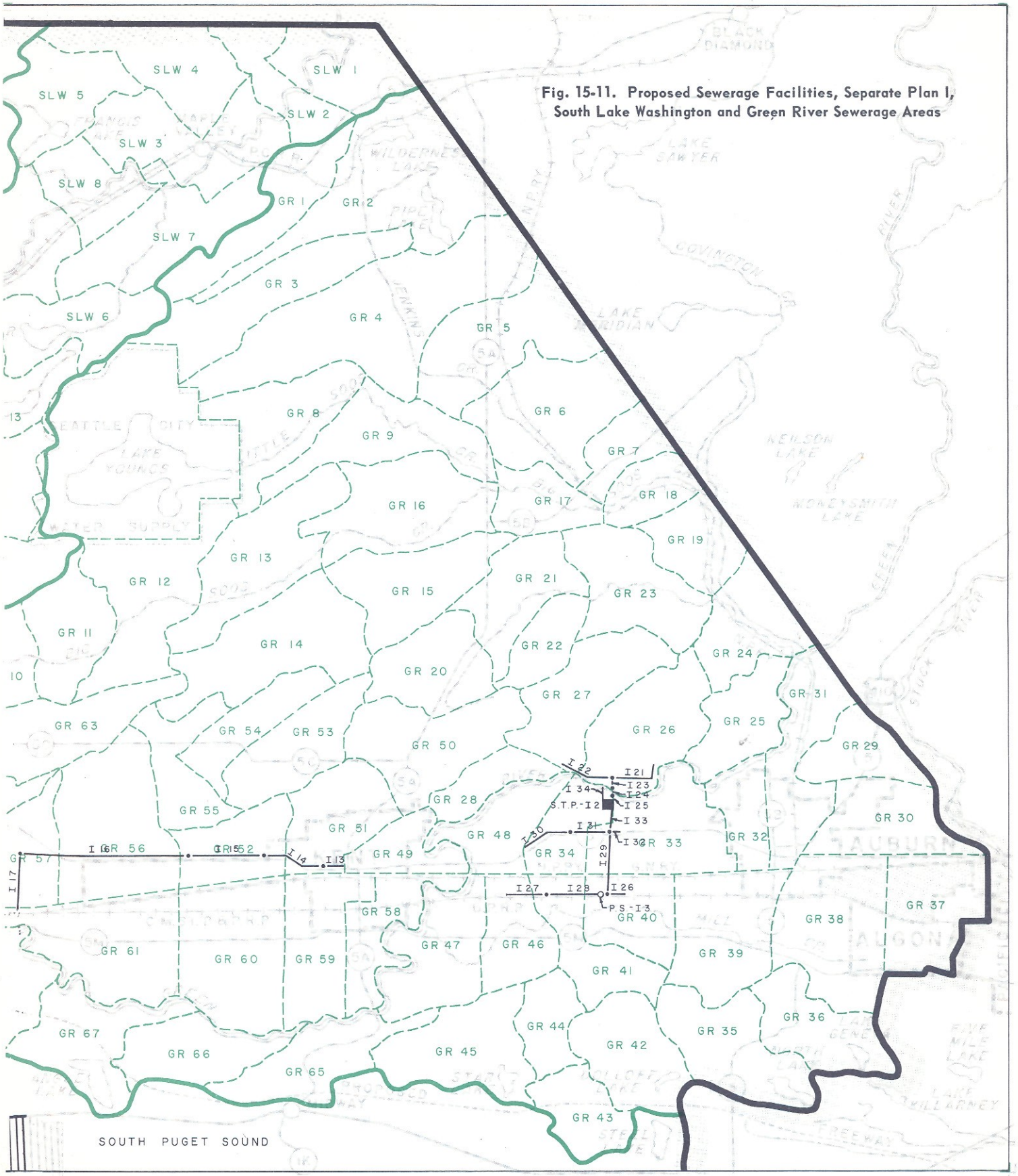


Fig. 15-11. Proposed Sewerage Facilities, Separate Plan I, South Lake Washington and Green River Sewerage Areas

Table 15-22. Description and Estimated Construction Costs, Separate Plan I, South Lake Washington and Green River Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
Renton System				
I-1	0.5	1.5	2,400 ft of 12-in. RC at 0.45%, average cut 10 ft, difficult wet, includes sheeting and dewatering	60,000
I-2	1.2	2.8	600 ft of 15-in. RC at 0.45%, average cut 16 ft, difficult wet, includes sheeting and dewatering	19,000
I-3	1.1	2.8	3,500 ft of 12-in. force main across Lake Washington	94,000
I-4	7.5	20	5,400 ft of 39-in. RC at 0.14%, average cut 7 - 8 ft, difficult wet, includes sheeting and dewatering	215,000
I-5	7.7	20	6,200 ft of 42-in. RC at 0.095%, average cut 8 - 10 ft, difficult wet, includes sheeting and dewatering	274,000
I-6	9.4	24	4,400 ft of 48-in. RC at 0.068%, average cut 10 ft, difficult wet, includes sheeting, dewatering and piling	228,000
I-7	9.4	24	400 ft of twin 20-in. inverted siphons, includes inlet and outlet structures	23,000
I-8	0.5	1.7	3,800 ft of 15-in. RC at 0.34%, average cut 6 - 8 ft, difficult wet, includes connections to existing sewers at Bryn Mawr - Lake Ridge sewage treatment plant which is to be abandoned	67,000
I-9	9.9	26	3,800 ft of 54-in. RC at 0.043%, average cut 21 ft, difficult wet, includes sheeting, dewatering and piling	317,000
I-10	12	30	1,600 ft of 54-in. RC at 0.06%, average cut 25 ft, difficult wet, includes sheeting, dewatering and piling	193,000
I-11	23	61	4,000 ft of 57-in. RC at 0.20%, average cut 20 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	391,000
I-12	24	64	5,300 ft of 57-in. RC at 0.20%, average cut 22 - 25 ft, difficult wet, includes sheeting, dewatering and piling	482,000
I-13	1.2	3.4	1,200 ft of 21-in. RC at 0.11%, average cut 9 ft, difficult wet, includes imported backfill, repaving and dewatering	30,000
I-14	1.9	5.5	4,200 ft of 24-in. RC at 0.15%, average cut 9 ft, difficult wet, includes imported backfill, repaving and dewatering	116,000
I-15	2.3	6.4	5,000 ft of 27-in. RC at 0.12%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	168,000
I-16	4.5-6.5	12-16	10,900 ft of 36-in. RC at 0.072 - 0.14%, average cut 9 - 12 ft, difficult wet, includes imported backfill, repaving and dewatering	360,000
I-17	9.8	24	4,000 ft of 39-in. RC at 0.21%, average cut 14 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	234,000
I-18	13	33	6,500 ft of 57-in. RC at 0.062%, average cut 22 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	675,000
I-19	17	41	6,700 ft of 63-in. RC at 0.045%, average cut 23 - 25 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	757,000
Subtotal, sewers, Renton system				4,703,000
PS-I-1	1.2	2.8	Pumping station, single stage, motor driven with gas engine standby, static lift 13 ft, total head at peak flow 20 ft, structure about 20 ft below ground, includes sheeting and dewatering	60,000

Continued on next page

Table 15-22. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
PS-I-2	12	30	Pumping station, single stage, motor and engine driven, static lift 18 ft, total head at peak flow 25 ft, structure about 35 ft below ground, includes sheeting and dewatering	248,000
Subtotal, pumping stations, Renton system				308,000
STP-I-1	41	101	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	6,234,000
I-20	41	101	3,100 ft of 60-in. RC effluent outfall to Duwamish River, difficult wet, includes sheeting, dewatering and railroad crossing	217,000
Total contract cost, Renton system				11,462,000
Engineering and contingencies, 25 per cent				2,866,000
Total construction cost, Renton system				14,328,000
Auburn System				
I-21	22	55	3,700 ft of 51-in. RC at 0.26%, average cut 17 ft, difficult wet, includes sheeting and dewatering	234,000
I-22	0.6	1.6	3,200 ft of 12-in. RC at 0.55%, average cut 6 - 7 ft, difficult wet, includes dewatering	49,000
I-23	23	57	200 ft of 51-in. RC at 0.28%, average cut 23 ft, difficult wet, includes sheeting and dewatering	16,000
I-24	23	57	400 ft of twin 30-in. inverted siphons, includes inlet and outlet structures	35,000
I-25	23	57	800 ft of 51-in. RC at 0.28%, average cut 28 ft, difficult wet, includes sheeting and dewatering	73,000
I-26	7.1	17	1,100 ft of 36-in. RC at 0.16%, average cut 9 ft, difficult wet, includes dewatering	33,000
I-27	3.4	8.7	2,700 ft of 33-in. RC at 0.067%, average cut 20 ft, difficult wet, includes sheeting and dewatering	140,000
I-28	4.1	11	3,900 ft of 36-in. RC at 0.06%, average cut 29 ft, difficult wet, includes sheeting and dewatering	292,000
I-29	11	27	4,300 ft of 36-in. RC at 0.42%, average cut 21 ft, difficult wet, includes sheeting, dewatering and railroad crossings	245,000
I-30	0.4	1.4	3,300 ft of 12-in. RC at 0.38%, average cut 15 ft, difficult wet, includes imported backfill, repaving and dewatering	97,000
I-31	0.6	1.8	2,500 ft of 15-in. RC at 0.19%, average cut 29 ft, difficult wet, includes imported backfill, repaving and dewatering	137,000
I-32	4.8	11	600 ft of 30-in. RC at 0.19%, average cut 8 ft, difficult wet, includes imported backfill, repaving and dewatering	18,000
I-33	16	36	2,100 ft of 42-in. RC at 0.38%, average cut 30 ft, difficult wet, includes sheeting and dewatering	173,000
Subtotal, sewers, Auburn system				1,542,000
PS-I-3	4.1	11	Pumping station, single stage, motor driven with gas engine standby, static lift 29 ft, total head at peak flow 35 ft, structure about 35 ft below ground, includes sheeting and dewatering	140,000

Continued on next page

Table 15-22. Continued

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
STP-I-2	36	85	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes special foundation	6,651,000
I-34	36	85	500 ft of 54-in. RC outfall to Green River	26,000
Total contract cost, Auburn system				8,359,000
Engineering and contingencies, 25 per cent				2,090,000
Total construction cost, Auburn system				10,449,000
Total construction cost, Plan I				24,777,000

See Fig. 15-11 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

participation in the central sewerage project. Under this project, sewage from the North Lake Sammamish sewerage area would be pumped into the East Lake Washington area, through which it would flow by gravity to the Renton plant of Core Plan B. Sewage from the North Lake Washington and Northwest Lake Washington areas would be conveyed to the West Point plant of Core Plan B for treatment and disposal.

South Lake Washington and Green River Sewerage Areas

Since all sewage from the South Lake Washington sewerage area can be conveyed to a point in the city of Renton at the southwest corner of the area, and since flow can be by gravity from that point into the Green River sewerage area, it is evident that both areas should be combined for the planning of a separate sewerage project. Only one basic alternative is available for such a project. This would involve construction of (1) a complete type treatment plant about one and one-half miles north of Auburn to serve the southern and eastern portions of the Green River sewerage area, and (2) construction of a complete type treatment plant at a site west of Renton, as proposed under Core Plan B, to serve the remainder of the Green River area, as well as the South Lake Washington area.

Two variations in the basic alternative were considered and are designated in the following discussion as Plan I and Plan II. Plan I provides capacity in the Renton plant for the South Lake Washington and the northern portion of the Green River areas only; Plan II provides capacity in the Renton plant for the North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, as well as for the South Lake Washington and the northern portion of the Green River areas.

Plan I. Locations of sewers, pumping stations, treatment works and outfalls proposed under this plan are shown in Fig. 15-11. These facilities are described in Table 15-22 and are estimated to cost $24,777,000.

Intercepting sewers for the Renton system include two branches, a north and a south. Starting at Hazelwood, the north branch would follow the route of the Northern Pacific Railroad to Renton and would go through Renton to the treatment plant west of the city. In addition to picking up sewage from the South Lake Washington sewerage area, this branch would intercept flow from the south half of Mercer Island in the East Lake Washington area and from the southern portion of the Southwest Lake Washington area. To avoid excessive cuts, two pumping stations would be required.

The south branch would extend southward from the treatment plant and would provide service to that portion of the Green River valley north of where the river flows from east to west. It would also serve areas to the east and west which drain directly into the valley.

Secondary treatment by the activated sludge process would be provided at the Renton plant for an ultimate average flow of 41 mgd, with a peak hydraulic capacity of 101 mgd. Except for capacity, and consequently the size of the units, this plant would be identical to the Renton plant proposed under Core Plan B. Chlorinated effluent would be discharged to Duwamish River through a 60-inch diffuser-equipped outfall.

Intercepting sewers for the Auburn plant were laid out to serve the Green River valley south of where the river flows from east to west. They were laid out also to serve areas on the east and west slopes of the valley, as well as the entire eastern portion of the Green River sewerage area, which portion is drained by Big

Soos Creek. Secondary treatment by the activated sludge process would be provided at the Auburn plant for an ultimate average dry weather flow of 36 mgd, with a peak hydraulic capacity of 85 mgd. Plant units would consist of preaeration and primary sedimentation tanks, aeration tanks, secondary sedimentation tanks, chlorine contact tanks, separate sludge digestion tanks and other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons. Chlorinated effluent would be discharged to Green River.

Plan II. Descriptions of facilities proposed under Plan II are given in Table 15-23. As indicated in this table, the apportioned construction cost of Plan II to the South Lake Washington and Green River sewerage areas amounts to a total of $20,343,000.

Two interceptor sewers, a north and a south, are required for the Renton system under Plan II. Of these, the north interceptor would be identical to that proposed for Core Plan B (Fig. 15-4), while the south interceptor would be identical to that proposed under Plan I (Fig. 15-11). Except for capacity, the Renton

Table 15-23. Description and Estimated Construction Costs, Separate Plan II, South Lake Washington and Green River Sewerage Areas

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
Renton System				
–	–	–	Sewers, same as B-2, Core Plan B, see Fig. 15-4 and Table 15-4	62,000[b]
–	–	–	Sewers, same as S-15, S-17, S-18, S-20, S-21, S-22, feeder sewers for Core Plan B, see Fig. 15-7 and Table 15-11	685,000[c]
–	–	–	Sewers, same as I-13 - I-19, Plan I, see Fig. 15-11 and Table 15-22	2,340,000
Subtotal, sewers, Renton system				3,087,000
STP	107	275	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as necessary operation, administration and laboratory facilities	4,674,000[d]
Outfall	107	275	3,100 ft of twin 72-in. RC outfall sewers, difficult wet, includes sheeting, dewatering and railroad crossing	154,000[e]
Total contract cost,[f] Renton system				7,915,000
Engineering and contingencies, 25 per cent				1,979,000
Total construction cost,[f] Renton system				9,894,000
Auburn System				
–	–	–	Sewers, same as I-21 - I-33, Plan I, see Fig. 15-11 and Table 15-22	1,542,000
–	–	–	Pumping station, same as PS-I-3, Plan I, see Fig. 15-11 and Table 15-22	140,000
–	–	–	Sewage treatment plant, same as STP-I-2, see Fig. 15-11 and Table 15-22	6,651,000
–	–	–	Outfall, same as I-34, Plan I, see Fig. 15-11 and Table 15-22	26,000
Total contract cost, Auburn system				8,359,000
Engineering and contingencies, 25 per cent				2,090,000
Total construction cost, Auburn system				10,449,000
Total construction cost,[f] Plan II				20,343,000

[a] *Expressed as average dry weather flow and maximum wet weather flow.*

[b] *Apportioned cost to the two sewerage areas; total cost of facilities $271,000.*

[c] *Apportioned cost to the two sewerage areas; total cost of facilities $7,936,000.*

[d] *Apportioned cost to the two sewerage areas; total cost of facility $13,127,000.*

[e] *Apportioned cost to the two sewerage areas; total cost of facility $467,000.*

[f] *Cost to the two sewerage areas.*

Table 15-24. Comparison of Construction Costs, Core Plan B and Separate Plans, South Lake Washington and Green River Sewerage Areas

Facility	Construction cost,[a] $1,000								
	Core Plan B[b]			Plan I[c]			Plan II[c]		
	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River
Sewers	11,485	905	10,580	7,272	2,391	4,881	5,786	905	4,881
Pumping stations	586	–	586	508	333	175	175	–	175
Sewage treatment plants	10,845	2,790	8,055	15,529	3,608	11,921	14,157	2,921	11,236
Outfalls	311	86	225	290	145	145	225	104	121
Totals	23,227	3,781	19,446	23,599	6,477	17,122	20,343	3,930	16,413

Plan I and II propose concentration of the sewage of the South Lake Washington and Green River Sewerage Areas in two secondary type treatment plants, one to be located north of Auburn and the second at the Renton site. The two plans differ only in the size of plant required at the Renton site.

[a]*Includes engineering and contingencies.*

[b]*Apportioned cost to the two areas, see Table 15-13; includes feeder sewers.*

[c]*Apportioned cost to the two areas.*

treatment plant would be identical to that proposed at this site under Core Plan B. Similarly, chlorinated effluent would be discharged to Duwamish River.

For the Auburn plant, intercepting sewers, pumping stations, treatment units and effluent disposal facilities would be identical to those proposed under Plan I.

Comparison of Plans. Apportioned construction costs to the two sewerage areas of the central sewerage project are given in Table 15-24, as are the estimated construction costs of the two separate plans just considered. Estimates of the total cost to the two areas range from $20,343,000 for Plan II to $23,599,000 for Plan I. For the South Lake Washington area, the costs range from $3,781,000 for participation in the central sewerage project to $6,477,000 for Plan I. For the Green River area, the costs range from $16,413,000 for Plan II to $19,446,000 for participation in the central project.

Estimated average annual costs for the core plan system and for each of two separate plans are given in Table 15-25. As there indicated, the total annual cost to the two areas is $1,586,000 for Plan II, $1,646,000 for the central project, and $1,841,000 for Plan I. For the South Lake Washington area, the cost for participation in the central sewerage project is the lowest and amounts to $296,000 per year. For the Green River area, Plan II is the least costly at $1,279,000 per year.

Selection of Most Acceptable Plan. On the basis of the cost estimates just given, it appears that Plan II is the most acceptable from the standpoint of over-all economy. Actually, however, the annual cost of this plan is only slightly lower than that for participation in the central sewerage project. This is true with respect not only to the total cost to the two sewerage areas but to the cost to the Green River area individually.

It will be seen that the difference in costs between Plan II and the central project amounts to $60,000 per year, or less than 4 per cent of the annual cost of Plan II. In the case of the Green River area, the difference is $71,000 per year, or less than 6 per cent of the Plan II cost. With annual costs close enough to minimize them as a decisive element, other pertinent factors must be taken into account and decisions made accordingly.

As shown in Table 15-25, operation and maintenance costs would be $69,000 per year lower for the central sewerage project than for Plan II. This difference is attributable to the higher cost of operating two plants rather than one. Furthermore, as the costs of labor and materials increase, operating costs for two plants would increase at a rate higher than that for a single plant and thus would tend to reduce the presently indicated difference in total annual costs.

In addition to lower operating costs, experience has shown generally that a single plant is more likely to produce a consistently satisfactory effluent and is less subject to plant upsets of a degree requiring the bypassing of raw sewage. These advantages, as they relate to the problem in question, are of particular importance.

Disposal conditions in Green River at the site of the Auburn plant are much less favorable than they are at Renton (Chapter 12). This means that plant upsets, as well as changed conditions with respect to anticipated loading, would cause a more serious impact at the Auburn location. In addition, discharge

of plant effluent at two locations would require a higher degree of BOD removal than would be the case if all the sewage were conveyed to a single plant at Renton (Chapter 12).

In conclusion, it is evident first that the choice between central sewerage and Plan II for the South Lake Washington and Green River sewerage areas tends to favor Plan II from the standpoint of total annual cost. On the other hand, it is equally evident that the advantages of more uniform operation and better receiving water conditions obtainable under the central project are sufficient to outweigh its slight disadvantage costwise. For that reason, it is recommended that the two areas be sewered to the Renton plant in the manner called for under Core Plan B.

Southwest Lake Washington, Elliott Bay and , and Lake Union Sewerage Areas

These three areas are now served for the most part by the city of Seattle. As such, the Lake Union and a part of the Southwest Lake Washington area are tributary to the North Trunk sewer of the city and thus would be tributary to the West Point plant proposed under Core Plan B. As a consequence, there are no feasible alternatives for the independent sewerage of these areas.

Since Elliott Bay and most of the remainder of the Southwest Lake Washington sewerage areas are sewered to the waterfront along Duwamish River and Elliott Bay, they are tributary to the required waterfront interceptor. On that basis, the only alternative to Core Plan B for these areas would be the provision of an independent plant at a site along the Duwamish River or Elliott Bay waterfront. This alternative was considered earlier as a part of Core Plan D, whereby it was determined that treatment at the West Point plant under Core Plan B would provide the most satisfactory solution.

South Puget Sound Sewerage Area

Because the topography of the South Puget Sound sewerage area requires high head pumping to con-

Table 15-25. Comparison of Annual Costs, Core Plan B and Separate Plans, South Lake Washington and Green River Sewerage Areas

Cost item	Average annual cost, $1,000								
	Core Plan B[a]			Plan I[b]			Plan II[b]		
	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River
Fixed costs[c]									
Sewers and outfalls	647	55	592	414	139	275	330	55	275
Pumping stations and treatment plants	744	182	562	1,043	256	787	932	190	742
	1,391	237	1,154	1,457	395	1,062	1,262	245	1,017
Maintenance and operation									
Sewers[d]	29	2	27	19	6	13	15	2	13
Pumping stations[e]	27	–	27	12	8	4	4	–	4
Sewage treatment plants[e]	162	46	116	308[f]	72	236	260[f]	49	211
Effluent chlorination[e]	37	11	26	45	11	34	45	11	34
	255	59	196	384	97	287	324	62	262
Total annual cost	1,646	296	1,350	1,841	492	1,349	1,586	307	1,279

Plans I and II propose concentration of the sewage of the South Lake Washington and Green River Sewerage Areas in two secondary type treatment plants, one to be located north of Auburn and the second at the Renton site. The two plans differ only in the size of plant required at the Renton site.

[a]*Apportioned cost to the two areas, see Table 15-14; includes feeder sewers.*

[b]*Apportioned cost to the two areas.*

[c]*Includes interest and depreciation calculated by the capital recovery method based on five per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[d]*0.25 per cent of construction cost.*

[e]*Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[f]*Includes reduction of $2.25 per dry ton for lagooning of digested sludge.*

vey sewage out of the area, no capacity was provided for it in the core plan system. Various plans for providing independent sewerage were studied and the plan found to be most feasible was compared with that of the central sewerage project for the area as a whole.

As stated in Chapter 14, the South Puget Sound sewerage area is divided topographically into five major subareas, namely, Redondo Beach, Des Moines, Miller Creek, Southwest Suburban and West Seattle. In the Redondo Beach and West Seattle subareas, the only possible independent project is that of providing for sewage collection, treatment and disposal within each subarea itself. For the other three subareas, studies were made of five possible alternatives. These are:

Plan I - Conveyance of sewage to a plant in each subarea, with effluent disposal in Puget Sound.

Plan II - Conveyance of sewage from the Southwest Suburban and Miller Creek subareas to a plant in the Miller Creek subarea and treatment of the sewage from the Des Moines subarea at a separate plant in that area. Effluent from both plants would be disposed of in Puget Sound.

Table 15-26. Summary of Construction and Annual Costs, Alternative Sewerage Plans, Des Moines, Miller Creek and Southwest Suburban Subareas, South Puget Sound Sewerage Area

Facility	Construction cost,[a] $1,000				
	Plan I	Plan II	Plan III	Plan IV	Plan V
Sewers	1,210	1,636	2,205	1,556	1,782
Pumping stations	–	325	970	594	510
Sewage treatment plants	3,767	3,598	3,914	3,675	4,035
Outfalls	651	659	275	270	308
Total construction cost	5,628	6,218	7,364	6,095	6,635
Cost Item	**Average annual cost, $1,000**				
	Plan I	Plan II	Plan III	Plan IV	Plan V
Fixed costs[b]					
Sewers and outfalls	102	126	136	100	114
Pumping stations and treatment plants	245	255	318	278	296
	347	381	454	378	420
Maintenance and operation					
Sewers[c]	5	6	6	5	5
Pumping stations[d]	–	26	79	35	53
Sewage treatment plants[d]	142	114	98	132	127
Effluent chlorination[d]	18	18	18	18	18
	165	164	201	190	203
Total annual cost	512	545	655	568	623

Plan I proposes concentration of the sewage in a separate plant in each subarea with effluent disposal to Puget Sound.

Plan II proposes concentration of the sewage of the Southwest Suburban and Miller Creek subareas in a plant located in the Miller Creek subarea and concentration of the sewage of the Des Moines subarea in a separate plant in that area.

Plan III proposes concentration of the sewage of all three subareas in a plant located in the Des Moines subarea with effluent disposal to Puget Sound.

Plan IV proposes concentration of the sewage of the northern portion of the Miller Creek subarea, along with the sewage of the Southwest Suburban subarea, in the existing plant of the Southwest Suburban Sewer District and concentration of the sewage of the southern portion of the Miller Creek subarea, along with the sewage of the Des Moines subarea, in a plant located in the Des Moines subarea.

Plan V proposes concentration of the sewage of the Southwest Suburban subarea in the existing plant of the Southwest Suburban Sewer District and concentration of the sewage of the Miller Creek and Des Moines subareas in a plant in the Des Moines subarea.

[a]*Includes engineering and contingencies.*

[b]*Includes interest and depreciation calculated by the capital recovery method based on 5 per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[c]*0.25 per cent of construction cost.*

[d]*Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

Plan III - Conveyance of sewage from all three subareas to a plant in the Des Moines subarea, with effluent disposal to Puget Sound.

Plan IV - Conveyance of sewage from the northern portion of the Miller Creek subarea, along with that from the Southwest Suburban subarea, to the existing plant of the Southwest Suburban Sewer District and treatment of sewage from the southern portion of the Miller Creek subarea, together with that from the Des Moines subarea, at a plant in the Des Moines subarea. Effluent from both plants would be disposed of to Puget Sound. Existing facilities of the Southwest Suburban Sewer District in the vicinity of Lake Burien in the Miller Creek subarea would continue in use as at present.

Plan V - Conveyance of sewage from the Southwest Suburban subarea to the present plant of the Southwest Suburban Sewer District, and treatment of sewage from the Miller Creek and Des Moines subareas at a plant in the Des Moines subarea. Effluent from both plants would be disposed of in Puget Sound.

Total construction costs for the five plans range from $5,628,000 for Plan I to $7,364,000 for Plan III (Table 15-26). Total annual costs range from $512,000 for Plan I to $655,000 for Plan III (Table 15-26). Based on these costs, it is apparent that the most satisfactory independent sewerage plan for the three subareas is that proposed under Plan I whereby sewage collection, treatment, and disposal facilities would be provided in each subarea.

Locations of all trunk sewers, pumping stations, treatment works and outfalls required to provide independent sewerage for the five subareas are shown in Fig. 15-12. Descriptions of the facilities and their estimated construction costs are given in Table 15-27.

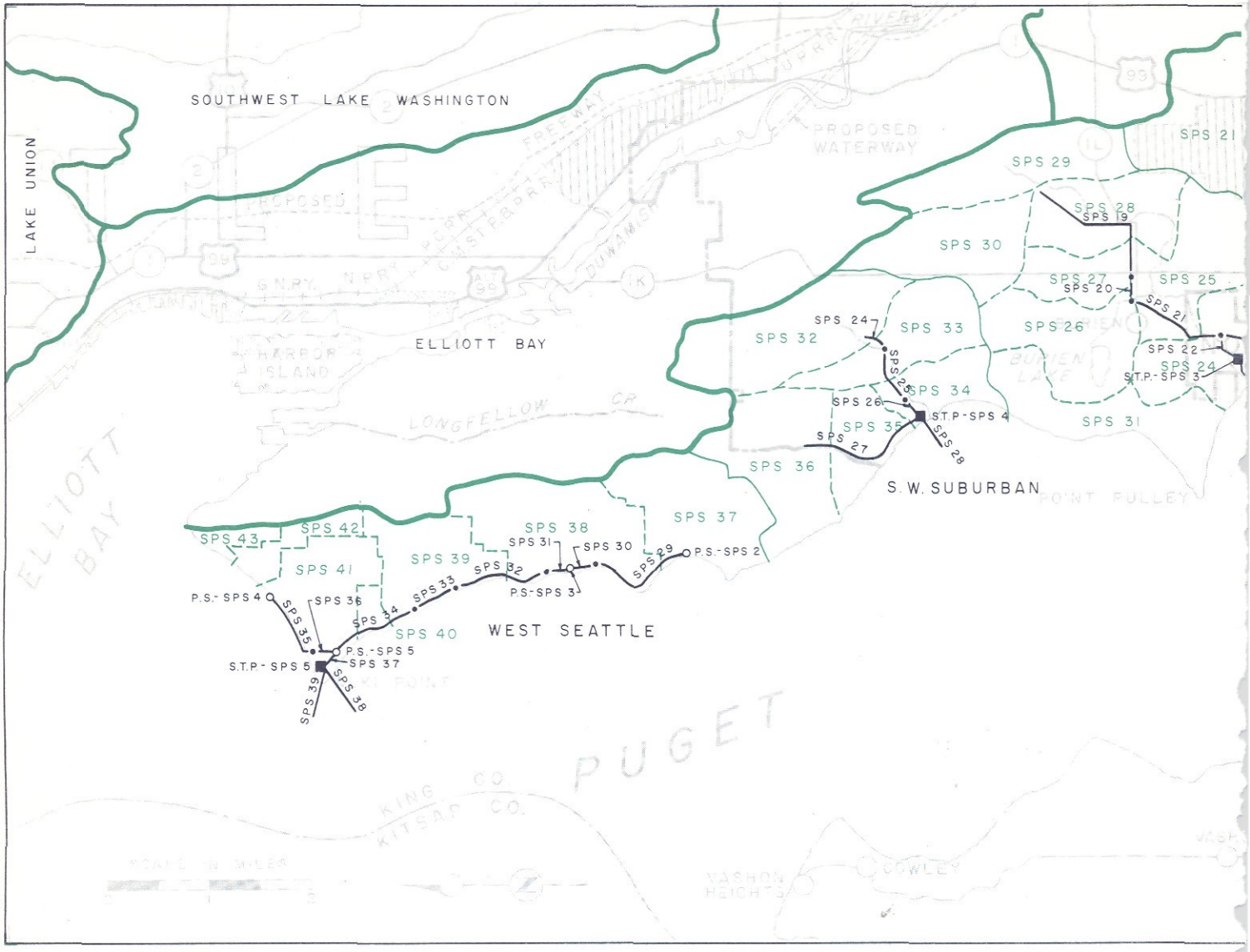
Redondo Beach Subarea. Treatment and disposal of the sewage of the Redondo Beach subarea would be obtained in a plant located about one and one-half miles from Lakota. Trunk sewers would consist of three branches, west, central and east. Of these, the west branch would be a high elevation sewer from the plant to Lakota, from which point it would turn south to serve the area south and west of Lakota. Because of the relatively high elevation of the sewer, local pumping would be required along the waterfront to bring in sewage from areas which cannot be served by gravity. The central branch would run generally southward from the plant and would serve Mirror Lake and surrounding areas. The east branch would consist of a waterfront interceptor to serve Redondo Beach and Woodmont Beach, a pumping station, and a high elevation interceptor to the plant. This branch would serve Steel Lake and surrounding areas. As in the west branch, some local pumping would be required.

Treatment would be provided in a primary type plant capable of accommodating an ultimate average dry weather flow of 5.0 mgd, with a peak hydraulic capacity of 14 mgd. Plant units would consist of influent pumps, preaeration and primary sedimentation tanks, separate sludge digestion tanks, chlorine contact tanks, and other necessary structures and appurtenances. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 3,000 feet offshore to a water depth of 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 1,500 feet offshore in water at a depth of about 120 feet.

Des Moines Subarea. Treatment and disposal of the sewage of the Des Moines subarea would be obtained in a plant located south of Des Moines. Trunk sewers would consist of a north and south branch, of which the south branch would extend southward from the plant through the community of Zenith and then eastward to U.S. Highway 99. Along much of its route, this branch would be laid at a high elevation, with the result that local pumping would be required in some areas. The north branch would be routed northward through the community of Des Moines and then northeastward to a point south of the Seattle-Tacoma International Airport. This branch would serve the airport and all its facilities.

Because of disposal conditions offshore from Des Moines, complete treatment would be required for an ultimate average flow of 6.5 mgd, with a peak hydraulic capacity of 16 mgd. Plant units would consist of influent pumps, preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since a detention time of over 20 minutes would be available in the outfall at ultimate average flow. Digested sludge, after passing through a washer, would be disposed of in Puget Sound through a submarine line extending approximately 2,700 feet offshore to a water depth of 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 1,300 feet offshore at a depth of about 60 feet.

Miller Creek Subarea. Treatment and disposal of the sewage of the Miller Creek subarea would be obtained in a plant located in the northern part of the city of Normandy Park. Two trunk sewers, a north and a south, would be required. Of these, the south trunk would serve the southern and central portions of Normandy Park as well as adjacent tributary areas. The north trunk would be laid generally along Miller Creek to beyond Five Corners and would serve the highly developed area around Lake Burien plus areas



to the north as far as White Center. It would also serve the northern portion of the city of Normandy Park. Some local pumping would be required to serve low areas along the routes of both trunks.

Treatment would be provided in a primary type plant laid out for an ultimate average flow of 7.5 mgd, with a peak hydraulic capacity of 18 mgd. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since a detention time of over 20 minutes would be available in the outfall at the ultimate average flow. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 3,300 feet offshore to a water depth of 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 2,900 feet offshore at a depth of about 200 feet.

Southwest Suburban Subarea. Treatment and disposal of the sewage of the Southwest Suburban subarea would continue to be obtained in the present plant of the Southwest Suburban Sewer District. Existing trunks extending eastward from the plant are of adequate capacity for expected future flows and would be fully utilized under the program herein proposed. A new trunk laid northward from the plant would be required to serve the Roxbury Heights area. The present primary type treatment plant has sufficient capacity for the expected ultimate average flow of 4.0 mgd and peak flow of 8.5 mgd.

Chlorinated effluent is presently being discharged to Puget Sound through a 36-inch submarine outfall terminating 600 feet offshore at a depth of 60 feet. As pointed out in Chapter 11, this method of disposal is inadequate in that it fails to provide the required degree of protection along the adjacent shoreline. That being the case, it appears that the present plant

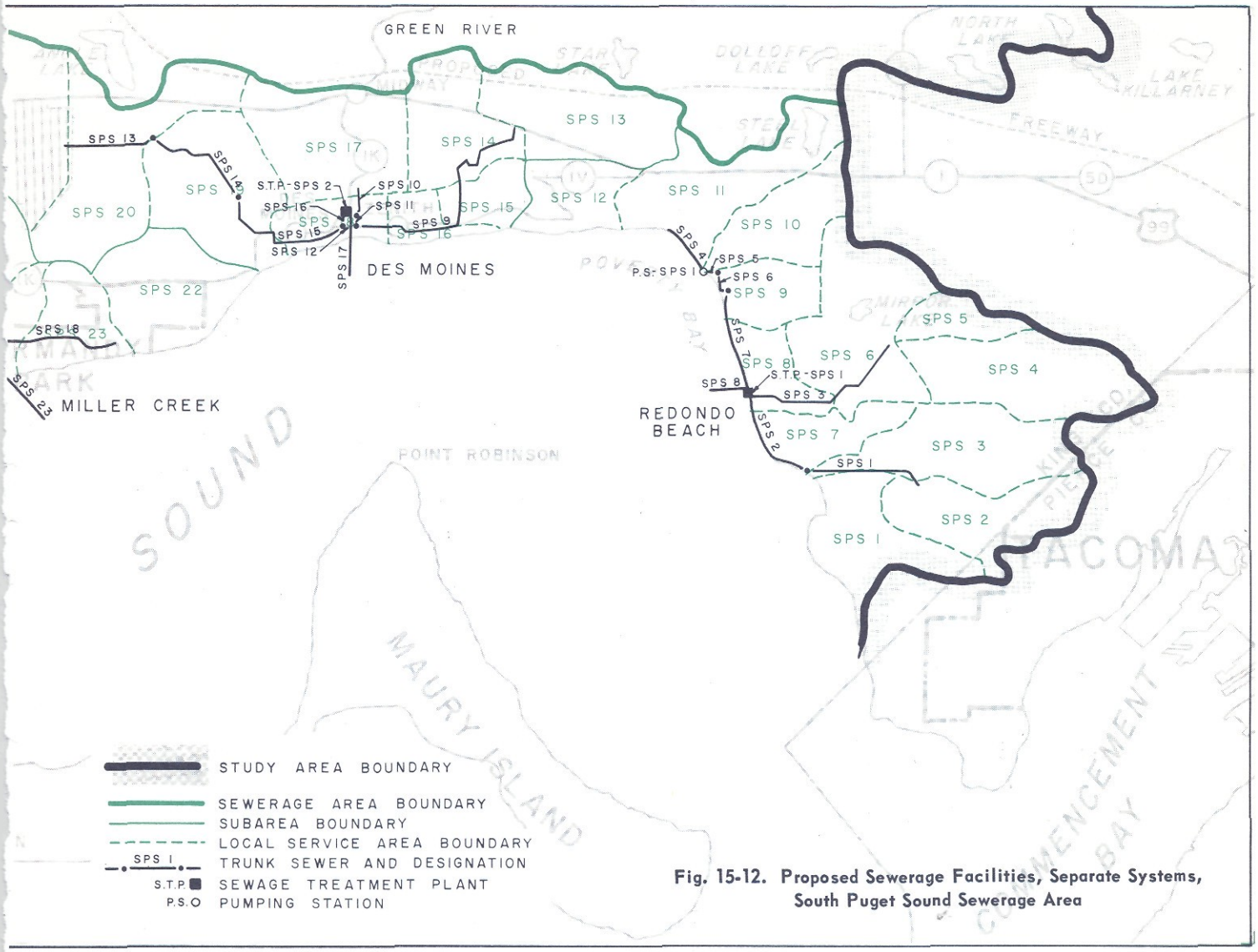


Fig. 15-12. Proposed Sewerage Facilities, Separate Systems, South Puget Sound Sewerage Area

will have to be expanded and modified to provide for secondary treatment. Before embarking on such a project, however, consideration should be given to the possibility of extending the outfall to a depth suitable for disposal of disinfected primary effluent.

In the event that an outfall extension is found to be impracticable or uneconomical, secondary treatment would be the only alternative and would require the addition of trickling filters and secondary sedimentation tanks. In addition, certain alterations to the existing plant would be required to permit the discharge of digested sludge to Puget Sound and the utilization of sludge gas for sludge heating purposes. These alterations would consist of piping changes, installation of necessary equipment, and construction of a sludge outfall line extending 3,300 feet offshore to a water depth of 400 feet. It would be necessary also to extend the effluent outfall an additional 200 feet to a water depth of 85 feet. Estimated costs of the required secondary facilities and the plant alterations amount to $568,000 exclusive of engineering and contingencies (Table 15-27).

West Seattle Subarea. Treatment and disposal of the sewage of the West Seattle subarea will be obtained in the Alki Point plant of the city of Seattle. This plant is now under construction and is scheduled to go into operation late in 1958. Two waterfront interceptors, one north and the other south of the plant, will serve this subarea. In addition, the north interceptor will receive sewage from the most westerly local service area of the Elliott Bay sewerage area. Three pumping stations will be required along the routes of the interceptors, two on the south and one on the north. In addition, a pumping station at 63rd Avenue S.W. will lift all sewage into the treatment plant. Since the West Seattle subarea is served by combined sewers, interceptor design is based on an overflow frequency of 12

Table 15-27. Description and Estimated Construction Costs, Separate Systems for South Puget Sound Sewerage Area

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Redondo Beach Subarea				
SPS-1	1.1	3.1	5,500 ft of 12-in. RC at 2.7%, average cut 6 ft, dry to moderately wet, includes access road	62,000
SPS-2	1.8	4.9	5,200 ft of 21-in. RC at 0.25%, average cut 6 ft, dry to wet, includes imported backfill, repaving and access road	86,000
SPS-3	0.9-1.6	2.4-4.3	8,400 ft of 18-in. RC at 0.3 - 4.9%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	111,000
SPS-4	0.9	2.5	3,400 ft of 18-in. RC at 0.18%, average cut 6 ft, dry to wet, includes imported backfill and repaving	53,000
SPS-5	1.3	3.4	800 ft of 12-in. force main	5,000
SPS-6	1.5	3.8	1,500 ft of 15-in. RC at 1.3%, average cut 6 ft, dry to moderately wet, includes imported backfill	19,000
SPS-7	1.5-1.8	4.0-4.8	6,000 ft of 18-in. RC at 0.43 - 2.5%, average cut 6 ft, dry to wet, includes imported backfill, repaving and access road	85,000
Subtotal, sewers				421,000
PS-SPS-1	1.3	3.4	Pumping station, single stage, motor driven, static lift 70 ft, total head at peak flow 90 ft, structure about 10 ft below ground	88,000
STP-SPS-1	5.0	14	Sewage treatment plant, primary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 3,000 ft of 6-in. outfall sludge line to a water depth of 400 ft	821,000
SPS-8	5.0	14	1,900 ft of 30-in. RC effluent outfall to water depth of 120 ft, includes 400 ft land section, and diffuser section over last 90 ft	217,000
Total contract cost, Redondo Beach subarea				1,547,000
Engineering and contingencies, 25 per cent				387,000
Total construction cost, Redondo Beach subarea				1,934,000
Des Moines Subarea				
SPS-9	0.7-1.8	1.8-4.3	14,900 ft of 15-in. RC at 0.24 - 4.8%, average cut 6 ft, dry to moderately wet, includes imported backfill, repaving and access road	189,000
SPS-10	1.0	2.5	1,700 ft of 12-in. RC at 2.2%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	19,000
SPS-11	1.2	2.8	600 ft of 15-in. RC at 0.5%, average cut 8 ft, dry to moderately wet	7,000
SPS-12	2.9	7.1	1,000 ft of 15-in. RC at 4.2%, average cut 6 ft, wet	15,000
SPS-13	1.3	3.2	5,200 ft of 15-in. RC at 2.0%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	62,000
SPS-14	2.5-3.0	6.1-7.2	5,400 ft of 21-in. RC at 0.74 - 3.2%, average cut 6 ft, dry to moderately wet	72,000
SPS-15	3.2-3.6	7.6-8.6	7,300 ft of 24-in. RC at 0.43 - 4.1%, average cut 6 - 8 ft, dry to wet, includes imported backfill and repaving	134,000
SPS-16	6.4	16	600 ft of 30-in. RC at 0.36%, average cut 12 ft, wet	15,000
Subtotal, sewers				513,000

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Table 15-27. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
STP-SPS-2	6.5	16	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 2,700 ft of 6-in. outfall sludge line to a water depth of 400 ft	1,479,000
SPS-17	6.5	16	2,500 ft of 30-in. RC effluent outfall to water depth of 60 ft, includes 1,200 ft land section, and diffuser section over last 45 ft	153,000
Total contract cost, Des Moines subarea				2,145,000
Engineering and contingencies, 25 per cent				536,000
Total construction cost, Des Moines subarea				2,681,000
Miller Creek Subarea				
SPS-18	1.2	2.8	7,000 ft of 12-in. RC at 1.5%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	79,000
SPS-19	2.0-2.6	4.5-6.0	8,400 ft of 21-in. RC at 0.22 - 0.9%, average cut 6 - 9 ft, dry to moderately wet, includes imported backfill and repaving	151,000
SPS-20	2.9	6.8	1,000 ft of existing 24-in. RC at 0.3%	Existing
SPS-21	4.1-5.9	9.4 - 14	5,300 ft of 24-in. RC at 0.65 - 2.1%, average cut 6 - 8 ft, dry to wet, includes imported backfill, repaving and connections to existing sewers at Southwest Suburban Sewer District pumping station No. 6 which is to be abandoned	109,000
SPS-22	7.4	17	1,700 ft of 27-in. RC at 1.2%, average cut 7 ft, wet, includes imported backfill and repaving	42,000
Subtotal, sewers				381,000
STP-SPS-3	7.5	18	Sewage treatment plant, primary type, includes facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as necessary operation and maintenance facilities, includes 3,300 ft of 6-in. outfall sludge line to a water depth of 400 ft	966,000
SPS-23	7.5	18	4,100 ft of 27-in. RC outfall to water depth of 200 ft, includes 1,200 ft land section, and diffuser section over last 150 ft	323,000
Total contract cost, Miller Creek subarea				1,670, 000
Engineering and contingencies, 25 per cent				418,000
Total construction cost, Miller Creek subarea				2,088,000
Southwest Suburban Subarea				
SPS-24	1.4	3.2	1,400 ft of existing 18-in. and 24-in. RC at 0.67 - 6.3%	Existing
SPS-25	1.9	4.2	2,000 ft of existing 30-in. RC at 5.2%	Existing
SPS-26	2.1	4.6	800 ft of existing 36-in. RC at 1.5%	Existing
SPS-27	1.0-1.5	2.1-3.3	6,600 ft of 12-in. RC at 1.5 - 2.4%, average cut 6 - 8 ft, dry to moderately wet, includes imported backfill, repaving and access road.	74,000
Subtotal, sewers				74,000
STP-SPS-4[c]	4.0	8.5	Sewage treatment plant, existing primary type to be converted to secondary type, includes facilities for preaeration and grit removal, comminution, primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and effluent chlorination, as well as facilities for operation and maintenance functions, includes 3,300 ft of 6-in. outfall sludge line to water depth of 400 ft	568,000

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Table 15-27. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
SPS-28	4.0	8.5	1,900 ft of 36-in. RC effluent outfall to water depth of 85 ft, includes 1,100 ft existing land section, 600 ft existing submarine section to water depth of 60 ft, and 200-ft extension with diffuser sections over last 90 ft	45,000
Total contract cost, Southwest Suburban subarea				687,000
Engineering and contingencies, 25 per cent				172,000
Total construction cost, Southwest Suburban subarea				859,000
West Seattle Subarea[d]				
SPS-29	1.6	23	6,200 ft of existing 30-in. force main	Existing
SPS-30	1.6	23	900 ft of existing 42-in. at 0.12%	Existing
SPS-31	3.5	39	1,400 ft of existing twin 27-in. force mains	Existing
SPS-32	3.5	39	5,000 ft of existing 36-in. pressure sewer	Existing
SPS-33	4.5	42	2,000 ft of existing 54-in. pressure sewer	Existing
SPS-34	4.5-4.7	42	4,800 ft of existing 54-in. at 0.11%	Existing
SPS-35	1.3	17	4,400 ft of existing 24-in. force main	Existing
SPS-36	1.3	17	1,400 ft of existing 30-in. at 0.45%	Existing
SPS-37	7.2	60	1,200 ft of existing parallel 24-in. and 42-in. force mains	Existing
PS-SPS-2	1.6	23	Pumping station, existing, static lift 30 ft, total head at peak flow 70 ft	Existing
PS-SPS-3	3.5	39	Pumping station, existing, static lift 43 ft, total head at peak flow 70 ft	Existing
PS-SPS-4	1.3	17	Pumping station, existing, static lift 17 ft, total head at peak flow 64 ft	Existing
PS-SPS-5	7.2	60	Pumping station, existing, static lift 26 ft, total head at peak flow 38 ft	Existing
STP-SPS-5[e]	7.2	60	Sewage treatment plant, existing primary type, includes facilities for preaeration and grit removal, comminution, primary sedimentation, sludge digestion, and effluent chlorination, as well as facilities for operation and maintenance functions. Cost is for 3,300 ft of 6-in. outfall sludge line to a water depth of 400 ft	66,000
SPS-38	7.2	60	1,400 ft of existing 42-in. RC effluent outfall to water depth of 85 ft	Existing
SPS-39	7.2	20	2,500 ft of 36-in. RC effluent outfall to water depth of 210 ft, includes diffuser section over last 500 ft	329,000
Total contract cost, West Seattle subarea				395,000
Engineering and contingencies, 25 per cent				99,000
Total construction cost, West Seattle subarea				494,000

See Fig. 15-12 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*No construction cost allowed for facilities already constructed or for facilities for which money has been allocated.*

[c]*Plant designed on basis of 1.3 hours detention in sedimentation tanks at 8.2 mgd flow. Detention time at ultimate average dry weather flow of 4.0 mgd will be 2.6 hours.*

[d]*West Seattle subarea presently served by combined sewers. Interceptor capacity is based on 12 overflows per summer.*

[e]*Plant designed on basis of 0.5 hours detention in sedimentation tanks at 60 mgd flow. Detention time at ultimate average dry weather flow of 7.2 mgd will be 5 hours.*

times per summer. If this frequency proves to be unsatisfactory because of beach contamination, additional interceptor capacity will have to be provided.

The treatment plant at Alki Point is a primary type with a capacity sufficient for an ultimate average flow of 7.2 mgd, with a peak hydraulic capacity of 60 mgd. Although present plans call for digested sludge to be hauled away, it is believed that this material could be safely disposed of in Puget Sound (Chapter 11). For that reason, the plan herein proposed calls for construction of a sludge outfall line which would extend approximately 3,300 feet into Puget Sound to a water depth of 400 feet.

Disinfected effluent is to be discharged through an outfall, presently under construction, which will terminate 1,400 feet offshore at a depth of 85 feet. As reported in Chapter 11, a chlorinated primary effluent discharged in water at that depth probably will not prevent some contamination of adjacent beaches. This implies that the outfall now going in should be extended to a depth sufficient to assure adequate dispersion at the point of discharge. Unfortunately, however, not enough head is available at the treatment plant to permit such an extension. It is proposed, therefore, that protection of the beaches be obtained by putting in a second outfall, which would be designed to carry the peak dry weather flow, and by utilizing the original outfall for wet weather overflows. Under this plan, the second outfall would extend directly west of Alki Point and would terminate approximately 1,100 feet offshore at a depth of about 210 feet.

Comparison of Independent Systems with Central Sewerage Project. High head pumping would be required to convey sewage of the four southern subareas in the South Puget Sound sewerage area into facilities of the core plan system. On the other hand, sewage from the northern subarea, West Seattle, could be conveyed by gravity northward from Alki Point along the waterfront through the Elliott Bay sewerage area and into the core plan system.

Table 15-28. Comparison of Construction and Annual Costs of Separate Plan and Central Sewerage Project, Redondo Beach Subarea, South Puget Sound Sewerage Area

	Separate plan, $1,000	Central sewerage[a], $1,000
Construction cost[b]	1,934[c]	1,209
Annual cost[d]	156	101

[a]*Does not include apportioned cost to subarea of any sewers within Green River Sewerage Area or of core plan facilities.*

[b]*Includes engineering and contingencies.*

[c]*From Table 15-27.*

[d]*Includes fixed costs and maintenance and operation costs.*

Facilities required in the Redondo Beach subarea for participation in the central project would include a high elevation interceptor to the south, a short high elevation interceptor to the north, and a pumping station at Redondo Beach. This station would have a capacity of 14 mgd at a total head of 260 feet and would pump through 2,700 feet of 24-inch force main to a second pumping station. The latter would operate against a total head of 290 feet and would pump through 4,400 feet of 24-inch force main to the crest of the ridge dividing the South Puget Sound and Green River sewerage areas. From there, sewage would flow by gravity through the Green River area to the Renton plant proposed under Core Plan B.

Table 15-28 gives estimated construction and annual costs both for the independent project and for participation in the central sewerage project. In the latter case, the only costs shown are those involved in collection of the sewage and pumping it to the crest of the ridge. They do not include the proportionate share of the Redondo Beach subarea either in the Renton treatment plant or in any sewers within the Green River sewerage area.

It will be seen that both construction and annual costs for the independent system are about 50 per cent higher than those for pumping to the crest of the ridge. No detailed estimates were made of Redondo Beach's share in central facilities beyond that point. A rough check indicates, however, that its share in Core Plan B treatment costs would amount to $712,000 for construction and to $57,000 per year for operation and maintenance and fixed charges. These costs, when added to those given in Table 15-28, bring the costs of the central sewerage project to about the same totals estimated for the independent system. If the proportionate share of sewers and pumping stations within the Green River sewerage area were added, the cost of central sewerage would be considerably higher. It is concluded, therefore, that the Redondo Beach subarea should be served by an independent system.

Two alternatives were considered for possible inclusion of the Des Moines, Miller Creek and Southwest Suburban subareas in the central sewerage project. Under Alternative 1, sewage from the Southwest Suburban subarea would be pumped into the Miller Creek subarea, through which it would flow by gravity to the mouth of Miller Creek. At that point, sewage from the two subareas would be pumped into the Des Moines subarea. Beyond Des Moines, flow from the three subareas would be pumped eastward to the crest of the ridge separating the South Puget Sound and Green River sewerage areas. From that point, flow would be by gravity through the Green River area to the Renton plant proposed under Core Plan B.

Under Alternative 2, sewage from each subarea would be pumped independently into sewers connecting

with core plan facilities. Sewage from the Southwest Suburban subarea would be conveyed to the existing sewage treatment plant, from which it would be pumped eastward through three pumping stations to the crest of the ridge separating the South Puget Sound and Elliott Bay areas. Flow from that point would be through service sewers in the Elliott Bay area to the West Point treatment plant of Core Plan B. Sewage from the Miller Creek subarea would be pumped into and would flow through the Green River sewerage area to the Renton plant of Core Plan B, as would sewage from the Des Moines subarea.

Estimated construction and annual costs for these two alternatives are given in Table 15-29, as are the costs for the independent systems. For the two alternatives, the indicated costs are limited to those involved in conveyance and pumping. They do not include the apportioned cost to the subareas of any facilities in the core plan system, nor do they include the cost of any service sewers within other sewerage areas necessary to convey the sewage to the core plan system.

Both the construction and annual costs for collection and pumping facilities required under the second of the two alternatives involving participation in the central sewerage project are substantially lower than those for the independent systems (Table 15-29). A preliminary estimate was made, therefore, of the costs which would be borne by the three subareas for the treatment plants involved in the central sewerage project. Distributions thus determined were as follows:

	Construction cost $1,000	Annual cost $1,000
Southwest Suburban	375	35
Miller Creek	997	88
Des Moines	865	75
Total	2,237	198

These costs, when added to those for Alternative 2 (Table 15-29), show that participation in the central sewerage project would cost about 10 per cent more than the independent systems. It is evident, therefore, that greater economy will be achieved by providing each of the three subareas with independent facilities for sewage collection, treatment and disposal.

In the West Seattle subarea, facilities for independent sewage collection, treatment and disposal are presently under construction by the city of Seattle. To determine whether detailed studies should be made of the possibility of participation by this subarea in the central sewerage project, estimates were made of the costs of sharing in the West Point treatment plant. These estimates amount to $624,000 for construction cost, and to $64,000 for annual cost, including fixed and operating charges. Additional costs which would be incurred for conveying sewage to West Point were not estimated. They would, however, increase the total annual cost by a substantial sum.

Table 15-29. Comparison of Construction and Annual Costs of Separate Plan and Central Sewerage Project, Southwest Suburban, Miller Creek and Des Moines Subareas, South Puget Sound Sewerage Area

	Separate plan[a], $1,000	Central sewerage[b], $1,000	
		Alternative 1	Alternative 2
Construction cost[c]	5,628	4,118	4,057
Annual cost[d]	512	422	350

[a]*From Table 15-26, Plan I.*

[b]*Does not include apportioned cost to subareas of any sewers within Green River and Elliott Bay Sewerage Areas or of core plan facilities.*

[c]*Includes engineering and contingencies.*

[d]*Includes fixed costs and maintenance and operation costs.*

Under either the central or the independent project, fixed costs for facilities presently under construction would have to be paid by the city. Such charges, therefore, need not be considered in comparing costs of the two possibilities. For comparison purposes, therefore, the only cost applicable to the independent project is that of plant maintenance and operation. This is estimated to be $63,000 per year.

Because participation in the central system at this time would involve higher annual costs than those applicable to the independent system, it is concluded that the West Seattle subarea can be served more economically by the independent system.

North Puget Sound Sewerage Area

The North Puget Sound sewerage area is topographically divided into three major subareas, namely, Seaview, Piper Creek and Boeing Creek. Since Seaview encompasses only 230 acres, its sewerage is a matter for local consideration. It should be noted, however, that construction of facilities to connect this area with the North Trunk sewer of the city of Seattle is now in progress. As a result, the Seaview subarea will be tributary to the West Point plant proposed under Core Plan B.

In the case of the Piper Creek and Boeing Creek subareas, plans were laid out under which each one would be served by independent facilities for sewage collection, treatment and disposal. Locations of all trunk sewers, pumping stations, treatment works and outfalls required for this purpose are shown in Fig. 15-13. Descriptions of these facilities and their estimated construction costs are given in Table 15-30.

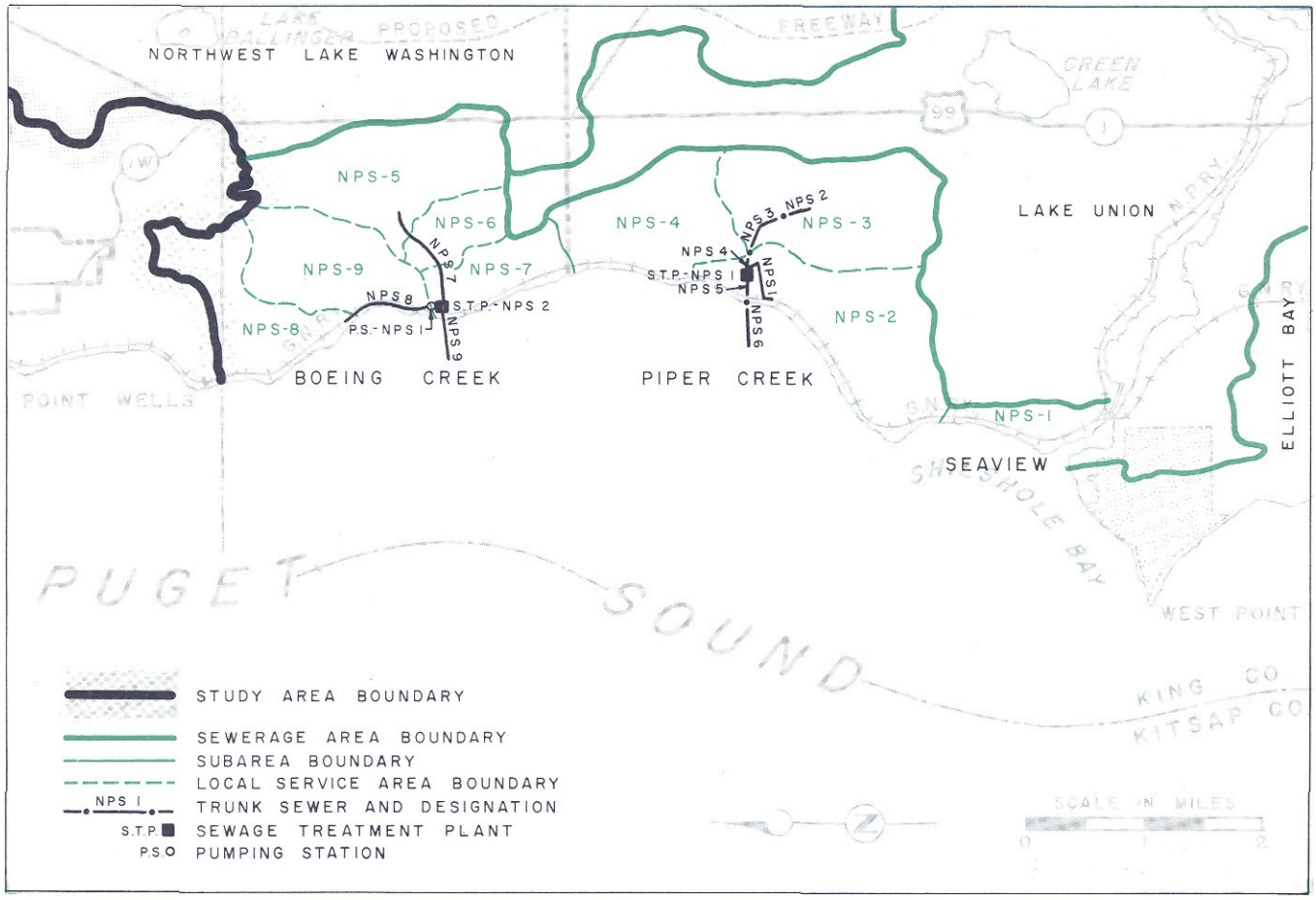


Fig. 15-13. Proposed Sewerage Facilities, Separate Systems, North Puget Sound Sewerage Area

Piper Creek Subarea. Treatment of sewage from the Piper Creek subarea is to be obtained in a plant constructed at the site of the present treatment installation of the Greenwood Sewer District. Trunk sewers would consist (1) of the existing sewer of the Greenwood Sewer District, which extends southeastward from the plant along Piper Creek to the Greenwood area, and (2) a high elevation interceptor to the south to serve the North Beach-Blue Ridge area. Some local pumping would be required in the south branch.

At the treatment plant, primary treatment would be provided for an ultimate average flow of 3.7 mgd, with a peak hydraulic capacity of 12 mgd. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since over 25 minutes detention time would be available in the outfall at ultimate average flow. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 3,300 feet offshore to a water depth of about 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 2,400 feet offshore at a depth of about 265 feet.

Boeing Creek Subarea. Treatment of the sewage of the Boeing Creek subarea would be obtained in a plant at the mouth of Boeing Creek. Trunk sewers would consist of two branches, an east and a north. The east branch would be laid eastward from the plant along Boeing Creek to the Ronald area, and the north would be laid northward along the waterfront to Richmond Beach. A pumping station would be required on the north branch to lift its flow into the treatment plant. The Highlands area to the south would be served by local sewers connecting to the plant.

The treatment plant would be a primary type having an ultimate average capacity of 2.6 mgd and a peak hydraulic capacity of 7.5 mgd. Plant units would consist of preparation and primary sedimentation tanks, separate sludge digestion tanks, and all other necessary structures and appurtenances. Chlorine contact tanks would not be required, since about 25 minutes detention time would be available in the outfall at ultimate average flow. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately

2,000 feet offshore to a water depth of about 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 1,400 feet offshore at a depth of 180 feet.

Comparison of Independent Systems with Central Sewerage Project. In determining the cost to the two subareas for participation in the central sewerage project, detailed layouts and cost estimates were made only for facilities required to collect the sewage and pump it to the crest of the ridge separating the North Puget Sound sewerage area from the Northwest Lake Washington and Lake Union sewerage areas. Costs thus obtained, both construction and annual, were compared with corresponding costs for the independent sewerage systems. As indicated in Table 15-31, esti-

Table 15-30. Description and Estimated Construction Costs, Separate Systems for North Puget Sound Sewerage Area

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
Piper Creek Subarea				
NPS-1	1.1	3.6	2,000 ft of 15-in. RC at 0.75%, average cut 6 ft, wet	35,000
NPS-2	1.0	3.1	1,400 ft of existing 21-in. at 0.28 - 3.7%	Existing
NPS-3	1.4	4.4	400 ft of existing 15-in. at 4.3 - 28% and 2,200 ft of existing 18-in. at 2.0 - 5.6%	Existing
NPS-4	2.3	7.3	300 ft of existing 21-in. at 2.5% and 500 ft of 24-in. at 1.6%	Existing
Subtotal, sewers				35,000
STP-NPS-1	3.7	12	Sewage treatment plant, primary type, includes facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 3,300 ft of 6-in. outfall sludge line to a water depth of 400 ft	576,000
NPS-5	3.7	12	2,000 ft of existing 27-in. effluent outfall, land section	Existing
NPS-6	3.7	12	2,400 ft of 33-in. RC submarine outfall to water depth of 265 ft, includes diffuser section over last 200 ft	188,000
Total contract cost, Piper Creek Subarea				799,000
Engineering and contingencies				200,000
Total construction cost, Piper Creek subarea				999,000
Boeing Creek Subarea				
NPS-7	0.7-1.0	1.8-2.9	4,900 ft of 12-in. RC at 0.65 - 10%, average cut 6 - 10 ft, wet	66,000
NPS-8	1.1-1.2	3.1-3.4	4,000 ft of 18-in. RC at 0.21 - 0.26%, average cut 10 - 15 ft, difficult wet	98,000
Subtotal, sewers				164,000
PS-NPS-1	1.2	3.4	Pumping station, single stage, motor driven, static lift 29 ft, total head at peak flow 35 ft, structure about 23 ft below ground	66,000
STP-NPS-2	2.6	7.5	Sewage treatment plant, primary type, includes facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 2,000 ft of 6-in. outfall sludge line to a water depth of 400 ft	452,000
NPS-9	2.6	7.5	1,600 ft of 24-in. RC effluent outfall to water depth of 180 ft, includes 200 ft land section, and diffuser section over last 135 ft	149,000
Total contract cost, Boeing Creek subarea				831,000
Engineering and contingencies, 25 per cent				208,000
Total construction cost, Boeing Creek subarea				1,039,000

See Fig. 15-13 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.* [b]*Does not include cost of acquiring existing facilities.*

mates for the independent systems are more than 40 per cent higher than those for pumping to the crest of the ridge. Because of these differences, an estimate was made of the proportionate costs to the two subareas of the central sewage treatment plant at West Point, proposed under Core Plan B. This indicated that the apportioned construction cost of the central plant would be $372,000 for the Piper Creek subarea and $279,000 for the Boeing Creek subarea, or a total of $651,000. The addition of these costs to the figures given in Table 15-31 makes the construction cost of central sewerage greater than that of the independent systems.

As for annual costs, the apportioned cost of the central treatment plant would be $36,000 per year to the Piper Creek subarea and $26,000 per year to the Boeing Creek subarea. It can thus be seen that addition of the apportioned costs of the West Point treatment plant only makes the annual costs for central sewerage about the same as those for the independent systems. By adding facilities required to convey sewage from the two subareas to West Point the cost of the central project would become considerably higher. It is apparent, therefore, that the more economical of the two plans for the Piper Creek and Boeing Creek subareas will be to provide independently in each area for sewage collection, treatment and disposal.

Table 15-31. Comparison of Construction and Annual Costs of Separate Plan and Central Sewerage Project, Piper Creek and Boeing Creek Subareas, North Puget Sound Sewerage Area

	Construction cost,[a] $1,000	Annual cost,[b] $1,000
Separate plan[c]		
Piper Creek	999	103
Boeing Creek	1,039	98
Total	2,038	201
Central sewerage[d]		
Piper Creek	637	68
Boeing Creek	796	70
Total	1,433	138

[a]*Includes engineering and contingencies.*

[b]*Includes fixed costs and maintenance and operation costs.*

[c]*From Table 15-30.*

[d]*Does not include apportioned cost to subarea of any sewers within Northwest Lake Washington and Lake Union Sewerage Areas or of core plan facilities.*

SERVICE SEWERS

In each sewerage area, sewers were laid out to serve a minimum tributary area of 1,000 acres. These sewers, designated herein as service sewers, will convey sewage from each local service area to the point of concentration in the area, at which point the sewage will be discharged to facilities provided under the central sewerage project.

In the case of the South Puget Sound and North Puget Sound areas, where independent sewerage projects were found to be the more satisfactory, service sewers required to convey sewage to the individual treatment plants were included in the layout of those systems. These sewers are shown in Figs. 15-12 and 15-13.

North Lake Sammamish Sewerage Area

Service sewers for the North Lake Sammamish sewerage area are laid out to convey the sewage of that area to a pumping station about two miles north of the city of Redmond (Fig. 15-14). Descriptions and estimated costs of these facilities are given in Table 15-32. Major elements of the system are as follows:

1. A waterfront interceptor (NLS 1-NLS 7) to serve areas draining directly into the lake from the east. A pumping station (PS-NLS 1) would be required to lift sewage from this interceptor into the trunk sewer serving areas to the north and east.

2. A trunk (NLS 8-NLS 11) to serve the Crystal and Cottage Lake areas to the north. A pumping station (PS-NLS 2) would be required at Cottage Lake to lift sewage from the east side of the lake into the main trunk.

3. A trunk (NLS 12-NLS 13) along Cottage Lake Creek to its confluence with Bear Creek. Sewage from NLS 11 would be discharged to this trunk.

4. A trunk (NLS 14-NLS 25) along Bear Creek to its confluence with Cottage Lake Creek. This trunk would serve areas north of Paradise Lake and, by means of branch trunks, would also serve areas along Stuve Creek and Seidel Creek. A pumping station (PS-NLS 3) would be required at Paradise Lake to avoid excessive cuts.

5. A trunk (NLS 26-NLS 28) along Bear Creek to its confluence with Evans Creek. This trunk would serve areas draining to Bear Creek.

6. A trunk (NLS 29-NLS 32) along Evans Creek to its confluence with Bear Creek. This trunk would serve areas to the east.

7. A trunk (NLS 33) along Bear Creek from the confluence of Bear and Evans Creek to the intersection with the east waterfront interceptor.

8. A trunk (NLS 34) along the south city limit of Redmond.

9. A waterfront interceptor (NLS 35-NLS 38) along the west shore of Lake Sammamish to serve areas draining directly into the lake.

10. A main trunk (NLS 39-NLS 41) along the Sammamish River valley to the pumping station (PS-S1) included in the feeder sewer system.

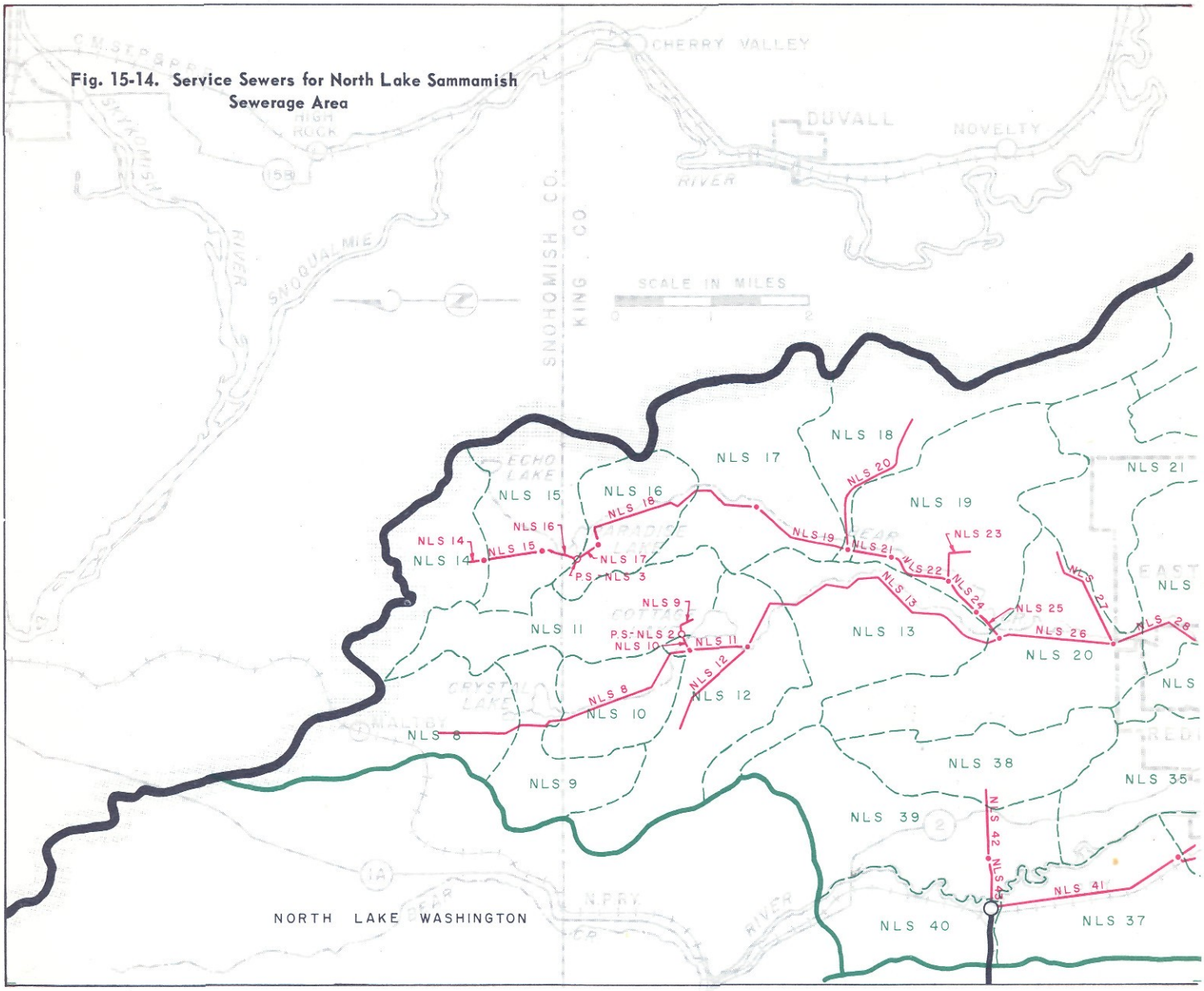


Fig. 15-14. Service Sewers for North Lake Sammamish Sewerage Area

11. A trunk (NLS 42-NLS 43) east of the pumping station (PS-S1) to serve areas along the eastern slope of Sammamish River valley.

South Lake Sammamish Sewerage Area

Service sewers for the South Lake Sammamish sewerage area are laid out to convey the sewage of that area to a pumping station situated on the waterfront of the lake east of Phantom Lake (Fig. 15-15). Descriptions and estimated costs of these facilities are given in Table 15-33. Major elements of the system are as follows:

1. A trunk (SLS 1-SLS 9) along Issaquah Creek to its confluence with the East Fork of Issaquah Creek. This trunk would serve the south portion of the area and most of the city of Issaquah.

2. A trunk (SLS 10) along the East Fork of Issaquah Creek to its confluence with Issaquah Creek.

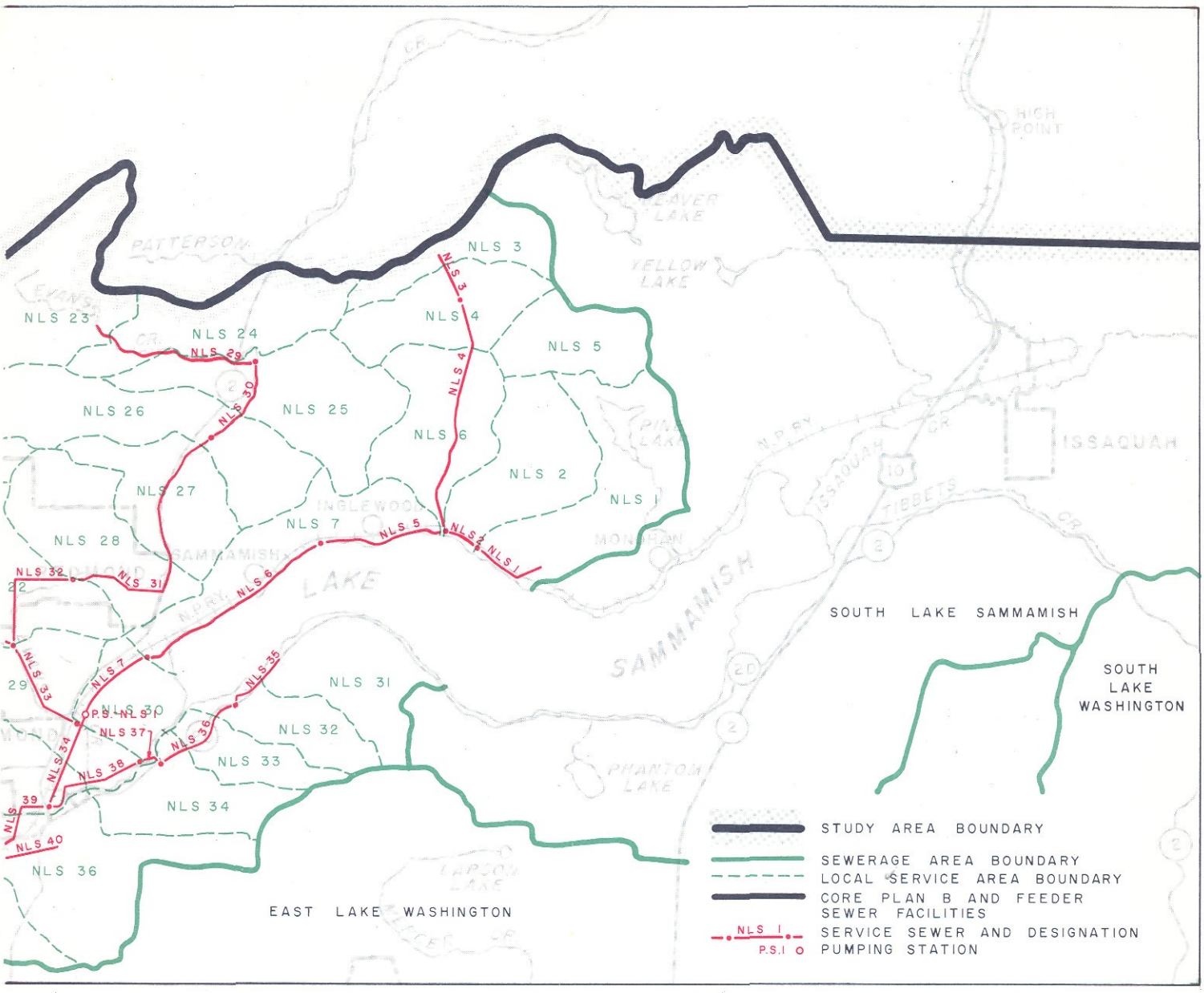
3. A trunk (SLS 11) along U.S. Highway 10.

4. A trunk (SLS 12-SLS 15) along Tibbetts Creek. Sewer SLS 11 would discharge into this trunk.

5. A trunk (SLS 16-SLS 20) to the north and east to serve the Beaver Lake area, as well as areas draining directly to Lake Sammamish.

6. A trunk (SLS 21) along the shore of Lake Sammamish to a pumping station (PS-SLS 1) situated on the lake front. The pumping station would discharge through a force main (SLS 22) to a high elevation waterfront interceptor (SLS 23).

7. A high elevation waterfront interceptor (SLS 23-SLS 24) to the pumping station (PS-S2) included in the feeder sewer system. This sewer would serve high



areas west of Lake Sammamish. Local pumping would be required for waterfront areas which could not sewer by gravity to the two pumping stations.

East Lake Washington Sewerage Area

Service sewers for the East Lake Washington sewerage area are laid out to convey the sewage of that area into the feeder sewer system (Fig. 15-15). Descriptions and estimated costs of these facilities are given in Table 15-33. Major elements of the system are as follows:

1. A high level waterfront interceptor (ELW 1-ELW 8) along the shore of Lake Washington and north of Juanita Bay to a pumping station (PS-ELW 2), which would discharge through a force main (ELW 9) into the feeder system. A pumping station (PS-ELW 1) would be required along the route of the sewer. Some local pumping would also be required.

2. A pumping station (PS-ELW 3) at the site of the existing sewage treatment plant of the city of Kirkland. This station would discharge through a force main (ELW 10) to the feeder sewers and would serve areas sewering to the Kirkland treatment plant.

3. A high elevation waterfront interceptor (ELW 11-ELW 13) to serve areas in Hunts Point, Medina and Clyde Hill. This interceptor would discharge to a pumping station (PS-ELW 5), which in turn would discharge through a force main ELW 14) into the feeder sewers. A pumping station (PS-ELW 4) would be required along the route of the sewer. Local pumping would be required for areas which could not be served by gravity.

Table 15-32. Description and Estimated Construction Costs, Service Sewers, North Lake Sammamish Sewerage Area

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
NLS-1	0.6	2.0	3,900 ft of 15-in. RC at 0.64%, average cut 6 ft, wet, includes railroad and highway crossings	62,000
NLS-2	0.9-1.2	2.5-3.4	2,700 ft of 18-in. RC at 0.14 - 0.25%, average cut 7 - 8 ft, wet	44,000
NLS-3	0.3	0.9	2,900 ft of 12-in. RC at 0.16%, average cut 8 ft, difficult wet, includes sheeting and special bedding	66,000
NLS-4	0.6-1.5	2.1-4.5	12,900 ft of 15-in. RC at 0.26 - 5.2%, average cut 9 - 24 ft, dry to difficult wet in swampy material, includes sheeting and special bedding in swamp and railroad and highway crossings	321,000
NLS-5	2.6	8.0	7,100 ft of 30-in. RC at 0.09%, average cut 10 - 12 ft, wet	182,000
NLS-6	3.0	9.0	12,200 ft of 33-in. RC at 0.064 - 0.075%, average cut 13 - 15 ft, wet to difficult wet in peat, includes sheeting and special bedding in peat	395,000
NLS-7	4.4	12	5,300 ft of 36-in. RC at 0.07%, average cut 18 ft, wet to difficult wet in peat, includes sheeting and special bedding in peat	260,000
NLS-8	0.9-1.8	2.4-4.9	15,700 ft of 18-in. RC at 0.13 - 1.5%, average cut 6 - 11 ft, wet to difficult wet in swampy material, includes sheeting and special bedding in swamp	349,000
NLS-9	0.5	1.5	1,400 ft of 12-in. RC at 0.45%, average cut 10 ft, difficult wet	26,000
NLS-10	0.8	1.9	400 ft of 8-in. force main, difficult wet	4,000
NLS-11	2.5	7.0	2,900 ft of 21-in. RC at 0.5%, average cut 7 - 8 ft, wet	52,000
NLS-12	0.7	2.0	5,300 ft of 12-in. RC at 1.1%, average cut 6 ft, dry to difficult wet	63,000
NLS-13	3.2-4.0	9.2 - 12	17,400 ft of 24-in. RC at 0.4 - 1.1%, average cut 7 - 10 ft, dry to difficult wet	325,000
NLS-14	0.6	1.6	800 ft of 12-in. RC at 0.62%, average cut 6 ft, difficult wet	12,000
NLS-15	0.8	2.1	3,400 ft of 15-in. RC at 0.26%, average cut 9 ft, difficult wet in swampy material, includes sheeting and special bedding	95,000
NLS-16	1.0	2.8	1,300 ft of 18-in. RC at 0.17%, average cut 14 ft, difficult wet in swampy material, includes sheeting and special bedding	47,000
NLS-17	1.5	4.0	1,100 ft of 12-in. force main	8,000
NLS-18	1.6-2.4	4.5-6.8	12,500 ft of 18-in. RC at 0.54 - 1.0%, average cut 8 - 11 ft, dry to moderately wet, includes imported backfill and repaving	171,000
NLS-19	2.7	7.6	4,300 ft of 21-in. RC at 0.68%, average cut 6 ft, dry to moderately wet	58,000
NLS-20	0.5-0.9	1.7-2.7	9,500 ft of 12-in. RC at 2.1 - 2.4%, average cut 6 ft, dry to moderately wet	88,000
NLS-21	3.4	10	2,600 ft of 21-in. RC at 1.2%, average cut 6 ft, dry to moderately wet.	35,000
NLS-22	3.6	10	4,000 ft of 24-in. RC at 0.5%, average cut 7 ft, dry to moderately wet.	65,000
NLS-23	0.3-0.6	1.0-1.9	3,400 ft of 12-in. RC at 1.3 - 2.0%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	34,000
NLS-24	4.2	12	2,500 ft of 24-in. RC at 0.8%, average cut 7 ft, dry to moderately wet.	41,000
NLS-25	4.3	12	1,900 ft of 27-in. RC at 0.53%, average cut 9 ft, dry to moderately wet	36,000
NLS-26	8.3-9.2	24-27	6,700 ft of 36-in. RC at 0.31 - 0.38%, average cut 10 - 11 ft, dry to moderately wet, includes imported backfill and repaving	166,000
NLS-27	0.6	2.0	6,400 ft of 12-in. RC at 2.0%, average cut 6 ft, dry to moderately wet.	60,000
NLS-28	10	29	5,000 ft of 39-in. RC at 0.29 - 0.41%, average cut 7 - 9 ft, dry to wet.	141,000
NLS-29	0.7	2.2	9,800 ft of 12-in. RC at 4.3%, average cut 6 ft, dry to moderately wet, includes access roads	98,000
NLS-30	0.9-1.6	2.9-4.9	5,200 ft of 18-in. RC at 0.25 - 0.5%, average cut 6 - 10 ft, difficult wet in peaty material, includes sheeting and special bedding	130,000

Continued on next page

Table 15-32. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
NLS-31	2.6	7.8	14,400 ft of 27-in. RC at 0.14 - 0.19%, average cut 10 - 12 ft, dry to difficult wet in peaty material, includes sheeting and special bedding in peat, imported backfill and repaving	360,000
NLS-32	3.4	10	7,600 ft of 30-in. RC at 0.15 - 0.35%, average cut 7 - 9 ft, wet, includes imported backfill and repaving	205,000
NLS-33	14-16	41-43	5,800 ft of 48-in. RC at 0.21 - 0.33%, average cut 8 ft, wet, includes railroad and highway crossings	228,000
NLS-34	19	56	5,300 ft of 72-in. RC at 0.045%, average cut 11 ft, difficult wet	360,000
NLS-35	0.6	2.2	3,400 ft of 15-in. RC at 0.2 - 0.37%, average cut 6 - 8 ft, wet, includes highway crossing	54,000
NLS-36	1.0-1.5	2.8-3.8	5,800 ft of 18-in. RC at 0.17 - 0.31%, average cut 10 - 11 ft, wet, includes imported backfill and repaving	114,000
NLS-37	2.0	5.2	1,100 ft of 21-in. RC at 0.26%, average cut 10 ft, difficult wet, includes Sammamish River crossing	27,000
NLS-38	2.8-3.0	6.7-7.5	6,100 ft of 30-in. RC at 0.065 - 0.082%, average cut 8 - 10 ft, difficult wet, includes sheeting and highway crossing	210,000
NLS-39	22-23	63-66	4,900 ft of 84-in. RC at 0.024 - 0.028%, average cut 12 - 13 ft, difficult wet, includes sheeting and railroad, highway and Sammamish River crossings	445,000
NLS-40	0.8	2.0	3,300 ft of 12-in. RC at 1.9%, average cut 6 ft, wet, includes railroad and highway crossings	48,000
NLS-41	24-25	68-70	11,300 ft of 84-in. RC at 0.03%, average cut 14 - 19 ft, difficult wet, includes sheeting	1,100,000
NLS-42	0.6	1.7	3,800 ft of 15-in. RC at 0.4%, average cut 6 ft, difficult wet, includes sheeting	90,000
NLS-43	1.5	4.3	2,800 ft of 18-in. RC at 0.4%, average cut 12 ft, difficult wet, includes sheeting and Sammamish River crossing	125,000
Subtotal, sewers				6,800,000
PS-NLS-1	4.4	12	Pumping station, single stage, motor driven with engine standby, static lift 16 ft, total head at peak flow 25 ft, structure about 25 ft below ground, includes sheeting and dewatering	148,000
PS-NLS-2	0.7	1.9	Pumping station, single stage, motor driven with engine standby, static lift 22 ft, total head at peak flow 40 ft, structure about 15 ft below ground, includes dewatering	47,000
PS-NLS-3	1.4	4.0	Pumping station, single stage, motor driven with engine standby, static lift 31 ft, total head at peak flow 60 ft, structure about 18 ft below ground, includes dewatering	73,000
Subtotal, pumping stations				268,000
Total contract cost				7,068,000
Engineering and contingencies, 25 per cent				1,767,000
Total construction cost				8,835,000

See Fig. 15-14 for location of facilities. [a]*Expressed as average dry weather flow and maximum wet weather flow.*

4. A trunk (ELW 17-ELW 19) along Richie Road and U.S. Highway 10 to a pumping station (PS-ELW 6), which would discharge through a force main (ELW 20) into the feeder sewers. This trunk would serve Factorial and areas to the east.

5. A waterfront interceptor (ELW 21-ELW 22) along the shore of Meydenbauer Bay to a pumping station (PS-ELW 7) at the site of the existing sewage treatment plant of the city of Bellevue. The pumping station would discharge through a force main and

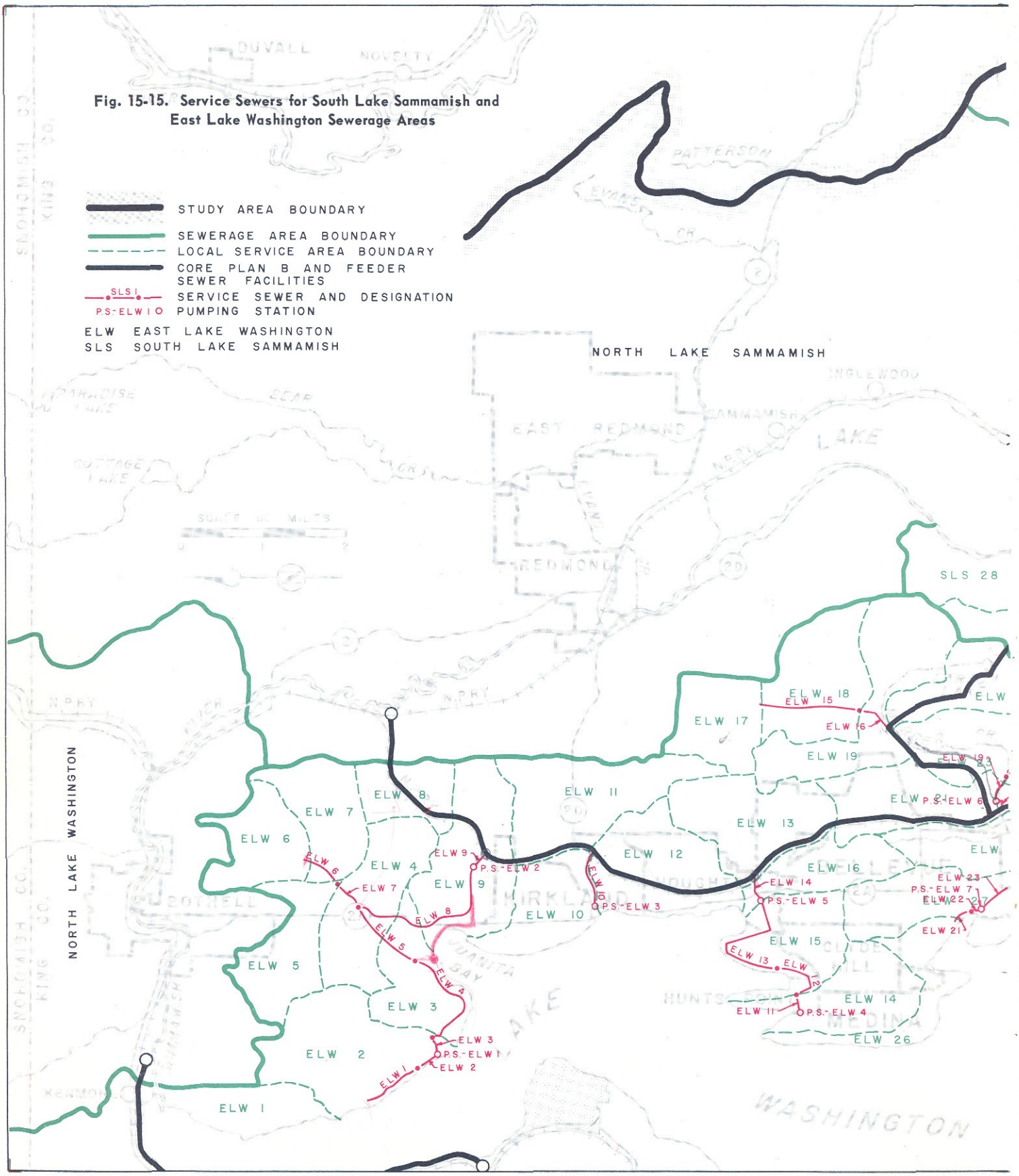
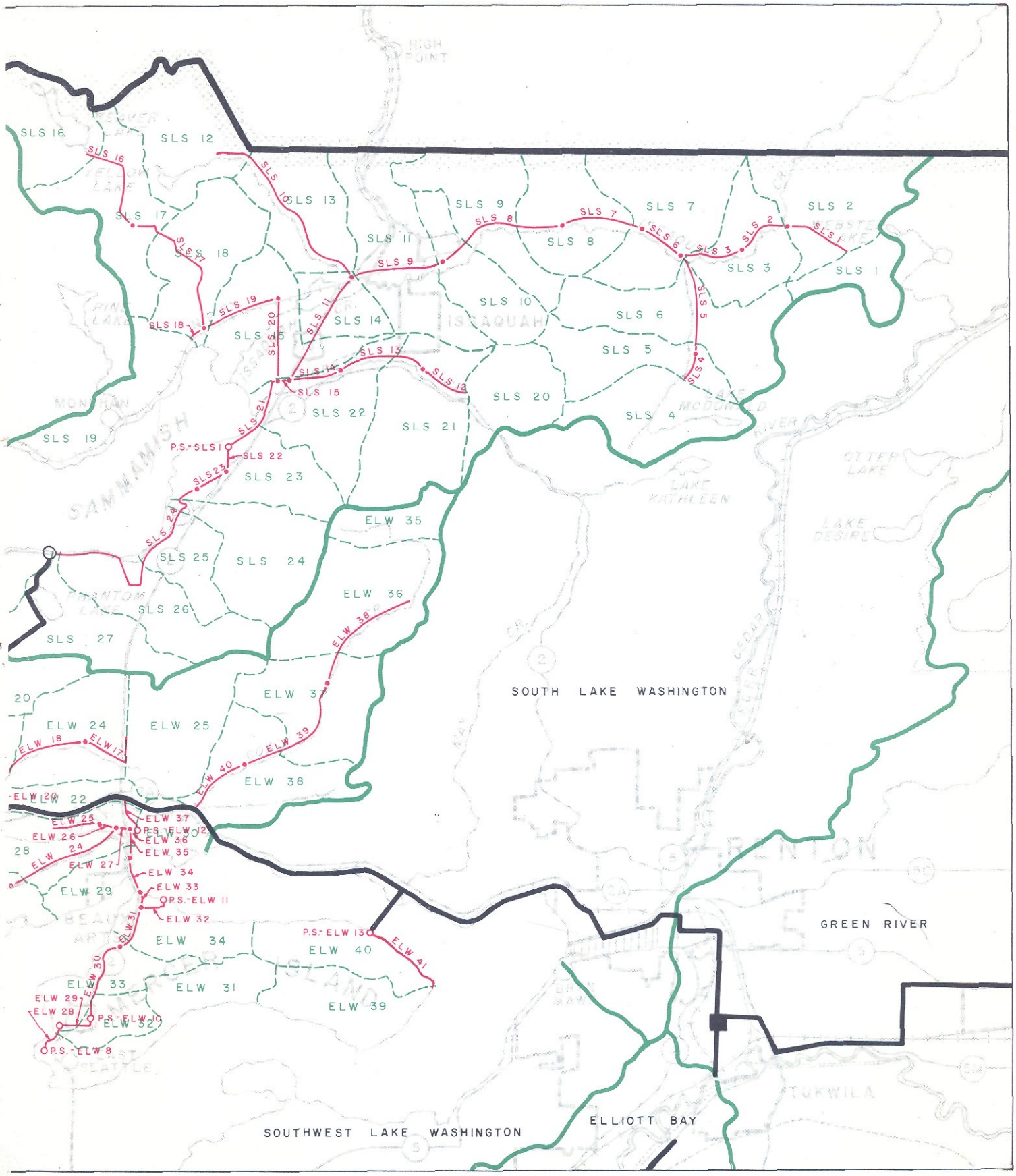


Fig. 15-15. Service Sewers for South Lake Sammamish and East Lake Washington Sewerage Areas


SLS 16
SLS 12
SLS 13
SLS 9
SLS 7
SLS 2
SLS 17
SLS 18
SLS 11
SLS 8
SLS 6
SLS 3
SLS 1
SLS 10
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ELW 20
ELW 18
ELW 17
ELW 26
ELW 27
P.S.-ELW 12
P.S.-ELW 11
P.S.-ELW 13
P.S.-ELW 10
P.S.-ELW 8
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
ELLIOTT BAY
SAMMAMISH
ISSAQUAH
RENTON
TUKWILA
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
PHANTOM LAKE
HIGH POINT

Table 15-33. Description and Estimated Construction Costs, Service Sewers, South Lake Sammamish and East Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
South Lake Sammamish Sewerage Area				
SLS-1	0.4	1.3	4,200 ft of 12-in. RC at 1.2%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	48,000
SLS-2	1.1	3.1	3,500 ft of 15-in. RC at 1.4%, average cut 6 ft, dry to moderately wet	38,000
SLS-3	1.6	4.3	4,700 ft of 18-in. RC at 0.85%, average cut 6 ft, dry to moderately wet	57,000
SLS-4	0.5	1.5	1,600 ft of 12-in. RC at 0.45%, average cut 7 ft, dry to moderately wet	15,000
SLS-5	0.8-1.6	2.8-4.5	6,300 ft of 15-in. RC at 0.45 - 1.8%, average cut 8 - 9 ft, dry to moderately wet	70,000
SLS-6	3.2-3.6	9.1-10	3,600 ft of 21-in. RC at 1.1%, average cut 7 - 9 ft, dry to moderately wet	50,000
SLS-7	3.8	11	5,200 ft of 24-in. RC at 0.6%, average cut 9 ft, dry to moderately wet, includes imported backfill and repaving	97,000
SLS-8	4.6	13	6,300 ft of 27-in. RC at 0.47%, average cut 7 - 8 ft, dry to wet, includes imported backfill and repaving	126,000
SLS-9	4.8-5.2	14-17	8,500 ft of 30-in. RC at 0.32-0.57%, average cut 7 - 8 ft, wet to difficult wet, includes imported backfill and repaving	251,000
SLS-10	0.4-0.8	1.7-2.8	13,200 ft of 12-in. RC at 0.55 - 4.4%, average cut 6 - 9 ft, dry to difficult wet, includes railroad and highway crossings	150,000
SLS-11	7.2	21	7,900 ft of 33-in. RC at 0.4%, average cut 7 - 10 ft, difficult wet, includes imported backfill and repaving	255,000
SLS-12	0.4-0.8	1.1-2.5	3,700 ft of 12-in. RC at 1.2 - 3.8%, average cut 6 ft, dry to moderately wet, includes repaving	38,000
SLS-13	1.4	3.9	5,100 ft of 15-in. RC at 2.5%, average cut 6 ft, wet, includes imported backfill and repaving	87,000
SLS-14	1.8	5.3	3,500 ft of 18-in. RC at 0.86%, average cut 7 ft, difficult wet	64,000
SLS-15	8.9	26	600 ft of 36-in. RC at 0.37%, average cut 13 ft, difficult wet, includes sheeting	24,000
SLS-16	0.4-0.6	1.1-2.1	7,300 ft of 12-in. RC at 0.37 - 0.87%, average cut 6 ft, dry to wet	97,000
SLS-17	0.9-1.4	2.7-4.4	10,700 ft of 15-in. RC at 0.43 - 6.0%, average cut 6 - 8 ft, dry to difficult wet, includes railroad and highway crossings	170,000
SLS-18	0.4	1.3	700 ft of 12-in. RC at 0.33%, average cut 9 ft, difficult wet	12,000
SLS-19	1.8	5.6	5,200 ft of 27-in. RC at 0.08%, average cut 13 ft, difficult wet, includes sheeting	185,000
SLS-20	2.4	7.3	5,200 ft of 30-in. RC at 0.07%, average cut 14 ft, difficult wet, includes sheeting and Issaquah Creek crossing	204,000
SLS-21	12	34	5,600 ft of 48-in. RC at 0.10%, average cut 14 - 15 ft, difficult wet, includes sheeting	312,000
SLS-22	12	36	1,900 ft of 30-in. force main	33,000
SLS-23	12	36	2,100 ft of 42-in. RC at 0.48%, average cut 7 ft, dry to moderately wet	60,000
SLS-24	13-14	38-40	15,300 ft of 48-in. RC at 0.18 - 0.27%, average cut 9 - 11 ft, dry to wet, includes imported backfill and repaving	621,000
Subtotal, sewers				3,064,000
PS-SLS-1	12	36	Pumping station, single stage, motor driven with engine standby, static lift 86 ft, total head at peak flow 115 ft, structure about 20 ft below ground, includes sheeting and dewatering	291,000

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Table 15-33. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
Total contract cost, South Lake Samamish Sewerage Area				3,355,000
Engineering and contingencies, 25 per cent				839,000
Total construction cost, South Lake Sammamish Sewerage Area				4,194,000
East Lake Washington Sewerage Area				
ELW-1	0.7	1.8	4,000 ft of 12-in. RC at 0.5 - 2.5%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	38,000
ELW-2	1.3	3.4	1,700 ft of 18-in. RC at 0.25%, average cut 8 ft, dry to moderately wet, includes imported backfill and repaving	27,000
ELW-3	1.3	3.4	1,400 ft of 12-in. force main, includes imported backfill and repaving	11,000
ELW-4	1.8	4.7	8,600 ft of 18-in. RC at 0.43 - 0.54%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	111,000
ELW-5	2.0-2.3	5.2-6.0	4,800 ft of 21-in. RC at 0.27 - 0.35%, average cut 7 - 11 ft, dry to moderately wet	67,000
ELW-6	0.6	1.4	2,400 ft of 12-in. RC at 1.2%, average cut 6 ft, dry to moderately wet	22,000
ELW-7	1.9	4.7	2,300 ft of 15-in. RC at 1.5 - 2.3%, average cut 6 - 9 ft, dry to moderately wet	25,000
ELW-8	4.1-5.0	11-13	10,800 ft of 33-in. RC at 0.10 - 0.15%, average cut 9 - 13 ft, dry to moderately wet, includes imported backfill and repaving	275,000
ELW-9	5.4	14	1,100 ft of 20-in. force main	12,000
ELW-10	0.9	3.3	4,100 ft of 12-in. force main, includes imported backfill and repaving	36,000
ELW-11	0.7	1.8	1,100 ft of 8-in. force main	7,000
ELW-12	0.8	2.0	3,900 ft of 15-in. RC at 0.28%, average cut 7 - 9 ft, wet	59,000
ELW-13	0.9-1.3	2.5-3.2	9,300 ft of 18-in. RC at 0.20%, average cut 6 - 9 ft, wet to difficult wet	158,000
ELW-14	1.9	4.9	1,500 ft of 14-in. force main	11,000
ELW-15	0.5	1.4	6,600 ft of 12-in. RC at 1.4%, average cut 6 ft, dry to moderately wet	61,000
ELW-16	1.4-1.9	3.8-5.0	2,600 ft of 15-in. RC at 0.85 - 1.8%, average cut 7 ft, dry to difficult wet, includes sheeting and highway crossing	42,000
ELW-17	0.4-0.8	1.1-2.3	4,400 ft of 12-in. RC at 1.0 - 4.7%, average cut 6 - 8 ft, dry to difficult wet, includes imported backfill, repaving and highway crossing	68,000
ELW-18	1.5	4.0	6,500 ft of 18-in. RC at 0.37%, average cut 8 ft, wet to difficult wet, includes sheeting	133,000
ELW-19	1.9	5.2	2,100 ft of 21-in. RC at 0.26%, average cut 8 ft, wet	38,000
ELW-20	2.2	5.8	700 ft of 14-in. force main, includes railroad crossing	9,000
ELW-21	0.6	1.7	1,600 ft of 12-in. RC at 0.55%, average cut 10 ft, difficult wet	29,000
ELW-22	1.4	4.2	500 ft of 18-in. RC at 0.38%, average cut 14 ft, difficult wet	13,000
ELW-23	1.5	4.4	2,500 ft of 14-in. force main, includes imported backfill and repaving	24,000
ELW-24	1.5	4.4	8,400 ft of 18-in. RC at 1.1%, average cut 7 ft, dry to difficult wet in peaty material, includes sheeting and special bedding in peat, imported backfill, repaving, and slough crossing	175,000
ELW-25	0.6	1.7	3,300 ft of 12-in. RC at 0.55%, average cut 13 ft, difficult wet in peaty material, includes sheeting, special bedding and piling	139,000
ELW-26	0.9	2.6	800 ft of 15-in. RC at 0.38%, average cut 22 ft, difficult wet in peaty material, includes sheeting, special bedding and piling	46,000
ELW-27	2.5	6.5	400 ft of 27-in. RC at 0.11%, average cut 23 ft, difficult wet in peaty material, includes sheeting, special bedding and piling	31,000

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Table 15-33. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
ELW-28	1.1	3.3	2,800 ft of existing 12-in. force main	Existing
ELW-29	1.5	4.5	2,700 ft of 10-in. force main to parallel existing 10-in. force main, includes imported backfill and repaving	22,000
ELW-30	1.8	5.6	6,400 ft of 12-in. force main to parallel existing 12-in. force main, includes imported backfill, repaving and highway crossing	64,000
ELW-31	1.8	5.6	2,500 ft of 18-in. RC at 0.22 - 2.4%, to parallel existing 12-in. and 18-in. sewers, includes imported backfill and repaving	36,000
ELW-32	0.5	1.8	2,100 ft of 10-in. force main, includes highway crossing	20,000
ELW-33	2.3	7.2	800 ft of existing 16-in. at 4.8 - 10.9%	Existing
ELW-34	2.3	7.2	1,600 ft of existing 16-in. inverted siphon across Lake Washington	Existing
ELW-35	2.3	7.2	2,500 ft of 18-in. RC at 1.2 - 1.8%, average cut 6 - 9 ft, wet to difficult wet in peaty material, includes sheeting, special bedding and piling in peat, imported backfill, repaving, slough crossing and connections to existing sewers at Mercer Island Sewer District pumping station No. 8 which is to be abandoned	74,000
ELW-36	5.0	15	1,600 ft of 39-in. RC at 0.08%, average cut 23 - 25 ft, difficult wet in peaty material, includes sheeting, special bedding, piling and slough crossing	158,000
ELW-37	5.5	16	1,100 ft of 24-in. force main, difficult wet in peaty material, includes sheeting, piling and special bedding	49,000
ELW-38	0.4-1.3	1.1-3.6	11,000 ft of 12-in. RC at 2.0 - 6.7%, average cut 6 ft, dry to moderately wet, includes access road	106,000
ELW-39	1.8	4.9	5,100 ft of 15-in. RC at 2.0%, average cut 6 ft, dry to moderately wet, includes access road	58,000
ELW-40	2.2	6.4	5,000 ft of 18-in. RC at 1.0 - 1.5%, average cut 6 - 8 ft, dry to wet, includes railroad crossing	76,000
ELW-41	0.5	1.4	4,300 ft of 12-in. RC at 0.4%, average cut 9 ft, wet	59,000
Subtotal, sewers				2,389,000
PS-ELW-1	1.3	3.4	Pumping station, single stage, motor driven with engine standby, static lift 80 ft, total head at peak flow 105 ft, structure about 14 ft below ground	63,000
PS-ELW-2	3.4	14	Pumping station, two stage, motor driven with engine standby, static lift 110 ft, total head at peak flow 135 ft, structure about 15 ft below ground	180,000
PS-ELW-3	0.9	3.3	Pumping station, two stage, motor driven with engine standby, static lift 110 ft, total head at peak flow 175 ft, structure about 20 ft below ground. Station located at site of existing Kirkland sewage treatment plant which is to be abandoned, includes sheeting, dewatering and connections to existing sewers	88,000
PS-ELW-4	0.7	1.8	Pumping station, single stage, motor driven with engine standby, static lift 40 ft, total head at peak flow 80 ft, structure about 15 ft below ground, difficult wet construction, includes sheeting and dewatering	50,000
PS-ELW-5	1.9	4.9	Pumping station, two stage, motor driven with engine standby, static lift 130 ft, total head at peak flow 155 ft, structure about 15 ft below ground, difficult wet construction, includes sheeting and dewatering	105,000
PS-ELW-6	2.2	5.8	Pumping station, single stage, motor driven with engine standby, static lift 92 ft, total head at peak flow 110 ft, structure about 14 ft below ground, includes sheeting and dewatering	100,000

Continued on next page

Table 15-33. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
PS-ELW-7	1.4	4.4	Pumping station, two stage, motor driven with engine standby, static lift 100 ft, total head at peak flow 135 ft, difficult wet construction. Station located at site of existing Bellevue sewage treatment plant which is to be abandoned, includes sheeting, dewatering and connections to existing sewers	111,000
PS-ELW-8	1.0	3.3	Pumping station, single stage, motor driven with engine standby, static lift 20 ft, total head at peak flow 65 ft. To replace existing Mercer Island Sewer District pumping station No. 3 which has a total installed capacity of 1.1 mgd, includes connection to existing sewers	62,000
PS-ELW-9	1.5	4.5	Pumping station, two stage, motor driven with engine standby, static lift 100 ft, total head at peak flow 150 ft. To replace existing Mercer Island Sewer District pumping station No. 2 which has a total installed capacity of 1.7 mgd, includes connections to existing sewers	90,000
PS-ELW-10	1.7	5.6	Pumping station, single stage, motor driven with engine standby, static lift 10 ft, total head at peak flow 80 ft. To replace existing Mercer Island Sewer District pumping station No. 6 which has a total installed capacity of 1.9 mgd, includes connections to existing sewers	85,000
PS-ELW-11	0.6	1.8	Pumping station, single stage, motor driven with engine standby, static lift 50 ft, total head at peak flow 80 ft, structure about 15 ft below ground. Station located at site of existing East Mercer Island Sewer District sewage treatment plant which is to be abandoned, includes connections to existing sewers	48,000
PS-ELW-12	5.6	16	Pumping station, single stage, motor and engine driven, static lift 80 ft, total head at peak flow 95 ft, structure about 30 ft below ground, difficult wet construction in peaty material, includes special foundations, sheeting and dewatering	212,000
PS-ELW-13	1.1	2.8	Pumping station, single stage, motor driven with engine standby, static lift 15 ft, total head at peak flow 55 ft, structure about 15 ft below ground	56,000
Subtotal, pumping stations				1,250,000
Total contract cost, East Lake Washington Sewerage Area				3,639,000
Engineering and contingencies, 25 per cent				910,000
Total construction cost, East Lake Washington Sewerage Area				4,549,000

See Fig. 15-15 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

gravity sewers (ELW 23, 24, 27 and 36) to a second pumping station (PS-ELW 12), which in turn would discharge through a force main (ELW 37) into the feeder sewers.

6. A trunk (ELW 25-ELW 26) in Mercer Slough to serve areas draining directly into the slough.

7. A trunk system (ELW 28-ELW 35) on Mercer Island, including the crossing to the east side of Lake Washington. This system would serve the northern half of Mercer Island and would incorporate existing sewers of the Mercer Island Sewer District. Three pumping stations (PS-ELW 8, PS-ELW 9 and PS-ELW 10) would be required along the routes of the trunk sewers. In addition a fourth pumping station (PS-ELW 11) would be required at the site of the existing sewage treatment plant of the East Mercer Sewer District to pump the sewage from that point into the trunk.

8. A trunk (ELW 38-ELW 40) along Coal Creek to serve areas to the south and east.

9. A waterfront interceptor (ELW 41) to a pumping station (PS-ELW 13) at the southeastern end of Mercer

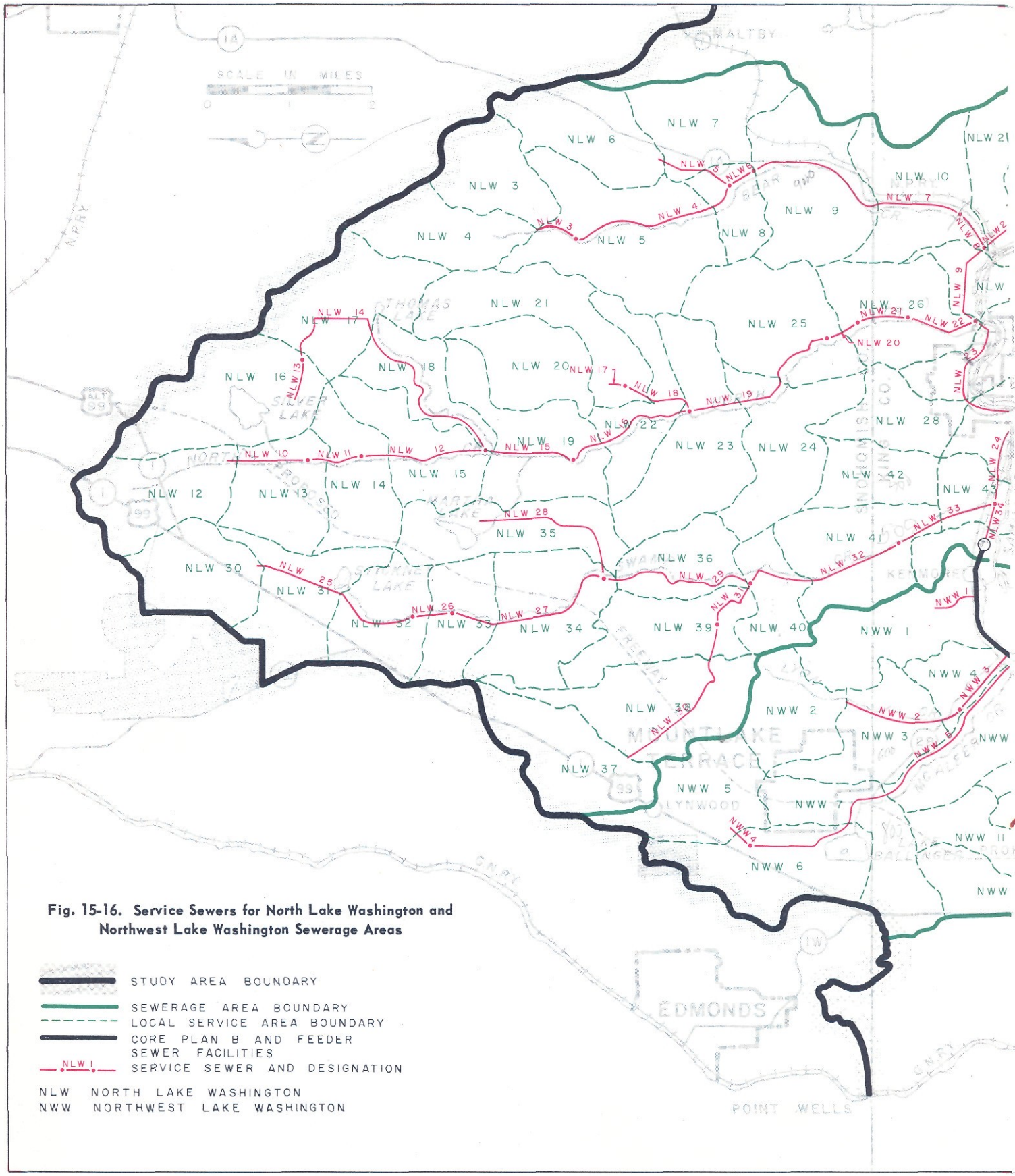


Fig. 15-16. Service Sewers for North Lake Washington and Northwest Lake Washington Sewerage Areas

STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
CORE PLAN B AND FEEDER SEWER FACILITIES
NLW 1 SERVICE SEWER AND DESIGNATION

NLW NORTH LAKE WASHINGTON
NWW NORTHWEST LAKE WASHINGTON

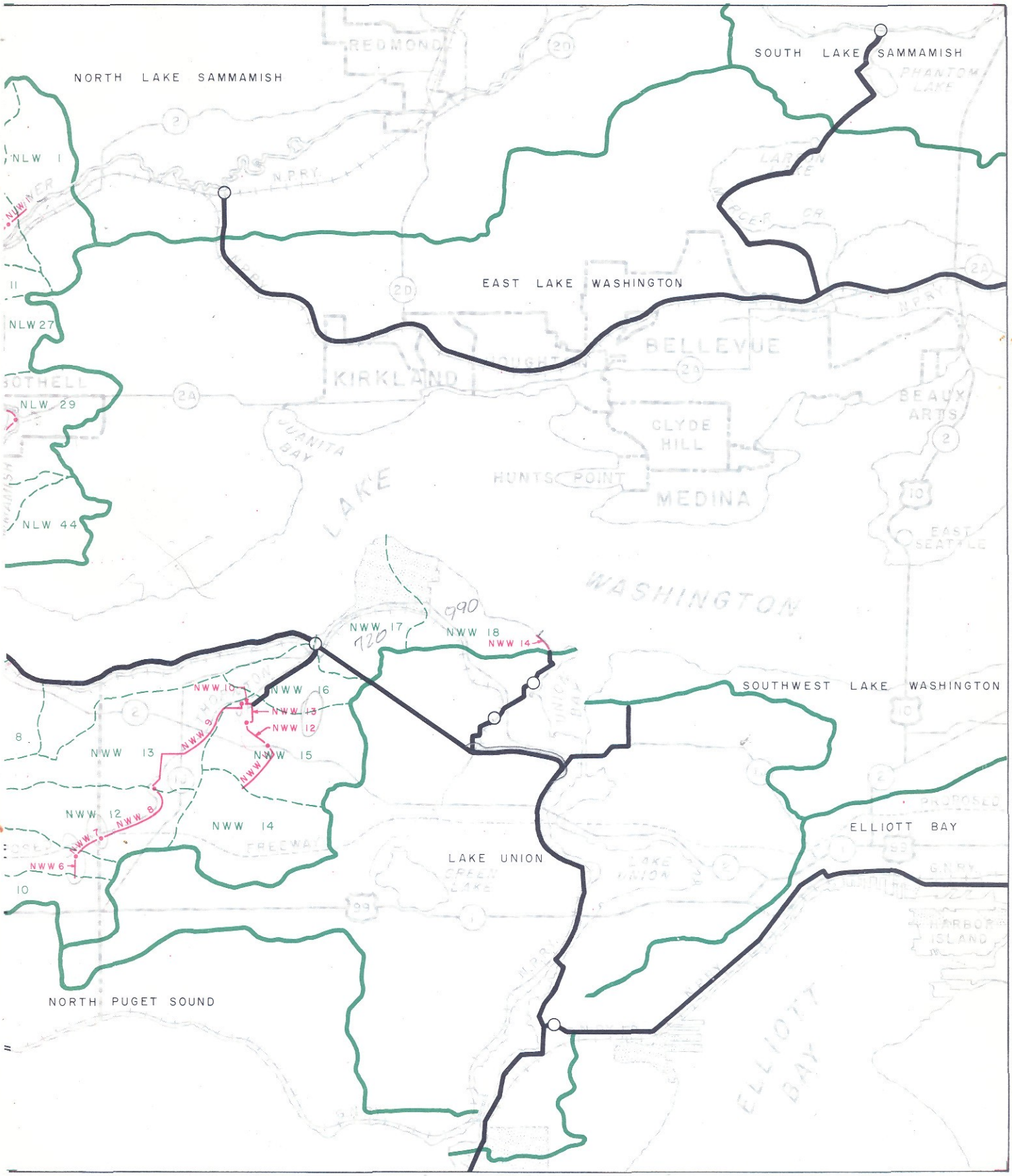

NORTH LAKE SAMMAMISH
SOUTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
SOUTHWEST LAKE WASHINGTON
LAKE UNION
ELLIOTT BAY
NORTH PUGET SOUND
REDMOND
KIRKLAND
BELLEVUE
BOTHELL
MEDINA
HUNTS POINT
CLYDE HILL
BEAUX ARTS
EAST SEATTLE
PHANTOM LAKE
JUANITA BAY
HARBOR ISLAND
NLW 1
NLW 11
NLW 27
NLW 29
NLW 44
NWW 6
NWW 7
NWW 8
NWW 9
NWW 10
NWW 12
NWW 13
NWW 14
NWW 15
NWW 16
NWW 17
NWW 18
720
990

Table 15-34. Description and Estimated Construction Costs, Service Sewers, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
North Lake Washington Sewerage Area				
NLW-1	0.7	2.0	1,600 ft of 15-in. RC at 0.24%, average cut 11 ft, difficult wet, includes sheeting	46,000
NLW-2	1.2	3.4	2,300 ft of 18-in. RC at 0.25%, average cut 13 ft, difficult wet, includes sheeting	84,000
NLW-3	1.2	3.0	2,200 ft of 15-in. RC at 0.52%, average cut 7 ft, difficult wet	37,000
NLW-4	1.9-2.7	5.1-7.2	12,100 ft of 24-in. RC at 0.12-1.0%, average cut 7 ft, difficult wet	270,000
NLW-5	1.0	2.7	5,100 ft of 12-in. RC at 2.3 - 3.4%, average cut 6 ft, dry to moderately wet	47,000
NLW-6	3.7	9.9	2,000 ft of 24-in. RC at 0.60%, average cut 7 ft, dry to moderately wet	33,000
NLW-7	4.0-6.7	11-18	15,900 ft of 27-in. RC at 0.33 - 1.5%, average cut 7 ft, wet to difficult wet	~~365,000~~
NLW-8	7.2	20	2,800 ft of 30-in. RC at 0.57 - 0.70%, average cut 9 - 12 ft, difficult wet, includes sheeting	92,000
NLW-9	8.8	24	6,900 ft of 39-in. RC at 0.20%, average cut 10 - 12 ft, difficult wet, includes sheeting	297,000
NLW-10	0.8	2.0	5,500 ft of 12-in. RC at 1.2%, average cut 8 ft, dry to moderately wet	53,000
NLW-11	1.3	3.6	3,900 ft of 15-in. RC at 1.5%, average cut 6 ft, dry to moderately wet	42,000
NLW-12	2.0-2.6	5.2-6.9	9,300 ft of 18-in. RC at 0.83 - 1.2%, average cut 6 ft, wet, includes special bedding	173,000
NLW-13	0.8	2.0	2,300 ft of 12-in. RC at 1.5%, average cut 8 ft, dry to moderately wet	30,000
NLW-14	1.0-2.3	2.6-6.1	21,600 ft of 18-in. RC at 0.15 - 1.4%, average cut 7 - 17 ft, wet to difficult wet in peaty material, includes special bedding in peat, imported backfill and repaving	464,000
NLW-15	5.1	14	5,800 ft of 24-in. RC at 0.90%, average cut 7 - 12 ft, wet, includes special bedding	143,000
NLW-16	5.9	16	8,900 ft of 27-in. RC at 0.70 - 1.1%, average cut 7 - 12 ft, wet	211,000
NLW-17	0.5	1.4	900 ft of 12-in. RC at 1.1%, average cut 6 ft, wet	12,000
NLW-18	1.6	4.3	5,300 ft of 15-in. RC at 1.5 - 1.9%, average cut 6 ft, wet	69,000
NLW-19	7.5-9.7	20-26	12,700 ft of 33-in. RC at 0.34 - 0.86%, average cut 7 - 8 ft, wet to difficult wet	354,000
NLW-20	10	27	2,400 ft of 36-in. RC at 0.40%, average cut 10 ft, difficult wet	74,000
NLW-21	11	28	3,000 ft of 39-in. RC at 0.27%, average cut 9 ft, difficult wet	101,000
NLW-22	11	30	4,900 ft of 42-in. RC at 0.21%, average cut 8-10 ft, difficult wet	178,000
NLW-23	20-21	54-57	11,600 ft of 66-in. RC at 0.065 - 0.07%, average cut 12 - 17 ft, difficult wet, includes sheeting, piling and highway crossing	911,000
NLW-24	21-22	58-59	6,500 ft of 72-in. RC at 0.047%, average cut 20 - 22 ft, difficult wet, includes sheeting, piling and railroad crossing	684,000
NLW-25	0.6-1.6	1.6-4.1	11,200 ft of 15-in. RC at 0.28 - 1.0%, average cut 6 - 12 ft, dry to difficult wet	195,000
NLW-26	1.9	5.1	3,300 ft of 18-in. RC at 0.79%, average cut 7 ft, difficult wet	61,000
NLW-27	2.3-3.1	6.2-8.3	11,800 ft of 21-in. RC at 0.37 - 0.76%, average cut 7-12 ft, wet to difficult wet, includes sheeting	292,000
NLW-28	0.4-0.8	1.1-2.1	10,200 ft of 12-in. RC at 1.5-1.6%, average cut 7 - 8 ft, wet to difficult wet	151,000
NLW-29	4.2	11	11,600 ft of 24-in. RC at 0.61 - 1.0%, average cut 7 ft, wet	235,000

Continued on next page

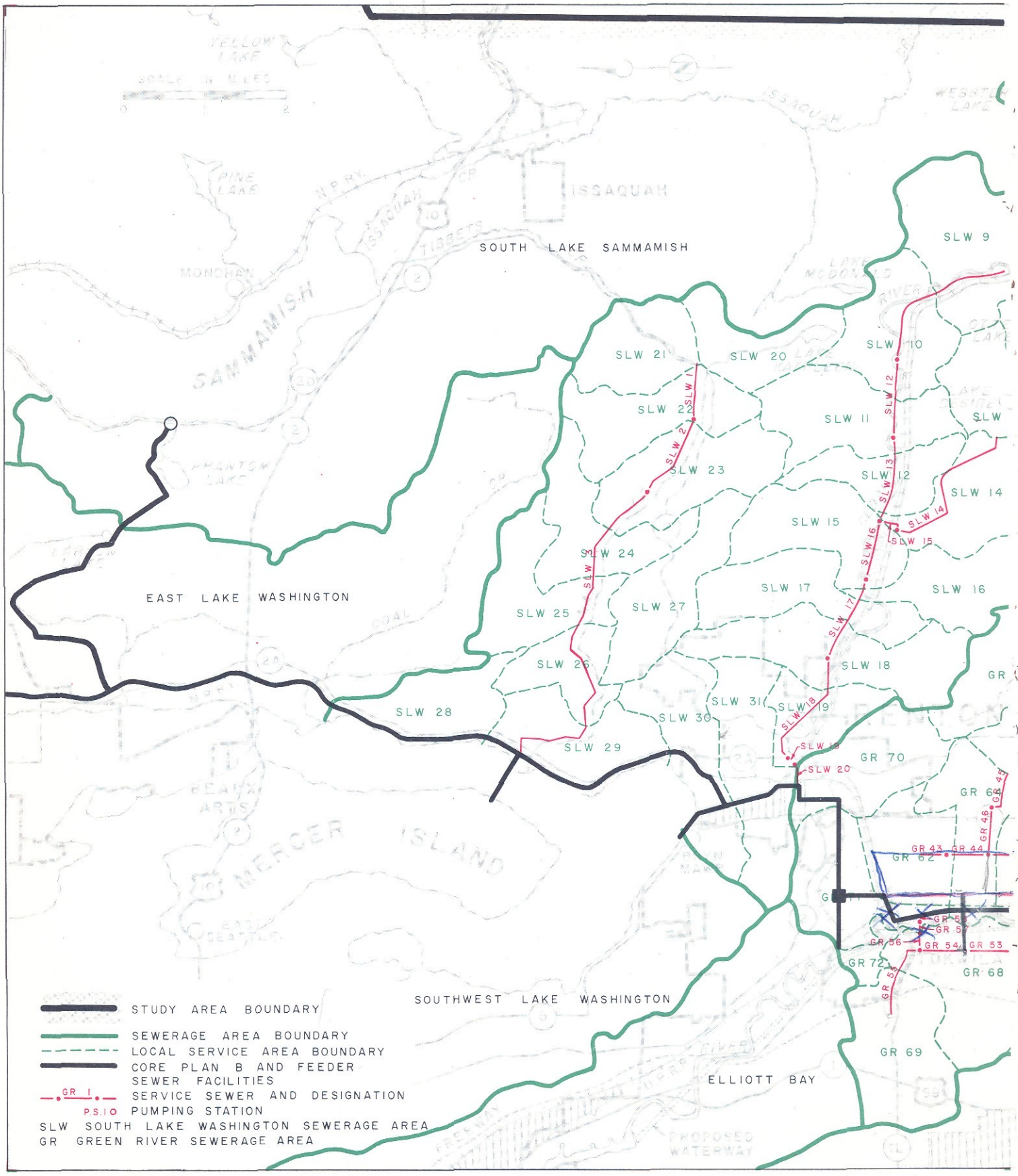
Table 15-34. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
NLW-30	0.8-1.7	2.1-4.6	13,100 ft of 15-in. RC at 0.29 - 1.3%, average cut 6 - 7 ft, difficult wet in peaty material, includes special bedding	245,000
NLW-31	2.4	6.3	4,700 ft of 18-in. RC at 1.2%, average cut 7 ft, wet to difficult wet in peaty material, includes special bedding	95,000
NLW-32	7.0	19	12,000 ft of 27-in. RC at 0.85 - 1.3%, average cut 11 ft, wet to difficult wet	303,000
NLW-33	8.0-8.8	22-23	7,500 ft of 30-in RC at 0.65 - 0.80%, average cut 9 - 14 ft, difficult wet, includes railroad and highway crossings	232,000
NLW-34	31	82	2,100 ft of 72-in. RC at 0.095%, average cut 23 ft, difficult wet, includes sheeting and piling	228,000
Total contract cost, North Lake Washington Sewerage Area				6,807,000
Engineering and contingencies, 25 per cent				1,702,000
Total construction cost, North Lake Washington Sewerage Area				8,509,000
Northwest Lake Washington Sewerage Area				
NWW-1	0.9	2.3	2,000 ft of 12-in. RC at 3.5%, average cut 8 ft, wet, includes railroad and highway crossings	35,000
NWW-2	0.6-1.4	1.6-3.6	7,500 ft of 12-in. RC at 1.6 - 3.1%, average cut 6 - 7 ft, wet, includes imported backfill and repaving	113,000
NWW-3	1.5-1.8	4.0-4.6	4,700 ft of 15-in. RC at 1.6%, average cut 6-7 ft, wet to difficult wet, includes railroad and highway crossings	80,000
NWW-4	0.6	1.6	2,500 ft of 12-in. RC at 0.56%, average cut 6 ft, wet	33,000
NWW-5	1.3-3.7	3.6-9.3	24,000 ft of 18-in. RC at 0.43 - 1.9%, average cut 6 - 19 ft, wet to difficult wet in peaty material, includes sheeting and special bedding in peat, imported backfill, repaving and railroad and highway crossings	568,000
NWW-6	0.9	2.2	1,500 ft of 15-in. RC at 0.33%, average cut 6 ft, difficult wet	24,000
NWW-7	1.7	4.0	2,700 ft of 18-in. RC at 0.56%, average cut 6 ft, difficult wet	48,000
NWW-8	2.1-2.6	5.4-6.8	6,100 ft of existing 30-in. RC at 0.5 - 3.6%	Existing
NWW-9	3.4-5.7	9.1-16	9,800 ft of existing 27-in. and 30-in. RC at 0.6 - 13.5%	Existing
NWW-10	5.9	17	1,000 ft of existing 42-in. RC at 0.3%	Existing
NWW-11	1.0-1.4	2.5-3.4	4,700 ft of 12-in. RC at 1.8 - 2.2%, average cut 6 - 7 ft, dry to moderately wet, includes highway crossing	48,000
NWW-12	2.2	5.6	2,700 ft of existing 30-in. RC at 0.9 - 2.7%	Existing
NWW-13	2.7	7.3	1,600 ft of existing 36-in. RC at 0.45%	Existing
NWW-14	1.5	4.4	700 ft of existing 42-in. at 0.05%. Cost is for 1,100,000, 400,000, 1,400,000, 2,150,000 and 640,000 gal. holding tanks on tributary local sewers	549,000
Total contract cost, Northwest Lake Washington Sewerage Area				1,498,000
Engineering and contingencies, 25 per cent				374,000
Total construction cost, Northwest Lake Washington Sewerage Area				1,872,000

See Fig. 15-16 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*


SCALE IN MILES
0 1 2
YELLOW LAKE
PINE LAKE
ISSAQUAH
WEBSTER LAKE
MONOHAN
SAMMAMISH
TIBBETS
SOUTH LAKE SAMMAMISH
LAKE McDONALD
PHANTOM LAKE
EAST LAKE WASHINGTON
COAL
MERCER ISLAND
BEAUX ARTS
EAST SEATTLE
SOUTHWEST LAKE WASHINGTON
ELLIOTT BAY
PROPOSED WATERWAY
SLW 9
SLW 10
SLW 11
SLW 12
SLW 13
SLW 14
SLW 15
SLW 16
SLW 17
SLW 18
SLW 19
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GR 68
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GR 72
GR 62
GR 43
GR 44
GR 46
GR 53
GR 54
GR 56
STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
CORE PLAN B AND FEEDER SEWER FACILITIES
GR 1 SERVICE SEWER AND DESIGNATION
P.S.10 PUMPING STATION
SLW SOUTH LAKE WASHINGTON SEWERAGE AREA
GR GREEN RIVER SEWERAGE AREA

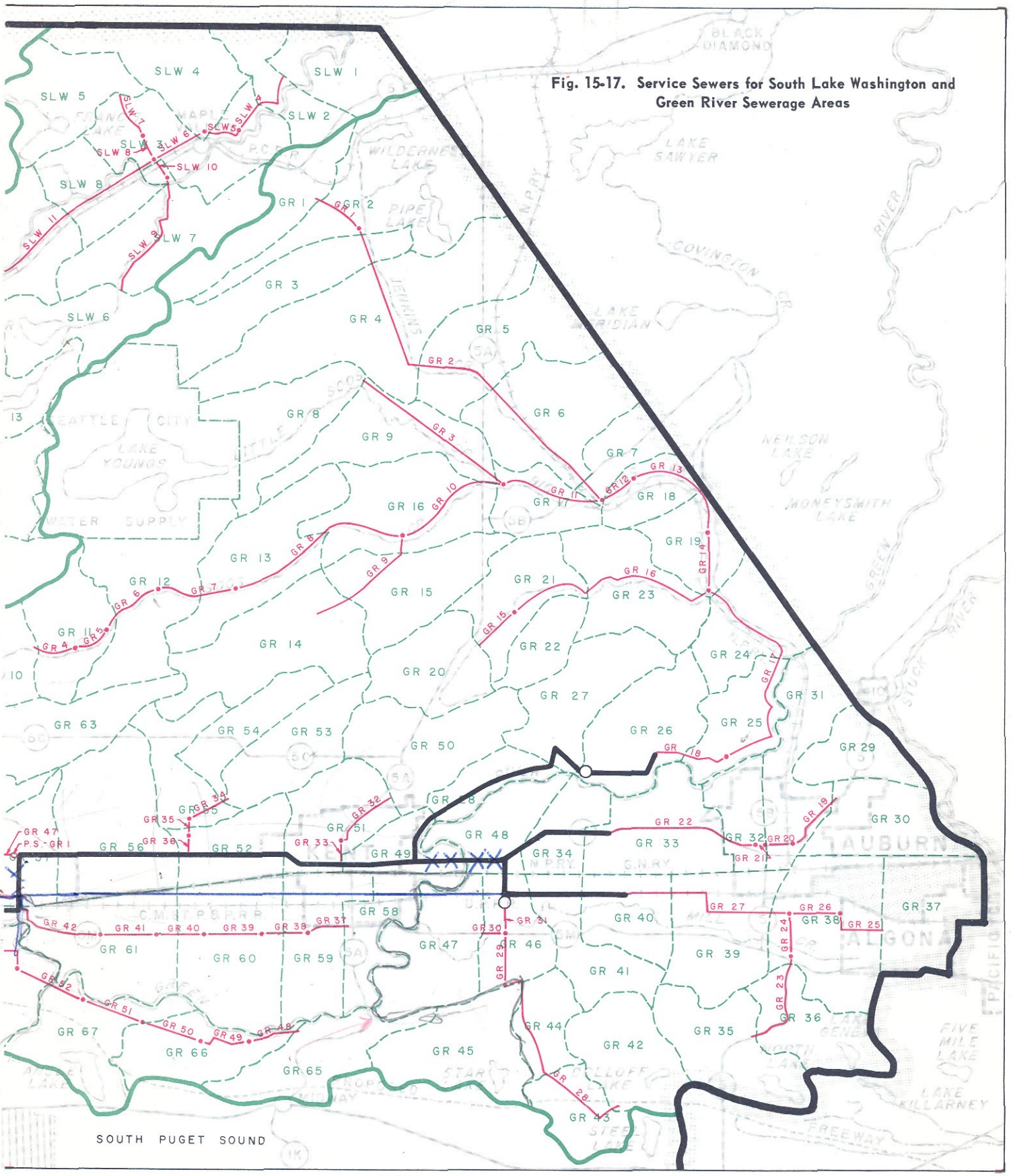

Fig. 15-17. Service Sewers for South Lake Washington and Green River Sewerage Areas

Island, which would discharge into a force main included under the feeder sewer system. The location of the pumping station may have to be changed during final design if it is decided that the route of the crossing to the east side of Lake Washington must avoid the submerged forests in the lake. This possibility does not affect the basic layouts either of the service sewers or of the central sewerage system. ELW 41 and PS-ELW 13 serve the southern portion of Mercer Island.

North Lake Washington Sewerage Area

Service sewers for the North Lake Washington sewerage area are laid out to convey the sewage of that area to a pumping station about two miles west of Bothell (Fig. 15-16). Descriptions and estimated costs of these facilities are given in Table 15-34. Major elements of the system are as follows:

1. A main trunk (NLW 1, 2, 9, 23, 24, and 34) along Sammamish River valley from about 4,000 feet southeast of Woodinville to the pumping station west of Bothell (PS-N 1). The latter is included in the feeder sewer system.
2. A trunk (NLW 3-NLW 8) along Bear Creek to serve the Bear Creek basin.
3. A trunk (NLS10-NLW22) along North and Penny creeks. This trunk would serve the Intercity and Silver Lake areas.
4. A trunk (NLW 25-NLW 33) along Swamp Creek. This trunk would serve the Fairmount, Mirror Lake and Lynwood areas.

Northwest Lake Washington Sewerage Area

Service sewers for the Northwest Lake Washington sewerage area consist of two systems (Fig. 15-16). Sewers for the first system are laid out to convey sewage of the northern portion of this area into the feeder sewers which are routed along the shore of Lake Washington. Sewers for the second system serve a small area bordering Lake Washington south of Sand Point Naval Air Station and discharge into the feeder sewers in the Lake Union sewerage area. Descriptions and estimated costs of the facilities in both systems are given in Table 15-34. Major elements are as follows:

1. A trunk (NWW 1) west of Kenmore.
2. A trunk (NWW 2-NWW 3) along Lyons Creek to serve a portion of Mountlake Terrace and surrounding areas.
3. A trunk (NWW 4-NWW 5) along McAleer and Hall creeks to serve the Lake Ballinger and surrounding areas, including the remainder of Mountlake Terrace.
4. A trunk (NWW 6-NWW 10) north from the existing Lake City sewage treatment plant of the city of Seattle. This trunk would incorporate existing sewers of the city of Seattle and would serve the northern part of the city, as well as the Ronald area and other areas to the north.
5. A trunk (NWW 11-NWW 13) west and south from the existing Lake City treatment plant. This trunk would incorporate existing sewers of the city of Seattle.
6. A trunk serving the waterfront area south of Sand Point Naval Air Station (NWW 14). This trunk would incorporate existing sewers of the city of Seattle and would receive sewage from the areas served by combined sewers. Storm water holding tanks sized to allow one overflow per summer would be provided on local sewers tributary to the trunk.

South Lake Washington Sewerage Area

Service sewers for the South Lake Washington sewerage area are laid out to convey the sewage of that area into the feeder sewer system (Fig. 15-17). Descriptions and estimated costs of these facilities are given in Table 15-35. Major elements of the system are as follows:

1. A trunk (SLW 1-SLW 3) along May Creek.
2. A trunk (SLW 4-SLW 20) along Cedar River to serve Maple Valley and other areas to the east, as well as all local areas draining to Cedar River.

Green River Sewerage Area

Service sewers for the Green River sewerage area are laid out to convey the sewage of that area into the feeder sewer system (Fig. 15-17). Descriptions and estimated costs of these facilities are given in Table 15-35. Major elements of the system are as follows:

1. A trunk (GR 1-GR 2) along Jenkins Creek to serve the Piper and Wilderness Lake areas. This trunk would also serve the major portion of the proposed Covington industrial area.
2. A trunk (GR 3-GR 14) along Big and Little Soos creeks. This trunk would serve areas to the north almost to the south city limit of Renton, as well as a portion of the proposed Covington industrial area.
3. A trunk (GR 15-GR 16) to serve areas west of Lake Meridian.
4. A trunk (GR 17-GR 18) along Green River which would discharge to the feeder sewer system. This trunk would serve areas east of Green river valley.
5. A trunk (GR 19-GR 22) along State Highway 5 to serve the city of Auburn and areas to the south which lie between Green River and the Great Northern railroad. This trunk would incorporate existing sewers of the city of Auburn.
6. A trunk (GR 23-GR 27) to serve areas west of the Great Northern railroad, including the city of Algona.
7. A trunk (GR 28-GR 31) to serve the Steel and Star Lake areas, as well as areas south of Green River.
8. Trunks (GR 32-GR 36) to serve part of the city of Kent and areas to the east of that city.

Table 15-35. Description and Estimated Construction Costs, Service Sewers, South Lake Washington and Green River Sewerage Areas

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
South Lake Washington Sewerage Area				
SLW-1	1.0-1.5	2.7-4.3	3,600 ft of 18-in. RC at 0.16 - 0.4%, average cut 7-11 ft, difficult wet, includes sheeting	95,000
SLW-2	1.8	5.2	5,400 ft of 21-in. RC at 0.26%, average cut 15 - 19 ft, difficult wet, includes sheeting	227,000
SLW-3	2.2-5.1	6.2-14	24,000 ft of 24-in. RC at 0.18 - 2.4%, average cut 6 - 24 ft, dry to difficult wet, includes sheeting and railroad and highway crossings	602,000
SLW-4	0.5-0.9	1.5-2.5	4,700 ft of 12-in. RC at 1.1 - 1.5%, average cut 6 - 9 ft, dry to moderately wet	45,000
SLW-5	0.9	2.5	2,900 ft of 15-in. RC at 0.36%, average cut 17 - 20 ft, dry to moderately wet	40,000
SLW-6	2.0	5.4	3,900 ft of 18-in. RC at 0.80%, average cut 12 ft, dry to moderately wet, includes imported backfill and repaving	77,000
SLW-7	0.5	1.5	2,800 ft of 12-in. RC at 3.8%, average cut 6 ft, dry to moderately wet	26,000
SLW-8	1.1	3.1	1,900 ft of 15-in. RC at 1.1%, average cut 6 ft, dry to moderately wet	21,000
SLW-9	0.6-0.9	1.6-2.7	8,300 ft of 12-in. RC at 1.0 - 3.0%, average cut 6 ft, dry to moderately wet, includes access road and Cedar River crossing	86,000
SLW-10	0.9	2.7	1,400 ft of 18-in. RC at 0.16%, average cut 7 ft, dry to moderately wet, includes railroad crossing	20,000
SLW-11	3.9-5.7	11-16	23,100 ft of 27-in. RC at 0.44 - 0.66%, average cut 7 - 8 ft, dry to wet, includes imported backfill, repaving, and Cedar River crossing	523,000
SLW-12	6.2	18	5,100 ft of 30-in. RC at 0.45%, average cut 8 ft, wet	125,000
SLW-13	7.0	20	5,800 ft of 33-in. RC at 0.35%, average cut 7 - 8 ft, wet	149,000
SLW-14	0.5-1.2	1.5-3.4	11,700 ft of 12-in. RC at 0.74 - 4.4%, average cut 6 ft, dry to moderately wet, includes access road	117,000
SLW-15	1.2	3.4	1,700 ft of 15-in. RC at 1.1%, average cut 7 ft, wet, includes railroad crossing	28,000
SLW-16	9.0	26	5,000 ft of 33-in. RC at 0.57 - 0.64%, average cut 7 ft, wet	127,000
SLW-17	10-11	28-31	5,100 ft of 36-in. RC at 0.4 - 0.5%, average cut 8 - 11 ft, wet, includes imported backfill, repaving, and Cedar River crossing	169,000
SLW-18	12-13	33-35	8,300 ft of 42-in. RC at 0.27 - 0.59%, average cut 10 - 13 ft, wet to difficult wet, includes imported backfill and repaving	364,000
SLW-19	13	35	400 ft of twin 24-in. inverted siphons, includes inlet and outlet structures	28,000
SLW-20	13	35	1,700 ft of 42-in. RC at 0.29%, average cut 15 ft, difficult wet, includes sheeting, dewatering, imported backfill and repaving	104,000
Total contract cost, South Lake Washington Sewerage Area				2,973,000
Engineering and contingencies, 25 per cent				743,000
Total construction cost, South Lake Washington Sewerage Area				3,716,000
Green River Sewerage Area				
GR-1	1.0	2.4	2,700 ft of 18-in. RC at 0.18%, average cut 6 ft, dry to moderately wet	33,000
GR-2	4.8-9.6	11-22	26,900 ft of 36-in. RC at 0.08 - 2.4%, average cut 7 ft, dry to difficult wet, includes railroad crossings	676,000
GR-3	2.2-2.9	4.5-6.3	10,400 ft of 18-in. RC at 0.44 - 1.4%, average cut 6 ft, dry to moderately wet	126,000

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Table 15-35. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
GR-4	0.7	1.8	2,500 ft of 12-in. RC at 0.72%, average cut 6 ft, dry to moderately wet	23,000
GR-5	1.2	3.0	2,300 ft of 18-in. RC at 0.22%, average cut 6 ft, difficult wet	41,000
GR-6	1.3	3.6	4,200 ft of 21-in. RC at 0.13%, average cut 8 ft, difficult wet	85,000
GR-7	2.0	5.3	5,200 ft of 27-in. RC at 0.075%, average cut 9 ft, wet to difficult wet	127,000
GR-8	2.4-2.8	6.3-7.3	11,600 ft of 30-in. RC at 0.06 - 0.24%, average cut 8 - 10 ft, wet to difficult wet	286,000
GR-9	0.9	2.6	9,400 ft of 24-in. RC at 1.3%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	94,000
GR-10	3.7-5.3	9.8-14	8,300 ft of 36-in. RC at 0.055 - 0.16%, average cut 7 ft, dry to moderately wet	183,000
GR-11	8.2-8.5	20-21	7,300 ft of 36-in. RC at 0.41 - 0.86% average cut 7 ft, dry to moderately wet	161,000
GR-12	18	44	3,100 ft of 36-in. RC at 1.6%, average cut 7 ft, dry to moderately wet	68,000
GR-13	18	44	8,100 ft of 39-in. RC at 0.86%, average cut 7 ft, dry to moderately wet	211,000
GR-14	19	46	4,200 ft of 48-in. RC at 0.25%, average cut 9 ft, dry to moderately wet	145,000
GR-15	0.6	1.6	2,900 ft of 12-in. RC at 0.5%, average cut 8 ft, dry to moderately wet	28,000
GR-16	1.0-2.0	2.9-5.3	15,000 ft of 15-in. RC at 0.7 - 1.9%, average cut 6 - 8 ft, dry to moderately wet, includes access road	176,000
GR-17	21-22	51-53	15,800 ft of 48-in. RC at 0.32 - 0.84%, average cut 12 - 23 ft, dry to moderately wet, includes access road	623,000
GR-18	22	54	6,500 ft of 51-in. RC at 0.25 - 0.53%, average cut 14 - 16 ft, wet, includes imported backfill and repaving	406,000
GR-19	0.3	0.8	3,500 ft of 12-in. RC at 0.57%, average cut 8 ft, difficult wet, includes imported backfill and repaving	45,000
GR-20	3.3	7.4	2,400 ft of existing 24-in. at 0.24%	Existing
GR-21	4.0	9.0	600 ft of existing 30-in. at 0.13%	Existing
GR-22	4.0-4.6	9.0-11	9,700 ft of 30-in. RC at 0.14 - 0.67%, average cut 7 - 8 ft, difficult wet, includes imported backfill and repaving	288,000
GR-23	0.8-1.4	2.0-3.6	6,500 ft of 12-in. RC at 2.5 - 5.0%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	73,000
GR-24	1.4	3.6	2,900 ft of 21-in. RC at 0.21%, average cut 8 ft, difficult wet, includes sheeting	81,000
GR-25	2.4-2.9	4.9-6.1	3,900 ft of 24-in. RC at 0.13 - 0.18%, average cut 7 - 9 ft, difficult wet, includes imported backfill, repaving and sheeting	126,000
GR-26	4.5	9.8	3,500 ft of 30-in. RC at 0.15%, average cut 10 ft, difficult wet, includes imported backfill, repaving and sheeting	145,000
GR-27	5.9-6.6	13-15	12,300 ft of 36-in. RC at 0.10 - 0.13%, average cut 8 - 10 ft, difficult wet, includes imported backfill and repaving	376,000
GR-28	0.7-1.5	1.8-4.0	13,900 ft of 12-in. RC at 2.0-- 4.5%, average cut 6 ft, dry to difficult wet, includes imported backfill, repaving and access road	156,000
GR-29	2.4	6.0	3,500 ft of 21-in. RC at 0.49%, average cut 10 ft, difficult wet	77,000
GR-30	0.6	1.6	1,500 ft of 15-in. RC at 0.16%, average cut 11 ft, difficult wet, includes imported backfill and repaving	38,000
GR-31	3.4	8.7	2,500 ft of 30-in. RC at 0.12%, average cut 15 ft, difficult wet, includes imported backfill, repaving and sheeting	126,000

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Table 15-35. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
GR-32	0.6	1.8	5,000 ft of 12-in. RC at 3.1%, average cut 6 ft, dry to moderately wet, includes access road	52,000
GR-33	1.1	2.9	1,500 ft of 15-in. RC at 0.55%, average cut 7 ft, difficult wet, includes imported backfill and repaving	30,000
GR-34	0.8	2.4	2,500 ft of 12-in. RC at 2.4%, average cut 6 ft, wet	33,000
GR-35	1.2	3.4	1,000 ft of 15-in. RC at 0.75%, average cut 7 ft, difficult wet	17,000
GR-36	2.1	5.3	1,200 ft of 18-in. RC at 0.65%, average cut 10 ft, difficult wet	25,000
GR-37	0.4	1.0	2,300 ft of 12-in. RC at 0.20%, average cut 10 ft, difficult wet, includes imported backfill and repaving	50,000
GR-38	0.8	2.0	2,900 ft of 15-in. RC at 0.24%, average cut 13 ft, difficult wet, includes imported backfill, repaving and sheeting	97,000
GR-39	1.5	4.1	3,700 ft of 21-in. RC at 0.17%, average cut 17 ft, difficult wet, includes imported backfill, repaving and sheeting	170,000
GR-40	1.9	5.0	2,900 ft of 24-in. RC at 0.12%, average cut 19 ft, difficult wet, includes imported backfill, repaving and sheeting	153,000
GR-41	2.1	5.7	4,000 ft of 27-in. RC at 0.085%, average cut 19 ft, difficult wet, includes imported backfill, repaving and sheeting	231,000
GR-42	2.4-2.8	6.8-7.4	6,100 ft of 30-in. RC at 0.067 - 0.085%, average cut 22 ft, difficult wet, includes imported backfill, repaving and sheeting	414,000
GR-43	1.5	3.2	1,700 ft of 18-in. RC at 0.22%, average cut 8 ft, difficult wet	32,000
GR-44	1.9	4.3	2,700 ft of 21-in. RC at 0.18%, average cut 11 ft, difficult wet	62,000
GR-45	0.6	1.6	2,300 ft of 12-in. RC at 4.3%, average cut 6 ft, wet	31,000
GR-46	0.8	2.2	3,100 ft of 15-in. RC at 0.42%, average cut 6 ft, difficult wet	50,000
GR-47	3.2	7.9	2,500 ft of 24-in. RC at 0.30%, average cut 20 ft, difficult wet, includes sheeting	110,000
GR-48	0.6	1.6	3,300 ft of 12-in. RC at 1.5%, average cut 6 ft, dry to moderately wet	31,000
GR-49	0.8	2.1	3,500 ft of 15-in. RC at 0.51%, average cut 12 ft, dry to moderately wet, includes imported backfill and repaving	62,000
GR-50	1.2	3.0	4,400 ft of 18-in. RC at 0.20%, average cut 12 ft, difficult wet, includes sheeting	123,000
GR-51	1.3	3.4	4,000 ft of 21-in. RC at 0.12%, average cut 8 ft, difficult wet	82,000
GR-52	1.8	4.9	5,000 ft of 24-in. RC at 0.12%, average cut 13 ft, difficult wet, includes sheeting	163,000
GR-53	3.1	7.5	5,000 ft of 30-in. RC at 0.08%, average cut 17 ft, difficult wet, includes sheeting	220,000
GR-54	4.5	10	2,700 ft of 36-in. RC at 0.06%, average cut 17 ft, difficult wet, includes sheeting	130,000
GR-55	0.5-0.8	1.4-2.2	5,600 ft of 12-in. RC at 1.7-2.3%, average cut 6 ft, dry to difficult wet, includes imported backfill and repaving	76,000
GR-56	6.5	15	1,200 ft of 42-in. RC at 0.055%, average cut 18 ft, difficult wet, includes sheeting	67,000
GR-57	6.5	15	400 ft of twin 18-in. inverted siphons across Duwamish Waterway, includes inlet and outlet structures	21,000
GR-58	6.5	15	800 ft of 42-in. RC at 0.055%, average cut 24 ft, difficult wet, includes sheeting	57,000
Subtotal, sewers				7,612,000

Continued on next page

Table 15-35. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
PS-GR-1	3.2	7.9	Pumping station, single stage, motor driven, static lift 24 ft, total head at peak flow 30 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering	111,000
Total contract cost, Green River Sewerage Area				7,723,000
Engineering and contingencies, 25 per cent				1,931,000
Total construction cost, Green River Sewerage Area				9,654,000

See Fig. 15-17 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

9. A trunk (GR 37-GR 42) to serve Green River valley west of the Great Northern railroad and east of Green River.

10. A trunk (GR 43-GR 47) to serve areas south of the city of Renton. A pumping station (PS-GR 1) would be required to lift the sewage from the trunk to the feeder sewer system.

11. A trunk (GR 48-GR 58) to serve areas west of Green River.

Southwest Lake Washington Sewerage Area

Service sewers for the Southwest Lake Washington sewerage area consist of three existing systems, all with combined sewers which serve the portions of this area lying within the city of Seattle (Fig. 15-18). Service to the southern portion, including the Bryn Mawr-Lake Ridge area, would be obtained by the feeder sewers. Descriptions and estimated costs of the required facilities are given in Table 15-36.

The first service sewer system serves lower Rainier valley from Seward Park on the north to the city limit of Seattle on the south. This system would incorporate existing facilities of the city of Seattle and would include the following major elements:

1. A waterfront interceptor (SWW 1-SWW 2) serving areas draining directly to Lake Washington. Storm water holding tanks sized to allow one overflow per summer would be provided on tributary local sewers.

2. A pumping station (PS-SWW 1), which would discharge through a force main (SWW 3) and gravity sewer (SWW 4) to the feeder sewer system.

3. A trunk (SWW 5-SWW 7) serving Rainier valley and discharging to the feeder sewer system. Since this trunk does not have sufficient capacity for the storm flow resulting from a rainfall with a recurrence interval of once in 10 years, partial separation of the tributary area (local service areas SWW 6 and SWW 5) would be required.

4. An overflow line (SWW 8-SWW 9) which receives storm water overflow both from sewer SWW 6 and from local sewers tributary to sewer SWW 4. A storm water holding tank sized to allow one overflow per summer would be provided at the discharge end of this line.

The second system serves upper Rainier valley from about Yesler Way on the north to Seward Park on the south. This system would incorporate existing facilities of the city of Seattle and would include the following major elements:

1. A trunk (SWW 10) serving areas draining directly to Lake Washington. Storm water holding tanks sized to allow one overflow per summer would be provided on tributary local sewers.

2. A pumping station (PS-SWW 2) which discharges through a gravity sewer (SWW 11) to the feeder sewer system. Partial separation of the tributary area (local service area SWW 10) would be required.

3. A trunk serving northern Rainier valley (SWW 12-SWW 13) and discharging to the feeder sewer system.

The third system serves the narrow waterfront strip bordering Lake Washington from Edgewater Park on the north to Coleman Park on the south. This system would incorporate existing facilities of the city of Seattle and would include the following major elements:

1. A pumping station (PS-SWW 3) which discharges to the feeder sewer system through a force main (SWW 14) and a high elevation gravity sewer (SWW 15, SWW 17-SWW 20). Storm water holding tanks sized to allow one overflow per summer would be provided on local sewers tributary to the pumping station. Partial separation of the tributary area (local service area SWW 15) would be required.

2. A pumping station (PS-SWW 4) which discharges through a force main (SWW 16) to the high elevation gravity sewer. Storm water holding tanks sized to

allow one overflow per summer would be provided on local sewers tributary to the pumping station.

Elliott Bay Sewerage Area

Service sewers for the Elliott Bay sewerage area are laid out to convey the sewage of that area into the central sewerage system. Descriptions and estimated costs of the service sewers are given in Table 15-36. Major elements are shown in Fig. 15-18 and are as follows:

1. A trunk (EB 1-EB 2) to serve industrial and residential areas east of West Duwamish.
2. A trunk system (EB 3-EB 18) to serve industrial and residential areas west of Duwamish River. To avoid excessive cuts, two pumping stations (PS-EB 1 and PS-EB 2) would be required. This system will include the Delridge trunk sewer (EB 4-EB 6), for which final designs have been prepared by the engineering department of the city of Seattle. Lengths, sizes and slopes of sewers included in this section of the trunk (Table 15-36) are exactly as designed by the city and are considered to be adequate.
3. A waterfront interceptor (EB 19) to serve the Magnolia Bluff area.

Lake Union Sewerage Area

Service sewers for the Lake Union sewerage area consist essentially of existing combined sewers of the city of Seattle which are tributary to the North Trunk (Fig. 15-18). The latter is to be incorporated in the central sewerage project and would convey sewage of the Lake Union area to the West Point treatment plant. Descriptions and estimated costs of the required facilities are given in Table 15-36. Major components are as follows:

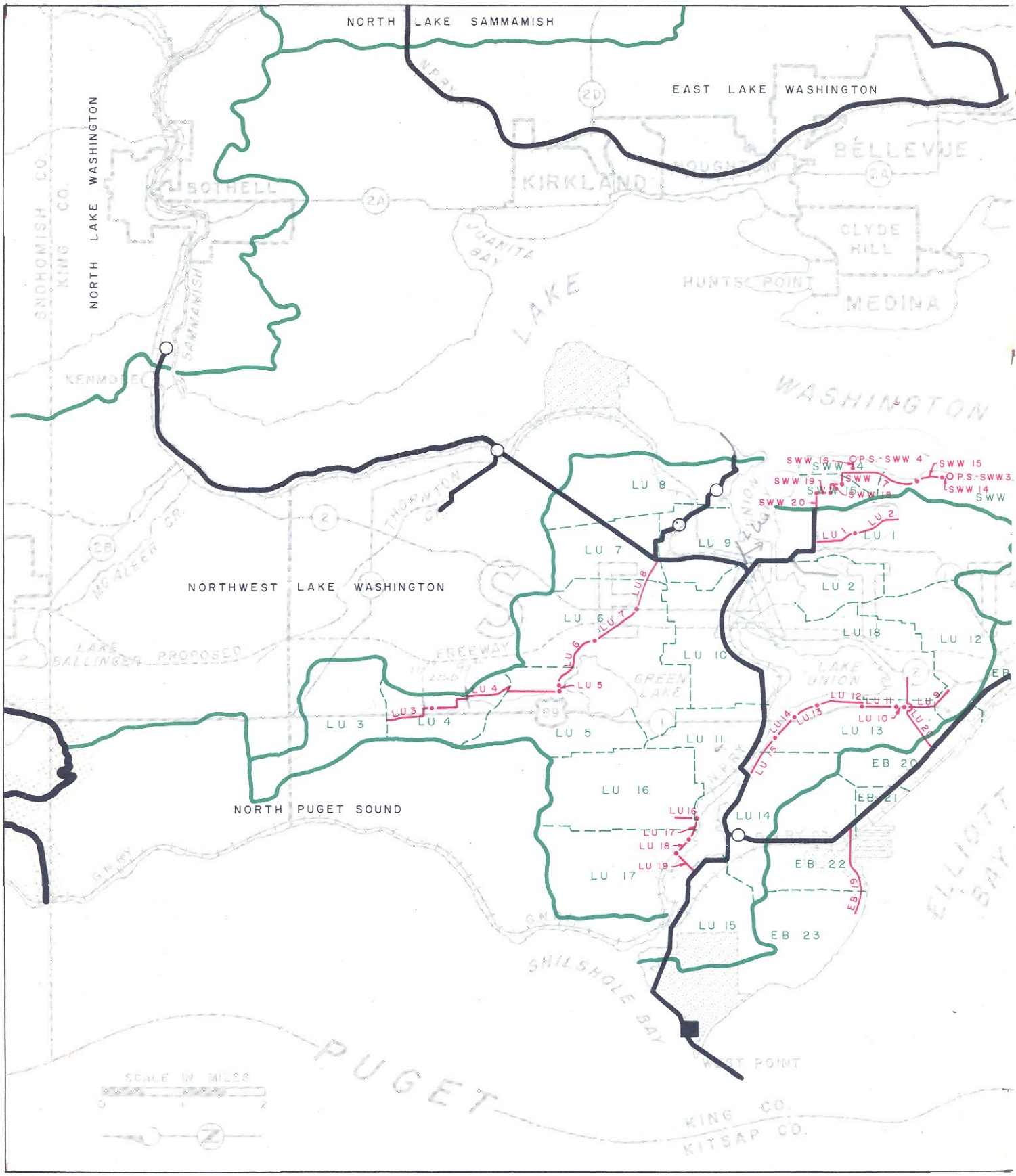
1. A trunk (LU 1-LU 2) to serve the Broadmoor and Madrona areas. Partial separation of the tributary area (local service area LU 1) would be required.
2. A trunk (LU 3-LU 4) to serve the Haller Lake area.
3. A trunk (LU 5-LU 6) to serve the area north of Green Lake.
4. A trunk (LU 7-LU 8) to serve all the area draining to Green Lake. Partial separation of a portion of the tributary area (local service area LU 6) would be required. Trunk LU 7-LU 8 includes the existing sewer of the city of Seattle (LU 8) which was damaged recently by a cave-in of overlying material. If inspection indicates that the damage is severe enough to require that this section be abandoned, the replacement sewer then would serve as a portion of the service sewer.
5. A high elevation trunk (LU 9-LU 11) to serve the higher areas of Capitol Hill. Partial separation of the tributary area (local service area LU 12) would be required.
6. A high elevation waterfront interceptor (LU 12-LU 15) to serve the higher areas draining to Lake Union and the Lake Washington Ship Canal.
7. A trunk and interceptor system (LU 16-LU 18) to serve the Ballard area. This system discharges through an inverted siphon (LU 19) to the North Trunk sewer. The existing inverted siphon consists of parallel 36-inch wood stave pipes, which undoubtedly will require replacement at some future date. Estimated costs presented in Table 15-36 allow for such a possibility. Partial separation of the tributary area (local service area LU 16) would be required.
8. A trunk (LU 20) which serves the lower areas draining to Lake Union and discharges to Core Plan B facilities in the Elliott Bay sewerage area. Partial separation of the tributary area (local service area LU 18) would be required.

OUTLINE OF PROPOSED SEWERAGE PROJECTS

It is evident from the foregoing analyses of costs and other controlling factors that collection, treatment and disposal of the sewage of the metropolitan Seattle area can be achieved most effectively by means of the central sewerage project designated herein as Core Plan B. Under this plan (Fig. 15-19), sewage from ten of the twelve individual sewerage areas would be conveyed to two plants for treatment and disposal.

Of the two treatment plants, one would be situated at Black River junction west of Renton and would provide complete treatment for sewage from the North Lake Sammamish, South Lake Sammamish, East Lake Washington, South Lake Washington and Green River sewerage areas. Chlorinated effluent from this plant would be discharged to Duwamish River. The second plant would be situated at West Point at the western extremity of Fort Lawton and would provide primary treatment for sewage from the North Lake Washington, Northwest Lake Washington, Southwest Lake Washington, Elliott Bay and Lake Union sewerage areas. Chlorinated effluent produced at this plant would be discharged to Puget Sound approximately 3,700 feet offshore at a water depth of 150 feet. Feeder and service sewers necessary to convey sewage from the ten sewerage areas to the Core Plan B facilities are described and their estimated costs are given in foregoing sections of this chapter.

In the South Puget Sound sewerage area, participation in the central sewerage project would not be economically feasible. It is proposed instead that sewage generated in this area be conveyed to and treated at five independent plants (Fig. 15-19). Each of these plants would serve a major sewerage subarea and would provide either primary treatment or secondary treatment, depending on receiving water requirements


NORTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
NORTH LAKE WASHINGTON
NORTHWEST LAKE WASHINGTON
NORTH PUGET SOUND
BOTHELL
KIRKLAND
BELLEVUE
CLYDE HILL
MEDINA
HUNTS POINT
KENMORE
SNOHOMISH CO.
KING CO.
LAKE WASHINGTON
PUGET
SHILSHOLE BAY
ELLIOTT BAY
WEST POINT
KITSAP CO.
LAKE BALLINGER PROPOSED FREEWAY
GREEN LAKE
LAKE UNION
SCALE IN MILES
LU 1
LU 2
LU 3
LU 4
LU 5
LU 6
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LU 8
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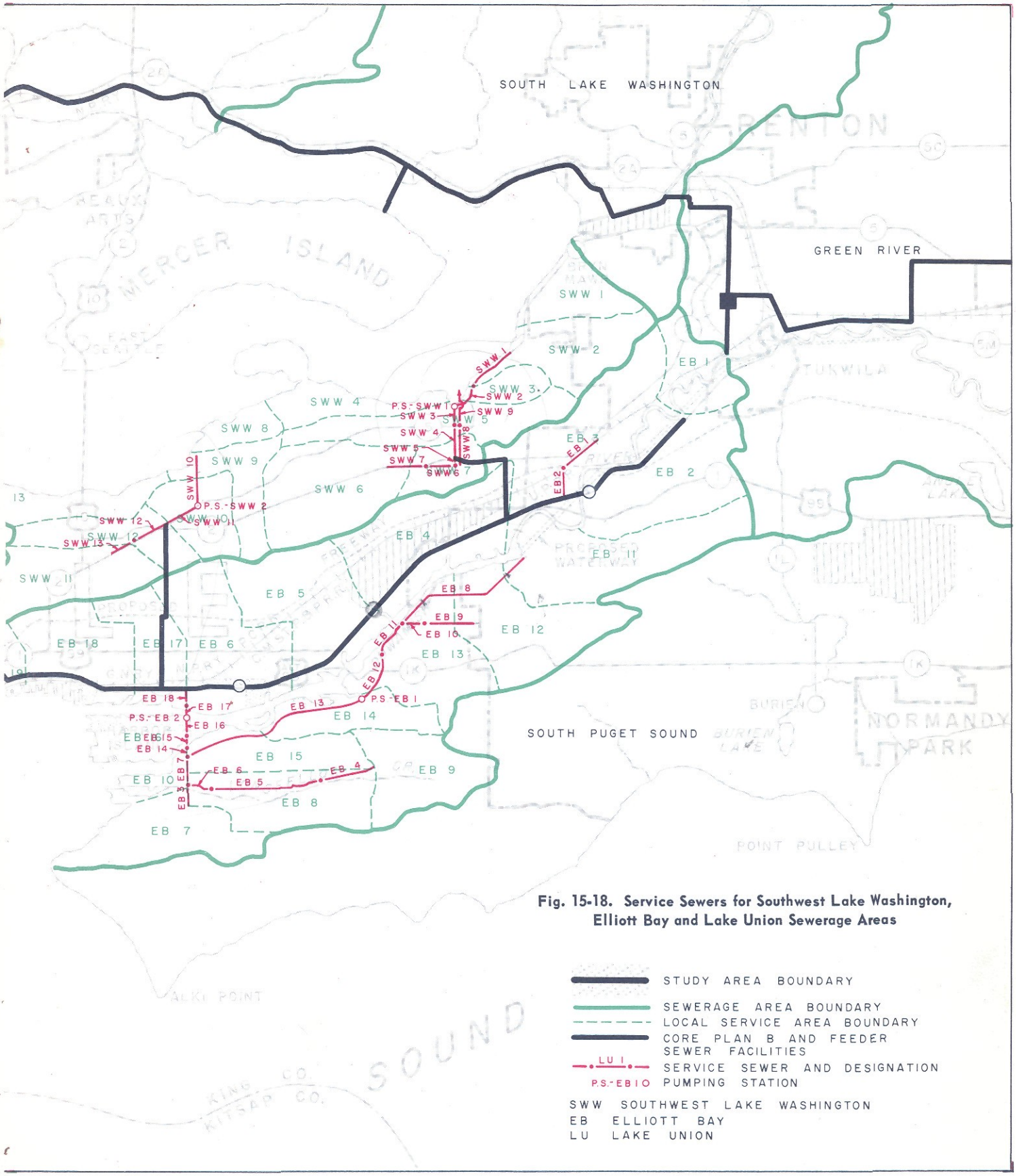

Fig. 15-18. Service Sewers for Southwest Lake Washington, Elliott Bay and Lake Union Sewerage Areas

Table 15-36. Description and Estimated Construction Costs, Service Sewers, Southwest Lake Washington, Elliott Bay and Lake Union Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Southwest Lake Washington Sewerage Area				
SWW-1	1.1	3.6	3,300 ft of 18-in. RC at 0.19%, wet, includes imported backfill and repaving. To parallel existing 18-in. sewer of insufficient capacity for peak wet weather sanitary flow, includes 4,800,000 gal. holding tank and overflow structure with overflow pipe and 1,000,000 gal. holding tank on tributary local sewers	609,000
SWW-2	1.5	4.5	1,100 ft of 18-in. RC at 0.19%, wet, includes imported backfill and repaving. To parallel existing 18-in. sewer of insufficient capacity for peak wet weather sanitary flow, includes 1,300,000 gal. holding tank on overflow from tributary local sewer	146,000
SWW-3	2.6	7.8	1,300 ft of existing twin 15-in. force mains	Existing
SWW-4	4.0	12	2,300 ft of existing 48-in. at 0.12%, 42-in. at 0.20%, and parallel 24-in. at 0.87% and 30-in. at 0.35%. Requires partial separation of a portion of local service area SWW-5	Existing
SWW-5	2.1	5.8, 85[c]	2,000 ft of existing 60-in. at 0.5%. Requires partial separation of local service area SWW-6	Existing
SWW-6	2.5	7.1, 107[c]	2,900 ft of existing 60-in. at 0.9%. Requires partial separation of a portion of local service area SWW-5	Existing
SWW-7	2.5	7.1	500 ft of existing 18-in.	Existing
SWW-8	–	307[d]	2,600 ft of existing 60-in. to 72-in. overflow pipe. Receives storm water overflow from sewer SWW-6	Existing
SWW-9	–	548[d]	1,300 ft of existing 84-in. overflow pipe. In addition to flow from sewer SWW-8, receives storm water overflow from local sewers tributary to sewer SWW-4. Cost is for 4,400,000 gal. holding tank at discharge end	370,000
SWW-10	1.2	3.6	3,500 ft of existing 42-in. at 0.1%. Cost is for 520,000, 350,000, 120,000, 520,000 and 3,400,000 gal. holding tanks on tributary local sewers	494,000
SWW-11	2.1	6.1, 50[c]	2,500 ft of existing 66-in. at 0.3%. Requires partial separation of local service area SWW-10	Existing
SWW-12	1.5	4.2, 173[e]	1,400 ft of existing 75-in. at 0.55%	Existing
SWW-13	2.3	6.5, 256[e]	2,700 ft of existing 102-in. at 0.24 - 0.3%	Existing
SWW-14	1.2	3.4	400 ft of existing 12-in. force main.	Existing
SWW-15	1.2	3.4	2,600 ft of existing 24-in. by 36-in. egg shaped sewer at 2.1% and 24-in. by 48-in. egg shaped sewer at 0.54%	Existing
SWW-16	0.4	1.2	1,000 ft of existing parallel 12-in. and 16-in. force mains	Existing
SWW-17	1.6	4.6	5,400 ft of existing 48-in. at 0.25%	Existing
SWW-18	1.6	4.6	1,900 ft of existing 54-in. at 0.15%	Existing
SWW-19	2.0	5.8, 30[d]	800 ft of existing 54-in. at 0.20%. Requires partial separation of local service area SWW-15	Existing
SWW-20	2.0	5.8, 30[d]	1,800 ft of existing 60-in. at 0.19%	Existing
			Holding tanks on local sewers tributary to PS-SWW-1. Tank capacities: 900,000, 1,400,000, 800,000 and 1,000,000 gals.	321,000
			Holding tanks on local sewers tributary to PS-SWW-3. Tank capacities: 230,000, 640,000, 230,000, 460,000, 520,000, 350,000, 350,000, 460,000, 870,000 and 290,000 gals.	506,000
			Holding tanks on local sewers tributary to PS-SWW-4. Tank capacities: 290,000, 640,000 and 520,000 gals.	178,000
Subtotal, sewers and holding tanks				2,624,000

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Table 15-36. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
PS-SWW-1	2.6	7.8	Pumping station, single stage, motor driven with engine standby, static lift 10 ft, total head at peak flow 25 ft. To replace existing sewage pumping station which has a total installed capacity of 5.2 mgd, includes connections to existing sewers	110,000
PS-SWW-2	1.2	3.6	Existing pumping station having total installed capacity of 10.7 mgd (3 pumps of 1.0, 4.7 and 5.0 mgd capacity)	Existing
PS-SWW-3	1.2	3.4	Pumping station, single stage, motor driven with engine standby, static lift 80 ft, total head at peak flow 90 ft. To replace existing sewage pumping station which has a total installed capacity of 2.65 mgd, includes connection to existing sewers	68,000
PS-SWW-4	0.4	1.2	Existing pumping station having total installed capacity of 3.9 mgd (2 pumps of 1.2 and 2.7 mgd capacity)	Existing
Subtotal, pumping stations				178,000
Total contract cost, Southwest Lake Washington Sewerage Area				2,802,000
Engineering and contingencies, 25 per cent				700,000
Total construction cost, Southwest Lake Washington Sewerage Area				3,502,000
Elliott Bay Sewerage Area				
EB-1	2.1	4.6	1,700 ft of 21-in. RC at 0.21%, average cut 8 ft, difficult wet	35,000
EB-2	4.9	10	2,000 ft of 33-in. RC at 0.09%, average cut 10 ft, difficult wet	60,000
EB-3	1.8	5.1	1,200 ft of 18-in. RC at 1.0%, average cut 15 ft, difficult wet, includes imported backfill, repaving, sheeting and railroad crossing	62,000
EB-4[f]	1.0	3.2	700 ft of 15-in. RC at 1.3 - 8.0%, average cut 9 - 14 ft, and 2,700 ft of 18-in. RC at 1.1 - 1.25%, average cut 9 - 11 ft, wet, includes repaving	60,000
EB-5[f]	1.9	6.3	1,200 ft of 18-in. RC at 3.0 - 4.5%, average cut 12 - 14 ft, 3,600 ft of 21-in. RC at 0.6 - 4.0%, average cut 9-16 ft, 2,100 ft of 24-in. RC at 0.4 - 0.8%, average cut 13 - 17 ft, and 1,300 ft of 30-in. RC at 0.15%, average cut 18 ft, wet to difficult wet, includes repaving and sheeting	234,000
EB-6[f]	2.3	7.4	200 ft of 24-in. RC at 1.0%, average cut 10 ft, and 900 ft of 30-in. RC at 0.54 - 0.85%, average cut 8 - 9 ft, difficult wet	30,000
EB-7	4.1	12	2,100 ft of 30-in. RC at 0.2%, average cut 19 - 20 ft, difficult wet, includes imported backfill, repaving and sheeting	130,000
EB-8	2.0	5.8	8,800 ft of 24-in. RC at 0.17 - 0.20%, average cut 10 - 11 ft, difficult wet, includes imported backfill and repaving	252,000
EB-9	2.3	5.9	3,200 ft of 21-in. RC at 0.5%, average cut 14 ft, wet, includes imported backfill and repaving	93,000
EB-10	3.6	8.4	1,500 ft of existing 42-in. at 0.11%. Requires complete separation of a portion of local service area EB-13	Existing
EB-11	5.5	14	2,700 ft of 39-in. RC at 0.07%, average cut 13 ft, difficult wet, includes imported backfill, repaving and sheeting	157,000
EB-12	6.4-6.7	16-17	3,200 ft of 42-in. RC at 0.07%, average cut 15 - 19 ft, difficult wet, includes imported backfill, repaving, sheeting and slough crossing	202,000
EB-13	6.9-7.4	17-19	12,600 ft of 39-in. RC at 0.13 - 0.24%, average cut 8 - 16 ft, difficult wet, includes imported backfill, repaving, sheeting and railroad crossing	532,000
EB-14	11	25	500 ft of 48-in. RC at 0.075%, average cut 18 ft, difficult wet, includes imported backfill, repaving and sheeting	39,000

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Table 15-36. Continued

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost,[b] dollars
EB-15	11	25	800 ft of parallel 16-in. and 30-in. inverted siphons across West Channel of Duwamish Waterway, includes inlet and outlet structures	54,000
EB-16	12	27	1,500 ft of 48-in. RC at 0.085%, average cut 15 ft, difficult wet, includes imported backfill, repaving and sheeting	103,000
EB-17	12	27	700 ft of parallel 16-in. and 24-in. force mains across East Channel of Duwamish Waterway	42,000
EB-18	12	27	900 ft of 36-in. RC at 0.39%, average cut 10 ft, difficult wet, includes imported backfill, repaving, sheeting and railroad crossing	42,000
EB-19	2.0-2.3	6.1-7.0	5,700 ft of 21-in. RC at 0.38 - 0.5%, average cut 7 - 11 ft, difficult wet, includes imported backfill, repaving, railroad crossing and overflow structures on tributary local sewers	177,000
Subtotal, sewers				2,304,000
PS-EB-1	6.7	17	Pumping station, single stage, motor driven, static lift 39 ft, total head at peak flow 45 ft, structure about 30 ft below ground, difficult wet, includes sheeting and dewatering	183,000
PS-EB-2	12	27	Pumping station, single stage, motor driven, static lift 10 ft, total head at peak flow 22 ft, structure about 20 ft below ground, difficult wet, includes sheeting, dewatering and special foundations	246,000
Subtotal, pumping stations				429,000
Total contract cost, Elliott Bay Sewerage Area				2,733,000
Engineering and contingencies, 25 per cent				683,000
Total construction cost, Elliott Bay Sewerage Area				3,416,000
Lake Union Sewerage Area				
LU-1	2.4	6.1, 86[c]	4,300 ft of existing 60-in. at 0.5%. Requires partial separation of local service area LU-1	Existing
LU-2	2.7	7.0, 86[c]	1,500 ft of existing 72-in. at 0.2%	Existing
LU-3	1.2	3.1	3,200 ft of 18-in. RC at 0.74%, average cut 8 ft, wet, includes imported backfill and repaving	60,000
LU-4	1.3-2.6	3.4-7.2	4,500 ft of existing 24-in. at 0.6 - 3.2% and 5,000 ft of existing 30-in. at 0.6 - 3.5%	Existing
LU-5	3.3	9.2, 103[e]	300 ft of existing 72-in. at 0.11%	Existing
LU-6	3.8	11, 167[e]	4,000 ft of existing 90-in. at 0.11-0.15%	Existing
LU-7	5.7	16, 230[c]	3,400 ft of existing 90-in. at 0.45%. Requires partial separation of local service area LU-6	Existing
LU-8	6.1	17, 230[c]	3,700 ft of existing 72-in. at 2.6%	Existing
LU-9	1.6	4.3, 70[c]	3,900 ft of existing 60-in. at 0.61%. Requires partial separation of local service area LU-12	Existing
LU-10	2.1	5.5, 99[c]	200 ft of existing parallel 24-in. and 66-in. inverted siphons	Existing
LU-11	2.1	5.5, 99[c]	2,600 ft of existing 84-in. at 0.21%	Existing
LU-12	2.4	6.5	2,900 ft of existing 48-in. at 0.2 - 0.26%	Existing
LU-13	2.6	6.9	1,000 ft of existing 54-in. at 0.18%	Existing
LU-14	2.8	7.6	2,800 ft of existing 60-in. at 0.13 - 0.14%	Existing
LU-15	3.1	8.4	2,700 ft of existing 72-in. at 0.19%	Existing
LU-16	2.5	6.4, 84[c]	1,400 ft of existing 69-in. by 116-in. rectangular shape at 0.075 - 0.09 0.09%. Requires partial separation of local service area LU-16	Existing

Continued on next page

Table 15-36. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
LU-17	2.5	6.4	2,100 ft of existing 54-in. at 0.04%	Existing
LU-18	2.5	6.4	1,000 ft of existing 66-in. at 0.035%	Existing
LU-19	4.7	12	1,300 ft of parallel 36-in. inverted siphons to replace existing wood stave inverted siphon, includes inlet and outlet structures	135,000
LU-20	3.0	7.8, 51[c]	5,600 ft of existing 72-in. at 0.08%. Requires partial separation of local service area LU-18	Existing
Total contract cost, Lake Union Sewerage Area				195,000
Engineering and contingencies, 25 per cent				49,000
Total construction cost, Lake Union Sewerage Area				244,000

See Fig. 15-18 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

[c]*Flow for 10-year storm from partially separated tributary area.*

[d]*Storm flow in excess of trunk capacity.*

[e]*Flow for 10-year storm from tributary area.*

[f]*Lengths, sizes and slopes of sewers are based on final design of trunk by the Seattle engineering department.*

at each site. Chlorinated effluent from the five plants would be discharged through submarine outfalls to Puget Sound.

Similarly, it would not be economically feasible for the North Puget Sound to participate in the central sewerage project. It is proposed, therefore, that sewage from this area be conveyed to and treated at two independent plants. These plants would serve the Piper Creek and Boeing Creek subareas and would provide primary treatment plus effluent disinfection. Effluent from each plant would be discharged through submarine outfalls to Puget Sound.

ADMINISTRATION OF PROPOSED SEWERAGE FACILITIES

It seems appropriate at this point to touch briefly on what are believed to be minimum administrative requirements for construction, maintenance and operation of the proposed sewerage facilities. Provision of an adequate administrative organization and enactment of proper administrative regulations will assure orderly and efficient development of the various projects.

Central Control

Sewerage problems of the metropolitan Seattle area are area-wide in scope. As such, they are the responsibility not only of the city of Seattle but of the area as a whole. Corrective measures must be formulated and carried out accordingly. This survey is the first step in that direction and is but one of the many which still remain to be taken.

Because of the physical and economic magnitude of the required sewerage projects, it is highly unlikely that any single existing political body would be in a position to assume responsibility for both their financing and their construction and operation. Further, it is equally unlikely that existing political bodies acting individually would be able to finance and construct the required facilities. Some means will have to be developed, therefore, whereby the necessary burdens can be equitably assumed, necessary funds can be raised, and necessary work can be undertaken on a systematic and truly economic basis.

To achieve the three objectives just noted and to assure area-wide participation in the solution of what is most assuredly an area-wide problem, it appears advisable to undertake the formation of a central agency encompassing the entire metropolitan area. Such an agency would be responsible for all administrative and engineering duties related to the financing, design, construction, maintenance and operation of all sewerage facilities herein recommended. It seems advisable also that this agency should take over and become responsible for the maintenance and operation of all existing sewage treatment plants in the metropolitan area. With full responsibility for major sewerage facilities vested in one agency, effective protection of all waters of the area will be assured.

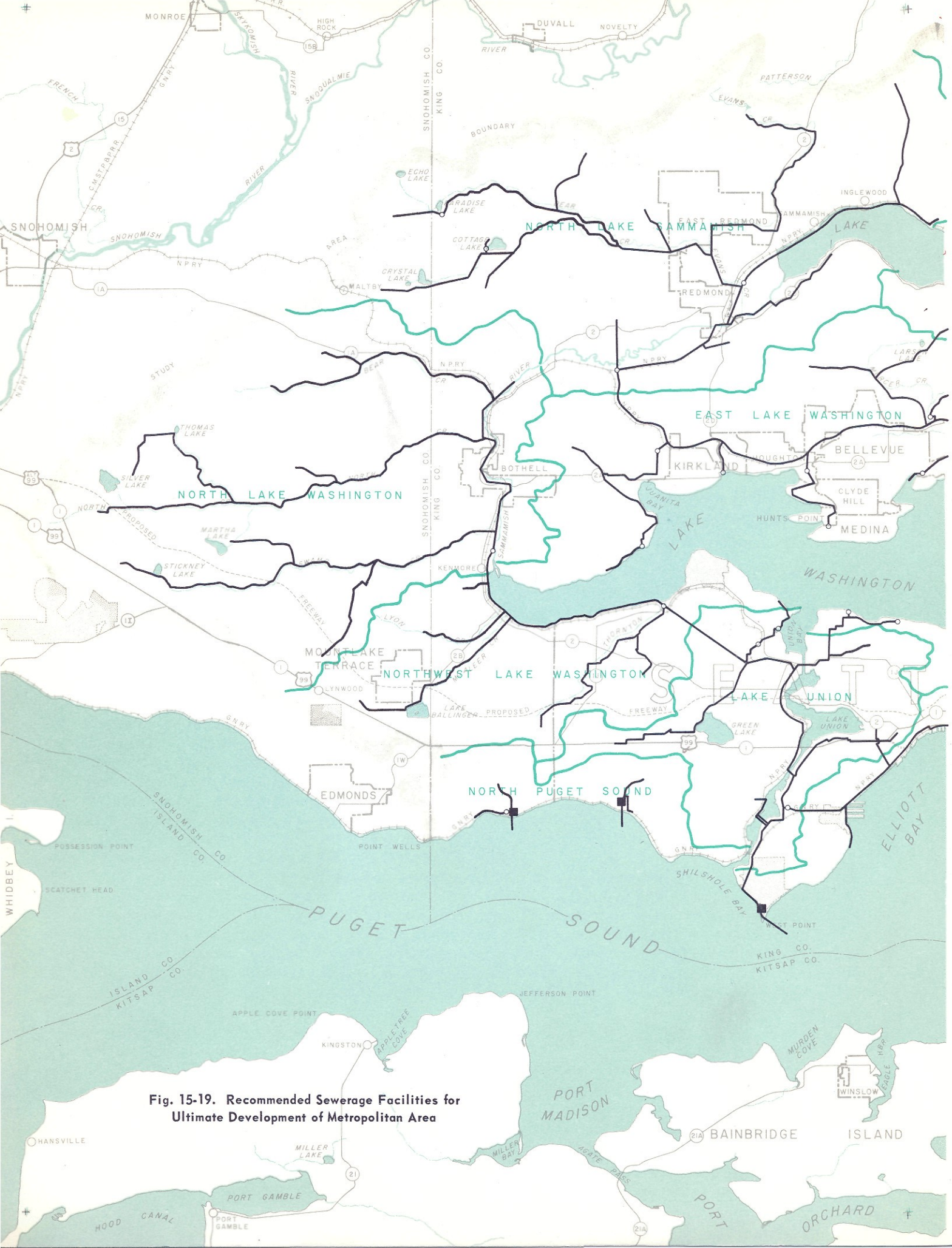

Fig. 15-19. Recommended Sewerage Facilities for Ultimate Development of Metropolitan Area

STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
MERCER ISLAND
VASHON ISLAND
MAURY ISLAND
BLAKE ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
RENTON
KENT
AUBURN
ALGONA
TACOMA
SEATTLE
BURIEN
NORMANDY PARK
DES MOINES
ZENITH
TUKWILA
ISSAQUAH
RAVENSDALE
BLACK DIAMOND
VASHON
DOCKTON
PORTAGE
COWLEY
VASHON HEIGHTS
HARPER
COLBY
MANCHESTER
RUSTON
GIG HARBOR
KI POINT
POINT ROBINSON
POINT PULLEY
RESTORATION POINT
KING CO.
KITSAP CO.
PIERCE CO.
SEWERAGE AREA BOUNDARY
TRUNK SEWERS
SEWAGE TREATMENT PLANTS
PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Predesign Investigations

In undertaking the design and construction of major sewerage facilities, it is necessary at the outset to know what soil conditions and what other conditions, particularly subsurface obstructions, are likely to be encountered during construction. This requirement is particularly applicable to the metropolitan Seattle area where geological formations are variable and major sewers will have to be routed through highly developed districts. One of the functions, therefore, of the central sewerage agency should be that of assembling essential information concerning soil conditions along the routes of all proposed intercepting sewers and tunnels, and at the sites of all treatment plants and pumping stations. Experience indicates that such information is useful not only in developing a sound and economical design but in obtaining favorable bids on construction.

Engineering Design and Control of Construction

In addition to administrative control, the central agency should have an engineering staff capable of performing both design work and inspection during construction. Design work will fall in two general categories, one more or less continuous and the other intermittent or occasional. Continuous work would include design of all trunk sewers and logically could be performed by the engineering staff of the central agency. Intermittent work would include design of the waterfront intercepting sewers, treatment plants, and major pumping stations, all of which require a design staff having specialized experience in this particular field. For that reason, and because the work load would be periodic rather than continuous, the central agency would possibly find it difficult to undertake all of the necessary engineering.

Rigid inspection will be required during construction of the proposed facilities. This will assure effective compliance with all specifications relating to the work and will guarantee construction in full accordance with the design requirements. Inspection generally should be performed by the engineering staff of the central agency. In the case, however, of facilities designed under contract by consulting engineering firms, it may be advisable to have such firms assume responsibility for resident engineering and inspection during construction.

Enforcement of Design Criteria

Successful operation of the facilities designed and constructed under the authority and control of the central agency will depend upon the cooperation of the many smaller agencies which are actually responsible for the local collection systems. Cooperation will be required in particular in two phases. Of these, the first is concerned with infiltration and storm inflow, and the second with industrial wastes.

Infiltration and Storm Inflow. As stated earlier in Chapter 13, present day construction materials and methods, coupled with adequate inspection, are such that infiltration and storm inflow quantities can be kept well within the allowances provided herein. Some method will have to be set up whereby the central agency can be assured that these allowances will not be exceeded in local systems where construction and operation are not within its direct jurisdiction.

A question may arise as to the necessity or economy of minimizing inflow from these sources. Benefits thus achieved may be illustrated by the following examples. Two areas, each of 1,000 acres, are to be served by separate sanitary sewerage systems. Design criteria for the first area include an allowance for infiltration and storm inflow of 1,000 gpad. For the second area, the allowance is four times that amount, or 4,000 gpad. Assuming a peak sanitary sewage flow from each area of 1.0 mgd, the total design flow for which sewer capacity must be provided is 2.0 mgd in the first case and 5.0 mgd in the second. Assuming also that the slope of the trunk sewers serving the areas is 0.7 per cent in both cases, the sewer required to serve the first area would be 12 inches in diameter, while that for the second area would be 18 inches in diameter. The difference in cost between these relatively small diameter sewers amounts to approximately three dollars per foot of sewer length. With larger tributary areas and similar infiltration ratios, the cost difference per foot of trunk sewer would be correspondingly greater.

As a further example, if roof leaders and foundation drains were permitted to be connected, maximum flows would be two to fifteen times those which would otherwise occur. In such an event, either sufficient sewer capacity would have to be provided or more frequent overflows would have to be allowed.

Obviously, it would not be feasible economically in a central sewerage project to provide collection, treatment and disposal facilities of a capacity sufficient to accommodate the excess flows illustrated by the two examples. Although the central agency cannot directly control construction of local sewers, it can exercise control by indirect means. First, it can require the adoption of standard specifications regarding the construction of local sewers, including house connections. These specifications should stipulate the class of construction required to maintain infiltration within the limits called for in this report. Second, the central agency should require

complete infiltration tests prior to the connection of any new local sewerage facilities. Should it be found that the infiltration limits are exceeded, a connection to the central system should be refused until the local system has been brought up to standard. And third, the central agency should maintain constant flow measurements to determine whether inputs from the local systems are within allowable limits. In cases where excess flows are consistently recorded, the local agency should be subject to an additional assessment and the charge thus imposed should be based on the capitalized cost of providing for the measured excess.

Industrial Wastes. A regulatory ordinance defining the characteristics of wastes acceptable to the sewerage systems should be adopted by the central agency. This should set forth allowable physical and chemical limits of all wastes which would produce undue loadings or would lead to deleterious effects either on collection and treatment facilities or on treatment and disposal functions.

Chapter 16
STAGE CONSTRUCTION OF SEWERAGE FACILITIES

As a final step in the development of a long-range program of sewerage improvements, it is necessary to establish a logical and orderly schedule for their construction on a stage or incremental basis. This in turn requires a determination of the most urgent immediate needs, as well as a determination of the times in the future when the recommended additions are likely to be required.

As stated earlier, the most urgent immediate needs include the removal of sewage and sewage effluents from the Lake Washington watershed, the interception of raw sewage and industrial waste outfalls discharging to Duwamish River, Elliott Bay and Puget Sound, and the provision of service to highly developed areas presently without public sewerage facilities. Scheduling of the many urgent projects must recognize the fact that the volume of work which can be undertaken in any one year is necessarily limited by the engineering and construction force which can be obtained and effectively utilized. Scheduling must recognize also the problems of financing and the limitations imposed by the time involved in obtaining necessary funds.

BASIS OF CONSTRUCTION PROGRAM

Forecasts of population and of average sewage flows for the various sewerage areas (Tables 14-1 and 15-1), coupled with estimated population distributions (Fig. 5-9), were used both to determine construction timing and to estimate treatment plant loadings. It should be realized, however, that urban development in specific areas may take place either sooner or later than here anticipated, and that metropolitan growth as a whole may occur at a slower rate. For that reason, later stages of the proposed construction program will be subject to adjustment.

Construction of the proposed improvements is to be undertaken in three stages. Stage I, includes facilities required to alleviate the most serious problems and assumes initial construction in 1960, with completion by 1970. This scheduling allows time for organization of a central sewerage agency to administer financing, design, construction, and operation of the recommended facilities. Stage II includes extension of facilities to serve additional areas and assumes construction during the period 1970 to 1980. Stage III includes all remaining facilities, the construction of which would be undertaken as required some time after 1980.

Because of limited development in adjoining areas and also because of financial considerations, construction in some areas presently in need of sewerage service has been deferred to Stages II or III. Such deferments require that local sewers and, in most cases, temporary sewage treatment plants be constructed and utilized until such a time as a connection to the central sewerage system becomes financially possible. Likewise, some of the existing treatment plants which serve small, individual areas will have to be retained until central sewerage can be made available.

STAGE I CONSTRUCTION, 1960-1970

Relief of the most urgent sewerage needs of the metropolitan area will be achieved upon completion of the Stage I construction program. Scheduled to begin by 1960 and to end by 1970, this program calls for construction of facilities (Fig. 16-1) estimated to cost a total of $83,215,000 (Table 16-1). Three basic systems are to be constructed, the first designated as the Renton system, the second as the West Point system, and the third as independent systems. Essential features of each are described in the following presentation.

Renton System Sewers

Renton system sewers to be constructed under Stage I include (1) the Core Plan B interceptor extending east from the proposed sewage treatment plant, (2) the north branch feeder sewer along the Northern Pacific railroad right-of-way to the point at which sewage presently discharged to the city of Kirkland treatment plant will be intercepted, and (3) service sewers required to intercept major sewage discharges to Lake Washington. Preliminary plans and profiles of all Core Plan B sewers and feeder sewers to be constructed under Stage I are presented in Appendix E.

Although initial flows in the north branch feeder sewer will be low as related to its design capacity, provision for ultimate flow requirements is necessary because of the difficult construction conditions along much of its route. These conditions are such that it would not be economically feasible to install a parallel line at some future date. Moreover, a large part of the cost of laying a sewer along this route is for preliminary items, such as the provision of adequate access and working space, and for contingencies as-

Table 16-1. Stage I Construction, Recommended Sewerage Facilities

Facility	Construction cost,[a] dollars
Core Plan B[b]	
Sewers	
B-2	339,000
B-4 – B-22[c]	13,911,000
Pumping stations	
PS-B-1 – PS-B-4	1,713,000
Treatment plants	
STP-B-1[d]	11,848,000
STP-B-2	11,524,000
Outfalls	
B-3[e]	318,000
B-23 – B-24	3,846,000
Total, Core Plan B	43,499,000
Core Plan B feeder sewers[f]	
S-4 – S-5	2,615,000
S-11 – S-15	5,762,000
S-17 – S-22	7,066,000
N-7 – N-9	6,209,000
N-10 – N-28[c]	358,000
PS-N-2[g]	738,000
PS-N-3 – PS-N-4[h]	–
Total, feeder sewers	22,748,000
Service sewers	
East Lake Washington[i]	
ELW-10	45,000
ELW-11 – ELW-14	294,000
ELW-21 – ELW-24	301,000
ELW-27	39,000
ELW-28 – ELW-31[j]	–
ELW-32	25,000
ELW-33 – ELW-34[j]	–
ELW-35 – ELW-37	351,000
PS-ELW-3 – PS-ELW-5	304,000
PS-ELW-7	139,000
PS-ELW-8 – PS-ELW-10[j]	–
PS-ELW-11 – PS-ELW-12	325,000
	1,823,000
Northwest Lake Washington[k]	
NWW-6 – NWW-7	90,000
NWW-8 – NWW-10[c]	–
NWW-11	60,000
NWW-12 – NWW-14[c]	686,000
	836,000
South Lake Washington[l]	
SLW-18 – SLW-20	620,000
	620,000
Green River[l]	
GR-20 – GR-21[c]	–
Southwest Lake Washington[m]	
SWW-1 – SWW-2[h, n]	770,000
SWW-3 – SWW-9[c, n]	462,000
SWW-10 – SWW-13[c, n]	618,000
SWW-14 – SWW-20[c]	–
PS-SWW-1[h,n]	401,000
PS-SWW-2[c]	–
PS-SWW-3[h, n]	632,000
PS-SWW-4[c, n]	222,000
	3,105,000
Elliott Bay[m]	
EB-1 – EB-19	2,880,000
PS-EB-1 – PS-EB-2	536,000
	3,416,000
Lake Union[m]	
LU-1 – LU-18[c]	75,000
LU-19[j]	–
LU-20[c]	–
	75,000
Total, service sewers	9,875,000
Separate systems	
South Puget Sound[o]	
Des Moines subarea	
SPS-15 – SPS-16	186,000
STP-SPS-2[p]	1,241,000
SPS-17	191,000
	1,618,000
Miller Creek subarea	
SPS-22	52,000
STP-SPS-3[q]	855,000
SPS-23	404,000
	1,311,000
Southwest Suburban subarea	
SPS-24 – SPS-27[c]	92,000
STP-SPS-4[c]	160,000
SPS-28[r]	–
	252,000
West Seattle subarea	
SPS-29 – SPS-37[c]	–
PS-SPS-2 – PS-SPS-5[c]	–
STP-SPS-5[c]	80,000
SPS-38[r]	–
	80,000
North Puget Sound[s]	
Piper Creek subarea	
NPS-1 – NPS-4[c]	44,000
STP-NPS-1	720,000
NPS-5 – NPS-6[c]	235,000
	999,000

(Continued on Page 452.)

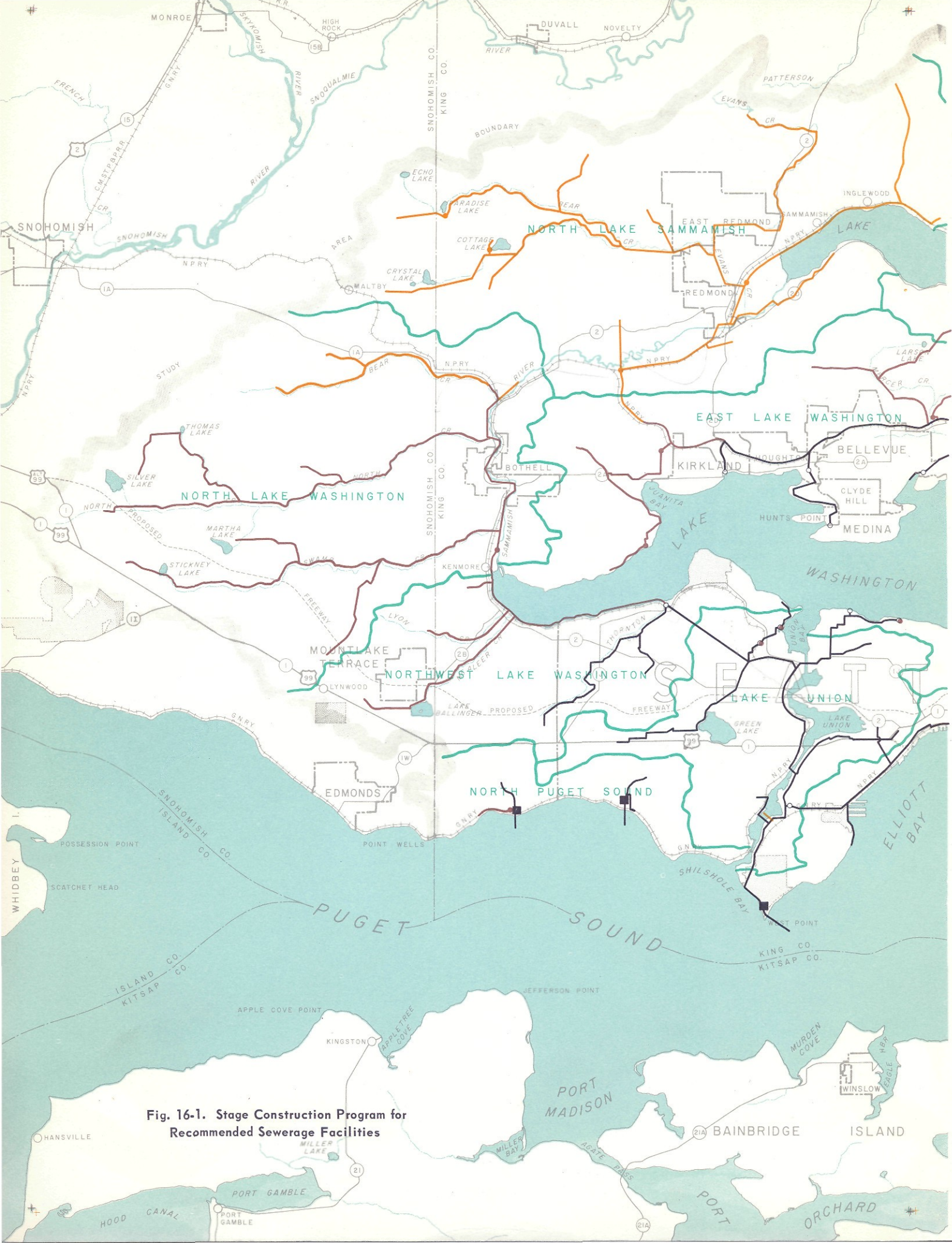

Fig. 16-1. Stage Construction Program for Recommended Sewerage Facilities

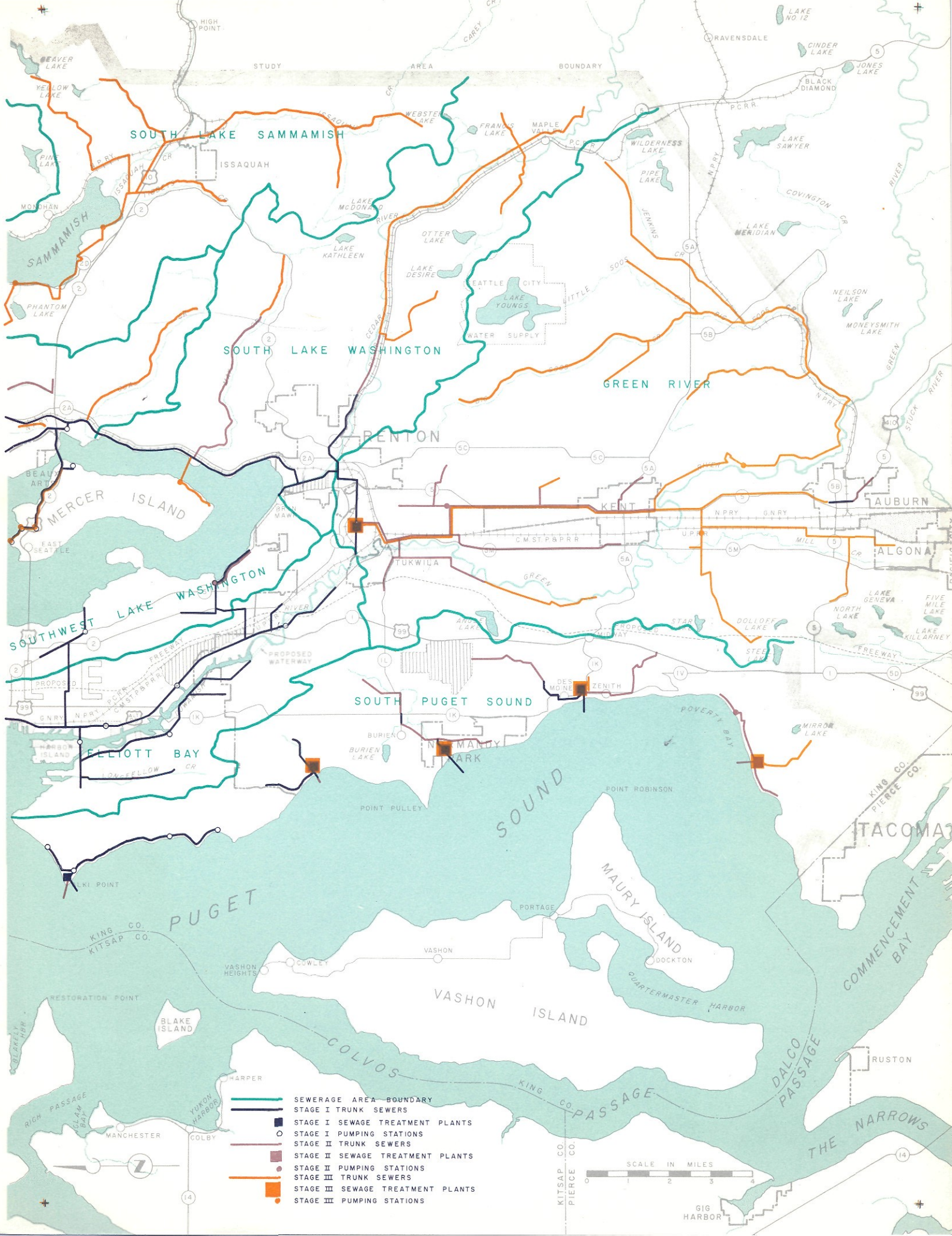
STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
PUGET SOUND
MERCER ISLAND
VASHON ISLAND
MAURY ISLAND
BLAKE ISLAND
RENTON
KENT
AUBURN
ALGONA
TACOMA
TUKWILA
NORMANDY PARK
ZENITH
ISSAQUAH
SAMMAMISH
KI POINT
POINT PULLEY
POINT ROBINSON
PORTAGE
DOCKTON
VASHON
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
QUARTERMASTER HARBOR
POVERTY BAY
KING CO.
KITSAP CO.
PIERCE CO.
SEWERAGE AREA BOUNDARY
STAGE I TRUNK SEWERS
STAGE I SEWAGE TREATMENT PLANTS
STAGE I PUMPING STATIONS
STAGE II TRUNK SEWERS
STAGE II SEWAGE TREATMENT PLANTS
STAGE II PUMPING STATIONS
STAGE III TRUNK SEWERS
STAGE III SEWAGE TREATMENT PLANTS
STAGE III PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 16-1. Continued

Facility	Construction cost[a] dollars	Facility	Construction cost[a] dollars
Boeing Creek subarea		Total, separate systems	5,093,000
NPS-7	82,000		
STP-NPS-2	565,000	Temporary sewage treatment plants	2,000,000
NPS-9	186,000		
	833,000	Total, Stage I construction	83,215,000

See Fig. 16-1 for location of facilities to be constructed under Stage I.

[a]*Includes engineering and contingencies.*

[b]*See Fig. 15-4 for location and Table 15-4 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[c]*Includes existing facilities.*

[d]*Initial construction: preaeration and primary sedimentation tanks – 24 mgd capacity (16.7% of ultimate); aeration tanks – 24 mgd capacity (16.7% of ultimate); secondary sedimentation tanks – 24 mgd capacity (16.7% of ultimate); and sludge digesters – 36 mgd capacity (25% of ultimate); includes cost of 100 acres of land.*

[e]*Initial construction, single 78-inch outfall.*

[f]*See Fig. 15-7 for location and Table 15-11 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[g]*Includes structure for ultimate requirements and equipment for 60 per cent of ultimate.*

[h]*Existing sewers and pumping stations; paralleling or replacing of inadequate sewers or pumping stations under Stage II construction.*

[i]*See Fig. 15-15 for location and Table 15-33 for description of facilities.*

[j]*Existing sewers and pumping stations; paralleling or replacing of inadequate sewers or pumping stations, Stage III construction.*

[k]*See Fig. 15-16 for location and Table 15-34 for description of facilities.*

[l]*See Fig. 15-17 for location and Table 15-35 for description of facilities.*

[m]*See Fig. 15-18 for location and Table 15-36 for description of facilities.*

[n]*Includes holding tanks on tributary local sewers.*

[o]*See Fig. 15-12 for location and Table 15-27 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[p]*Initial construction, primary plant for 3.25 mgd capacity (50 per cent of ultimate).*

[q]*Initial construction for 3.75 mgd capacity (50 per cent of ultimate.*

[r]*Existing outfall; extension Stage II construction.*

[s]*See Fig. 15-13 for location and Table 15-30 for description of facilities.*

sociated with difficult working conditions. Since these costs would remain more or less constant regardless of the size of line to be installed, about the only saving that would accrue in laying a smaller sewer initially would be the relatively small difference in pipe costs. It would thus be false economy to start with a sewer smaller than that required for ultimate development of the tributary area.

Intercepting sewers to serve the Green River sewerage area south of the proposed treatment plant are not included in the Stage I construction program. This is because developed areas to the south, namely, Kent and Auburn, are presently served by independent treatment and disposal works which should remain adequate or can feasibly be enlarged during the period covered by this program. Sewers necessary to connect these areas to the central system are provided for under subsequent stages and would be constructed and extended to keep pace with industrial and residential development of Green River valley and adjoining areas. Similarly, feeder sewers to serve the North Lake Sammamish and South Lake Sammamish sewerage areas are not included in Stage I because those areas are not yet developed to an extent sufficient to justify such a project.

Renton System Treatment Plant

Secondary treatment by the activated sludge process will be provided at the Renton plant. Initially, this plant will serve the Southwest Lake Washington, East Lake Washington and South Lake Washington sewerage areas. Ultimately, it will receive sewage also from the North Lake Sammamish, South Lake Sammamish and Green River sewerage areas.

Stage I construction of treatment plant facilities calls for provision of a capacity sufficient to last for approximately 10 to 15 years. Following that period, the capacity will be increased by adding parallel units.

Based on an analysis of predicted average flows to the Renton plant (Table 16-2), it appears feasible to plan construction in three increments, with each increment providing for a capacity of 48 mgd, or one-third of the ultimate requirement. Since the first increment would provide sufficient capacity until 1985-1990, a further division of construction is required in order to keep the initial outlay to the lowest practi-

Table 16-2. Estimated Average Sewage Flows, Renton Sewage Treatment Plant

Year	Average flow, mgd
1965	7
1970	13
1975	19
1980	30
1985	43
1990	60
1995	74
2000	87

cable level. Initial construction, therefore, will consist of providing sedimentation and aeration capacity for 24 mgd and sludge digestion capacity for a plant flow of 36 mgd. These facilities will be adequate until 1975-1980, at which time additions will be made to increase the total capacity of the plant to 48 mgd.

Flow Diagram and Design Data. A diagrammatic arrangement of the necessary treatment facilities is shown in Fig. 16-2. Basic data relating to the various structures and items of equipment are presented in Table 16-3. A detailed layout cannot be made at this time because orientation and final arrangement of the various units will depend on the physical characteristics of the plant site. Nevertheless, a tentative layout is shown in Fig. 16-3 and assumes construction on a 50-acre site about 1,000 feet in width by 2,200 feet in length. A perspective view of the Renton plant based on this layout is shown in Fig. 16-4.

Required structures at the plant will include sedimentation tanks, aeration tanks, sludge digestion tanks, power building, administration building, chlorination building, sludge control building, and miscellaneous items such as passageways, pipe chases, and meter boxes. The power building will contain mechanical bar screens, influent pumping equipment, aeration blowers, power generation equipment, lavatory and locker room, day room, store rooms, workshops and garage, offices, and miscellaneous mechanical equipment. The administration building will contain offices, laboratories and other necessary facilities. It is possible that the administration building may be eliminated, or its construction delayed to some later stage, by combining it initially with the power building. For preliminary estimating purposes, however, the administration building is included under Stage I construction. The sludge control building will contain all sludge heating and sludge handling equipment. The chlorination building will contain all chlorination equipment and a bulk chlorine storage tank.

The estimated initial cost of the treatment plant provides for the purchase of 100 acres of land, or sufficient for ultimate development. Of this total, approximately 50 acres will be used for plant structures and facilities, while the remaining 50 will be used for sludge lagoons. While the latter need not necessarily be contiguous to the plant, they should at least be within an economical pumping distance. Only a portion of the total land will be used under initial construction. Nevertheless, the entire 100 acres should be purchased immediately to allow for economic and orderly expansion of plant facilities as the need arises.

Pretreatment Facilities. Prior to pumping and treatment, raw sewage will be screened through bar racks to remove large objects and rags which otherwise would damage or clog pumps and interfere with operation.

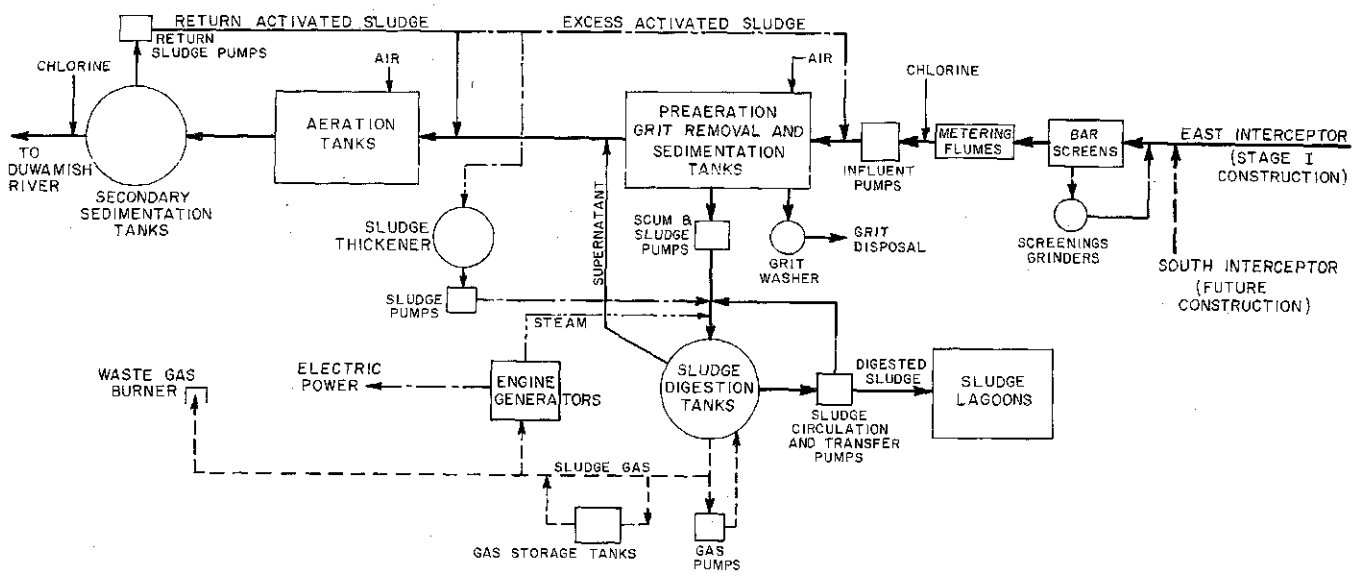

Fig. 16-2. Flow Diagram, Renton Sewage Treatment Plant

Table 16-3. Design Factors, Renton Sewage Treatment Plant

Population in thousands	
Initial design	170
Ultimate	995
Industrial, population equivalent in thousands	
Initial design	80
Ultimate	665
Total design population in thousands	
Initial design	250
Ultimate	1,660
Loading	
Average dry weather flow, mgd	
Initial design	24
Ultimate	143
Peak wet weather flow, mgd	
Initial design	100
Ultimate	360
Present minimum flow, mgd	3.5
BOD, 1,000 pounds per day	
Initial design	50
Ultimate	332
Suspended solids, 1,000 pounds per day	
Initial design	60
Ultimate	415
Bar screens	
Number	
Initial design	2
Ultimate	4
Clear spacing, inches	0.75
Influent pumps	
Number	
Initial design	3
Ultimate	5
Installed capacity, mgd	
Initial design	150
Ultimate	450
Aeration blowers	
Number	
Initial design	2
Ultimate	5
Installed capacity, 1,000 cfm	
Initial design	50
Ultimate	125
Preaeration and grit removal tanks	
Number	
Initial design	3
Ultimate	18
Length, feet	26
Width, feet	38
Average water depth, feet	15
Detention time at average DWF, minutes	20
Hydraulic capacity per tank, mgd	35
Primary sedimentation tanks	
Number	
Initial design	3
Ultimate	18
Length, feet	130
Width, feet	38
Average water depth, feet	9
Detention time at average DWF, minutes	60
Overflow rate at average DWF, gal. per sq ft per day	1,600
Hydraulic capacity per tank, mgd	35
Primary treatment efficiency	
Assumed BOD reduction, per cent	25
Assumed suspended solids reduction, per cent	45
Aeration tanks	
Number	
Initial design	3
Ultimate	18
Length, feet	700
Width, feet	38
Average water depth, feet	15
Detention time at average DWF, hours	8.9
Return activated sludge, per cent of average DWF	25
Air supplied, cu ft per gal. at average DWF	1.0
Hydraulic capacity per tank, mgd	35
Secondary sedimentation tanks	
Number	
Initial design	8
Ultimate	48
Diameter, feet	70
Average effective water depth, feet	8.5
Detention time at average DWF, hours	2.0
Overflow rate at average DWF, gal. per sq ft per day	800
Hydraulic capacity per tank, mgd	12.5
Secondary treatment efficiency	
Assumed BOD reduction, per cent	93
Assumed suspended solids reduction, per cent	82
Total treatment efficiency	
Assumed BOD reduction, per cent	95
Assumed suspended solids reduction, per cent	90
Sludge digestion tanks	
Number	
Initial design	2
Ultimate	8
Diameter, feet	100
Side water depth, feet	30
Volume per tank, 1,000 cubic feet	233
Loading, pounds per cubic foot per day	0.20
Sludge lagoons	
Volume, acre-feet	
Initial design	100
Ultimate	600
Storage, years (ultimate construction at design loading, 1960 – 2030, 80 per cent moisture)	10

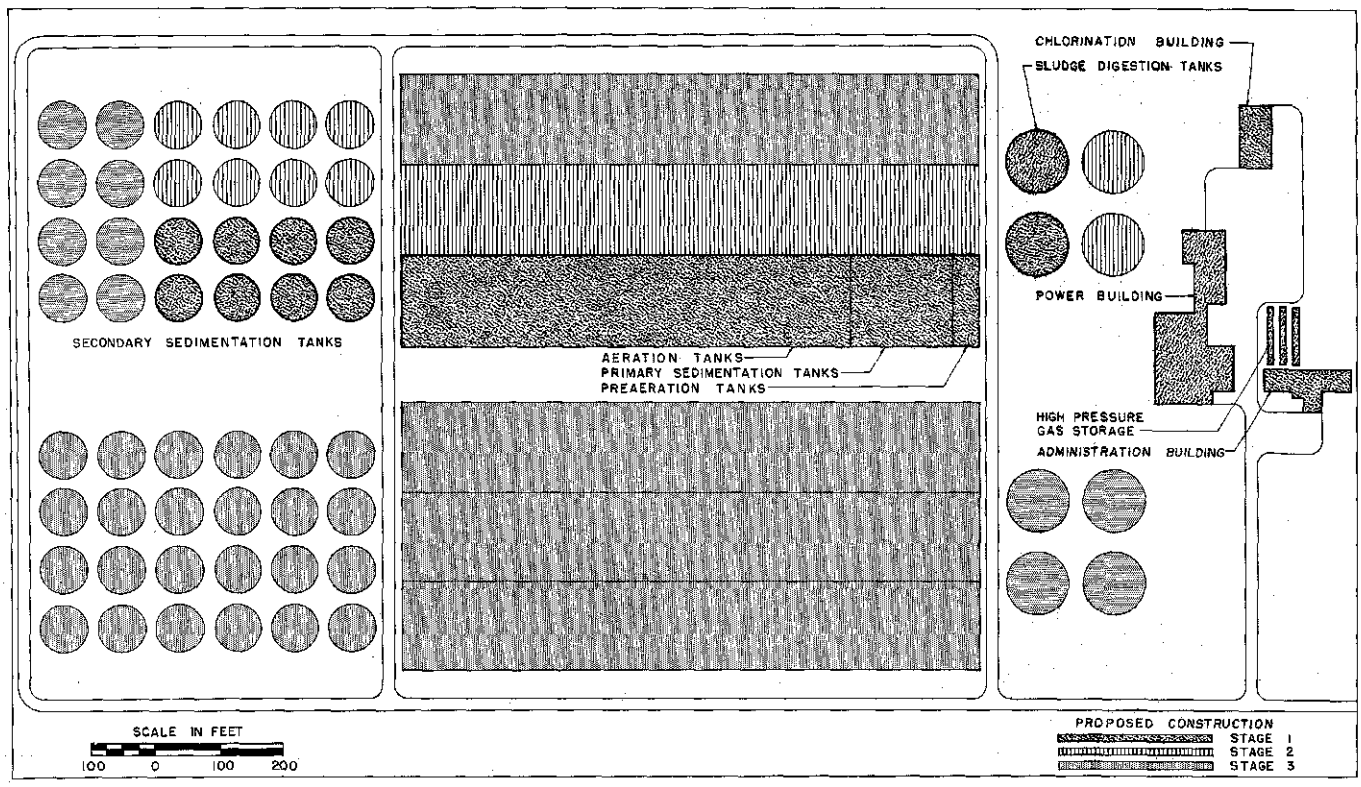

Fig. 16-3. Possible Layout of Sewage Treatment Facilities, Renton Plant

of the sedimentation tanks. Equipment for this purpose will consist of four mechanically cleaned bar screens, two of which will be installed initially. Material removed by the screens will be ground by one of two grinders and returned to the sewage flow upstream from the screens. Each screen will be installed in a suitable influent channel. Water surface elevations downstream from the bar screens will be controlled by parshall flumes, which will act also as primary metering elements for flow measurements.

Influent Pumping. Screened raw sewage will be lifted about 30 feet by engine driven pumps. Pump speed and discharge rate will be controlled by flow-responsive pneumatic equipment. This arrangement permits elimination of the usually costly raw sewage sump and results in high sedimentation efficiency through prevention of pumping surges in the sedimentation tanks.

Stage I construction calls for the installation of three pumps, two of which will be capable of accommodating the estimated design peak flow, with the third unit reserved as a standby. Subsequent stages of construction call for replacement of the three initial units with pumps of larger capacity, as well as the installation of two additional units.

Power Generation. Engines for driving the raw sewage pumps and aeration blowers, and for supplying other major power demands, will be either dual-fuel or spark ignition type, using as a source of fuel sludge gas produced in the sludge digestion process. Waste heat rejected through the cooling system and in the exhaust gases will be used to heat the digester and plant buildings.

Preaeration and Grit Removal. Combination preaeration and grit removal will be obtained in reinforced concrete tanks, each with a detention period of 20 minutes at the design flow of 8 mgd, and a maximum hydraulic capacity of 35 mgd. Three tanks will be provided initially, with 18 required for ultimate capacity. Under initial design, all tanks will have to be in operation during peak flows. At ultimate development, however, it will be possible to take one or more tanks out of service at any time. Grit will be collected in hoppers within the tanks, pumped to a grit washer by water ejectors, and finally disposed of by filling low areas around the plant.

Primary Sedimentation. Primary sedimentation will be obtained in reinforced concrete tanks, which structurally, will be a continuation of the preaeration tanks. As in the preaeration system, three tanks will be constructed initially and 18 ultimately. Rectangular in plan, each tank will provide a detention period of 60 minutes at design flow and a maximum hydraulic capacity of 35 mgd. All of the tanks will be equipped

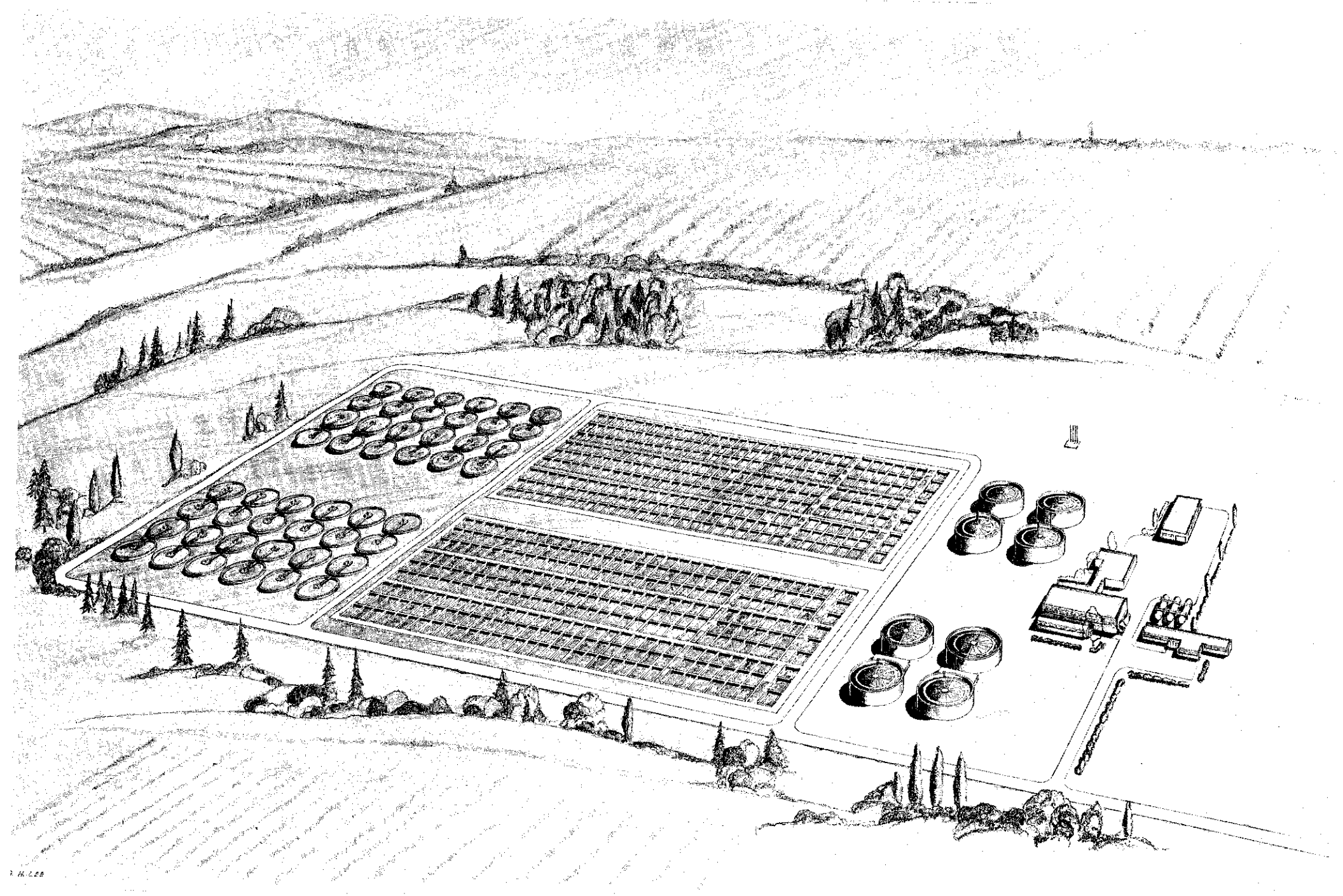

Fig. 16-4. Perspective View of Renton Sewage Treatment Plant

with mechanical sludge collecting and scum removing mechanisms.

Aeration. Aeration of the mixture consisting of primary settled sewage and activated sludge will be obtained in reinforced concrete tanks which will provide a detention period of 8.9 hours at design flow and a maximum hydraulic capacity of 35 mgd. Three of these tanks will be constructed initially and 18 ultimately.

Aeration capacity will be provided at a rate of 1.0 cubic foot of air per gallon of sewage, average dry weather flow. Five engine driven blowers will be installed, two of them under Stage I construction. One blower will be available as a standby unit, both initially and in the future.

Return activated sludge will be introduced at the inlet to each aeration tank. Input rates will be regulated automatically and the flow will be distributed to each tank, using pneumatic controllers. Excess activated sludge either will be returned to the plant influent upstream from the raw sewage metering flumes, or will be discharged directly to the digesters after passing through a sludge thickener.

Secondary Sedimentation. Secondary sedimentation will be obtained in circular reinforced concrete tanks having a detention time of two hours at design flow and a maximum hydraulic capacity of 12.5 mgd. These tanks will be constructed in batteries of four, of which two will be installed under Stage I and twelve ultimately. A distribution structure will be provided at each battery to distribute the flow equally to each tank. The rate of withdrawal of sludge from each tank will be regulated by pneumatic controllers.

Effluent Chlorination. Effluent will be chlorinated to an extent sufficient to obtain adequate disinfection. Chlorine doses will be regulated automatically, using flow-responsive pneumatic control equipment. Provision will be made for bulk chlorine storage and a railroad siding will be constructed to enable purchase of chlorine in tank car lots. Since adequate detention time will be available in the outfall, chlorine contact tanks will not be required during initial operation. Contact tanks to provide an additional detention time of 10 minutes will be required in the future.

Raw Sludge Pumping. Combined raw sludge and scum removed from the primary sedimentation tanks will be conveyed to an external sump, from which it will be pumped directly to the digesters by means of heavy duty pumps. Operation of these pumps will be automatically regulated by means of pneumatic control equipment and will depend on the level of sludge in the sump.

Sludge Digestion and Disposal. Sludge digestion will be obtained in circular reinforced concrete, brick veneered tanks. Eight tanks will be required for ultimate capacity, with two being constructed under the Stage I program. Each tank will be equipped with a floating steel cover and will be designed for utmost flexibility of operation. No pipes or equipment of any kind will be suspended in the tanks. Sludge inlets and outlets, as well as the gas take-offs, will be carried outside the tanks either in the ground or over the tops in suitable flexible conduits. All outdoor lines will be adequately protected against freezing. Although the two initial tanks will be identical, sludge piping will be so arranged that either one may be used for primary digestion and the other for secondary.

Temperature of the digesting sludge will be maintained at 95° F, using an automatic control system and circulation of the sludge through an external heat exchanger. In addition to sludge circulation, a separate gas circulating system will be provided to break up scum formations.

Supernatant liquor removed from the digestion tanks will be returned to the inlet of the aeration tanks. Digested sludge will be removed from the digesters periodically and discharged to the lagoons. A lagoon capacity of 100 acre-feet is expected to be sufficient for approximately 10 years under initial loading.

Effluent Outfall. Effluent from the treatment plant will be discharged to Duwamish River through a 78-inch reinforced concrete outfall. Ultimately, a parallel 78-inch line will be required.

West Point System Sewers

Sewers to be constructed in the West Point system under the Stage I program include all those of Core Plan B, plus feeder sewers to the point at which flow to the Lake City treatment plant can be intercepted. Stage I construction provides also for service sewers which are required in the Elliott Bay, Northwest Lake Washington and Lake Union sewerage areas both to intercept existing raw sewage and industrial waste outfalls, and to provide service to

Table 16-4. Estimated Average Sewage Flows, West Point Sewage Treatment Plant

Year	Average flow, mgd
1965	57
1970	67
1975	78
1980	83
1985	89
1990	95
1995	100
2000	104

presently developed areas. Preliminary plans and profiles of Core Plan B sewers and feeder sewers to be constructed under Stage I are presented in Appendix E.

West Point System Treatment Plant

Primary treatment will be provided in the West Point plant. Initially, this plant will serve the Southwest Lake Washington, Elliott Bay, Lake Union and

Table 16-5. Design Factors, West Point Sewage Treatment Plant

Population in thousands	958
Industrial, population equivalent in thousands	532
Total design population in thousands	1,490
Loading	
Average dry weather flow, mgd	~~118~~
Peak wet weather flow, mgd	~~302~~ 323
Present minimum flow, mgd	28
BOD, 1,000 pounds per day	298
Suspended solids, 1,000 pounds per day	372
Bar screens	
Number	4
Clear spacing, inches	0.75
Influent pumps	
Number	4
Installed capacity, mgd	400
Preaeration blowers	
Number	3
Installed capacity, 1,000 cfm	12
Preaeration and grit removal tanks	
Number	12
Length, feet	48
Width, feet	38
Average water depth, feet	15
Detention time at average DWF, minutes	30
Hydraulic capacity per tank, mgd	30
Sedimentation tanks	
Number	12
Length, feet	240
Width, feet	38
Average water depth, feet	9
Detention time at average DWF, hours	1.5
Overflow rate at average DWF, gal. per sq ft per day	1,100
Hydraulic capacity per tank, mgd	30
Treatment efficiency	
Assumed BOD reduction, per cent	30
Assumed suspended solids reduction, per cent	60
Sludge digestion tanks	
Number	4
Diameter, feet	100
Side water depth, feet	36
Volume per tank, 1,000 cubic feet	280
Loading, pounds per cubic foot per day	0.2

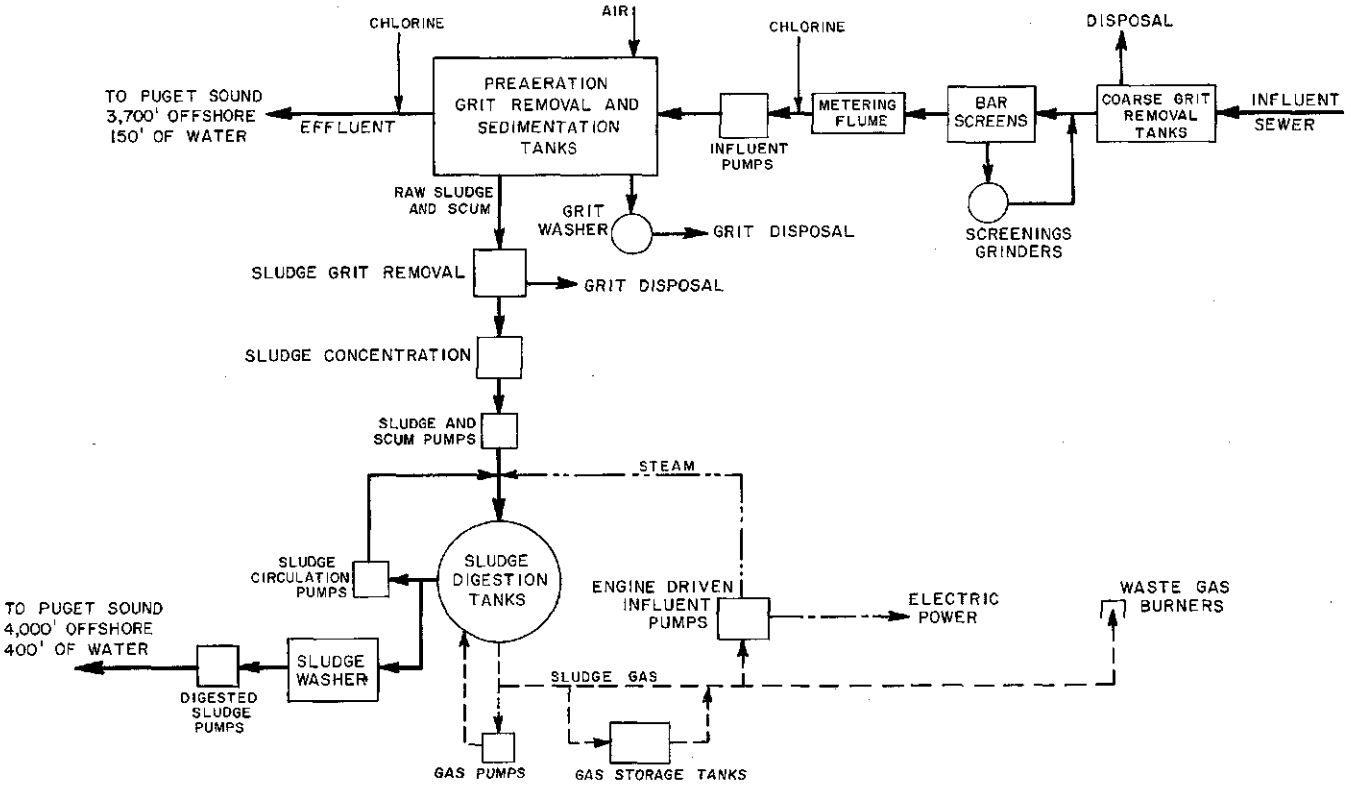

Fig. 16-5. Flow Diagram, West Point Sewage Treatment Plant

Northwest Lake Washington sewerage areas. At some future date, probably in about 10 to 15 years, it will receive sewage also from the North Lake Washington area.

An analysis of predicted average flows to the West Point plant (Table 16-4) indicates that the initial flow will be 57 mgd, or about 50 per cent of the ultimate dry weather capacity of 118 mgd. By 1975, the flow will increase to 78 mgd, or about 70 per cent of the ultimate. It is obvious, therefore, that any program designed for stage or incremental construction would require enlargement of the plant within a relatively short period. On that basis, the plant should be constructed initially to meet ultimate needs.

Flow Diagram and Design Data. Basic design data relating to the various structures and items of equipment to be incorporated in the West Point treatment plant are presented in Table 16-5. A flow diagram is shown in Fig. 16-5, a tentative layout is shown in Fig. 16-6, and a sketch of the treatment works as it might appear when completed is shown in Fig. 16-7.

Required structures will include sedimentation tanks, sludge digestion tanks, plant control building, chlorination building, sludge control building, and miscellaneous units such as passageways, pipe chases, and necessary in the future, planning should be such that a 50-acre site could be developed.
power generation equipment, lavatory and locker room, day room, store rooms, workshops and garage, offices, laboratories, and miscellaneous mechanical equipment. The sludge control building will contain all sludge heating and handling equipment. The chlorination building will contain all chlorination equipment and chlorine storage facilities.

An area of 20 acres will be required for the units to

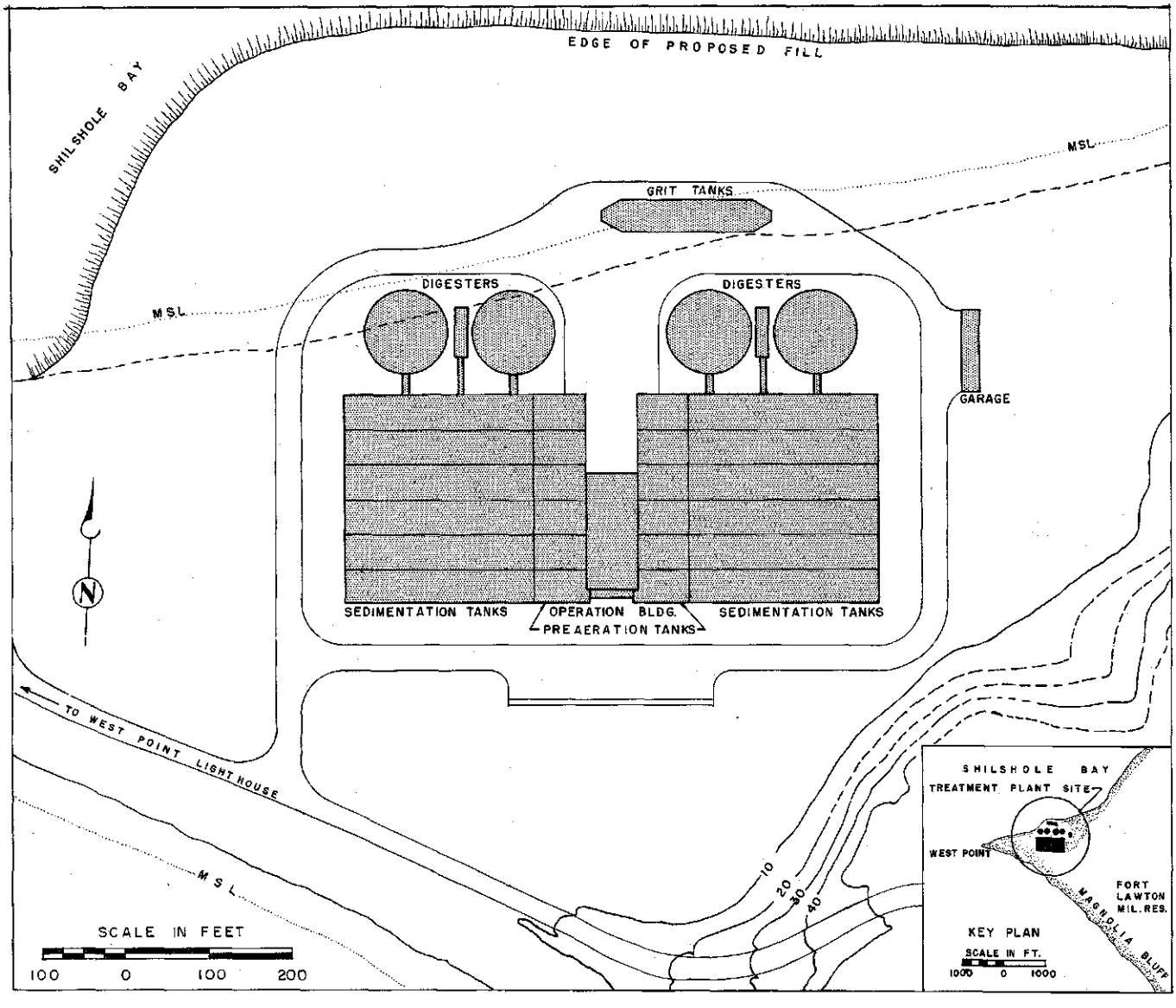

Fig. 16-6. Suggested Layout of Sewage Treatment Facilities, West Point Plant

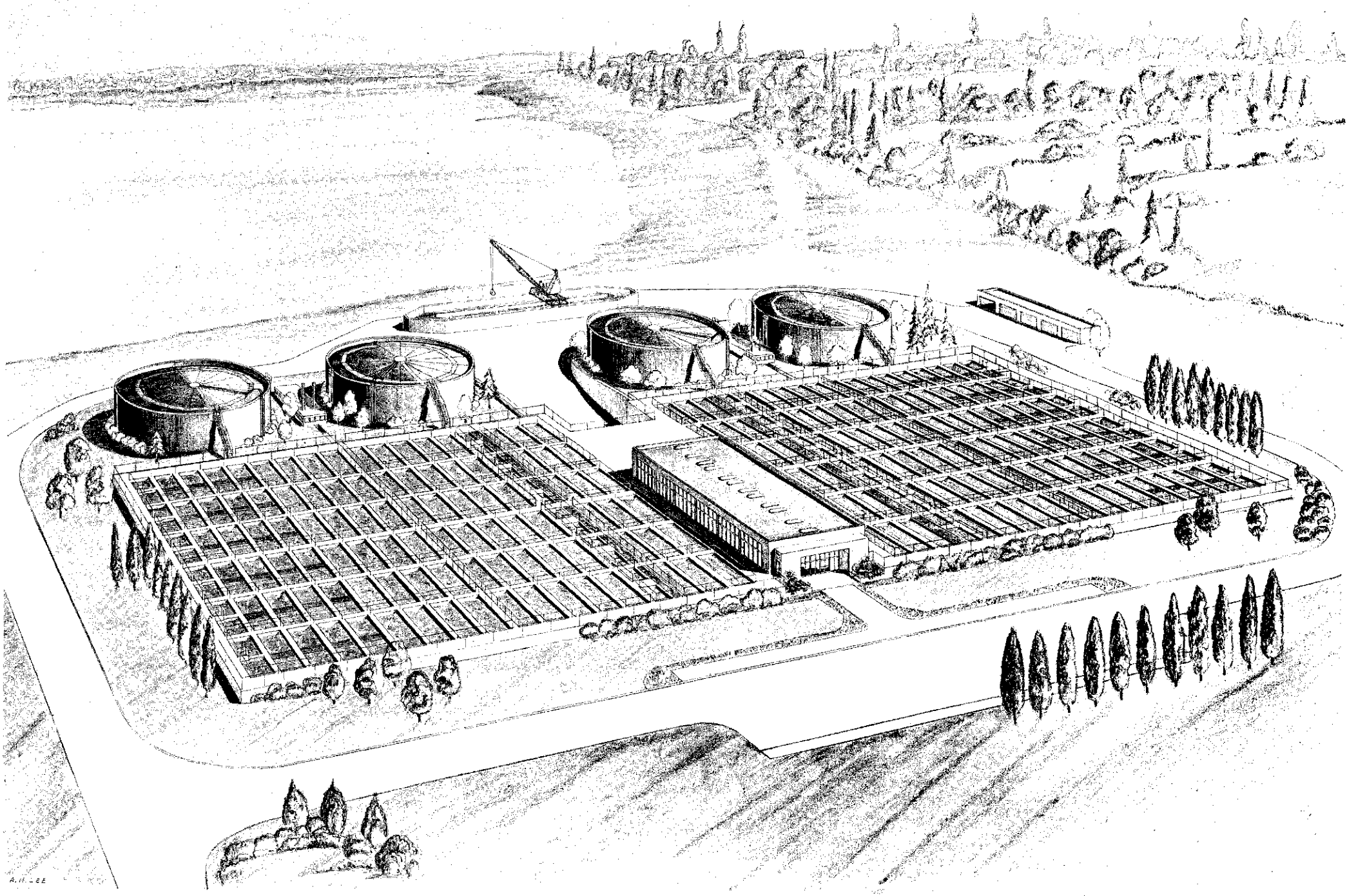

Fig. 16-7. Perspective View of West Point Sewage Treatment Plant

be constructed initially. In view, however, of the possibility that secondary treatment could become necessary in the future planning should be such that a 50-acre site could be developed.

Coarse Grit Removal. Because much of the area tributary to the plant is served by combined sewers, the incoming sewage will contain large particles of grit. This material is capable of damaging mechanical equipment and must therefore be removed. For that purpose, two rectangular channels will be provided in which gravel and other objects heavier than water and larger than 1/2 inch in diameter will be removed. A clam shell bucket, operating from a top-riding, electric crane, will dredge accumulated coarse grit from the channels for hauling away by dump truck.

Pretreatment Facilities. Prior to pumping and treatment, raw sewage will be screened through bar racks to removal large objects and rags which otherwise would damage or clog pumps and interfere with operation of the sedimentation tanks. Equipment for this purpose will consist of four mechanically cleaned bar screens, each installed in a suitable influent channel. Material removed by the screens will be ground by one of two grinders and returned to the sewage flow upstream from the screens. Water surface elevations downstream from the screens will be controlled by parshall flumes, which will act also as the primary metering elements for flow measurement.

Influent Pumping. After passing through the metering flumes, the incoming sewage will be lifted to the sedimentation tanks by four identical engine-driven pumps. Each pump will be designed to discharge a maximum flow of 100 mgd against a total dynamic head of approximately 30 feet. Thus, at the peak capacity of 302 mgd, one pump will be available as a standby unit. Pump speeds and discharge rates will be controlled by pneumatic equipment to equal exactly the rate of raw sewage input.

Power Generation. Four gas-burning diesel or spark ignition engines will be provided to drive the raw sewage pumps and preaeration blowers and to supply other major power demands. Gas produced in the sludge digestion process will be used as a source of fuel for the engines, and waste heat rejected through the cooling system and in the exhaust gases will be used to heat the digesters and plant buildings.

Preaeration and Grit Removal. Combination preaeration and grit removal will be obtained in 12 reinforced concrete tanks, which will provide a detention period of 30 minutes at design flow and a maximum hydraulic capacity of 30 mgd. It will be possible for at least one tank to be out of service at all times. Grit will be collected in hoppers within the tanks, pumped to a grit washer by water ejectors, and finally disposed of by filling low areas around the plant.

Sedimentation. Sedimentation will be provided in 12 reinforced concrete tanks which, structurally, will be a continuation of the preaeration tanks. Rectangular in plan, each tank will provide a detention period of 90 minutes at design flow and a maximum hydraulic capacity of 30 mgd. All of the tanks will be equipped with mechanical sludge collection and scum removing mechanisms.

Effluent Chlorination. Effluent will be chlorinated during the recreational season, May to September, to insure adequate protection of beaches. Chlorine doses will be regulated automatically, using flow-responsive pneumatically controlled equipment. Provision will be made for storage and handling of chlorine in ton containers. Chlorine contact tanks will not be required since adequate detention times will be available in the outfall.

Raw Sludge Handling. It is expected that a fairly large amount of fine sand not readily removable by conventional grit removal methods will be carried to the plant and will settle with organic sludge in the sedimentation tanks. To prevent deposition of this material in the digestion tanks, provision will be made for its separation from the sludge before the sludge enters the tanks. For that purpose, sludge will be pumped continuously from the sedimentation tanks by means of air lift units and will pass through a constant velocity channel of such design that organic solids will be kept in suspension and grit will separate and deposit. Material thus deposited will be removed from the channel and disposed of by filling low areas around the plant site. Thin sludge will flow to a sump where it will be concentrated by a thickening operation to 5 or 6 per cent solids. From the sump, the thickened material will be delivered to the digesters by means of heavy duty pumps.

Sludge Digestion and Disposal. Sludge digestion will be obtained in four circular reinforced concrete, brick veneered tanks, each equipped with a floating cover. Although all tanks will be identical, sludge piping will be so designed that any of them may be used for either primary or secondary stage digestion. Each tank will be designed for utmost flexibility and no pipes or equipment of any kind will be suspended inside. Sludge inlets and outlets, as well as gas take-offs, will be carried outside the tanks either in the ground or over the tops in suitable flexible conduits. All outdoor lines will be adequately protected against freezing.

Table 16-6. Estimated Average Sewage Flows, Sewage Treatment Plants, South Puget Sound Sewerage Area

Year	Average flow, mgd				
	Redondo Beach	Des Moines	Miller Creek	Southwest Suburban	West Seattle
1965	–	0.8	0.6	2.5	6.7
1970	–	1.6	1.0	3.0	6.8
1975	0.6	2.7	3.1	2.2	6.9
1980	1.0	3.4	3.8	2.5	6.9
1985	1.4	4.0	4.6	2.7	7.0
1990	1.9	4.5	5.2	3.0	7.0
1995	2.4	4.9	5.7	3.2	7.0
2000	3.0	5.2	6.1	3.4	7.1

Temperature of the digesting sludge will be maintained at 95° F, using an automatic control system and circulating of the sludge through an external heat exchanger. In addition to sludge circulation, a separate gas circulating system will be provided to break up scum formations.

Digested sludge will be removed from the digesters periodically and, after passing through a washer, will be pumped to Puget Sound through a sludge outfall line extending 4,000 feet offshore to a water depth of 400 feet. The sludge washer will be of the counterflow type and will be designed to remove any floating material remaining in the digested material. Wash water will be returned to the plant influent. Supernatant liquor removed from the digestion tanks will be disposed of to Puget Sound through the sludge outfall line.

Effluent Outfall. Effluent from the treatment plant will be discharged through a gravity outfall sewer terminating 3,700 feet from shore at a depth of 150 feet. This line will consist of 1,900 feet of 120-inch reinforced concrete pipe land section and 3,700 feet of twin 78-inch reinforced concrete pipe submarine section. Each of the latter will have a diffuser section with a total of twelve 18-inch diameter outlets.

Independent Systems

Independent systems to be constructed under the Stage I program include facilities required both to in-intercept raw sewage discharges to Puget Sound and to serve areas in which sewerage service is presently needed. Units to be constructed include trunk sewers, pumping stations, treatment works and outfalls.

Des Moines Subarea, South Puget Sound Sewerage Area. Construction of a sewage treatment plant, submarine outfall and trunk sewers to serve the rapidly developing area in the immediate vicinity of the community of Des Moines is scheduled under Stage I. Because of financial considerations, construction of the sewer connecting the Seattle-Tacoma International Airport to the plant is deferred to Stage II. This presupposes continued operation of the small treatment plant at the airport. Should it prove desirable to abandon the plant at an earlier date, the construction date of the connecting sewer would have to be advanced.

A primary type treatment plant would be constructed initially. Since it will not be possible under ultimate flow conditions to maintain adequate protection of the shoreline with a primary effluent, the plant will be so designed that secondary units can be added in the future.

To determine the required initial capacity as well as construction dates of subsequent enlargements, an analysis was made of predicted flows to the treat-

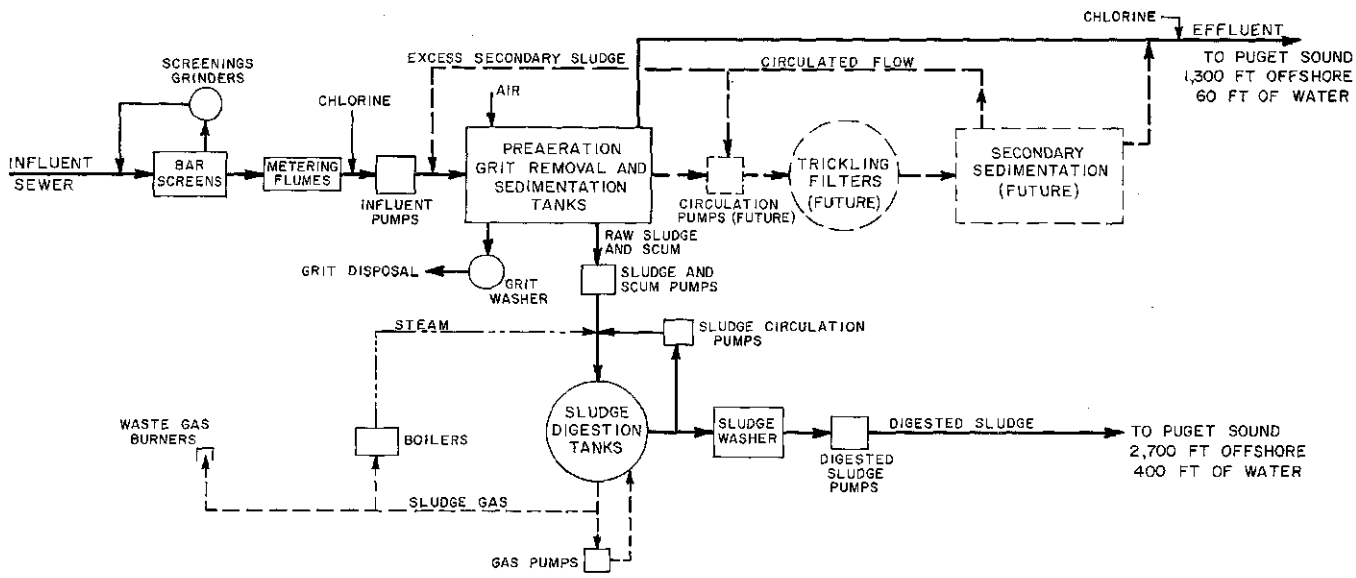

Fig. 16-8. Flow Diagram, Des Moines Sewage Treatment Plant

ment plant (Table 16-6). From this it appears that construction of a primary treatment plant with an initial capacity of 3.25 mgd will be satisfactory. Expansion to an ultimate capacity of 6.5 mgd will be required between 1975 and 1980. Construction of secondary units will also be required at that time.

As shown schematically in Fig. 16-8, the plant to be constructed initially includes facilities for pretreatment, influent pumping, preaeration and grit removal, primary sedimentation, separate sludge digestion, sludge disposal and effluent chlorination. Ultimately, facilities for trickling filtration and secondary sedimentation will have to be provided. To facilitate orderly future development, approximately 10 acres of land should be obtained for the plant site. Design factors are presented in Table 16-7.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 1,300 feet from shore at a depth of 60 feet. This sewer will consist of 1,200 feet of 30-inch reinforced concrete pipe land section and 1,300 feet of 30-inch reinforced concrete pipe submarine section.

Miller Creek Subarea, South Puget Sound Sewerage Area. Construction of a sewage treatment plant, submarine outfall and trunk sewers to serve the area in the immediate vicinity of the city of Normandy Park is scheduled under Stage I. Because of financial considerations, construction of the sewer connecting the Burien Lake area to the plant is deferred to Stage II. This presupposes continued operation of the sewerage system in that area, with discharge of the sewage therefrom to the existing sewage treatment plant of the Southwest Suburban Sewer District. Should it prove advisable to modify the existing system at an earlier date, the construction date of the connecting sewer would have to be advanced.

Treatment works to serve the Miller Creek subarea will be of the primary type and will be designed with an initial capacity of 3.75 mgd, or 50 per cent of the ultimate requirement. As determined from Table 16-6, expansion to its ultimate capacity of 7.5 mgd will be required between 1975 and 1980.

Facilities will be provided for pretreatment, preaeration and grit removal, sedimentation, separate sludge digestion, sludge disposal, and effluent chlor-

Table 16-7. Design Factors, Des Moines Sewage Treatment Plant

Population in thousands	
Initial design	28
Ultimate	60
Industrial, population equivalent in thousands	
Initial design	6
Ultimate	9
Total design population in thousands	
Initial design	34
Ultimate	69
Loading	
Average dry weather flow, mgd	
Initial design	3.25
Ultimate	6.5
Peak wet weather flow, mgd	
Initial design	8.0
Ultimate	16
Present minimum flow, mgd	0.4
BOD, 1,000 pounds per day	
Initial design	7
Ultimate	14
Suspended solids, 1,000 pounds per day	
Initial design	8
Ultimate	17
Bar screens	
Number	
Initial design	1
Ultimate	2
Clear spacing, inches	0.75
Influent pumps	
Number	3
Installed capacity, mgd	24
Preaeration and grit removal tanks	
Number	
Initial design	2
Ultimate	4
Detention, minutes	30
Hydraulic capacity per tank, mgd	6
Primary sedimentation tanks	
Number	
Initial design	2
Ultimate	4
Detention, minutes	90
Overflow rate, gallons per square foot per day	1,080
Hydraulic capacity per tank, mgd	6
Trickling filters (future)	
Number	2
Rock volume per filter, cubic yard	1,550
Circulation ratio	1.5:1
Volumetric loading per filter, mgd	8.0
BOD loading, pounds per cubic yard	2.5
Secondary sedimentation tanks (future)	
Number	4
Detention, minutes	120
Overflow rate, gallons per square foot per day	800
Hydraulic capacity per tank, mgd	6
Sludge digestion tanks	
Number	
Initial design	2
Ultimate	3
Volume per tank, 1,000 cubic feet	25
Loading, pounds per cubic foot per day	0.2

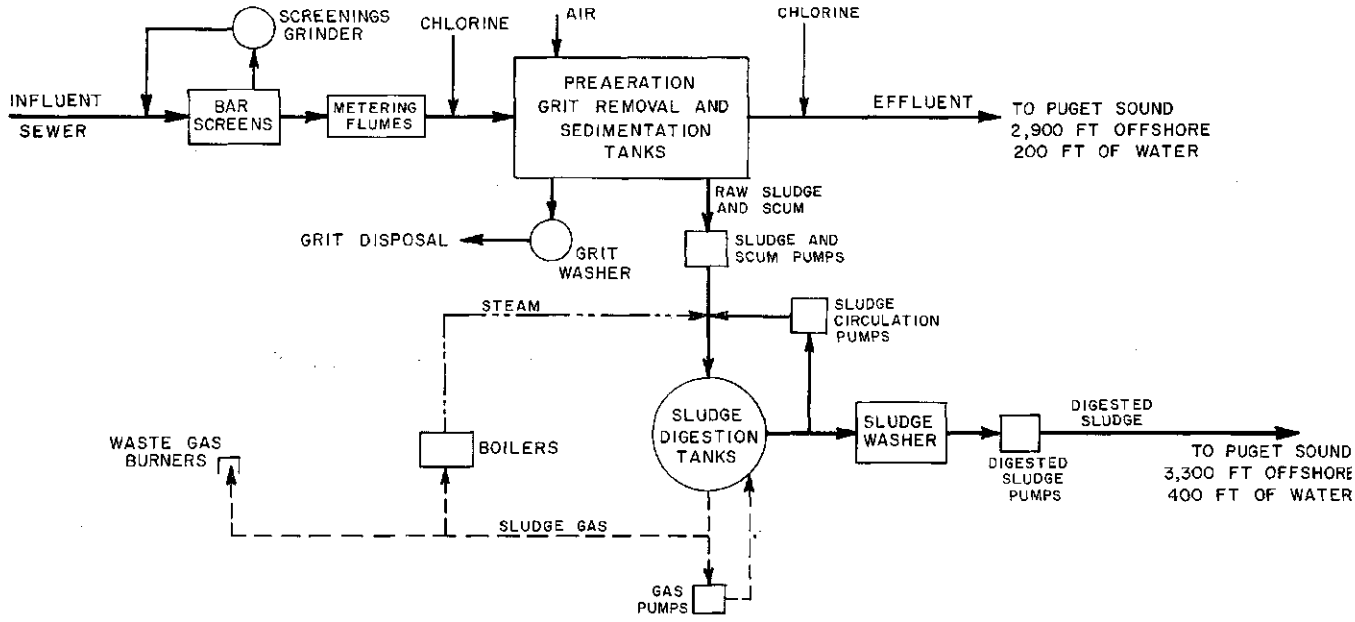

Fig. 16-9. Flow Diagram, Miller Creek Sewage Treatment Plant

ination. Approximately five acres of land will be required for ultimate development. A schematic flow diagram of the recommended plant is shown in Fig. 16-9, while design factors are given in Table 16-8.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 2,900 feet from shore at a depth of 200 feet. This line will consist of a land section comprising 1,200 feet of 27-inch reinforced concrete pipe and a submarine section comprising 2,900 feet of 27-inch reinforced concrete pipe.

Southwest Suburban Subarea, South Puget Sound Sewerage Area. The Stage I program calls for acquisition by the central agency of existing sewerage facilities of the Southwest Suburban Sewer District. Also called for is construction of a trunk sewer to serve the Roxbury Heights area.

Since the Stage I schedule for the Miller Creek subarea does not provide for construction of the sewer from the Miller Creek treatment plant to the Burien Lake area, the existing system serving the latter will

Table 16-8. Design Factors, Miller Creek Sewage Treatment Plant

Population in thousands	
Initial design	42
Ultimate	85
Loading	
Average dry weather flow, mgd	
Initial design	3.75
Ultimate	7.5
Peak wet weather flow, mgd	
Initial design	9.0
Ultimate	18
Present minimum flow, mgd	0.3
BOD, 1,000 pounds per day	
Initial design	8
Ultimate	17
Suspended solids, 1,000 pounds per day	
Initial design	10
Ultimate	21
Bar screens	
Number	
Initial design	1
Ultimate	2
Clear spacing, inches	0.75
Preaeration and grit removal tanks	
Number	
Initial design	2
Ultimate	4
Detention, minutes	30
Hydraulic capacity per tank, mgd	6
Sedimentation tanks	
Number	
Initial design	2
Ultimate	4
Detention, minutes	90
Overflow rate, gal. per sq ft per day	1,150
Hydraulic capacity per tank, mgd	6
Sludge digestion tanks	
Number	2
Volume per tank, 1,000 cubic feet	18
Loading, pounds per cubic foot per day	0.2

continue to function as presently laid out. It is expected that this sewer will be constructed under the Stage II program, at which time the entire Lake Burien area will become tributary to the Miller Creek plant.

As reported earlier, the existing treatment plant of the Southwest Suburban Sewer District is of the primary type and has ample capacity for the predicted ultimate flow from the tributary area. On the other hand, receiving water conditions are such that a disinfected primary effluent cannot be safely discharged offshore from the plant. It will be necessary, therefore, to add facilities for secondary treatment. This improvement, though presently scheduled for Stage II construction, may be required at an earlier date if conditions in the receiving waters are found to be unsatisfactory.

For the treatment plant, the only work scheduled under Stage I is that of revamping sludge handling and heating facilities to enable ocean disposal of sludge and use of digester gas for heating purposes. Boilers utilizing digester gas will be installed, and sludge heating will be accomplished by external heat exchangers, using hot water produced in the boilers. Sludge will be circulated through the heat exchangers, using nonclog centrifugal pumps, and hot water will be circulated by means of a centrifugal pump. Heating rates will be controlled by regulating the temperature of the hot water in the heat exchanger. With these alterations, the existing heat pump presently used for sludge heating purposes will be abandoned, but all useable parts will be salvaged. Digested sludge, after passing through a washer, will be pumped to Puget Sound through a sludge outfall line extending 3,300 feet offshore to a water depth of 400 feet. The sludge washer will be of the counterflow type and will be designed to remove any floating material remaining in the digested sludge. Wash water will be returned to the plant influent. A schematic flow diagram of the existing plant is presented in Fig. 6-11.

Effluent is presently discharged through a 36-inch submarine outfall sewer terminating 600 feet offshore at a depth of 60 feet. As shown in Chapter 11, disposal this near shore and at the relatively shallow depth will not provide the required degree of shoreline protection, even with secondary treatment. It is proposed, therefore, to extend the outfall an additional 200 feet to a water depth of 85 feet, which would be sufficient to assure safe disposal of secondary effluent. Although extension of the outfall is not scheduled until Stage II, earlier construction may be required if bacterial contamination of the shoreline occurs in the meanwhile.

West Seattle Subarea, South Puget Sound Sewerage Area. The Stage I program calls for acquisition by the central agency of existing sewerage facilities of the city of Seattle, including intercepting sewers, pumping stations and treatment and disposal works.

Facilities now under construction will have ample capacity for the predicted ultimate flows from the tributary area. Under the Stage I program, the only work to be undertaken will be the construction of an outfall sludge line to a distance of 3,300 feet offshore and a water depth of 400 feet. Addition of the sludge line, together with a sludge washer, will enable disposal of digested sludge in Puget Sound. A schematic flow diagram of the existing plant is presented in Fig. 6-37.

In the project now under construction, effluent from the treatment plant will be discharged through a 42-inch submarine outfall sewer terminating 1,400 feet offshore at a depth of 85 feet. As shown in Chapter 11, it probably will not be possible with such an outfall to maintain adequate protection of the nearby beaches.

Since sufficient head is not available in the plant as constructed, pumping would be required in order to extend the existing outfall beyond its present depth. It is proposed, therefore, to construct a second outfall which will carry the peak sanitary flow and to use the existing line as a storm water overflow. The new line will terminate directly west of Alki Point and will discharge approximately 1,100 feet offshore at a depth of 210 feet. Although construction of the new outfall is not scheduled until Stage II, earlier construction may be required if bacterial contamination of the beaches occurs in the meanwhile.

Piper Creek Subarea, North Puget Sound Sewerage Area. Construction of a sewage treatment plant, trunk sewers, and a submarine outfall are scheduled under Stage I. Also scheduled is acquisition by the central agency of certain existing sewers of the city of Seattle.

Since initial flows will exceed 60 per cent of the ultimate flow (Table 16-9), the treatment plant will be constructed initially to meet ultimate needs. This plant will be of the primary type and will have a capacity of 3.7 mgd. Facilities will be provided for

Table 16-9. Estimated Average Sewage Flows, Sewage Treatment Plants, North Puget Sound Sewerage Area

Year	Average flow, mgd	
	Piper Creek	Boeing Creek
1965	2.3	1.0
1970	2.7	1.4
1975	3.0	1.9
1980	3.2	2.2
1985	3.3	2.4
1990	3.5	2.5
1995	3.6	2.5
2000	3.6	2.6

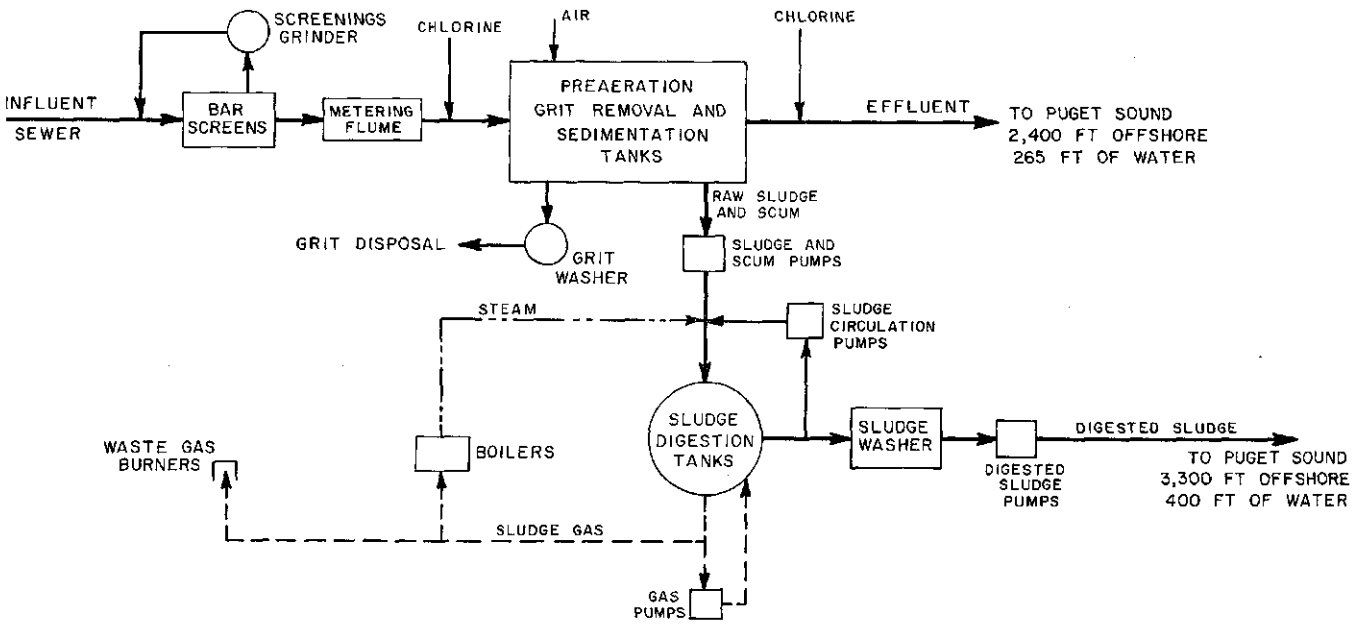

Fig. 16-10. Flow Diagram, Piper Creek Sewage Treatment Plant

pretreatment, preaeration and grit removal, sedimentation, separate sludge digestion, sludge disposal, and effluent chlorination. Approximately three acres of land will be required for the plant site. A schematic flow diagram is shown in Fig. 16-10, and design factors are given in Table 16-10.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 2,400 feet from shore at a depth of 265 feet. This sewer will consist of a land section comprising 2,000 feet of existing 27-inch pipe and a submarine section comprising 2,400 feet of 33-inch reinforced concrete pipe.

Boeing Creek Subarea, North Puget Sound Sewerage Area. Construction of a sewage treatment plant, submarine outfall and trunk sewers to serve the Ronald and Highlands area is scheduled under the Stage I program. Because of financial considerations, construction of the trunk sewer and pumping station to serve Richmond Beach is deferred to Stage II. Due, however, to the fact that raw sewage is presently discharged to Puget Sound at Richmond Beach, construction of facilities required to intercept this discharge should be undertaken at the earliest practicable date.

A primary type treatment plant will be required to serve the Boeing Creek subarea and will have a capacity of 2.6 mgd. An analysis of the predicted average sewage flows (Table 16-9) indicates that initial flows will be 1.0 mgd, or about 40 per cent of ultimate. By 1975, the flow will have increased to 1.9 mgd, or 70 per cent of ultimate. It will be necessary, therefore, to provide initially for ultimate needs.

As shown diagrammatically in Fig. 16-11, the recommended plant will contain facilities for pretreatment, preaeration and grit removal, sedimentation, separate sludge digestion, sludge disposal, and effluent chlorination. Approximately three acres will

Table 16-10. Design Factors, Piper Creek Sewage Treatment Plant

Population in thousands	25	Preaeration and grit removal tanks	
		Number	3
Loading		Detention, minutes	30
Average dry weather flow, mgd	3.7	Hydraulic capacity per tank, mgd	6
Peak wet weather flow, mgd	12	Sedimentation tanks	
Present minimum flow, mgd	0.9	Number	3
		Detention, minutes	90
BOD, 1,000 pounds per day	5	Overflow rate, gal. per sq ft per day	1,070
Suspended solids, 1,000 pounds per day	6	Hydraulic capacity per tank, mgd	6
Bar screens		Sludge digestion tanks	
Number	1	Number	2
		Volume per tank, 1,000 cubic feet	5
Clear spacing, inches	0.75	Loading, pounds per cubic foot per day	0.2

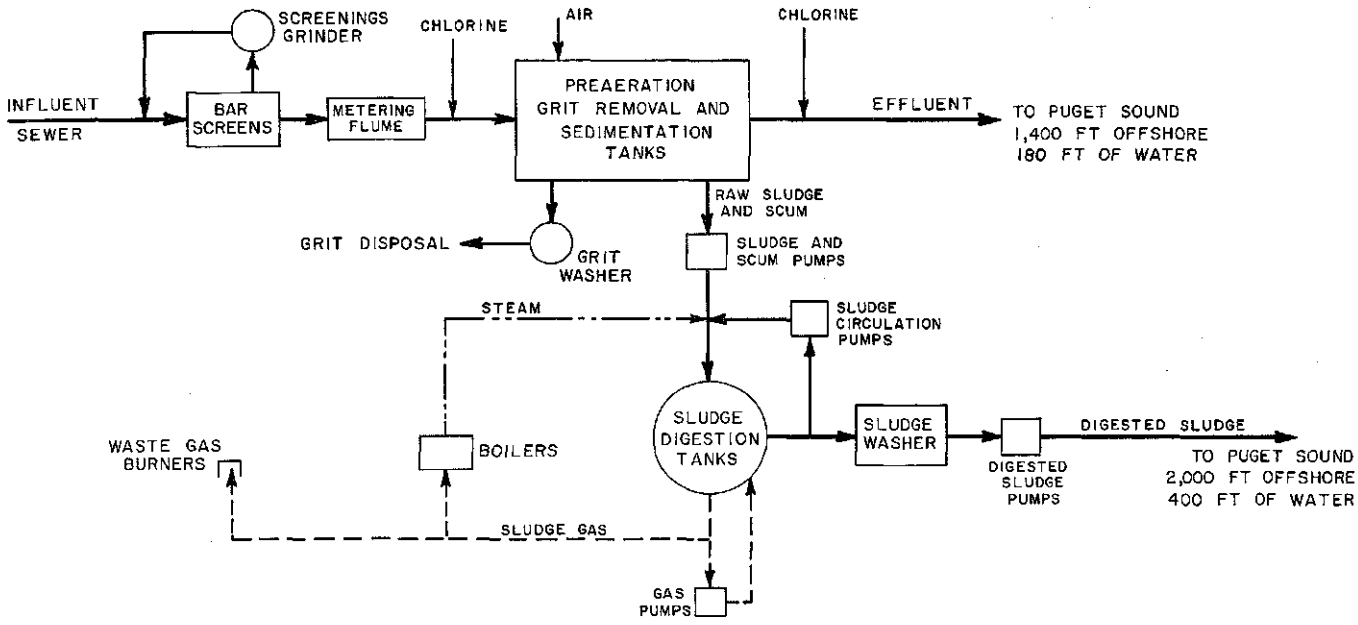

Fig. 16-11. Flow Diagram, Boeing Creek Sewage Treatment Plant

be required for the plant site. Design factors for the plant are presented in Table 6-11.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 1,400 feet from shore at a depth of 180 feet. This line will consist of a land section comprising 200 feet of 24-inch reinforced concrete pipe and of a submarine section comprising 1,400 feet of 24-inch reinforced concrete pipe.

Temporary Sewage Treatment Plants

Although it is difficult to determine exactly where new temporary treatment plants or other disposal facilities will be required, it is apparent at this time that such facilities will be necessary in the Bothell, Redmond and Mountlake Terrace areas. Similar facilities may be required also to serve isolated developments during the period covered by Stage I construction. Stage I estimates, therefore, include an allowance of $2,000,000 for the construction of temporary treatment plants.

Decisions as to whether temporary plants should be provided during subsequent stages of construction properly should be made as the area develops. It will not be possible until then to determine whether it would be more economical to construct a temporary facility or to obtain service from the central system.

STAGE II CONSTRUCTION, 1970-1980

Stage II construction is scheduled for the years 1970 to 1980. In this period, central sewerage facilities will be extended to the North Lake Washington sewerage area and to the Green River sewerage area. Additionally, sewerage service will be made available to new areas as they develop. For this stage, the total estimated construction costs are $35,417,000 (Table 16-12).

Table 16-11. Design Factors, Boeing Creek Sewage Treatment Plant

Population in thousands	20	Preaeration and grit removal tanks	
		Number	2
Loading		Detention, minutes	30
Average dry weather flow, mgd	2.6	Hydraulic capacity per tank, mgd	7.5
Peak wet weather flow, mgd	7.5	Sedimentation tanks	
Present minimum flow, mgd	0.4	Number	2
BOD, 1,000 pounds per day	4	Detention, minutes	90
Suspended solids, 1,000 pounds per day	5	Overflow rate, gallons per square feet per day	1,080
		Hydraulic capacity per tank, mgd	7.5
Bar screens		Sludge digestion tanks	
Number	1	Number	2
		Volume per tank, 1,000 cubic feet	4
Clear spacing, inches	0.75	Loading, pounds per cubic feet per day	0.20

Table 16-12. Stage II Construction, Recommended Sewerage Facilities

Facility	Construction cost,[a] dollars
Core Plan B[b]	
B-1[c]	1,120,000
STP-B-1[d]	1,638,000
Total, Core Plan B	2,758,000
Core Plan B feeder sewers[e]	
S-2 – S-3	939,000
S-8 – S-10	1,040,000
S-32[f]	3,090,000
S-33[g]	1,280,000
N-1 – N-6	5,245,000
PS-N-1	676,000
PS-N-3 – PS-N-4[h]	1,192,000
Total, feeder sewers	13,462,000
Service sewers	
East Lake Washington[i]	
ELW-1 – ELW-9	735,000
ELW-15 – ELW-16	129,000
ELW-17 – ELW-20	310,000
PS-ELW-1 – PS-ELW-2	304,000
PS-ELW-6	125,000
	1,603,000
North Lake Washington[j]	
NLW-8 – NLW-34	7,406,000
Northwest Lake Washington[j]	
NWW-1 – NWW-5	1,036,000
South Lake Washington[k]	
SLW-3	752,000
SLW-16 – SLW-17	370,000
	1,122,000
Green River[k]	
GR-19	56,000
GR-32 – GR-33	102,000
GR-35 - GR-47	1,802,000
GR-52 – GR-58	918,000
PS-GR-1	139,000
	3,017,000
Southwest Lake Washington[l]	
SWW-1 – SWW-2[m]	174,000
PS-SWW-1[n]	138,000
PS-SWW-3[n]	85,000
	397,000
Total, service sewers	14,581,000
Separate systems	
South Puget Sound[o]	
Redondo Beach subarea	
SPS-2	108,000
SPS-4 – SPS-7	202,000
PS-SPS-1	110,000
STP-SPS-1[p]	709,000
SPS-8	271,000
	1,400,000
Des Moines subarea	
SPS-9	236,000
SPS-12 – SPS-14	185,000
STP-SPS-2[q]	800,000
	1,221,000
Miller Creek subarea	
SPS-18 – SPS-21	424,000
STP-SPS-3[r]	350,000
	774,000
Southwest Suburban subarea	
STP-SPS-4[s]	550,000
SPS-28[t]	56,000
	606,000
West Seattle subarea	
SPS-38[u]	411,000
North Puget Sound[v]	
Boeing Creek subarea	
NPS-8	122,000
PS-NPS-1	82,000
	204,000
Total, separate systems	4,616,000
Total, Stage II construction	35,417,000

See Fig. 16-1 for location of facilities to be constructed under Stage II.

[a] *Includes engineering and contingencies.*

[b] *See Fig. 15-4 for location and Table 15-4 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[c] *Stage II construction: 63 in. at 0.05%, capacity 42 mgd (33% of ultimate). To be paralleled under Stage III construction.*

[d] *Enlargement to capacity of 48 mgd (33.3% of ultimate).*

[e] *See Fig. 15-7 for location and Table 15-11 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[f] *Stage II construction: 60-in. at 0.036 - 0.05%, capacity 31 to 38 mgd (33% of ultimate). To be paralleled under Stage III construction.*

[g] *Stage II construction: 60 in. at 0.063%, capacity 40 mgd (33% of ultimate). To be paralleled under Stage III construction.*

[h] *New stations to replace inadequate existing stations.*

[i] *See Fig. 15-15 for location and Table 15-33 for description of facilities.*

[j] *See Fig. 15-16 for location and Table 15-34 for description of facilities.*

(footnotes continued on next page)

Renton System Sewers

Intercepting sewers for the Renton system to be constructed under the Stage II program include: the Core Plan B interceptor extending south from the treatment plant; the south branch feeder sewer to the city of Kent; and extension of the north branch feeder sewer to serve the Juanita Bay area in the East Lake Washington sewerage area; and the branch feeder sewer to serve the eastern portion of the East Lake Washington sewerage area. Service sewers to be constructed comprise those required to extend sewerage service to developing areas.

Because the rate and extent of population and industrial development in the Green River sewerage area, particularly in the eastern portion, cannot be predicted accurately at the present time, construction of the south Core Plan B interceptor (B1) and the south feeder sewer (S32-33) is planned on an incremental basis. An analysis of predicted flows into these sewers indicates that about one-third the ultimate flow can be expected by 1990. It seems advisable, therefore, to provide initially for one-third the ultimate requirement. When the capacity of the original lines is reached, parallel lines can be constructed. At that time it will be possible to predict more precisely both the rate and extent of future development.

The advisability of incremental construction can be demonstrated by analyzing the estimated costs. If the Core Plan interceptor (B1) were constructed initially for ultimate needs, it would cost $1,504,000 (Table 15-14). On the other hand, if it were constructed for only one-third the ultimate capacity, the total cost would be $1,120,000 (Table 16-12). If that difference in capital outlay, amounting to $384,000, were invested at five per cent interest, it would yield $1,019,000 at the end of 20 years and $1,300,000 at the end of 25 years. The cost of paralleling the interceptor with another sewer having a capacity of two-thirds the ultimate need is estimated at $1,275,000, or about the amount that would accrue at the end of 25 years. Similar values would obtain in the case of the feeder sewers (S32-33).

Renton System Treatment Plant

Enlargement of the Renton sewage treatment plant to a capacity of 48 mgd is scheduled under Stage II construction. New facilities required will include three preaeration and primary sedimentation tanks, three aeration tanks, eight secondary sedimentation tanks, two digesters, and additional sludge lagoon capacity. At the same time, both influent pumping and aeration blower capacity will have to be increased.

West Point System Sewers

Sewers for the West Point system to be constructed under the Stage II program include extension of the north branch feeder sewer to its terminus at the pumping station (PS-N1) in the North Lake Washington sewerage area. With this extension, the entire North Lake Washington sewerage area and the northern portion of the Northwest Lake Washington sewerage area will be tributary to the West Point sewage treatment plant. Service sewers will be constructed as required to extend service to additional developing areas.

West Point System Treatment Plant

Since the West Point sewage treatment plant will be constructed initially with a capacity sufficient for ultimate needs, no additional construction will be required under Stage II.

Independent Systems

With the exception of the Rendondo Beach subarea, construction of facilities to extend service within the various subareas is scheduled under Stage II. In the Redondo Beach subarea, Stage II calls for the construction of a sewerage system, including trunk sewers, treatment works, and a submarine outfall, to serve that area.

STAGE III CONSTRUCTION

Construction of all remaining facilities herein recommended (Fig. 16-1) is provided for under Stage III. This program will be undertaken some time after 1980 as the need develops. Additions and improvements

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Table 16-12 footnotes continued.

[k] *See Fig. 15-17 for location and Table 15-35 for description of facilities.*

[l] *See Fig. 15-18 for location and Table 15-36 for description of facilities.*

[m] *Parallel to existing sewer.*

[n] *Replacement of inadequate existing station.*

[o] *See Fig. 15-12 for location and Table 15-27 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[p] *Initial construction for 1.67 mgd capacity (33% of ultimate).*

[q] *Enlargement to ultimate capacity of 6.5 mgd and provision of secondary treatment facilities.*

[r] *Enlargement to ultimate capacity of 7.5 mgd.*

[s] *Provision of secondary treatment facilities.*

[t] *Extension of submarine outfall.*

[u] *New outfall designed for maximum dry weather sanitary flow (20 mgd).*

[v] *See Fig. 15-13 for location and Table 15-30 for description of facilities.*

Table 16-13. Stage III Construction, Recommended Sewerage Facilities

Facility	Construction costs,[a] dollars
Core Plan B[b]	
B-1[c]	1,275,000
STP-B-1[d]	8,216,000
B-33[e]	318,000
Total, Core Plan B	9,809,000
Core Plan B feeder sewers[f]	
S-1	486,000
S-6 – S-7	305,000
S-16	118,000
S-23 – S-31	2,526,000
S-32[g]	4,099,000
S-33[h]	1,609,000
PS-S-1 – PS-S-4	1,669,000
PS-N-2[i]	100,000
Total, feeder sewers	10,912,000
Service sewers	
North Lake Sammamish[j]	
NLS-1 – NLS-43	8,500,000
PS-NLS-1 – PS-NLS-3	335,000
	8,835,000
South Lake Sammamish[k]	
SLS-1 – SLS-24	3,830,000
PS-SLS-1	364,000
	4,194,000
East Lake Washington[k]	
ELW-25 – ELW-26	231,000
ELW-29 – ELW-31[l]	152,000
ELW-38 – ELW-41	374,000
PS-ELW-8 – PS-ELW-10[m]	296,000
PS-ELW-13	70,000
	1,123,000
North Lake Washington[n]	
NLW-1 – NLW-7	1,102,000
Green River[o]	
GR-1 – GR-18	4,365,000
GR-22 – GR-31	1,858,000
GR-34	41,000
GR-48 – GR-51	372,000
	6,636,000
South Lake Washington[o]	
SLW-1 – SLW-2	402,000
SLW-4 – SLW-15	1,571,000
	1,973,000
Lake Union[p]	
LU-19[q]	169,000
Total, service sewers	24,032,000
Separate systems	
South Puget Sound[r]	
Redondo Beach subarea	
SPS-1	78,000
SPS-3	139,000
STP-SPS-1[s]	364,000
	581,000
Des Moines subarea	
SPS-10 – SPS-11	32,000
Total, separate systems	613,000
Total, Stage III construction	45,366,000

See Fig. 16-1 for location of facilities to be constructed under Stage III.

[a] *Includes engineering and contingencies.*

[b] *See Fig. 15-4 for location and Table 15-4 for description of facilities.*

[c] *84 in. at 0.05% (capacity 90 mgd) to parallel sewer constructed under Stage II.*

[d] *Enlargement in 2 increments to ultimate capacity of 143 mgd.*

[e] *Parallel to outfall constructed under Stage I.*

[f] *See Fig. 15-7 for location and Table 15-11 for description of facilities.*

[g] *78 in. at 0.036 - 0.05% (capacity 63 - 75 mgd) to parallel sewer constructed under Stage II.*

[h] *78 in. at 0.063% (capacity 80 mgd) to parallel sewer constructed under Stage II.*

[i] *Enlargement to ultimate capacity.*

[j] *See Fig. 15-14 for location and Table 15-32 for description of facilities.*

[k] *See Fig. 15-15 for location and Table 15-33 for description of facilities.*

[l] *Parallel to existing sewers.*

[m] *New stations to replace inadequate existing stations.*

[n] *See Fig. 15-16 for location and Table 15-34 for description of facilities.*

[o] *See Fig. 15-17 for location and Table 15-35 for description of facilities.*

[p] *See Fig. 15-18 for location and Table 15-36 for description of facilities.*

[q] *Replacement of existing inverted siphon.*

[r] *See Fig. 15-12 for location and Table 15-27 for description of facilities.*

[s] *Enlargement in 2 increments to ultimate capacity of 5.0 mgd.*

called for under Stage III are estimated to cost a total of $45,366,000 (Table 16-13).

SUMMARY OF STAGE CONSTRUCTION COSTS

As a matter of interest, it is desirable in concluding this chapter to total the construction costs for the three stages and thus to emphasize the magnitude of the expenditures which will be required for sewerage service in the years ahead. For the three stages, the estimated costs are $83,215,000 under Stage I, $35,417,000 under Stage II and $45,366,000 under Stage III. In all, therefore, the metropolitan Seattle area is faced with the prospect over the next 40 or 50 years of having to finance an estimated expenditure of $163,998,000 for facilities which will be required not only to protect the waters of Lake Washington but to bring an end to health and other hazards associated with any failure to provide properly for the sewerage needs of the entire community.

Chapter 17
DEVELOPMENT OF STORM DRAINAGE PLANS

Storm drainage is regarded and undertaken as a municipal function in every modern community throughout the world. The extent of its provision is related to and dependent upon many factors, among which are:

1. Intensity and duration of heavy rains.
2. Topography as related to ground slopes and available natural watercourses.
3. Soil conditions, particularly with respect to perviousness.
4. Proportionate area of paved surfaces and roofs.
5. Value of property to be protected against flooding.
6. Ability and willingness of taxpayers to meet the necessary costs.

Storm drainage serves the dual purpose of preventing damage due to flooding and of providing convenience in the use of city streets. It has little public health or esthetic significance, in which respect it is quite different from sanitary sewerage. Storm drainage facilities, moreover, are relatively expensive because the capacity of adequate conduits to service a given area must be many times that of sewers required to collect and convey sewage from the same area.

Despite the relatively high cost of storm drainage, its provision to the maximum possible extent is becoming regarded as a fundamental necessity. While it is difficult in many cases to justify the required expenditure on purely economic grounds, the insistent demands of personal and public comfort and convenience cannot be ignored. Further, the provision of an adequate storm drainage system will minimize storm water inflow to the sanitary sewerage system.

Unlike the disposal of sewage, the disposal of urban storm drainage does not involve a consideration either of treatment requirements or of dilution capacity available in receiving waters. Changes in land use from rural to urban functions may, however, have a significant effect on the drainage characteristics of a watershed and thus have to be taken into account in drainage planning.

Development of urban improvements in a watershed brings about a decrease in the amount of water entering the ground and thereby in the ground water level. This in turn affects both the outflow from springs and the level of spring-fed lakes. During wet weather, urban improvements serve to increase not only the total volume of surface water runoff but the maximum rate of runoff.

USE OF NATURAL WATERCOURSES

Storm water from most of the metropolitan Seattle area is now disposed of in numerous natural watercourses. In general, continued use of these channels will be essential to any project involving the collection, transportation and disposal of storm water runoff.

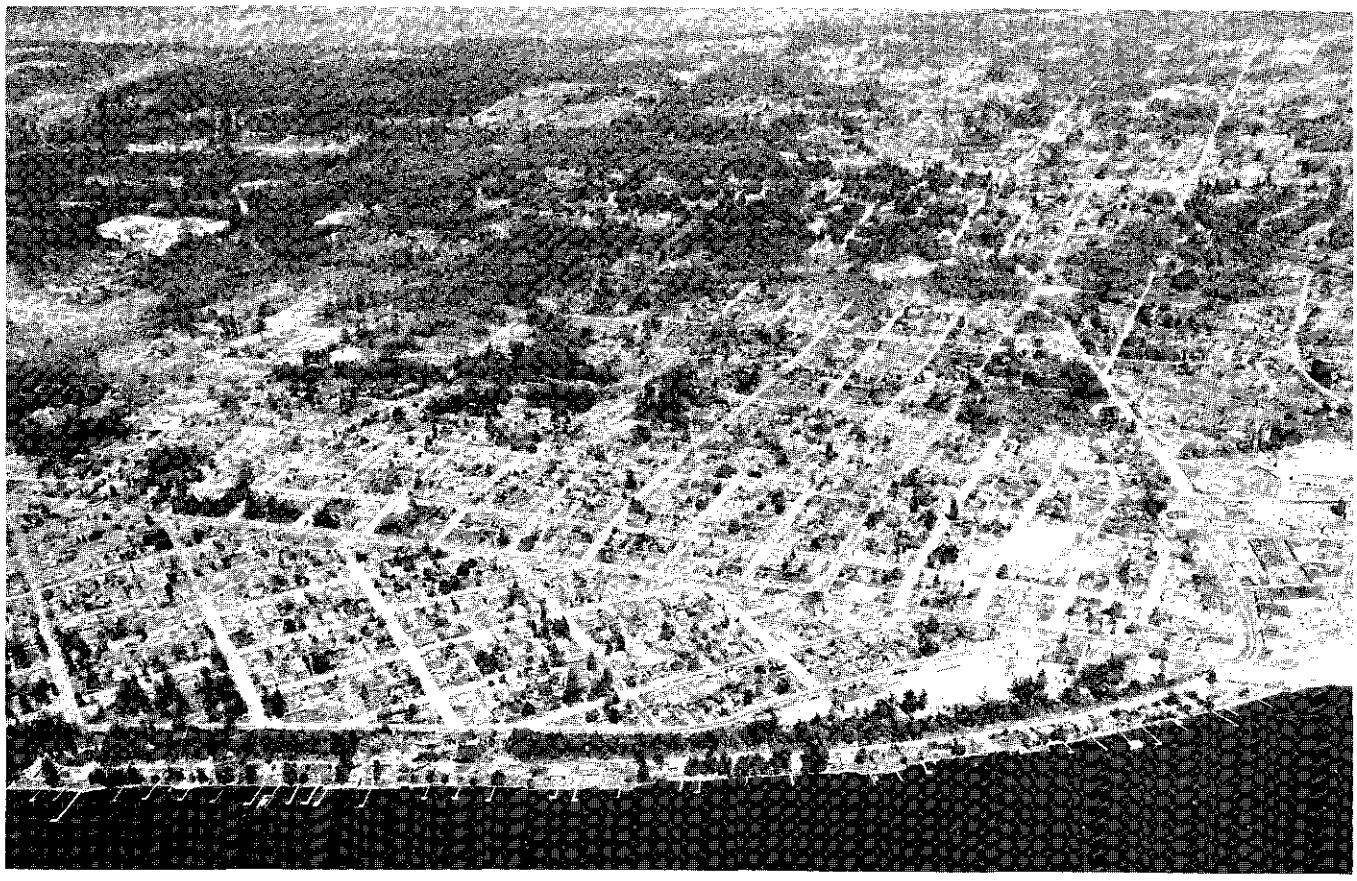
Use of natural watercourses as storm drainage channels may affect other beneficial uses. For the most part, however, these other uses are of secondary or incidental importance compared to the storm drainage function. In fact, some channels and stream beds may be utilized exclusively for storm water conveyance, even to the extent of converting them to closed conduits.

While impairment of other beneficial uses is inevitable in some cases, it does not necessarily follow that the storm drainage function is inconsistent with continued use of a watercourse for other purposes. This is particularly true in the case of wooded ravines and similar areas where channel improvements for storm drainage purposes can be kept to a minimum consistent with the protection of adjoining and upstream properties. In other words, many natural streams can be used for the conveyance of storm water without detracting from their value as recreational and esthetic assets.

From the standpoint of preliminary planning, it is obvious that the many lakes of the metropolitan area can be utilized as storage or holding basins for storm water runoff. Proper integration of these lakes in an over-all program of drainage improvements will, of course, make it possible to achieve maximum economy in the provision of facilities for downstream conveyance and disposal. By using a holding basin, it is possible to reduce the maximum rate of flow and thus the size of the downstream facilities.

LEGAL ASPECTS OF STORM DRAINAGE

Based upon a long record of court decisions, common law holds that watercourses and natural drainage channels may not be blocked to the detriment of upstream property. Likewise, surface runoff resulting from precipitation on upstream property can be disposed of to such watercourses, even when the rate of discharge is increased as a result of ordinary improvements on the property. Each owner, therefore, must accept surface waters naturally tributary to his



CITY OF KIRKLAND in the Kirkland-Houghton drainage area lies on the east shore of Lake Washington. Because of good ground slopes and short distances to final points of disposal, the cost of providing storm drainage facilities in areas such as this is reasonably low.

property. In turn, he has the right of unimpaired drainage of his own land, and of the improvements thereon, through properties downstream from him. This concept, by extension, applies also to political entities. On the other hand, if surface runoff is diverted from its natural watershed to another watershed, those responsible for the diversion also become responsible for any damage which may thus be caused.

CENTRAL CONTROL OF STORM DRAINAGE FACILITIES

In most instances, natural watershed boundaries transcend political boundaries. When, as a consequence, two or more political entities lie within a common watershed, cooperative action is required in order both to protect watercourses and to prevent damage by storm water runoff of public and private property. Such cooperation would be assured if the administration of all major drainage facilities were made the responsibility of the central agency proposed herein for the administration of major sewerage facilities.

A central agency would serve not only to provide adequate drainage for all areas but would achieve that objective at a cost which, in general, would be lower than if drainage functions were taken over on a piece-meal basis by individual communities or districts. Furthermore, the provision of adequate storm drainage facilities, as the need for them develops, reduces interruption to travel and communication and thereby benefits not only the area being served but the metropolitan area as a whole.

PRELIMINARY DESIGN OF STORM DRAINAGE FACILITIES

In the layout of drainage facilities, precise information regarding local topography and locations of individual drainage areas is required before even preliminary plans of drainage structures can be evolved. Normally such information cannot be obtained until an area has developed to the extent that drainage patterns as modified by street layouts and other factors can be fully evaluated.

Because much of metropolitan Seattle is presently undeveloped, it is virtually impossible at this time to delineate the over-all drainage areas and subareas required for design purposes. It was not considered

feasible, therefore, to attempt a layout of drainage facilities for the entire area. Instead, preliminary designs were developed for four typical areas in which drainage patterns could be properly ascertained. These preliminary layouts are presented in subsequent sections of this chapter and illustrate the methods that should be employed in the design of drainage facilities. A study of storm drainage requirements, as related to separation of the present combined systems serving Seattle, is discussed later in Chapter 18.

When and as the design of drainage projects is undertaken, planning should encompass the entire tributary area. Moreover, each drainage area should be subdivided into appropriate subareas and storm water contributions should be calculated accordingly. This procedure was followed in developing preliminary designs for the four typical areas herein considered.

Design Period

Design of storm drainage facilities to serve a particular area should be based on conditions which are expected to prevail at ultimate or saturation development of that area. This in turn calls for the use of runoff coefficients and times of concentration anticipated in the future. While it is evident that only a portion of a drainage system has to be constructed initially, facilities then provided should have a capacity sufficient for ultimate needs. Subsequent additions may be made on a stage or stepwise basis, timed to keep pace with the growth of the tributary area.

Design Criteria

Criteria to be used in the design of storm drainage facilities were presented in Chapter 13. Briefly, design of urban storm drainage systems is based on the rational formula. This is represented by the formula $Q = ciA$ wherein Q is the runoff rate in cubic feet per second, c is a selected coefficient of runoff expressed as the ratio of runoff to rainfall, i is the mean intensity of rainfall expressed in inches per hour, and A is the tributary area expressed in acres. Values for c were based on projected development of the area and were obtained from Table 13-5. Values of i for calculated times of concentration were based on a storm having a recurrence interval of 10 years (Fig. 13-1 and Tables 13-6 and 13-7).

Storm Water Conduits

Three general categories of storm water conduits were used in developing preliminary layouts for the four typical areas. These were reinforced concrete pipe, open concrete lined channels, and open improved earth channels. Selection of the type of conduit to be used was based on a determination of the expected extent and type of areal development, modified to some degree by economic considerations.

Reinforced concrete pipe was used in areas where a high degree of development, either residential, commercial or industrial, is to be expected. In such areas, closed conduits offer definite advantages with respect to maintenance, safety, community appearance, space requirements, and general convenience.

Concrete lined channels were used in areas where soil conditions are such that scouring would occur at the required velocities. They were used also where the design flow is such that provision of pipe conduits would be economically unsound. Fencing of the channels was provided to the extent deemed necessary for public safety.

Improved earth channels were used in areas where development can be expected to be relatively light, where scouring velocities would not develop, and where the flows are such as to preclude the use of pipe conduits. In general, this type of conduit involved the use of natural watercourses, improved only to the degree necessary to obtain the desired capacity. In all open channels, whether lined or unlined, reinforced concrete box or pipe culverts were provided at all street and railroad crossings.

Storm Drainage for Local Service Areas

Storm drainage systems herein considered were laid out in each case to serve local tributary areas of not less than 160 acres (Chapter 13). As such, they include only those facilities necessary to establish a basic framework in the four areas selected for preliminary study. Laterals and street inlets will have to be constructed to serve smaller areas and connections will have to be made to the basic network.

Design of storm drainage systems for local service areas is beyond the scope of this report. Consideration should be given, nevertheless, to some of the standards applicable to the provision of lateral drains and street inlets.

Use of Street Gutters for Conveying Storm Flows. It is obvious that maximum economy in the design of a storm drainage system can be attained only if street gutters are utilized to the fullest possible extent for the collection and conveyance of the storm runoff. Only thus can the lengths and costs of necessary underground drains be reduced to a minimum.

Street gutters can be utilized to within about one inch of the tops of their curbs, provided pedestrian traffic is not heavy nor the velocity of flow is so great as to be a potential hazard to public safety. In areas where gutters are to be used, roof leaders may discharge either directly onto the ground or through a pipe laid under the sidewalk and curb into the gutter. For design purposes, the extent of gutter use should be determined for each particular street.

Storm Drain Laterals for Foundation Drains. In areas where the local governing body accepts responsibility for the gravity collection of water from foundation drains, storm drain laterals must be provided together with sanitary sewer laterals. Except in areas served by existing combined sewers, design of the sewerage system proposed for the metropolitan area is such that no roof leader or foundation drain connections can be tolerated. Obviously, therefore, storm water from these sources will have to be collected in separate storm drain laterals.

Street Inlets. Street inlets should be provided where necessary to pick up gutter water. In general, good practice requires that flow in gutters be picked up at intervals of about 1,200 feet. In flat areas, the spacing may have to be somewhat less, while that in steep areas may be considerably greater. In commercial and downtown business areas at main street intersections, public convenience requires that very little water be permitted to flow in gutters. Hence, street inlets should be provided at each intersection. Usually four, or sometimes eight, inlets are required at such locations.

Street inlets should be located on the "return", that is, at the beginning of the curve of the corner curb. This eliminates the high curb which results when the inlet is placed at the midpoint of the curve. By using two inlets or a cross curb drain, the entire pedestrian crossing area can be kept free of water. Common practice calls for one inlet in residential areas and two inlets in downtown areas.

Design of street inlets should be such as to facilitate the entrance of water without clogging. A satisfactory design provides for grating bars parallel to the gutter. Provision should be made also for a vertical opening along the curb face both to accommodate increased flow and to permit entrance of water if the grating becomes clogged. To serve these purposes, the horizontal grated section should have a minimum area of six square feet, while the vertical curb face opening should be a minimum of three feet long by four inches high.

Street inlets should be depressed a minimum of two inches below the gutter level. Further, they should be connected to the main drainage system by a pipe at least 10 inches in diameter. The bottom of the inlet should be sloped to the invert of the pipe.

Each inlet must be checked for hydraulic capacity by comparing the design flow which is to be picked up with the known capacity of the inlet. Where the expected flow is greater than the capacity of the inlet, suitable modifications must be made. These include provision of a longer curb face opening, use of two inlets, and provision of a depressed gutter in the vicinity of the inlet.

In storm drainage systems properly designed to prevent deposition in the lines, there is no need for catch basins wherein sand and grit are trapped and accumulated. Catch basin maintenance is usually such that the accumulated material is rarely removed, with the result that the basin soon fails to perform its intended function.

STORM DRAINAGE PLANS FOR SELECTED AREAS

Preliminary designs of storm drainage facilities for four selected drainage areas in metropolitan Seattle were made (1) to demonstrate the method of runoff computation and conduit size selection, and (2) to provide a basis for estimating the cost of a drainage program. Selection of the study areas was based on several factors, among which are:

1. Different problems with respect to drainage design.
2. Similarity to other drainage areas in metropolitan Seattle.
3. Present development to a degree that design factors can be properly ascertained.

Since topography governs the general route, as well as the point of discharge of storm drainage, design alternatives are limited generally to minor deviations in routes and, in some cases, to a choice between an open channel or a closed conduit. In a few instances, the situation with respect to topography is such that it is possible to divert runoff from one drainage area to another. Comparative costs of such alternatives are discussed later in connection with the individual projects to which they apply.

Kirkland - Houghton Area

Occupying a total of 4,510 acres, the Kirkland-Houghton drainage area (Fig. 17-1) lies along the eastern shore of Lake Washington and extends from Juanita Bay on the north to Yarrow Bay on the south. Included within its boundaries are the cities of Kirkland and Houghton. Dense development has taken place in parts of the area and is expected to spread eventually over the entire area.

The Kirkland-Houghton drainage area comprises a number of small subareas, each draining individually to Lake Washington. Surface slopes, which are generally from east to west, and distances to points of disposal are such as to enable effective and low-cost drainage with a minimum of local complications.

All major storm water conduits serving the area follow natural watercourses or grades. Except for possible minor changes during final design, no alternatives are apparent for the suggested routing of storm drains. Because of present and anticipated future developments, pipe drains are mandatory in all but the northernmost subarea. In the latter, some sec-

Table 17-1. Description and Estimated Construction Costs, Drainage Plans KH-A and KH-B, Kirkland – Houghton Area

Number	Design flow, cfs	Description	Construction cost, dollars
Plan KH-A			
KH-1	115	2,200 ft of 30-in. RC at 7.1%, dry, includes repaving and railroad and highway crossings	52,000
KH-2	181	2,100 ft of 42-in. RC at 3.0%, dry, includes railroad and highway crossings	64,000
KH-3	332	3,500 ft of 54-in. RC at 2.7%, dry to moderately wet, includes railroad and highway crossings	132,000
KH-4	228	2,700 ft of 45-in. RC at 3.2%, dry to moderately wet, includes railroad and highway crossing	82,000
KH-5	122	1,800 ft of 33-in. RC at 5.0%, dry	34,000
KH-6	680	1,700 ft of 78-in. RC at 1.6%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	135,000
KH-7	476	2,300 ft of 66-in. RC at 1.9%, dry to wet	110,000
KH-8	219	1,100 ft of 45-in. RC at 3.0%, dry to moderately wet, includes railroad crossing	30,000
KH-9	299	400 ft of 51-in. RC at 3.0%, dry to moderately wet	13,000
KH-10	72	2,600 ft of 27-in. RC at 5.2%, dry	40,000
KH-11	545	2,600 ft of improved open channel at 0.07%, average cross-sectional area 110 sq ft, includes street crossings	58,000
KH-12	465	2,500 ft of improved open channel at 0.07%, average cross-sectional area 93 sq ft, includes street crossing	16,000
KH-13	398	1,800 ft of improved open channel at 0.55%, average cross-sectional area 40 sq ft, includes street crossing	14,000
KH-14	324	1,400 ft of improved open channel at 0.52%, average cross-sectional area 35 sq ft, includes railroad crossing	17,000
KH-15	268	2,200 ft of improved open channel at 0.53%, average cross-sectional area 30 sq ft, includes street crossing	31,000
KH-16	181	1,000 ft of improved open channel at 0.60%, average cross-sectional area 22 sq ft, includes outlet from Forbes Lake	6,000
KH-17	64	2,600 ft of 33-in. RC at 1.6%, dry to wet	50,000
KH-18	126	2,600 ft of 36-in. RC at 4.0%, wet to difficult wet, includes inlet to Forbes Lake	57,000
KH-19	72	3,100 ft of 30-in. RC at 3.1%, dry	53,000
Total contract cost, Plan KH-A			994,000
Engineering and contingencies, 25 per cent			248,000
Total construction cost, Plan KH-A			1,242,000
Plan KH-B			
KH-1 – KH-10	–	Same as KH-1 – KH-10, Plan KH-A	692,000
KH-11	570	2,600 ft of 90-in. RC at 0.54%, difficult wet, includes sheeting, dewatering and highway crossings	299,000
KH-12	485	2,500 ft of 66-in. RC at 2.0%, includes sheeting, dewatering, and highway crossing	149,000
KH-13	414	1,800 ft of 60-in. RC at 2.4%, dry to wet, includes highway crossing	85,000
KH-14	336	1,400 ft of 57-in. RC at 2.1%, dry to wet, includes railroad crossing	66,000
KH-15	268	2,200 ft of 57-in. RC at 1.5%, dry to wet, includes highway crossing	101,000

Continued on next page

Table 17-1. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
KH-16	181	1,000 ft of 45-in. RC at 2.2%, dry to wet	30,000
KH-17 – KH-19	–	Same as KH-17 – KH-19, Plan KH-A	162,000
Total contract cost, Plan KH-B			1,584,000
Engineering and contingencies, 25 per cent			396,000
Total construction cost, Plan KH-B			1,980,000

See Fig. 17-1 for location of facilities.

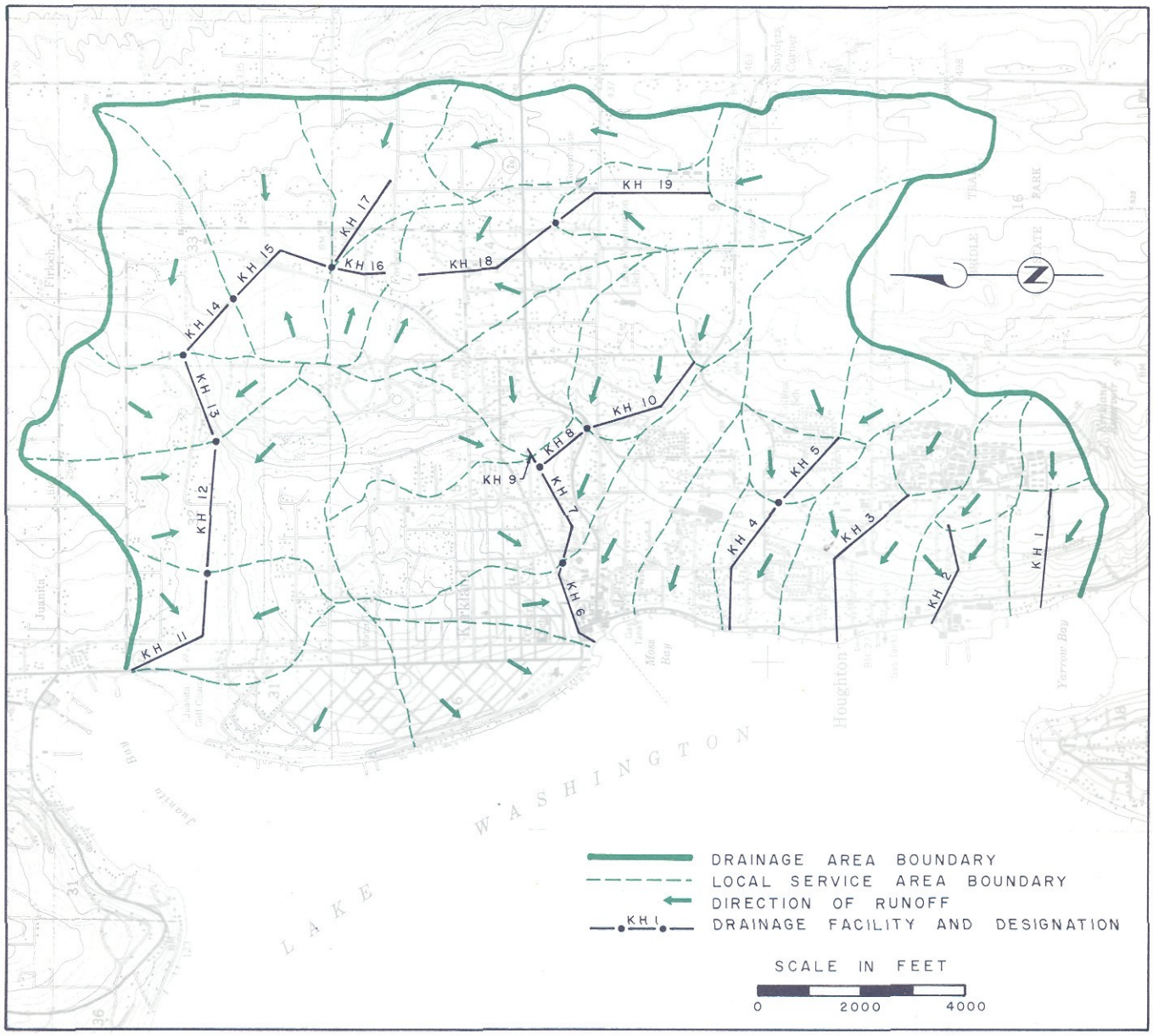


Fig. 17-1. Storm Drainage Plans KH-A and KH-B, Kirkland-Houghton Area

tions could be drained by means of unlined earth channels.

Two plans were laid out to determine the relative costs of closed versus open conduits. The first of these plans, designated as Plan KH-A, provides for using improved earth channels to the fullest possible extent. The second, designated as Plan KH-B, is based on the use of reinforced concrete pipe throughout the entire system.

Locations of major storm drainage facilities for the Kirkland-Houghton drainage area are shown in Fig. 17-1. Descriptions and estimated construction costs of facilities required under each of the two plans are given in Table 17-1. For Plan KH-A, the estimated cost is $1,242,000, while that for Plan KH-B amounts to $1,980,000. Based on a total tributary area of 4,510 acres, the cost of providing trunk storm drainage facilities in areas similar to Kirkland-Houghton will vary between $270 and $440 per acre, depending on the extent to which open channels can be used.

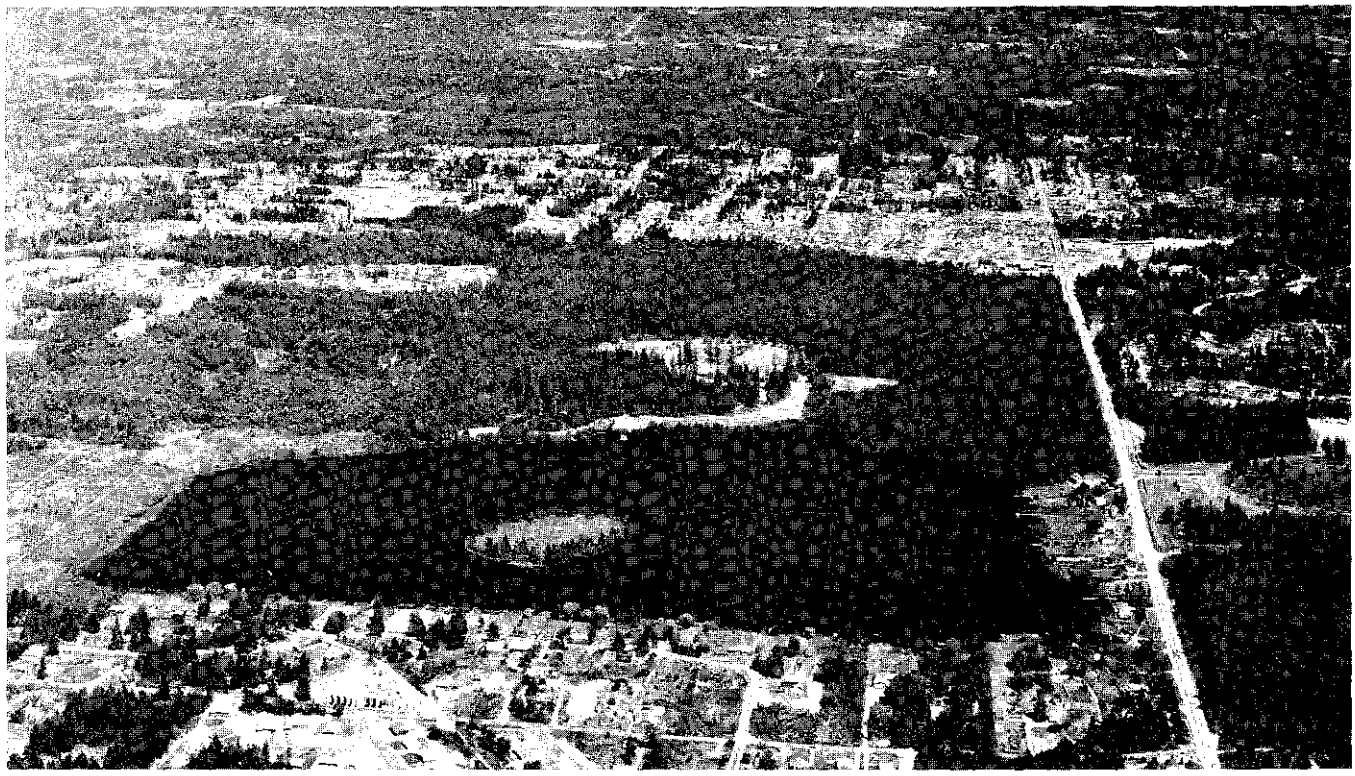
Mountlake Terrace Area

This moderately well developed area contains 7,610 acres, of which 4,550 acres are in Snohomish County. Residential communities are growing rapidly and are expected to continue developing until the area is fully occupied.

Topographically, the Mountlake Terrace area consists of two drainage basins, Lyon Creek and McAleer Creek (Fig. 17-2). Lyon Creek basin, containing a total of 2,510 acres, slopes steeply from north to south and is drained by Lyon Creek which discharges to Lake Washington at the southern end of the basin. McAleer Creek basin is drained by McAleer Creek, which originates at Lake Ballinger and also discharges to Lake Washington. Lake Ballinger occupies an area of approximately 100 acres. Of the total of 5,100 acres in this basin, 3,300 acres are tributary to Lake Ballinger and the remainder directly to McAleer Creek.

Both of the two creeks flow through relatively steep ravines, the sides of which are occupied throughout much of their length by attractive, well landscaped homes. Along their lower reaches, the two creeks traverse a flat, low-lying area potentially valuable for commercial development. For a distance of about one and one-half miles above their points of discharge, the creeks run a few hundred yards apart and parallel to each other.

In the Mountlake Terrace area, storm water runoff originating in Snohomish County must flow into King County. Drainage facilities to be constructed in King County must be designed, therefore, to accommodate both its own runoff plus that from Snohomish County. Such a project should be the joint responsibility of



LAKE BALLINGER in the Mountlake Terrace drainage area acts as a storm water holding basin, thus reducing downstream flows. Similar lakes in the metropolitan area should be utilized as holding basins to achieve economical design of storm drainage facilities.

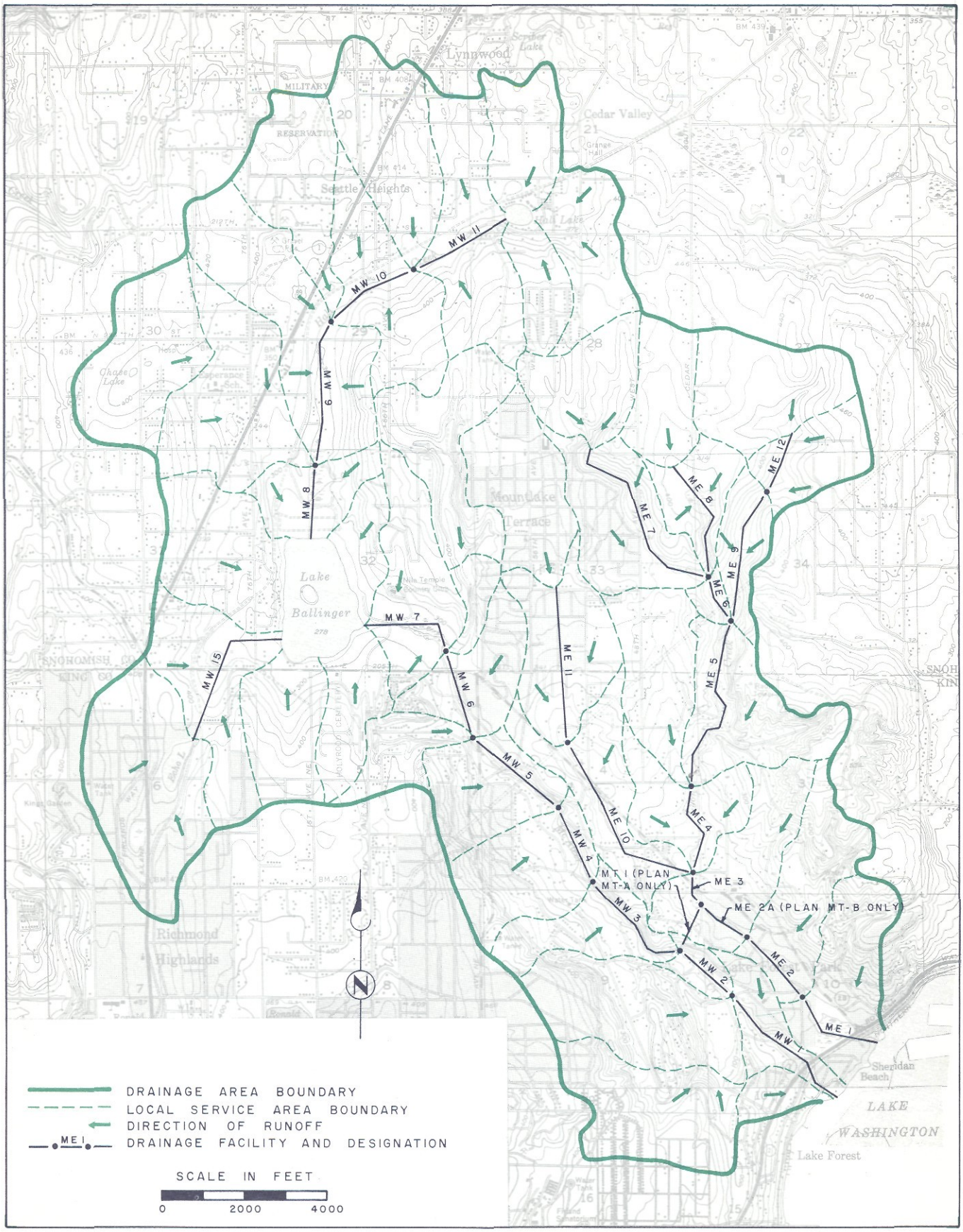


Fig. 17-2. Storm Drainage Plans MT-A and MT-B, Mountain Terrace Area

Table 17-2. Description and Estimated Construction Costs, Drainage Plans MT-A and MT-B, Mountlake Terrace Area

Number	Design flow, cfs	Description	Construction cost, dollars
Plan MT-A			
ME-1	48	2,200 ft of 30-in. RC at 1.8%, difficult wet, includes railroad and highway crossings	65,000
ME-2	13	2,100 ft of 18-in. RC at 1.7%, wet to difficult wet	31,000
ME-3	366	700 ft of improved open channel at 0.09%, average cross-sectional area 75 sq ft, includes street crossings	13,000
ME-4	317	2,100 ft of improved open channel at 0.09%, average cross-sectional area 65 sq ft, includes street crossing	10,000
ME-5	271	4,500 ft of improved open channel at 0.06%, average cross-sectional area 55 sq ft, includes street crossings	28,000
ME-6	127	1,200 ft of improved open channel at 2.85%, average cross-sectional area 9 sq ft, includes street crossing	3,000
ME-7	37	4,600 ft of 27-in. RC at 1.35%, wet, includes street crossing	87,000
ME-8	30	2,800 ft of 24-in. RC at 2.9%, dry to difficult wet, includes street crossings	47,000
ME-9	88	3,200 ft of improved open channel at 1.2%, average cross-sectional area 8 sq ft	5,000
ME-10	64	4,800 ft of 33-in. RC at 1.9%, dry to wet	95,000
ME-11	30	3,600 ft of 21-in. RC at 4.7%, dry, includes repaving and street crossing	45,000
ME-12	48	1,600 ft of 30-in. RC at 1.3%, dry to wet, includes street crossing	29,000
MW-1	570	3,200 ft of improved open channel at 0.06%, average cross-sectional area 115 sq ft, includes street crossings	39,000
MW-2	560	1,700 ft of improved open channel at 0.06%, average cross-sectional area 115 sq ft, includes street crossing	12,000
MT-1	400	1,100 ft of 66-in. RC tunnel at 1.5%, includes allowance of 20% for uncertainties	290,000
MW-3	199	2,900 ft of improved open channel at 0.51%, average cross-sectional area 23 sq ft, includes street crossings	15,000
MW-4	163	2,000 ft of improved open channel at 0.47%, average cross-sectional area 20 sq ft	4,000
MW-5	133	2,800 ft of improved open channel at 0.55%, average cross-sectional area 17 sq ft, includes street crossings	14,000
MW-6	113	1,300 ft of improved open channel at 0.69%, average cross-sectional area 15 sq ft, includes street crossing	7,000
MW-7	105[a]	2,000 ft of 60-in. RC at 0.15%, wet to difficult wet, includes street crossings and outlet structure from Lake Ballinger	93,000
MW-8	326	1,700 ft of 72-in. RC at 0.53%, difficult wet, includes street crossings and inlet structure to Lake Ballinger	122,000
MW-9	206	3,400 ft of 63-in. RC at 0.53%, wet to difficult wet, includes street crossings	154,000
MW-10	128	2,400 ft of 51-in. RC at 0.54%, wet, includes street crossing	85,000
MW-11	64	2,600 ft of 39-in. RC at 0.76%, wet, includes street crossing	65,000
MW-14	73	1,900 ft of 36-in. RC at 1.05%, dry, includes street crossing	36,000
MW-15	41	3,600 ft of 24-in. RC at 3.2%, dry to wet, includes repaving and street crossings	55,000
MW-16	50	3,600 ft of 27-in. RC at 2.6%, dry, includes street crossing	59,000
MW-17	46	2,500 ft of 27-in. RC at 2.2%, wet to difficult wet	45,000
Total contract cost, Plan MT-A			1,553,000
Engineering and contingencies, 25 per cent			388,000
Total construction cost, Plan MT-A			1,941,000

Continued on next page

Table 17-2. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Plan MT-B			
ME-1	455	2,200 ft of 66-in. RC at 1.9%, difficult wet, includes railroad and highway crossings	196,000
ME-2	417	2,100 ft of 66-in. RC at 1.7%, wet to difficult wet	135,000
ME-2A	388	1,300 ft of 66-in. RC at 1.6%, wet	62,000
ME-3 – ME-12	–	Same as ME-3 – ME-12, Plan MT-A	362,000
MW-1	226	3,200 ft of improved open channel at 0.11%, average cross-sectional area 45 sq ft, includes street crossings	38,000
MW-2	205	1,700 ft of improved open channel at 0.11%, average cross-sectional area 45 sq ft, includes street crossing	8,000
MW-3 – MW-17	–	Same as MW-3 – MW-17, Plan MT-A	754,000
Total contract cost, Plan MT-B			1,555,000
Engineering and contingencies, 25 per cent			389,000
Total construction cost, Plan MT-B			1,944,000

See Fig. 17-2 for location of facilities.

[a]*Outflow from Lake Ballinger which will act as a holding basin.*

both counties, preferably acting through a central agency.

Since the two streams draining the area flow parallel to and close to each other along their lower reaches, diversion of a part of the runoff from the Lyon Creek basin to the McAleer Creek basin would be a feasible alternative to separate disposal in each basin. Two projects were laid out to show their comparative costs. Plan MT-A designates that project wherein part of the flow in the Lyon Creek basin is diverted to the McAleer Creek basin. Under Plan MT-B, all drainage is conveyed and disposed of in each basin separately.

Location of the major storm drainage facilities are shown in Fig. 17-2. Descriptions and estimated construction costs of the facilities required under both plans are given in Table 17-2.

Facilities upstream from the point of diversion of Lyon Creek to McAleer Creek are the same for both plans. Further, both plans propose the use of Lake Ballinger as a storm water holding basin. While no detailed estimates were made of the value of the lake for storm drainage impoundment, this can be readily demonstrated.

As shown in Table 17-2, the flow to be provided for at the lake outlet would be reduced from 326 to 105 cfs by utilizing the lake for storage purposes. This reduction permits the use of a 60-inch line in the first section immediately downstream from the lake (MW 7), whereas a 90-inch line would be required if the total upstream flow were to be accommodated. On that basis, the difference in cost for this section alone would amount to about $100,000. It is apparent, therefore, that continued use of Lake Ballinger for storm drainage impoundment is an economic necessity. Wherever possible, other lakes throughout the metropolitan area should be fully utilized for the same purpose. In the case of Lake Ballinger, storage requirements are such that a water level change of about two feet will obtain under winter storm conditions.

Estimated construction costs of the two plans are almost identical. Plan MT-A is estimated to cost $1,941,000, as compared to $1,944,000 for Plan MT-B. It is obvious, therefore, that the decision as to which of the two plans should be adopted for the Mountlake Terrace area should be based on factors other than cost. These factors would include value of land released for other than storm drainage purposes, magnitude of construction work in congested areas, and reduction of flood potential in business and other districts of high value.

Based on a total tributary area of 7,610 acres, the cost of providing major storm drainage facilities in areas similar to Mountlake Terrace will be about $260 per acre.

Des Moines Area

Situated on the western slope of the metropolitan area, the Des Moines drainage area drains to Puget Sound. This area comprises two drainage basins, both of which empty into the sound within the community of Des Moines (Fig. 17-3). Of the two basins, the northern contains 3,580 acres and drains the greater portion of the Seattle-Tacoma International Airport. Discharge is to the sound through a narrow, steep-sided, heavily wooded natural watercourse. In the southern basin, which contains 1,270 acres, sur-

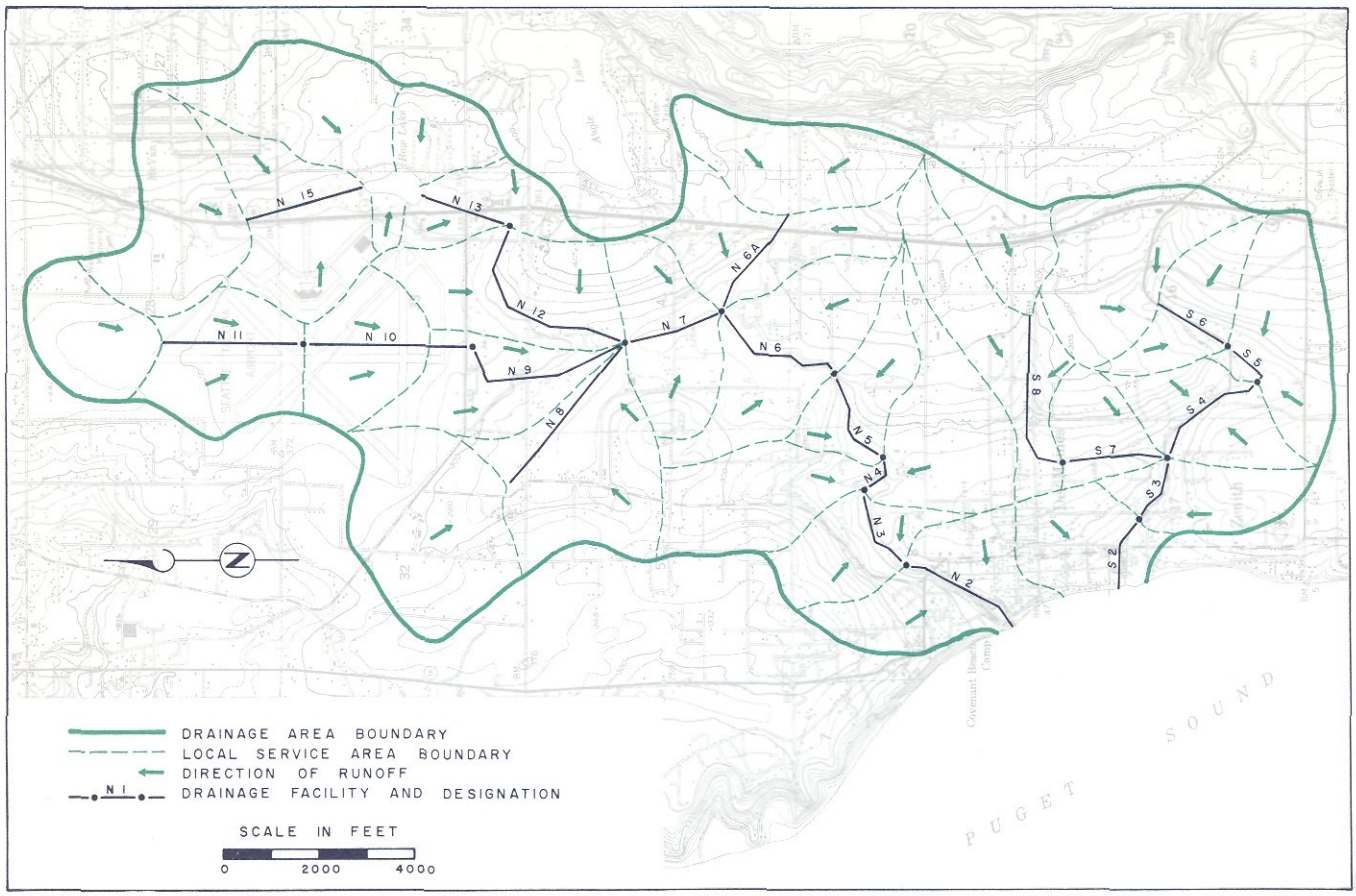


Fig. 17-3. Storm Drainage Plan, Des Moines Area

face slopes and distances to the point of disposal are such that minimum size conduits can be utilized.

Development in the area is presently centered around the community of Des Moines and the airport. Continued residential growth can be expected throughout the area, with some industrial development adjacent to the airport.

The drainage plan proposed for the Des Moines area calls for the use of improved earth channels to the fullest possible extent. Enclosed conduits are to be utilized only where extensive commercial development can be expected, or in areas where elimination of open channels will release land suitable for residential development. No alternatives either as to route or type of facility were considered.

Locations of all major storm drainage facilities required for the Des Moines area are shown in Fig. 17-3. Descriptions and estimated construction costs are given in Table 17-3.

Estimated construction costs amount to a total of $2,655,000. Based on a total tributary area of 4,850 acres, the cost of providing major storm drainage facilities in areas similar to Des Moines will amount to approximately $550 per acre.

Kent Area

Situated in the Green River valley, the Kent drainage area (Fig. 17-4) is bounded on the south and west by Green River, on the east by the ridge separating the Green River and Big Soos Creek drainage basins, and on the north by an arbitrary line along the present north city limit of the city of Kent. Present development is centered around Kent, the only incorporated city in the area. Large scale industrial development can be expected in and near the city. Residential growth undoubtedly will occur in the hilly regions along the eastern boundary of the area. Of the total of 11,480 acres within the drainage area, 6,190 acres lie in the valley and 5,290 acres in the hilly regions to the east.

For a distance of about one and one-half miles to the east, the Kent area is below the flood stage of Green River. For that reason, dikes have been constructed to prevent flooding of adjacent lands. Periodically, however, the river tops the dikes and flooding occurs. While it is expected that flooding may be controlled by the construction of Eagle Gorge dam, the fact still remains that storm water runoff from the floor of the valley cannot be drained to the river by gravity flow. For instance, information obtained

from the Seattle office of the U.S. Army Corps of Engineers indicates that the estimated water surface elevation in Green River at Kent was about 40 feet above mean sea level during the flood of 1951. At that time, the measured flow in the river was 19,000 cfs. Even with a controlled maximum flow of 12,000 cfs, the water surface elevation at Kent would be about 35 feet above mean sea level. Ground surface elevations in the valley along the river range from about 40 feet above mean sea level on the south to 22 feet on the north.

Continued development of suburban areas at higher elevations to the east will result not only in increased runoff but in a greatly reduced time of concentration. In consequence, storm water will accumulate rapidly in the low-lying areas. Obviously, therefore, the provision of drainage facilities to prevent flooding these areas by local surface runoff will be a difficult and costly undertaking.

With the valley floor sloping from south to north as well as east to west, several alternatives are pos-

Table 17-3. Description and Estimated Construction Costs, Drainage Plan for Des Moines Area

Number	Design flow, cfs	Description	Construction cost, dollars
S-2	425	1,500 ft of 78-in. RC at 0.8%, difficult wet, includes repaving and street crossings	110,000
S-3	404	1,300 ft of 57-in. RC at 3.1%, wet, includes street crossings	54,000
S-4	176	2,700 ft of 45-in. RC at 2.2%, wet, includes street crossings	87,000
S-5	117	1,000 ft of improved open channel at 0.57%, average cross-sectional area 16 sq ft, includes street crossing	9,000
S-6	74	1,800 ft of improved open channel at 0.70%, average cross-sectional area 11 sq ft	3,000
S-7	118	2,100 ft of 39-in. RC at 2.9%, wet, includes street crossings	58,000
S-8	61	3,600 ft of 27-in. RC at 4.9%, dry to wet, includes repaving and street crossing	64,000
N-2	637	2,300 ft of improved open channel at 0.53%, average cross-sectional area 58 sq ft, includes street crossing	30,000
N-3	589	1,900 ft of improved open channel at 0.55%, average cross-sectional area 54 sq ft	6,000
N-4	502	800 ft of improved open channel at 0.51%, average cross-sectional area 50 sq ft	3,000
N-5	484	2,100 ft of improved open channel at 0.70%, average cross-sectional area 48 sq ft, includes street crossing	14,000
N-6	436	2,800 ft of improved open channel at 0.65%, average cross-sectional area 44 sq ft, includes street crossing	15,000
N-6A	66	2,600 ft of 30-in. RC at 3.3%, dry to moderately wet, includes highway crossing	45,000
N-7	376	1,900 ft of improved open channel at 0.58%, average cross-sectional area 39 sq ft, includes street crossing	16,000
N-8	67	4,100 ft of 42-in. RC at 0.49%, wet	123,000
N-9	156	4,000 ft of 45-in. RC at 1.9%, dry, includes highway crossing	114,000
N-10	80	3,100 ft of 39-in. RC at 2.1%, dry, includes airport runway crossing	83,000
N-11	56	3,100 ft of 39-in. RC at 0.48%, dry, includes airport runway crossing	100,000
N-12	159	4,200 ft of 45-in. RC at 1.8%, dry to wet	114,000
N-13	128	1,900 ft of 51-in RC at 0.53%, wet to difficult wet, includes highway crossing and outlet from Bow Lake	82,000
N-15	59	3,000 ft of 45-in. RC at 0.23%, wet to difficult wet, includes inlet to Bow Lake	124,000
Total contract cost			2,124,000
Engineering and contingencies, 25 per cent			531,000
Total construction cost			2,655,000

See Fig. 17-3 for location of facilities.

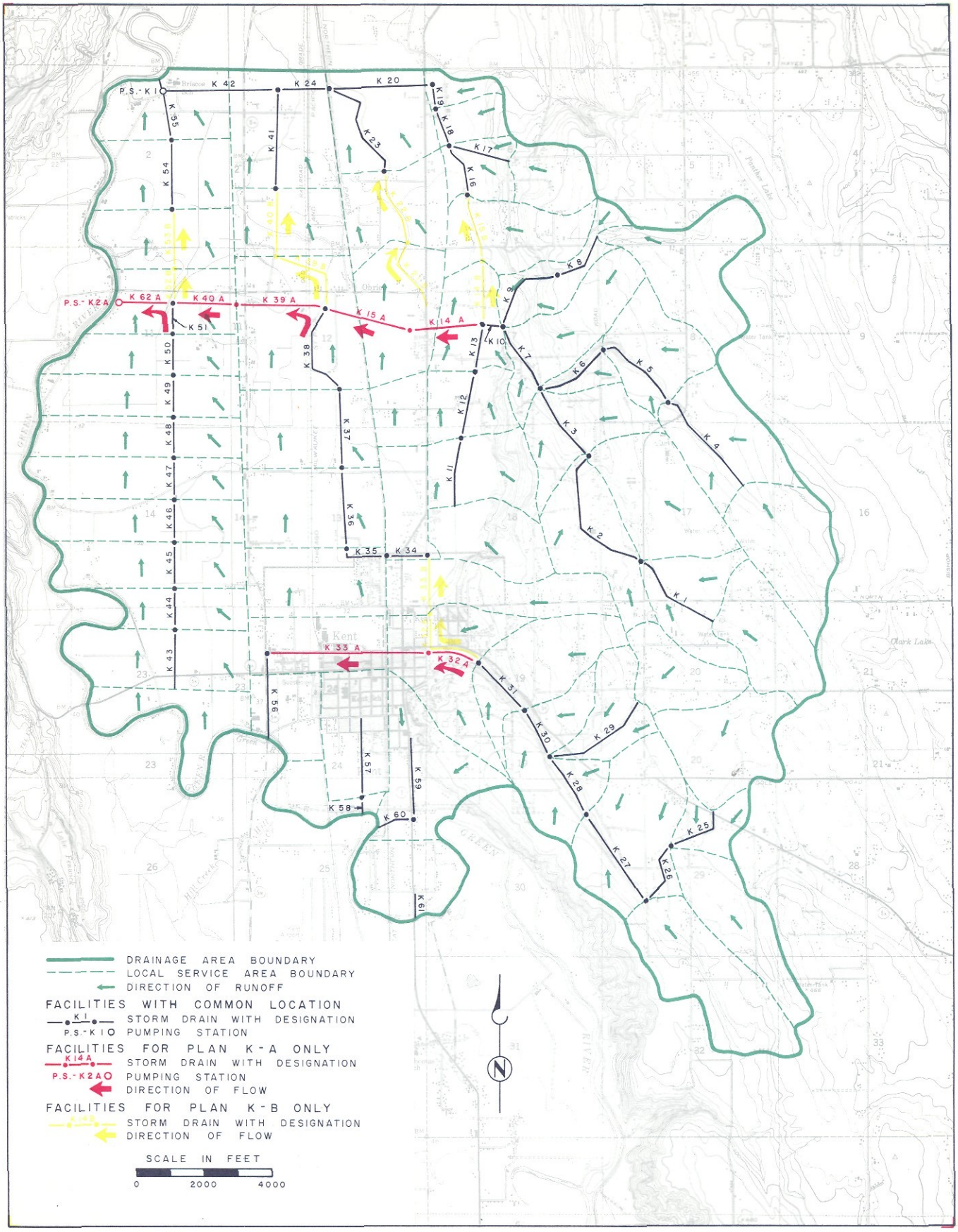


Fig. 17-4. Storm Drainage Plans K-A and K-B, Kent Area

Table 17-4. Description and Estimated Construction Costs, Drainage Plans K-A and K-B, Kent Area

Number	Design flow, cfs	Description	Construction cost, dollars
Plan K-A			
K-1	41	2,800 ft of 30-in. RC at 1.0%, dry, includes street crossings	52,000
K-2	83	4,200 ft of 33-in. RC at 3.7%, dry to wet, includes street crossings	80,000
K-3	168	2,500 ft of 39-in. RC at 5.6%, wet	64,000
K-4	55	3,400 ft of 30-in. RC at 2.1%, dry	58,000
K-5	95	2,500 ft of improved open channel at 0.72%, average cross-sectional area 13 sq ft, includes street crossings	6,000
K-6	123	2,300 ft of improved open channel at 0.72%, average cross-sectional area 16 sq ft	5,000
K-7	306	2,400 ft of improved open channel at 0.54%, average cross-sectional area 34 sq ft, includes street crossings	12,000
K-8	71	1,600 ft of 30-in. RC at 4.4%, dry to wet	27,000
K-9	96	2,300 ft of 30-in. RC at 8.5%, wet	44,000
K-10	402	500 ft of conc lined ditch at 0.4%, average cross-sectional area 42 sq ft	19,000
K-11	59	2,400 ft of 51-in. RC at 0.14%, difficult wet, includes street crossings	89,000
K-12	96	2,000 ft of 60-in. RC at 0.14%, difficult wet, includes street crossings	90,000
K-13	153	1,400 ft of 72-in. RC at 0.14%, difficult wet, includes street crossings	82,000
K-14A	550	2,300 ft of conc lined and fenced ditch at 0.07%, average cross-sectional area 102 sq ft, includes street crossings	166,000
K-15A	587	2,600 ft of conc lined and fenced ditch at 0.08%, average cross-sectional area 102 sq ft, includes street and railroad crossings	188,000
K-34	87	1,100 ft of 57-in. RC at 0.16%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	62,000
K-35	127	1,500 ft of 66-in. RC at 0.16%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	98,000
K-36	189	2,200 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 40 sq ft, includes street crossings	119,000
K-37	244	2,500 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 44 sq ft, includes street crossings	140,000
K-38	286	2,600 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 52 sq ft, includes street crossings	156,000
K-39A	760	2,700 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 150 sq ft, includes street crossings	280,000
K-40A	846	1,600 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 165 sq ft, includes street and railroad crossings	180,000
K-43	55	1,500 ft of 51-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	71,000
K-44	104	1,500 ft of 63-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	86,000
K-45	150	1,400 ft of 72-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	94,000
K-46	191	1,200 ft of 84-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	98,000
K-47	228	1,200 ft of 84-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	98,000
K-48	268	1,200 ft of 90-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	109,000

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Table 17-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
K-49	305	1,200 ft of 96-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	118,000
K-50	340	1,200 ft of 102-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	146,000
K-51	375	900 ft of 102-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	110,000
K-62A	1,160	1,600 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 230 sq ft	194,000
PS-K-2A	1,160	Storm water pumping station	755,000
K-16	47	1,400 ft of 45-in. RC at 0.15%, difficult wet, includes sheeting	98,000
K-17	42	2,000 ft of 24-in. RC at 6.8%, dry to difficult wet	33,000
K-18	98	1,100 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 36 sq ft, includes street crossing	52,000
K-19	116	800 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 43 sq ft, includes street crossing	40,000
K-20	120	3,300 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 43 sq ft, includes street and railroad crossings	156,000
K-23	32	3,200 ft of 45-in. RC at 0.07%, difficult wet	99,000
K-24	159	1,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 55 sq ft, includes street crossings	84,000
K-41	37	3,000 ft of 48-in. RC at 0.067%, difficult wet	100,000
K-42	216	3,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 60 sq ft, includes street crossings	224,000
K-54	70	1,500 ft of 60-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	84,000
K-55	127	1,500 ft of 78-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	110,000
PS-K-1	300	Storm water pumping station	225,000
K-25	40	2,000 ft of 27-in. RC at 2.0%, dry, includes street crossing	34,000
K-26	66	1,700 ft of 30-in. RC at 3.1%, dry	29,000
K-27	177	3,200 ft of 48-in. RC at 1.8%, wet, includes street crossing	123,000
K-28	201	1,800 ft of improved open channel at 0.46%, average cross-sectional area 23 sq ft	4,000
K-29	49	3,000 ft of 24-in. RC at 5.8%, dry to wet, includes street crossing	56,000
K-30	268	1,900 ft of improved open channel at 0.54%, average cross-sectional area 30 sq ft	5,000
K-31	285	2,000 ft of improved open channel at 0.52%, average cross-sectional area 32 sq ft	7,000
K-32A	324	2,800 ft of 78-in. RC at 0.54%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	220,000
K-33A	362	3,600 ft of 96-in. RC at 0.17%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	381,000
K-56	400	2,800 ft of 102-in. RC at 0.11%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	334,000
K-57	37	2,300 ft of 39-in. RC at 0.22%, difficult wet, includes repaving, sheeting and dewatering	79,000

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Table 17-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
K-58	82	800 ft of 42-in. RC at 0.63%, difficult wet, includes sheeting and dewatering	32,000
K-59	70	2,500 ft of 51-in. RC at 0.2%, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	120,000
K-60	164	900 ft of 60-in. RC at 0.45%, difficult wet, includes sheeting and dewatering	52,000
K-61	28	1,000 ft of 36-in. RC at 0.2%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	32,000
Total contract cost, Plan K-A			6,709,000
Engineering and contingencies, 25 per cent			1,677,000
Total construction cost, Plan K-A			8,386,000
Plan K-B			
K-1 – K-13	–	Same as K-1 – K-13, Plan K-A	628,000
K-14B	401	2,100 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft	126,000
K-15B	401	1,700 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossings	126,000
K-16	401	1,400 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossings	134,000
K-17	42	Same as K-17, Plan K-A	33,000
K-18	401	1,100 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossing	87,000
K-19	401	800 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossing	53,000
K-20	410	3,300 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 93 sq ft, includes street and railroad crossings	251,000
K-21B	15	2,900 ft of 30-in. RC at 0.14%, difficult wet, includes street crossing	62,000
K-22-B	35	2,000 ft of 45-in. RC at 0.10%, difficult wet	64,000
K-23	59	3,200 ft of 57-in. RC at 0.07%, difficult wet	127,000
K-24	470	1,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 104 sq ft, includes street crossing	143,000
K-25 – K-31	–	Same as K-25 – K-31, Plan K-A	258,000
K-32B	311	2,600 ft of 78-in. RC at 0.8%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	188,000
K-33-B	352	1,600 ft of 84-in. RC at 0.38%, difficult wet, includes sheeting and dewatering	114,000
K-34	383	1,100 ft of 96-in. RC at 0.18%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	114,000
K-35	408	1,500 ft of 102-in. RC at 0.16%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	177,000
K-36	414	2,200 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 74 sq ft, includes street crossings	163,000
K-37	434	2,500 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 74 sq ft, includes street crossings	180,000
K-38	460	2,600 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 94 sq ft, includes street crossings	215,000

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Table 17-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
K-39B	468	2,400 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 94 sq ft, includes street and railroad crossings	206,000
K-40B	483	1,900 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 120 sq ft, includes street crossings	188,000
K-41	510	3,000 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 130 sq ft, includes street crossing	261,000
K-42	972	3,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 177 sq ft, includes street crossings	370,000
K-43	55	1,500 ft of 54-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	75,000
K-44	102	1,500 ft of 72-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	102,000
K-45	147	1,400 ft of 78-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	103,000
K-46	187	1,200 ft of 90-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	109,000
K-47	224	1,200 ft of 90-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	109,000
K-48	262	1,200 ft of 96-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	118,000
K-49	299	1,200 ft of 102-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	134,000
K-50	333	1,200 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	150,000
K-51	366	900 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	113,000
K-52B	375	1,000 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	125,000
K-53B	384	1,800 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	225,000
K-54	412	2,000 ft of 114-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	272,000
K-55	436	1,400 ft of 114-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	190,000
PS-K-1	1,300	Storm water pumping station	845,000
K-56	58	2,800 ft of 51-in. RC at 0.15%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	122,000
K-57 – K-61	–	Same as K-57 – K-61, Plan K-A	315,000
Total contract cost, Plan K-B			7,375,000
Engineering and contingencies, 25 per cent			1,844,000
Total construction cost, Plan K-B			9,219,000

See Fig. 17-4 for location of facilities.

sible with respect to the routing of storm drainage facilities. Preliminary layouts and cost estimates were prepared for two such alternatives. Designated as Plan K-A, the first provides for three major points of discharge to Green River, one at the southern end of the area, the second at the center, and the third at the north end. Under the second alternative, designated as Plan K-B, the greater portion of the drainage would be conveyed to a point in the northern end of the area and discharged to the river. Both plans provide for pumping where necessary and assume a maximum water surface elevation of

20 feet above mean sea level at the pumping station.

Gravity drains serving local drainage areas are also provided under both plans. Design of these drains is based on a controlled water surface elevation in the river of 35 feet above mean sea level.

Locations of all major storm drainage facilities for the Kent area are shown in Fig. 17-4. Descriptions and estimated construction costs are given in Table 17-4. For Plan K-A, the estimated cost is $8,386,000, while that for Plan K-B amounts to $9,219,000. Based on a total tributary area of 11,480 acres, the cost of providing major storm drainage facilities in areas similar to Kent can be expected to vary between $730 and $800 per acre.

SUMMARY OF DRAINAGE COSTS

While it is impossible at this time to estimate accurately the cost of providing major storm drainage facilities throughout the entire metropolitan Seattle area, an approximation based on the unit costs just developed may be of value for planning purposes. To determine the approximate total cost, areas for which drainage facilities are to be provided were divided into five general categories, as follows:

A. An area with favorable surface slopes in which distances to the point of disposal are short and improved earth channels can be used to the maximum extent. Unit cost, $275 per acre.

B. An area with favorable surface slopes in which holding basins and improved earth channels can be utilized. Unit cost, $250 per acre.

C. An area with favorable surface slopes, but which requires a large proportion of enclosed conduits. Unit cost, $450 per acre.

D. An area in which distances to the point of disposal are relatively long. Unit cost, $550 per acre.

E. An area with difficult drainage problems and where pumping may be required. Unit cost, $750 per acre.

In developing the approximate ultimate cost of major storm drainage facilities for the entire metropolitan area, a portion of the city of Seattle was excluded. This portion is presently served by combined sewers. With that exclusion, the area in which storm drainage facilities will ultimately be required amounts to a total of approximately 330,000 acres.

Table 17-5. Approximate Storm Drainage Costs for Ultimate Development in the Metropolitan Seattle Area

Category	Area, acres	Cost per acre,[a] dollars	Total cost,[a] dollars
A	118,000	275	32,450,000
B	33,000	250	8,250,000
C	59,000	450	26,550,000
D	63,000	550	34,650,000
E	57,000	750	42,750,000
Total	330,000	–	144,650,000

[a]*Includes engineering and contingencies.*

Based upon a study of photographic maps and on information obtained by field inspection, the total area was divided into the five drainage categories outlined above. Estimated construction costs were then developed accordingly and are listed for each category in Table 17-5. As thus determined, the approximate cost of providing major drainage facilities in the metropolitan area for conditions of ultimate or saturation development amounts to a total of $145 million.

It is apparent, of course, that construction of all facilities required at ultimate development need not be undertaken immediately. On the other hand, construction should be undertaken as soon as possible in presently developed areas lacking storm drainage facilities and in other areas where development is limited due to inadequate drainage. This program will involve construction within the next ten years of approximately one-fourth to one-third of the system required for ultimate development. Thereafter, construction can be scheduled as the need for additional facilities develops.

Chapter 18
SEPARATION OF COMBINED SEWERS IN SEATTLE

In common with many other cities throughout the country, the city of Seattle is faced with a diversity of problems brought about by the use of combined sewers for the conveyance of sewage and storm water. These problems are manifested in many ways. For example, heavy rains lead to gross overloading of sewers and thus to sewage overflows in streets, gutters, and basements. Furthermore, contamination of waterways occurs during even moderate rains and is brought about by sewage discharges from overflow structures in combined systems. Correction of these conditions can be achieved only by separation of sanitary sewage and storm water in both trunk and local collection systems. The degree to which separation is required depends, of course, not only on the severity of these problems in local areas but on the extent to which they are brought about by sewage and storm water inputs from upstream locations.

EXTENT OF SEPARATION REQUIREMENTS

As set forth in Chapter 15, the most economic means of preventing bacterial contamination in Lake Washington by storm water overflows is that of constructing holding tanks at overflow locations. These tanks would be designed to store excess water during rainfall periods and to discharge the stored water back to intercepting sewers after the rain has stopped falling.

Beneficial uses of other major bodies of water, namely, Lake Washington Ship Canal, Elliott Bay and Duwamish River, are largely commercial in nature. As such they are not affected adversely by storm water overflows from the combined system. On that basis, and with the provision of holding tanks to protect Lake Washington, the separation program in Seattle need be undertaken only to the extent necessary to relieve overloaded trunk and local sewers.

Design Criteria

Design flows for the facilities required to obtain storm water separation were established on the basis of the criteria presented in Chapter 13. Storm water runoff was calculated by means of the rational formula, using a storm recurrence interval of 10 years. Runoff coefficients and times of concentration were based on ultimate development of each of the selected areas. A proportionate runoff coefficient of two-thirds of the total was used for street and yard drainage, while that used for roof drainage was one-third of the total. These values are based on a detailed analysis by the Seattle engineering department of approximately 20 acres in the Laurelhurst area. Maximum rates of sanitary sewage flow were determined by multiplying the average flow by a factor related to the tributary population and by adding an appropriate allowance for infiltration.

Analysis of Existing System

To determine the extent to which separation is required, it is necessary to analyze the capacity of the combined system. Information thus obtained determines whether the system is adequate (1) to perform its intended function, (2) to carry a part of the storm water flow, as well as all the sanitary sewage flow, (3) to carry only the sanitary sewage flow, or (4) to carry only the storm water runoff.

Previous Studies. In 1951, the Seattle engineering department conducted an overload study of the existing system to determine which sewers were incapable of carrying both storm water and sanitary sewage. This study assumed a uniform storm water runoff rate of 15 cfm per acre, which is equivalent on the average to a storm having a recurrence frequency of once every two years. Even on that basis, which represents only a moderate storm, a substantial number of the combined sewers were found to be deficient in storm flow capacity. Further, this condition was not confined to any particular area or locality but was prevalent throughout the city as a whole.

Present Studies. Detailed studies of separation requirements were made in six areas. In general, it was found that none of the existing systems in their entirety and that very few sewers within these systems have a capacity sufficient for the flow resulting from a 10-year storm. Without exception, all of the systems are capable of carrying the peak flow of sanitary sewage. In most instances, the addition of some relief sewers will enable them also to carry a portion of the storm flow.

Type of Separation

Two complete analyses were made and preliminary layouts and cost estimates were prepared accordingly for each of the six areas. The first analysis assumed partial separation whereby (1) a new storm drainage system to accommodate runoff from streets and yards would be constructed, and (2) runoff from roofs and foundation drains, together with sanitary sewage, would continue to go to the existing system. The second analysis assumed complete separation whereby (1) a new storm drainage system of sufficient capacity to accommodate all storm water runoff, including that from roofs and foundation drains, would be constructed, and (2) the existing system would carry sanitary sewage only. In one area, a study was made also of the possibility of providing a new drainage system to carry runoff from streets and roofs and of using the existing system for sanitary sewage and drainage from foundation drains.

Routes of Storm Drains

In general, the new drainage systems were laid out to follow the routes of the existing combined sewers. For this purpose, use was made of maps prepared by the city engineer's office during the 1951 overload studies.

Since it is possible to discharge storm water to the nearest available body of water, economy in construction could be achieved in some instances by rerouting drains on the basis of using minimum distances to points of disposal. This would make it possible both to shorten the lengths of pipe and to use pipe of smaller diameter. With these advantages in mind, a possible rerouting of storm drains was investigated in one of the six areas.

In the layout of local storm drains for partial separation, it was assumed that, to intercept street inlets, such drains would be required to the next to last intersection along each run. If street gutters are utilized to the fullest possible extent, the lengths of local drains required can, of course, be reduced considerably. In the case of complete separation, it was assumed that local storm drains would be required to the last house on each run to intercept roof leaders and foundation drains.

BASIS OF COST ESTIMATES

Estimates of costs for sewers and storm drains required for the two types of separation are based on the data presented in Chapter 13. As a part of these projects, it will be necessary also to reconnect house sewers and storm water catch basins, to change connections at manholes and, in the case of complete separation, to install new house connections for either storm water drains or sanitary sewers. Unit costs for these purposes are as follows:

House sewer reconnections	$ 40.00
Catch basin reconnections	$ 80.00
Manhole connections	$100.00
New house sewers	$300.00

Drains designed to pick up street drainage only (partial separation) were assumed to be laid at minimum depth, allowing about three feet of cover. Drains designed to pick up all storm runoff, including that from roof leaders and foundation drains (complete separation), were assumed to be laid at a depth of 12 feet, which is about the average required to intercept foundation drains. In one area, a system was laid out which would provide essentially complete separation in that runoff from streets and roofs but not from foundation drains would discharge to the storm drains. With the foundation drains eliminated, the storm drains were assumed to be at minimum depth. Sanitary sewers required for relief of the existing system were assumed to be a minimum of 12 feet deep.

Wherever deemed possible, storm drains were assumed to be laid in parking strips adjacent to the streets. Otherwise, the cost estimates include allowances for repaving. These vary from $1.50 per lineal foot for 8-inch pipe to $4.80 per lineal foot for 60-inch pipe. In paved areas where settlement would be objectionable and where excavated material was considered to be unsuitable for backfill, the costs allow for use of imported granular backfill material.

Construction conditions were assumed to range from dry to moderately wet. In sections where conditions are known to be unfavorable, the costs allow for such items as water control and use of sheet piling.

SEPARATION REQUIREMENTS IN SELECTED AREAS

Development of an actual program of storm water separation in the city of Seattle will require a detailed analysis of all component parts of the existing system. In a preliminary survey of the nature here reported, it is not necessary, nor is it possible, due to lack of topographic and other essential information, to make such an analysis. It is necessary, however, to analyze a sufficient number of typical areas to demonstrate the methods to be employed and to provide a basis for preliminary planning and cost estimating. With this information, it is possible also to reach certain conclusions as to the best procedure to follow in undertaking the separation program.

Selection of Areas for Study

As a first step in the separation study, a number of areas throughout the city were selected as having typical separation problems. These areas were selected

in consultation with the engineering department of the city and are among those in which serious overloading of combined sewers is presently occurring. As a second step, six of these areas were selected for detailed study and analysis.

Sewer systems in each of the six areas are shown in detail in maps prepared for the overload studies made in 1951. These maps give sewer sizes and, in most cases, sewer lengths and invert elevations. Additional information was obtained from individual sewer cards on file in the city engineer's office. District numbers used herein in discussing the separation problem coincide with the drawing numbers of the maps prepared for the 1951 study.

District 6 - Briarcliff

Situated on the west side of Magnolia immediately south of Fort Lawton, the Briarcliff district encompasses an area of 150 acres and has a predicted ultimate population of 1,600. Drainage is to the west to Puget Sound and ground slopes are generally steep. This district lies in a major slide area, with slides occurring practically every winter. Subsurface drains have been installed to control and prevent slides and are generally connected to the combined system.

Existing trunk sewers consist of one along Perkins Lane and one along West Ray Street, both of which discharge through an outfall to Puget Sound at the foot of West Ray Street. Under the recommended sewerage plan (Chapter 15), a local intercepting sewer will be required to convey the sewage from the present point of discharge to the central sewerage system at Thirty-Second Avenue West. Lateral sewers range in size from 8-inch to 15-inch.

With a few minor exceptions, principally at the extremity of each lateral, the system does not have a capacity sufficient for combined storm water and sanitary sewage flows. It does, however, have ample capacity for peak sanitary flow. With the addition of a relatively small number of relief sewers,

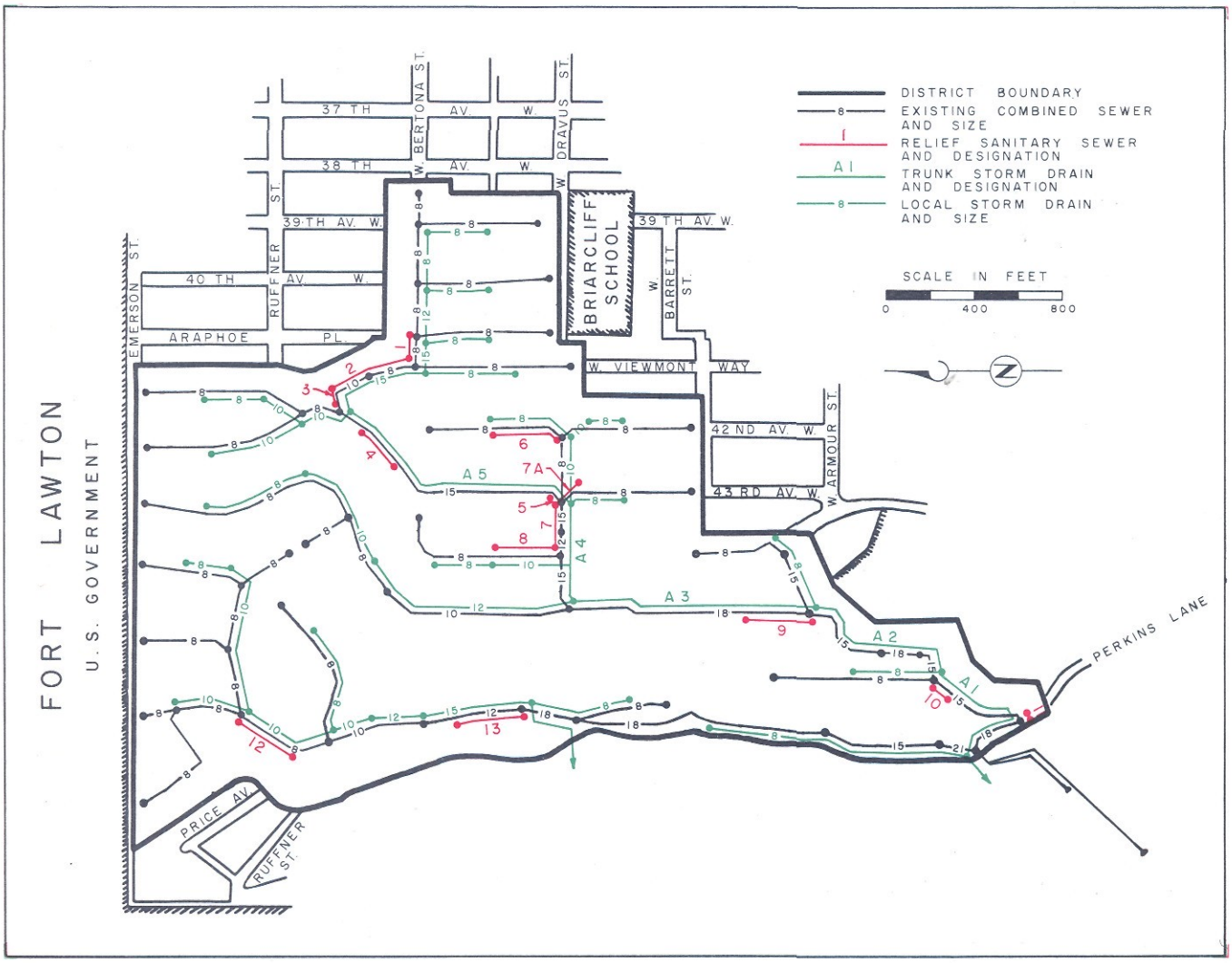

Fig. 18-1. Layout of Facilities for Partial Separation, District 6 - Briarcliff

Table 18-1. Description and Estimated Construction Costs for Separation of District 6 – Briarcliff

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	0.9, 2.7[b]	140 ft of 8-in. conc at 2.1%	1,600
2	1.2, 4.1[b]	410 ft of 8-in. conc at 3.3%	4,700
3	3.2, 4.1[b]	80 ft of 15-in. RC at 0.5%	1,400
4	3.2, 6.0[b]	210 ft of 18-in. RC at 0.2%	4,200
5	2.8, 6.2[b]	70 ft of 15-in. RC at 0.3%	1,300
6	0.6, 1.3[b]	380 ft of 8-in. conc at 0.6%	4,400
7	8.2, 8.2[b]	300 ft of 15-in. RC at 1.5%, required to permit necessary upstream modifications	5,600
7A	0.8	120 ft of 8-in. conc, required to prevent excessive surcharging	1,300
8	0.3, 1.2[b]	310 ft of 8-in. conc at 0.5%	3,600
9	2.0, 13[b]	300 ft of 10-in. conc at 1.1%	3,700
10	5.5, 14[b]	70 ft of 12-in. RC at 2.0%	1,200
11	3.4, 14[b]	30 ft of 10-in. conc at 3.0%	400
12	1.0, 4.5[b]	320 ft of 8-in. conc at 8.4%	3,700
13	1.6, 6.1[b]	260 ft of 8-in. conc at 1.6%	2,900
–	–	Reconnect 30 house connections	1,200
–	–	Reconnect 20 manholes	2,000
Subtotal, relief sanitary sewers			43,200
Trunk storm drains			
A-1	29	640 ft of 24-in. RC at 1.0% minimum slope	10,700
A-2	28	790 ft of 24-in. RC at 2.3% minimum slope	10,600
A-3	25	1,130 ft of 24-in. RC at 1.1% minimum slope	16,300
A-4	18	550 ft of 24-in. RC at 1.8% minimum slope	7,900
A-5	12	1,150 ft of 21-in. RC at 0.6% minimum slope	14,600
Subtotal, trunk storm drains			60,100
Local storm drains			
–	–	7,430 ft of 8-in.	34,400
–	–	4,460 ft of 10-in.	24,600
–	–	2,000 ft of 12-in.	16,000
–	–	600 ft of 15-in.	5,800
–	–	50 intersection crossings, includes catch basin reconnections	18,800
Subtotal, local storm drains			99,600
Total contract cost, partial separation			202,900
Engineering and contingencies, 25 per cent			50,700
Total construction cost, partial separation			253,600
Complete separation[c]			
Trunk storm drains			
A-1	42	640 ft of 30-in. RC at 1.0% minimum slope	13,900

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Table 18-1. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
A-2	41	790 ft of 30-in. RC at 2.3% minimum slope	14,800
A-3	37	1,130 ft of 30-in. RC at 1.1% minimum slope	23,400
A-4	24	550 ft of 30-in. RC at 1.8% minimum slope	11,400
A-5	18	1,150 ft of 30-in. RC at 0.6% minimum slope	23,900
Subtotal, trunk storm drains			87,400
Local storm drains			
–	–	7,430 ft of 8-in.	51,500
–	–	3,460 ft of 10-in.	27,400
–	–	3,620 ft of 12-in.	38,600
–	–	1,800 ft of 15-in.	22,300
–	–	460 ft of 18-in.	6,600
–	–	430 new house connections	129,000
–	–	50 intersection crossings, includes catch basin reconnections	18,800
Subtotal, local storm drains			294,200
Total contract cost, complete separation			381,600
Engineering and contingencies, 25 per cent			95,400
Total construction cost, complete separation			477,000

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-1 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. See Fig. 18-1 for location of trunk storm drains.*

this system could carry the peak sanitary sewage flow plus the storm flow contributed by roof leaders and foundation drains.

Present facilities in the Briarcliff district are shown in Fig. 18-1, together with the relief sewers and storm drains required to effect partial separation. Descriptions and estimated construction costs are given in Table 18-1. The total cost of partial separation amounts to $253,600, or about $1,690 per acre. Of this total, approximately 21 per cent is for relief sanitary sewers, 30 per cent for trunk storm drains, and 49 per cent for local storm drains.

No layout is shown of the facilities required for complete separation. For that purpose, storm drain locations would be substantially the same as those in Fig. 18-1, but local drains would be extended to the end of each existing sewer. No relief sewers would be required because the present system has adequate capacity for the peak flow of sanitary sewage. For complete separation, the estimated cost is $477,000 (Table 18-1), of which about 23 per cent is for trunk storm drains and 77 per cent is for local storm drains, including new house connections.

District 17 - Wedgewood

Occupying a total area of 480 acres, the Wedgewood district lies west of the Sand Point Naval Air Station and north of the Laurelhurst district. Its ultimate population is expected to reach 9,000. Surface slopes are moderate to steep and drain generally to the south.

At present, sewage from this area is discharged through a 36-inch trunk to the North Trunk. Under storm flow conditions, excess flow is diverted by an overflow structure at East 55th Street and 40th Avenue Northeast and is discharged through an outfall line to Union Bay at the foot of 38th Avenue Northeast. It is reported that overflows occur during even relatively light rainfall.

In general, the existing system lacks sufficient capacity to accommodate the flows resulting from a 10-year storm. With a minor amount of relief sewer construction, however, it would have capacity for the peak sanitary flow plus storm water runoff from roof leaders.

District 17 includes the site of the old Cedar Vale

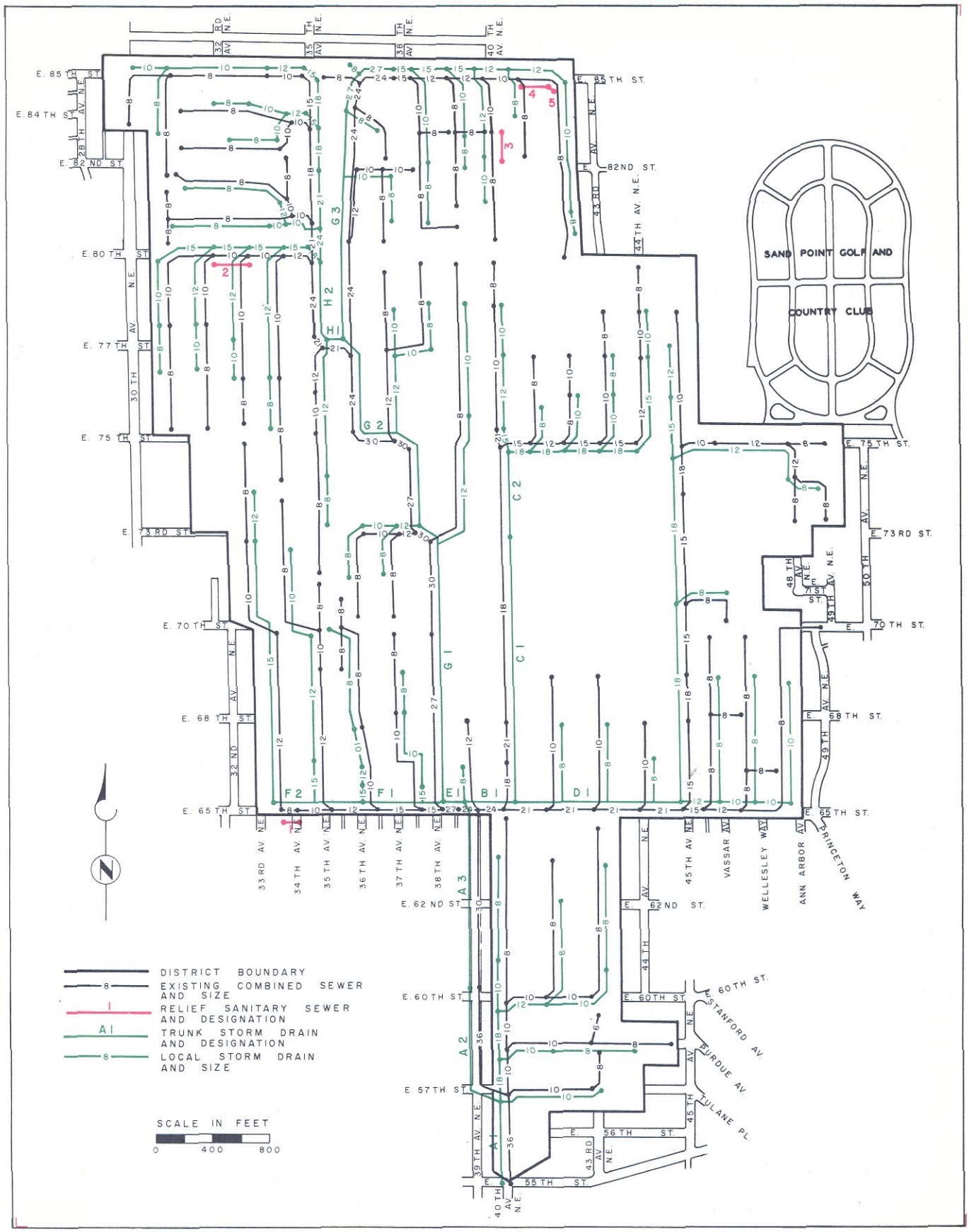

Fig. 18-2. Layout of Facilities for Partial Separation, District 17 - Wedgewood

Table 18-2. Description and Estimated Construction Costs for Separation of District 17 – Wedgewood

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	0.7, 3.6[b]	120 ft of 8-in. conc at 5.8%	800
2	1.4, 4.6[b]	250 ft of 8-in. conc at 2.2%	1,800
3	0.6, 1.8[b]	210 ft of 8-in. conc at 1.0%	1,400
4	0.4, 1.3[b]	240 ft of 8-in. conc at 1.0%	1,500
5	0.2, 1.3[b]	60 ft of 8-in. conc at 2.8%	400
–	–	Reconnect 8 house connections	300
–	–	Reconnect 10 manholes	1,000
Subtotal, relief sanitary sewers			7,200
Trunk storm drains			
A-1	100	660 ft of 39-in. RC at 2.3%	21,000
A-2	98	880 ft of 39-in. RC at 1.9 – 2.8%	28,000
A-3	98	1,220 ft of 33-in. RC at 3.3 – 7.3%	34,300
B-1	49	200 ft of 24-in. RC at 4.9 – 5.4%	3,100
C-1	23 – 26	1,960 ft of 21-in. RC at 3.0 – 5.2%	25,000
C-2	10 – 12	560 ft of 18-in. RC at 0.7 – 5.0%	6,200
D-1	21 – 27	1,300 ft of 21-in. RC at 1.3 – 3.9%	20,000
E-1	55	250 ft of 30-in. RC at 5.0 – 7.7%	5,200
F-1	15 – 18	460 ft of 15-in. RC at 7.7 – 12.1%	5,500
F-2	7 – 12	680 ft of 15-in. RC at 5.8 – 8.8%	8,000
G-1	41 – 44	1,910 ft of 30-in. RC at 1.1 – 4.5%	48,100
G-2	35 – 40	1,550 ft of 30-in. RC at 0.73 – 4.0%	32,400
G-3	13 – 14	1,780 ft of 30-in. RC at 0.1 – 0.4%	31,900
H-1	25	220 ft of 24-in. RC at 3.5%	4,000
H-2	22	640 ft of 24-in. RC at 0.4%	10,800
Subtotal, trunk storm drains			283,500
Local storm drains			
–	–	14,200 ft of 8-in.	69,800
–	–	14,050 ft of 10-in.	83,700
–	–	10,660 ft of 12-in.	89,600
–	–	5,010 ft of 15-in.	49,200
–	–	4,980 ft of 18-in.	56,100
–	–	360 ft of 21-in.	5,200
–	–	280 ft of 24-in.	4,500
–	–	480 ft of 27-in.	8,600
–	–	149 intersection crossings, includes catch basin reconnections	41,000
Subtotal, local storm drains			407,700
Total contract cost, partial separation			698,400
Engineering and contingencies, 25 per cent			174,600
Total construction cost, partial separation			873,000

Continued on next page

Table 18-2. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Complete separation[c]			
Trunk storm drains			
A-1, A-2	155	1,540 ft of 45-in. RC at 1.9 – 2.8%	84,700
A-3	150	1,220 ft of 39-in. RC at 3.3 – 7.3%	60,500
B-1	75	200 ft of 33-in. RC at 4.9 – 5.4%	6,300
C-1	33 – 37	1,960 ft of 24-in. RC at 4.9 – 5.4%	33,800
C-2	16 – 19	560 ft of 21-in. RC at 0.7 – 5.0%	8,700
D-1	42 – 51	1,300 ft of 30-in. RC at 1.3 – 3.9%	36,700
E-1	86	250 ft of 36-in. RC at 5.0 – 7.7%	8,900
F-1, F-2	12 – 25	1,140 ft of 18-in. RC at 5.8 – 12.1%	21,400
G-1, G-2	54 – 67	3,280 ft of 36-in. RC at 1.1 – 4.5%	125,900
G-2, G-3	19 – 54	1,960 ft of 33-in. RC at 0.1 – 2.9%	49,400
H-1, H-2	34 – 40	860 ft of 33-in. RC at 0.4 – 3.5%	30,200
Subtotal, trunk storm drains			466,500
Local storm drains			
–	–	19,100 ft of 8-in.	109,100
–	–	13,000 ft of 10-in.	89,000
–	–	14,400 ft of 12-in.	136,600
–	–	11,300 ft of 15-in.	130,700
–	–	4,600 ft of 18-in.	64,000
–	–	2,600 ft of 21-in.	37,600
–	–	400 ft of 24-in.	6,900
–	–	300 ft of 27-in.	5,800
–	–	440 ft of 33-in.	9,600
–	–	2,200 new house connections	660,000
–	–	149 intersection crossings, includes catch basin reconnections	41,000
Subtotal, local storm drains			1,290,300
Total contract cost, complete separation			1,756,800
Engineering and contingencies, 25 per cent			439,200
Total construction cost, complete separation			2,196,000

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-2 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. See Fig. 18-2 for location of trunk storm drains.*

housing project east of 40th Avenue Northeast and south of East 75th Street. This site of 45 acres, which formerly was utilized for housing service personnel, is now being subdivided and is more than 50 per cent occupied by new homes. Since detailed layouts of the new sewers being constructed in this area were not available, the entire 45 acres were taken as tributary to the same point in the sewerage system as the old housing project (Fig. 18-2).

Locations of facilities required for partial separation of the Wedgewood district are shown in Fig. 18-2, as are the location and sizes of existing sewers. Descriptions and estimated construction costs are given in Table 18-2. As there listed, the total cost of partial separation amounts to $873,000, or about $1,820 per acre. Of this total, approximately 1 per cent is for relief sanitary sewers, 41 per cent for trunk storm drains, and 58 per cent for local storm drains.

Also given in Table 18-2 are descriptions and estimated construction costs of facilities required for complete separation in the Wedgewood district. For this, the total cost amounts to $2,196,000, or about $4,570 per acre, of which approximately 27 per cent is for trunk storm drains and 73 per cent is for local storm drains, including new house connections. Location of facilities required for complete separation are generally as shown in Fig. 18-2.

District 23 - Southwest Seattle

Lying along the shores of Puget Sound northeast of Lowman Beach Park, the Southwest Seattle district contains 960 acres and is expected to have an ultimate population of 14,500. Drainage in the district is generally to the south and west to Puget Sound through a number of ravines, the most prominent of which runs through the district in a northeasterly direction from Lowman Beach Park.

In this district, the existing combined system consists of one principal trunk in the ravine from Lowman Beach Park and of local sewers connected to that trunk. At present, raw sewage from the area is discharged without treatment to Puget Sound at the foot of Murray Avenue. A pumping station is now under construction, however, which will lift this flow into the waterfront interceptor for conveyance to the Alki Point treatment plant.

Although the principal trunk has sufficient capacity in most sections to accommodate the flow resulting from a 10-year storm, this capacity is not available generally in the local sewers connected to it. Ample capacity is available, however, for the peak sanitary flow. By the addition of some relief sewers, the system would be able also to accommodate the storm flow contributed by roof leaders and foundation drains.

Locations of relief sanitary sewers and of the storm drain additions required to effect partial separation are shown in Fig. 18-3. This figure also shows the locations of the facilities presently in use. Descriptions and estimated costs of the additions required for partial separation are given in Table 18-3.

Under the proposed layout, the principal trunk in the present system would be utilized as a storm drain to the fullest possible extent. There are two reasons for this. First, the trunk is capable of accommodating storm water runoff from streets and yards and, second, the size of the sewer required for the sanitary flow plus the storm flow from roofs is smaller than that required for street drainage.

Estimated construction costs for partial separation in the Southwest Seattle district amount to $1,735,000, or about $1,810 per acre. Of this total, approximately 12 per cent is for relief sanitary sewers, 19 per cent for trunk storm drains, and 69 per cent for local storm drains.

Locations of facilities required for complete separation are substantially the same as those shown in Fig. 18-3. Descriptions and estimated construction costs of these facilities are also given in Table 18-3. Again, the existing principal trunk would be utilized to the fullest possible extent as a storm drain. In this case, construction of relief storm drains would be required in certain sections and some surcharging could be expected of the existing line along Fairmount Avenue. It is estimated that about $85,000 would be saved by using the existing sewer as a storm drain and constructing a new sanitary sewer.

Estimated construction costs for complete separation in the Southwest Seattle district amount to $3,495,800, or about $3,640 per acre (Table 18-3). Of the total cost, approximately 3 per cent is for sanitary sewers, 10 per cent for trunk storm drains, and 87 per cent for local storm drains, including new house connections. In both the partial and complete separation projects, the relatively low percentages of the total cost of trunk drains results from the use of existing sewers for that purpose.

District 29 - South Magnolia

Situated on Puget Sound at the south end of Magnolia, the South Magnolia district occupies an area of 530 acres and has a predicted ultimate population of 5,700. Drainage is variable, with the western portion draining directly to Puget Sound and the remainder draining to the east and west to a flat area culminating in a ravine along 32nd Avenue West. Surface slopes, except in the flat area along 32nd Avenue West, are steep.

The existing combined system is tributary to a principal trunk which is laid along 32nd Avenue West and discharges raw sewage to the sound at the foot of that avenue. Under the sewerage plan recommended in Chapter 15, this discharge will be intercepted and the sewage conveyed to the West Point treatment plant. With a few minor exceptions, the existing system lacks capacity for the combined storm and sanitary flows. It does, however, have capacity for the peak sanitary flow.

Studies were made of two plans for storm water separation. The first, designated as Plan I, provides for the conveyance of storm water to one point of disposal, while the second, Plan II, provides for conveyance to three points of disposal. Under Plan I, storm drain routes would follow the routes of existing sewers. Under Plan II, the routes would be such that storm water would be conveyed the shortest possible distance to the points of disposal.

Under Plan II, a study was made also of an alternative method of separation which would provide for the collection of all runoff from streets and roofs but not for the interception of foundation drains. The latter

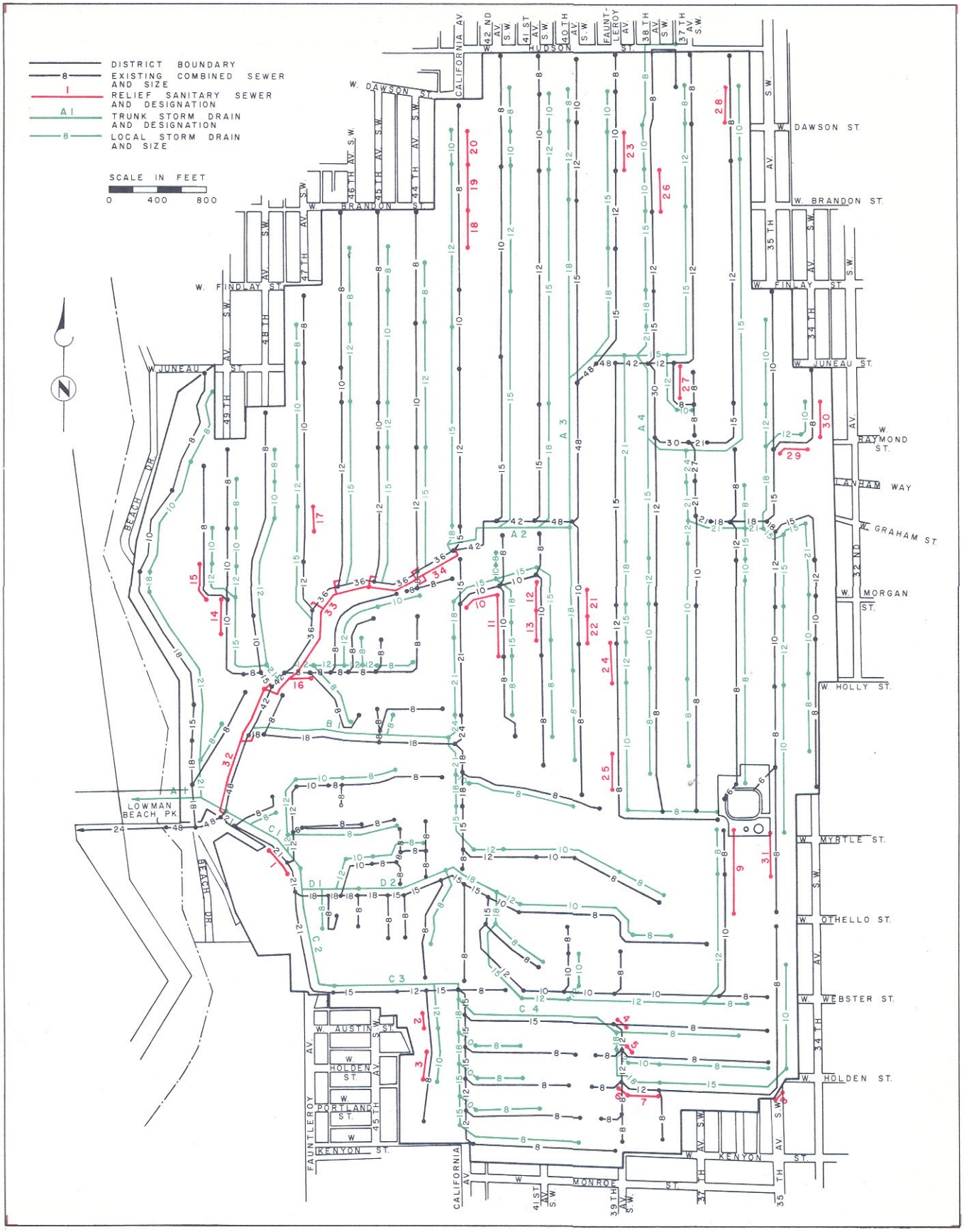

Fig. 18-3. Layout of Facilities for Partial Separation, District 23 - Southwest Seattle

Table 18-3. Description and Estimated Construction Costs for Separation of District 23 – Southwest Seattle

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	15, 30[b]	300 ft of 21-in. RC at 1.0%	6,600
2	0.7, 2.8[b]	100 ft of 8-in. conc at 1.5%	800
3	0.5, 1.7[b]	230 ft of 8-in. conc at 1.0%	1,600
4	1.5, 6.5[b]	80 ft of 8-in. conc at 2.2%	500
5	0.5, 1.7[b]	70 ft of 8-in. conc at 1.0%	500
6	3.9	80 ft of 15-in. RC at 0.4% to replace existing 12-in.	1,100
7	1.5, 3.9[b]	260 ft of 10-in. conc at 0.5%	2,200
8	2.0	90 ft of 15-in. RC at 0.2% to replace existing 8-in.	1,600
9	0.3, 1.1 – 1.8[b]	690 ft of 8-in. conc at 0.4 – 0.6%	4,800
10	1.2, 5.2[b]	280 ft of 8-in. conc at 3.1%	2,600
11	0.4, 1.7[b]	610 ft of 8-in. conc at 0.3 – 0.9%	4,200
12	2.8, 4.1[b]	240 ft of 15-in. RC at 0.3%	3,300
13	0.9, 2.1[b]	240 ft of 10-in. conc at 0.3%	2,000
14	1.0, 3.1[b]	290 ft of 8-in. conc at 0.9%	2,000
15	0.5, 1.5[b]	340 ft of 8-in. conc at 0.7 – 1.0%	2,400
16	2.0, 6.2[b]	220 ft of 8-in. conc at 12.9%	1,200
17	1.4, 3.5[b]	370 ft of 10-in. conc at 0.9%	3,100
18	0.7 – 1.0, 2.7 – 3.0[b]	660 ft of 8-in. conc at 2.4 – 2.8%	7,200
19	1.3, 2.1[b]	380 ft of 10-in. conc at 0.4%	4,900
20	0.5, 1.4[b]	280 ft of 8-in. conc at 0.6%	3,000
21	0.8, 2.9[b]	240 ft of 8-in. conc at 0.9%	1,700
22	1.3, 2.7[b]	240 ft of 10-in. conc at 0.4%	2,000
23	0.3, 1.7[b]	300 ft of 8-in. conc at 0.4%	3,200
24	0.5, 4.3[b]	330 ft of 8-in. conc at 9.7%	2,300
25	0.4, 3.1[b]	290 ft of 8-in. conc at 4.8%	2,000
26	1.0, 2.1[b]	320 ft of 8-in. conc at 0.9%	1,800
27	0.2, 1.3[b]	340 ft of 8-in. conc at 0.8%	2,400
28	0.4, 1.0[b]	280 ft of 8-in. conc at 0.4%	1,900
29	0.4 – 0.8,[b] 1.6	300 ft of 8-in. conc at 0.4 – 0.9%	2,000
30	0.2, 1.0[b]	290 ft of 8-in. conc at 0.4%	2,000
31	0.9, 1.6[b]	360 ft of 10-in. conc at 0.4%	4,600
32	58 – 66	1,100 ft of 30-in. RC at 3.2 – 6.0% to replace existing 42 – 48-in. which is to be used as storm drain	26,000
33	50 – 55	1,200 ft of 27-in. RC at 3.0 – 15% to replace existing 36 – 42-in. which is to be used as storm drain	29,700
34	46 – 47	640 ft of 24-in. RC at 5.6 – 33% to replace existing 36-in. which is to be used as storm drain	13,300
–	–	Reconnect 80 house connections	3,200

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Table 18-3. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
–	–	Reconnect 64 manholes	6,400
–	–	Reconnect 9 lateral sewers	1,800
Subtotal, relief sanitary sewers			161,900
Trunk storm drains			
A-1	220 – 225	600 ft of 54-in. RC at 1.8%	27,600
A-2	86 – 100	1,140 ft of 30-in. RC at 4.5 – 7.0%	21,300
A-3	80 – 83	1,560 ft of 48-in. RC at 0.3 – 0.6%	55,200
A-4	44 – 66	1,200 ft of 33-in. RC at 0.7 – 3.9%	27,300
B-1	32 – 36	1,680 ft of 21-in. RC at 5.6 – 10.5%	22,800
C-1	60	860 ft of 36-in. RC at 1.0 – 20%	21,800
C-2	32	1,170 ft of 27-in. RC at 1.2 – 18%	20,100
C-3	27 – 32	1,280 ft of 18-in. RC at 7.5 – 17%	14,100
C-4	15 – 17	1,260 ft of 18-in. RC at 2.6 – 5.1%	15,300
D-1	25 – 30	520 ft of 24-in. RC at 1.6 – 5.0%	8,300
D-2	23 – 25	710 ft of 18-in. RC at 6.2 – 15%	8,700
–	–	Connections to existing trunk to be utilized as storm drain	2,000
Subtotal, trunk storm drains			244,500
Local storm drains			
–	–	25,200 ft of 8-in.	125,800
–	–	17,600 ft of 10-in.	103,900
–	–	17,700 ft of 12-in.	148,200
–	–	24,200 ft of 15-in.	238,500
–	–	9,800 ft of 18-in.	113,500
–	–	5,700 ft of 21-in.	81,200
–	–	600 ft of 24-in.	9,600
–	–	227 intersection crossings, includes catch basin reconnections	80,900
Subtotal, local storm drains			901,600
Total contract cost, partial separation			1,308,000
Engineering and contingencies, 25 per cent			427,000
Total construction cost, partial separation			1,735,000
Complete separation[c]			
Relief sanitary sewers[d]			
–	3.0 – 4.5	2,940 ft of 15-in. RC at 3.0 – 33% to replace existing 36 – 48-in. which is to be utilized as storm drain	42,600
–	1.9 – 2.4	1,200 ft of 12-in. RC at 3.8 – 7.0% to replace existing 42 – 48-in. which is to be utilized as storm drain	13,100
–	1.5 – 1.9	1,480 ft of 12-in. RC at 0.3 – 0.6% to replace existing 48-in. which is to be utilized as storm drain	16,900
–	–	15 new house connections	4,500
–	–	Reconnect 18 lateral sewers	3,600
Subtotal, relief sanitary sewers			80,700

Continued on next page

Table 18-3. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Trunk storm drains			
A-1	365 – 375	600 ft of 63-in. RC at 1.8%	32,400
–	250 – 260	710 ft of existing 42 – 48-in.	–
–	60, 240[b]	390 ft of 30-in. RC at 3.2% to parallel existing 42-in.	8,300
–	215 – 225	670 ft of existing 42-in.	–
–	190 – 200	540 ft of 45-in. RC at 3.0% to replace existing 36-in.	16,800
–	170 – 190	630 ft of 39-in. RC at 5.6 – 9.2% to replace existing 36-in.	16,700
A-2	135 – 165	1,220 ft of existing 42-in.	–
A-3	100 – 135	1,750 ft of existing 42 – 48-in.	–
A-4	68 – 80	930 ft of 39-in. RC at 0.7 – 3.9%	25,300
B-1	45 – 50	1,680 ft of 27-in. RC at 5.6 – 10.5%	32,500
C-1	100	860 ft of 42-in. RC at 1.0 – 20%	45,300
C-2	55 – 100	1,170 ft of 33-in. RC at 1.2 – 18%	33,400
C-3	50 – 55	1,280 ft of 24-in. RC at 7.5 – 17%	20,300
C-4	28 – 30	1,260 ft of 24-in. RC at 2.6 – 5.1%	21,700
D-1	40 – 45	520 ft of 27-in. RC at 1.6 – 5.0%	10,600
D-2	35	710 ft of 21-in. RC at 6.2 – 15%	11,000
–	–	Connections to existing trunk to be utilized as storm drain	6,000
Subtotal, trunk storm drains			280,300
Local storm drains			
–	–	37,600 ft of 8-in.	226,700
–	–	11,800 ft of 10-in.	84,800
–	–	17,300 ft of 12-in.	170,100
–	–	19,300 ft of 15-in.	224,100
–	–	19,700 ft of 18-in.	264,600
–	–	6,000 ft of 21-in.	98,900
–	–	6,900 ft of 24-in.	125,500
–	–	240 ft of 27-in.	4,600
–	–	340 ft of 30-in.	7,900
–	–	227 intersection crossings, includes catch basin reconnections	80,900
–	–	3,825 new house connections	1,147,500
Subtotal, local storm drains			2,435,600
Total contract cost, complete separation			2,796,600
Engineering and contingencies, 25 per cent			699,200
Total construction cost, complete separation			3,495,800

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-3 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Routes of trunk storm drains are generally as shown in Fig. 18-3.*

[d]*Routes approximately same as relief sanitary sewers 32-34 and trunk storm drains A2and A3, partial separation.*

would continue to discharge to the existing system, which has ample capacity to take this flow plus the peak sanitary flow. An obvious advantage of such an alternative is that the storm drains could be laid at minimum depth.

Locations of facilities required for partial separation under Plan I are shown in Fig. 18-4, as are the locations and sizes of existing sewers. Under this plan, all storm drains would follow the routes of existing sewers and storm water would be conveyed to a single point of disposal at the foot of 32nd Avenue West. Descriptions and estimated construction costs are given in Table 18-4. As there noted, the total cost amounts to $958,200, or about $1,810 per acre. Of that total, approximately 19 per cent is for relief sanitary sewers, 48 per cent for trunk storm drains and 33 per cent for local storm drains.

Descriptions and estimated costs of facilities required for complete separation under Plan I are given in Table 18-4. It will be seen that the total cost amounts to $1,940,600, of which 32 per cent is for trunk storm drains, and 68 per cent is for local storm drains, including new house connections. Locations of the required facilities would, in general, be as shown in Fig. 18-4.

With three points of storm water disposal, as called for under Plan II, the locations of trunk drains required for separation would be as shown in Fig. 18-5. This figure also shows the locations and sizes of local storm drains required for the separation alternative which calls for the collection of runoff from streets and roofs but not from foundation drains.

Descriptions and estimated construction costs of the facilities required for each of the three alternatives available under Plan II are given in Table 18-4. One alternative involves partial separation, while the other two involve complete separation. For partial separation, the estimated cost amounts to a total of $893,500, or about $1,690 per acre. Of that total, approximately 20 per cent is for relief sanitary sewers, 45 per cent for trunk storm drains, and 35 per cent for local storm drains.

For complete separation under Alternative 1 of Plan II, the estimated cost totals $972,200, or about $1,830 per acre. Approximately 49 per cent of this total is for trunk storm drains and 51 per cent is for local storm drains.

For complete separation under Alternative 2 of Plan II, the estimated cost totals $1,890,000, or about $3,570 per acre. Trunk storm drains account for approximately 29 per cent of the total, while local storm drains, including new house connections, account for 71 per cent.

A summary of the estimated costs for the two plans of separation is presented in Table 18-5. As far as partial separation is concerned, the saving achieved by using three points of disposal rather than a single point is indicated by the lower costs of Plan II. Of more interest, however, is the fact that complete separation could be obtained at a slight additional cost by providing a drainage system designed to take all flow from street gutters and roof leaders. In districts such as South Magnolia, where extensive construction of relief sanitary sewers is required for partial separation, the advantage of such an alternative is that local drains can be extended to critical areas and roof leaders can be connected as required to alleviate overloading of the sanitary system. This advantage is obtained because removal of roof drainage from the sanitary system eliminates the need for relief sanitary sewers.

In areas where the existing system is capable of carrying both sanitary sewage and storm flow runoff from roof leaders, the drainage system may be extended only as required to pick up street inlets. In that manner, construction can be tailored to meet the requirements of any particular area and can be spread over a number of years. A further saving can be achieved by utilizing street gutters to the fullest possible extent for the conveyance of storm water runoff. By so doing, the length of local storm drains can be reduced in areas where only street drainage has to be removed from the existing combined system.

District 33 - Madison Park

Containing a total of 100 acres, the Madison Park district is situated on the shores of Lake Washington and Union Bay and is expected to have an ultimate population of 1,500. Topographically, it is fairly flat, with the surface sloping to Lake Washington on the east and Union Bay on the north.

The existing sewerage system is partly combined and partly separated. In the separated portion, a storm drain on McGilvra Boulevard picks up street drainage only and discharges to Union Bay. This drain is laid about five feet shallower than the sanitary sewer and thus cannot be utilized to intercept foundation drains.

Sewage from both the combined sewers and the sanitary sewers in the separated portion is discharged to a pumping station at 43rd Avenue North and East Lynn Street. At this station, dry weather flows are pumped to a station on East Lee Street, which in turn pumps to the North Trunk sewer. Wet weather flow in excess of the pumping capacity overflows at East Lynn Street.

Although the combined system has ample capacity for the peak flow of sanitary sewage, it is not capable of accommodating the flow resulting from a 10-year storm. Construction of a substantial number of relief sanitary sewers will be required in order to handle the peak sanitary flow plus the

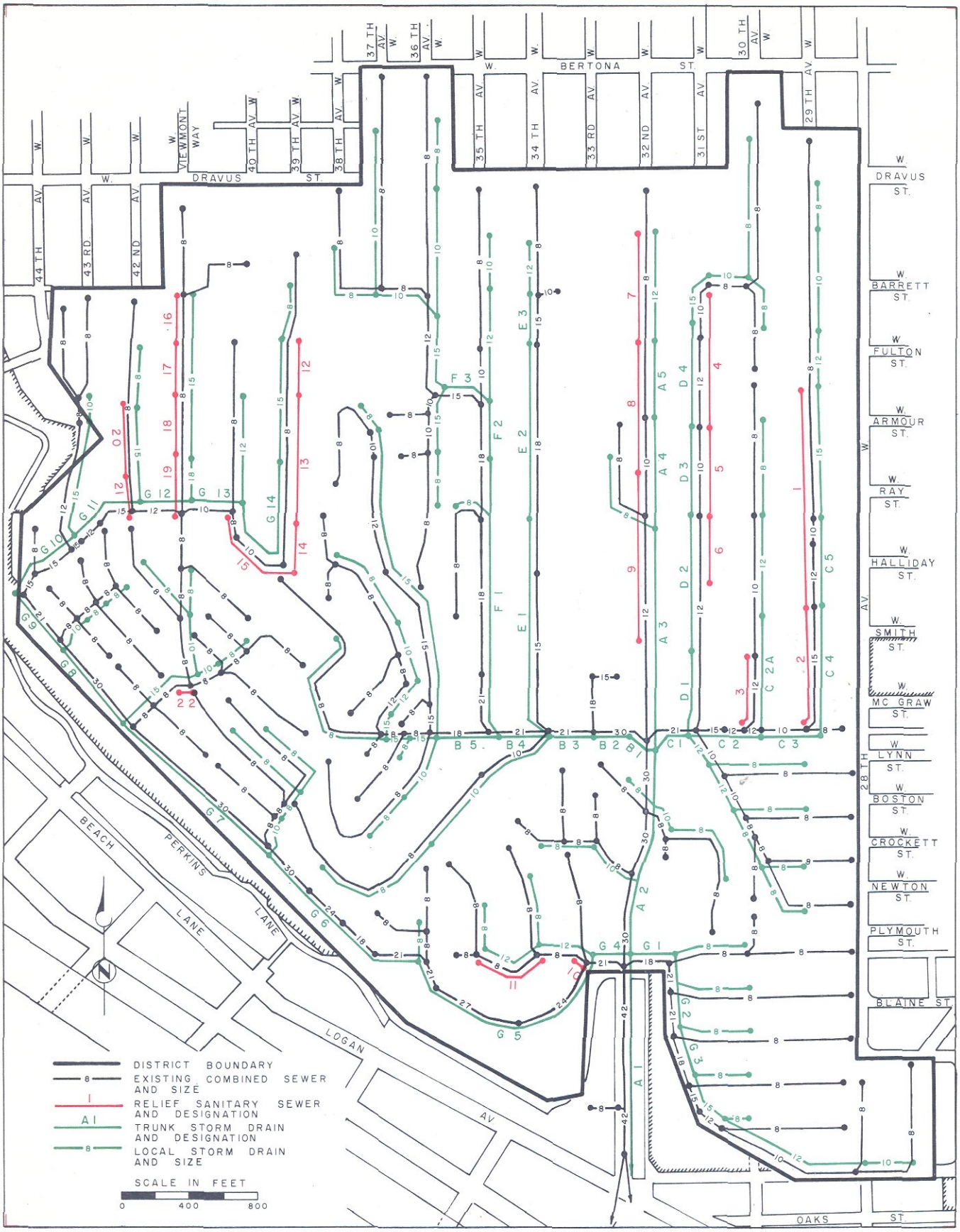

Fig. 18-4. Layout of Facilities for Plan I, Partial Separation, District 29 - South Magnolia

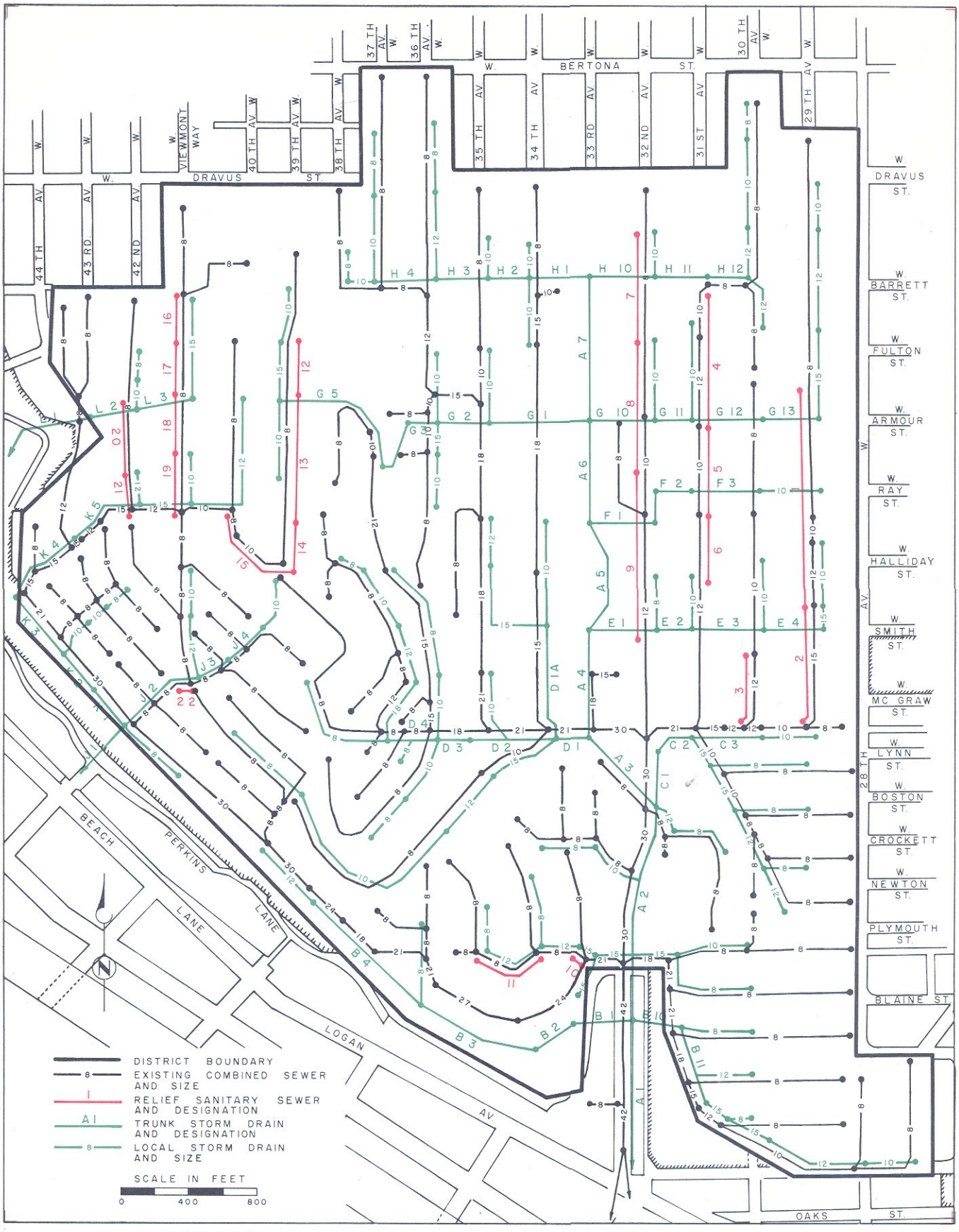

Fig. 18-5. Layout of Facilities Plan II, Complete Separation, Alternative 1, District 29 - South Magnolia

Table 18-4. Description and Estimated Construction Costs for Separation of District 29 — South Magnolia

Number	Design flow, cfs	Description	Construction cost, dollars
Plan I[a]			
Partial separation[b,c]			
Relief sanitary sewers			
1	2.2, 5.3[d]	1,300 ft of 8-in. conc at 3.0%	15,000
2	1.2, 6.2[d]	750 ft of 10-in. conc at 0.6%	9,200
3	1.6, 6.5[d]	500 ft of 10-in. conc at 0.5%	6,200
4	1.2, 4.9[d]	800 ft of 8-in. conc at 1.0%	9,200
5	2.0, 5.1[d]	530 ft of 8-in. conc at 3.0%	6,100
6	1.5, 5.1[d]	370 ft of 8-in. conc at 1.5%	4,300
7	1.2, 2.0[d]	650 ft of 8-in. conc at 2.0%	7,500
8	2.2, 3.5[d]	780 ft of 10-in. conc at 1.0%	9,600
9	1.5, 4.9[d]	1,000 ft of 8-in. conc at 1.5%	11,500
10	1.5, 2.1[d]	100 ft of 8-in. conc at 1.5%	1,200
11	0.8, 1.1[d]	450 ft of 8-in. conc at 0.4%	5,200
12	0.7, 1.6[d]	310 ft of 8-in. conc at 0.5%	3,600
13	1.6, 2.5[d]	750 ft of 10-in. conc at 0.5%	9,200
14	1.9, 2.9[d]	260 ft of 12-in. RC at 0.6%	4,400
15	1.0, 2.9[d]	500 ft of 8-in. conc at 0.8%	5,800
16	1.6, 2.5[d]	340 ft of 8-in. conc at 1.8%	3,800
17	2.1, 3.7[d]	330 ft of 10-in. conc at 1.8%	4,100
18	1.9, 3.9[d]	320 ft of 8-in. conc at 2.8%	3,700
19	3.2, 4.4[d]	320 ft of 12-in. RC at 1.0%	5,400
20	0.7, 1.2[d]	400 ft of 10-in. conc at 0.2%	4,900
21	0.9, 1.7[d]	200 ft of 8-in. conc at 0.5%	2,300
22	1.7, 4.1[d]	100 ft of 8-in. conc at 4.0%	1,200
–	–	Reconnect 106 house connections	4,200
–	–	Reconnect 45 manholes	4,500
Subtotal, relief sanitary sewers			142,100
Trunk storm drains			
A-1	120	1,100 ft of 36-in. RC at 5.0%	20,800
A-2	94	1,400 ft of 36-in. RC at 5.0%	29,300
A-3	12	1,350 ft of 18-in. RC at 2.0%	16,500
A-4	8	570 ft of 18-in. RC at 0.8%	5,800
B-1	46	100 ft of 36-in. RC at 0.6%	2,300
B-2	46	240 ft of 30-in. RC at 1.2%	4,600
B-3	45	280 ft of 27-in. RC at 4.0%	5,200
B-4	33	360 ft of 24-in. RC at 6.0%	6,000
B-5	16	340 ft of 18-in. RC at 5.5%	4,200
C-1	40	350 ft of 24-in. RC at 3.3%	5,100
C-2	22	270 ft of 24-in. RC at 5.0%	3,900
C-3	15	370 ft of 24-in. RC at 3.0%	5,300

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
C-4	14	700 ft of 24-in. RC at 0.6%	10,000
C-5	11	560 ft of 18-in. RC at 1.1%	6,100
C-2A	8	670 ft of 21-in. RC at 0.4%	8,500
D-1	13	460 ft of 18-in. RC at 3.0%	5,000
D-2	12	800 ft of 18-in. RC at 1.5%	8,800
D-3	11	470 ft of 18-in. RC at 2.5%	5,200
D-4	10	620 ft of 18-in. RC at 0.9%	6,800
E-1	12	1,310 ft of 24-in. RC at 2.0%	19,000
E-2	8	1,050 ft of 24-in. RC at 0.2%	15,200
E-3	4	370 ft of 21-in. RC at 0.14%	4,700
F-1	19	1,600 ft of 24-in. RC at 1.0%	23,100
F-2	17	300 ft of 21-in. RC at 1.3%	3,800
F-3	13	320 ft of 18-in. RC at 3.0%	3,500
G-1	16	300 ft of 24-in. CMP at 20%	8,200
G-2	13	450 ft of 24-in. RC at 0.9%	7,500
G-3	8	310 ft of 21-in. RC at 0.3%	4,000
G-4	35	220 ft of 33-in. RC at 5.0%	4,200
G-5	32	1,350 ft of 33-in. RC at 0.7%	26,800
G-6	30	1,200 ft of 33-in. RC at 0.4%	23,600
G-7	28	1,080 ft of 33-in. RC at 0.3%	21,200
G-8	22	700 ft of 33-in. RC at 0.2%	13,800
G-9	22	340 ft of 24-in. RC at 0.9%	4,900
G-10	22	450 ft of 18-in. RC at 9.0%	5,000
G-11	18	460 ft of 18-in. RC at 4.8%	5,100
G-12	16	320 ft of 18-in. RC at 9.7%	3,500
G-13	9	300 ft of 18-in. RC at 4.0%	3,300
G-14	6	1,100 ft of 18-in. RC at 0.8%	12,100
Subtotal, trunk storm drains			371,900
Local storm drains			
–	–	11,970 ft of 8-in.	55,500
–	–	7,210 ft of 10-in.	39,700
–	–	4,570 ft of 12-in.	36,600
–	–	8,040 ft of 15-in.	76,400
–	–	118 intersection crossings, includes catch basin reconnections	44,400
Subtotal, local storm drains			252,600
Total contract cost, partial separation, Plan I			766,600
Engineering and contingencies, 25 per cent			191,600
Total construction cost, partial separation, Plan I			958,200
Complete separation[e,f]			
Trunk storm drains			
A-1	180	1,100 ft of 42-in. RC at 5.0%	27,400
A-2	140	1,400 ft of 42-in. RC at 5.0%	37,800

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
A-3	18	1,350 ft of 21-in. RC at 2.0%	21,900
A-4	11	570 ft of 21-in. RC at 0.8%	8,000
B-1	69	100 ft of 42-in. RC at 0.6%	3,100
B-2	69	240 ft of 36-in. RC at 1.2%	6,000
B-3	68	280 ft of 30-in. RC at 4.0%	6,200
B-4	50	360 ft of 27-in. RC at 6.0%	7,200
B-5	24	340 ft of 18-in. RC at 5.5%	4,800
C-1	60	350 ft of 30-in. RC at 3.3%	7,300
C-2	33	270 ft of 27-in. RC at 5.0%	5,100
C-3	23	370 ft of 27-in. RC at 3.0%	7,000
C-4	21	700 ft of 27-in. RC at 0.6%	13,200
C-5	16	560 ft of 21-in. RC at 1.1%	8,500
C-2A	12	670 ft of 24-in. RC at 0.4%	11,400
D-1	20	460 ft of 24-in. RC at 3.0%	7,800
D-2	18	800 ft of 24-in. RC at 1.5%	13,600
D-3	17	470 ft of 24-in. RC at 2.5%	8,000
D-4	16	620 ft of 24-in. RC at 0.9%	10,500
E-1	17	1,310 ft of 27-in. RC at 2.0%	24,800
E-2	13	1,050 ft of 27-in. RC at 0.2%	19,800
E-3	6	370 ft of 24-in. RC at 0.14%	6,300
F-1	29	1,600 ft of 27-in. RC at 1.0%	30,200
F-2	26	300 ft of 27-in. RC at 1.3%	5,700
F-3	20	320 ft of 21-in. RC at 3.0%	4,900
G-1	24	320 ft of 27-in. CMP at 20%	12,000
G-2	20	450 ft of 27-in. RC at 0.9%	9,000
G-3	13	310 ft of 27-in. RC at 0.3%	5,800
G-4	53	220 ft of 39-in. RC at 5.0%	5,500
G-5	48	1,350 ft of 39-in. RC at 0.7%	36,200
G-6	45	1,200 ft of 39-in. RC at 0.4%	32,200
G-7	42	1,080 ft of 39-in. RC at 0.3%	29,000
G-8	33	700 ft of 39-in. RC at 0.2%	18,800
G-9	33	340 ft of 30-in. RC at 0.9%	7,100
G-10	33	450 ft of 24-in. RC at 9.0%	7,600
G-11	28	460 ft of 21-in. RC at 4.8%	7,000
G-12	24	320 ft of 21-in. RC at 9.0%	4,900
G-13	14	300 ft of 21-in. RC at 4.0%	4,600
G-14	9	1,100 ft of 21-in. RC at 0.8%	16,800
Subtotal, trunk storm drains			503,000
Local storm drains			
–	–	33,050 ft of 8-in.	228,600
–	–	7,930 ft of 10-in.	62,600
–	–	4,730 ft of 12-in.	50,400

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
–	–	6,020 ft of 15-in.	75,000
–	–	7,630 ft of 18-in.	108,500
–	–	1,600 new house connections	480,000
–	–	118 intersection crossings, includes catch basin reconnections	44,400
Subtotal, local storm drains			1,049,500
Total contract cost, complete separation, Plan I			1,552,500
Engineering and contingencies, 25 per cent			388,100
Total construction cost, complete separation, Plan I			1,940,600
Plan II[g]			
Partial separation[b,h]			
–	–	Relief sanitary sewers[i]	142,100
Trunk storm drains			
A-1	147	820 ft of 36-in. RC at 5.0%	15,500
A-2	131	1,200 ft of 36-in. RC at 5.0%	22,800
A-3	101	580 ft of 36-in. RC at 3.5%	13,300
A-4	76	600 ft of 36-in. RC at 1.2%	13,800
A-5	67	750 ft of 36-in. RC at 1.2%	15,400
A-6	61	570 ft of 36-in. RC at 1.0%	10,800
A-7	31	750 ft of 30-in. RC at 0.7%	12,900
B-1	10	400 ft of 15-in. CMP at 10%, anchored to slopes	7,600
B-2	9	250 ft of 18-in. RC at 1.8%	3,100
B-3	7	760 ft of 15-in. RC at 3.3%	7,200
B-4	4	900 ft of 10-in. conc at 4.0%	5,000
B-10	12	350 ft of 18-in. CMP at 10%, anchored to slopes	7,300
B-11	10	480 ft of 18-in. RC at 2.2%	5,300
C-1	20	430 ft of 18-in. RC at 4.0%	4,700
C-2	17	340 ft of 18-in. RC at 4.0%	3,700
C-3	7	300 ft of 12-in. RC at 13%	2,400
D-1	17	380 ft of 18-in. RC at 4.0%	4,700
D-1A	7	650 ft of 15-in. RC at 2.0%	6,200
D-2	17	520 ft of 18-in. RC at 6.0%	6,400
D-3	14	170 ft of 15-in. RC at 5.5%	1,900
D-4	7	300 ft of 12-in. RC at 10%	2,900
E-1	12	300 ft of 18-in. RC at 2.0%	3,300
E-2	11	320 ft of 15-in. RC at 5.0%	3,000
E-3	8	350 ft of 12-in. RC at 11%	2,800
E-4	4	350 ft of 10-in. conc at 15%	1,900
F-1	7	500 ft of 18-in. RC at 0.5%	5,500
F-2	5	320 ft of 10-in. conc at 8.0%	2,600
F-3	4	320 ft of 10-in. conc at 11%	1,800
G-1	19	660 ft of 18-in. RC at 4.5%	7,200

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
G-2	16	320 ft of 18-in. RC at 4.0%	3,500
G-3	11	150 ft of 15-in. RC at 17%	1,400
G-4	11	350 ft of 15-in. RC at 6.0%	3,300
G-5	7	750 ft of 12-in. RC at 10%	6,000
G-10	13	300 ft of 18-in. RC at 1.3%	3,300
G-11	11	320 ft of 15-in. RC at 10%	3,000
G-12	9	320 ft of 15-in. RC at 13%	3,000
G-13	8	320 ft of 15-in. RC at 8.7%	3,000
H-1	17	320 ft of 21-in. RC at 1.6%	4,100
H-2	14	330 ft of 18-in. RC at 8.0%	3,600
H-3	11	320 ft of 15-in. RC at 10%	3,000
H-4	7	250 ft of 12-in. RC at 16%	2,000
H-10	11	320 ft of 18-in. RC at 1.6%	3,500
H-11	9	320 ft of 15-in. CMP at 29%	6,000
H-12	7	320 ft of 12-in. RC at 12%	2,600
J-1	27	600 ft of 24-in. CMP at 20%, anchored to slopes	15,300
J-2	13	500 ft of 15-in. RC at 11%	4,900
J-3	9	200 ft of 15-in. RC at 10%	1,900
J-4	4	400 ft of 10-in. conc at 7.5%	2,200
K-1	15	340 ft of 30-in. RC at 0.2%	6,200
K-2	15	340 ft of 30-in. RC at 0.2%	6,200
K-3	10	340 ft of 18-in. RC at 1.0%	3,700
K-4	10	410 ft of 15-in. RC at 10%	3,900
K-5	8	460 ft of 15-in. RC at 4.8%	4,400
L-1	13	700 ft of 18-in. CMP at 20%, anchored to slopes	14,700
L-2	11	300 ft of 15-in. CMP at 20%	5,800
L-3	8	350 ft of 12-in. RC at 10%	2,800
Subtotal, trunk storm drains			324,300
Local storm drains			
–	–	17,190 ft of 8-in.	80,000
–	–	11,000 ft of 10-in.	60,500
–	–	4,370 ft of 12-in.	35,000
–	–	3,000 ft of 15-in.	28,500
–	–	118 intersection crossings, includes catch basin reconnections	44,400
Subtotal, local storm drains			248,400
Total contract cost, partial separation, Plan II			714,800
Engineering and contingencies, 25 per cent			178,700
Total construction cost, partial separation, Plan II			893,500
Complete separation, Alternative 1[j]			
Trunk storm drains			
A-1	220	820 ft of 42-in. RC at 5.0%	20,400

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
A-2	197	1,200 ft of 42-in. RC at 5.0%	29,800
A-3	152	580 ft of 42-in. RC at 3.5%	17,000
A-4	114	600 ft of 42-in. RC at 1.2%	17,300
A-5	101	750 ft of 39-in. RC at 1.2%	18,200
A-6	92	570 ft of 39-in. RC at 1.0%	12,900
A-7	46	750 ft of 33-in. RC at 0.7%	14,200
B-1	15	400 ft of 18-in. CMP at 10%, anchored to slopes	8,000
B-2	14	250 ft of 18-in. RC at 1.8%	3,100
B-3	11	760 ft of 15-in. RC at 3.3%	7,200
B-4	6	900 ft of 12-in. RC at 4.0%	7,200
B-10	18	350 ft of 21-in. CMP at 10%, anchored to slopes	8,400
B-11	15	480 ft of 18-in. RC at 2.2%	5,200
C-1	30	430 ft of 24-in. RC at 4.0%	6,200
C-2	25	340 ft of 21-in. RC at 4.0%	3,700
C-3	10	300 ft of 12-in. RC at 13%	2,400
D-1	26	380 ft of 21-in. RC at 4.0%	5,800
D-1A	11	650 ft of 18-in. RC at 2.0%	7,100
D-2	26	520 ft of 21-in. RC at 6.0%	7,800
D-3	21	170 ft of 18-in. RC at 5.5%	2,100
D-4	11	300 ft of 15-in. RC at 10%	3,300
E-1	18	300 ft of 21-in. RC at 2.0%	3,800
E-2	16	320 ft of 18-in. RC at 5.0%	3,500
E-3	12	350 ft of 15-in. RC at 11%	3,300
E-4	6	350 ft of 12-in. RC at 15%	2,800
F-1	10	500 ft of 21-in. RC at 0.5%	6,400
F-2	7	320 ft of 12-in. RC at 8.0%	2,600
F-3	6	320 ft of 12-in. RC at 11%	2,600
G-1	28	660 ft of 24-in. RC at 4.5%	9,500
G-2	24	320 ft of 21-in. RC at 4.0%	4,100
G-3	16	150 ft of 18-in. RC at 17%	1,600
G-4	16	350 ft of 18-in. RC at 6.0%	3,800
G-5	11	750 ft of 15-in. RC at 10%	7,100
G-10	20	300 ft of 21-in. RC at 1.3%	3,800
G-11	17	320 ft of 18-in. RC at 10%	3,500
G-12	13	320 ft of 18-in. RC at 13%	3,500
G-13	12	320 ft of 15-in. RC at 8.7%	3,000
H-1	26	320 ft of 24-in. RC at 1.6%	4,500
H-2	21	330 ft of 21-in. RC at 8.0%	4,200
H-3	17	320 ft of 18-in. RC at 10%	3,500
H-4	11	250 ft of 15-in. RC at 16%	2,400
H-10	16	320 ft of 21-in. RC at 1.6%	4,100
H-11	14	320 ft of 18-in. CMP at 29%	7,000
H-12	11	320 ft of 15-in. RC at 12%	3,000

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
J-1	40	600 ft of 27-in. CMP at 20%, anchored to slopes	16,200
J-2	20	500 ft of 18-in. RC at 11%	5,600
J-3	13	200 ft of 15-in. RC at 10%	1,900
J-4	6	400 ft of 12-in. RC at 7.5%	3,200
K-1	22	340 ft of 33-in. RC at 0.2%	6,800
K-2	22	340 ft of 33-in. RC at 0.2%	6,800
K-3	15	340 ft of 21-in. RC at 1.0%	4,300
K-4	15	410 ft of 18-in. RC at 10%	4,500
K-5	12	460 ft of 15-in. RC at 4.8%	4,400
L-1	20	700 ft of 21-in. CMP at 20%, anchored to slopes	16,800
L-2	16	300 ft of 18-in. CMP at 20%	6,600
L-3	12	350 ft of 15-in. RC at 10%	3,300
Subtotal, trunk storm drains			381,300
Local storm drains			
–	–	8,810 ft of 8-in.	41,000
–	–	13,530 ft of 10-in.	74,500
–	–	7,110 ft of 12-in.	57,000
–	–	4,860 ft of 15-in.	46,000
–	–	1,240 ft of 18-in.	13,600
–	–	118 intersection crossings, includes catch basin reconnections	44,400
–	–	400 new house connections[k]	120,000
Subtotal, local storm drains			396,500
Total contract cost, complete separation, Plan II, Alternative 1			777,800
Engineering and contingencies, 25 per cent			194,400
Total construction cost, complete separation, Plan II, Alternative 1			972,200
Complete separation, Alternative 2[e,h]			
–	–	Trunk storm drains[l]	443,800
–	–	Local storm drains[l]	366,200
–	–	32,250 ft of additional 8-in. local drains to extend to last house on each block	222,000
–	–	1,600 new house connections	480,000
Total contract cost, complete separation, Plan II, Alternative 2			1,512,000
Engineering and contingencies, 25 per cent			378,000
Total construction cost, complete separation, Plan II, Alternative 2			1,890,000

[a]*Plan I provides for installation of storm drains along routes of existing sewers with one point of disposal.*

[b]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers.*

[c]*See Fig. 18-4 for location of facilities.*

[d]*First flow is required relief capacity, second is total design flow.*

[e]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Assumes that storm drains will be laid at sufficient depth to intercept all foundation drains.*

[f]*See Fig. 18-4 for location of trunk storm drains.*

(Footnotes continued on next page.)

Table 18-5. Summary of Separation Costs, District 29 – South Magnolia

Facility	Construction cost,[a] dollars				
	Plan I[b]		Plan II[c]		
	Partial separation[d]	Complete separation[e]	Partial separation[d]	Complete separation	
				Alternative 1[f]	Alternative 2[e]
Relief sanitary sewers	177,600	–	177,600	–	–
Trunk storm drains	464,900	628,700	405,400	476,600	554,800
Local storm drains[g]	315,700	711,900	310,500	345,600	735,200
New house connections	–	600,000	–	150,000[h]	600,000
Total	958,200	1,940,600	893,500	972,200	1,890,000

[a]*Includes engineering and contingencies.*

[b]*Plan I provides for installation of storm drains along routes of existing combined system with one point of disposal.*

[c]*Plan II provides for rerouting of storm drains with 3 points of disposal.*

[d]*Provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers.*

[e]*Provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers.*

[f]*Provides for removal of all street drainage and roof leaders from sanitary sewers, but permits continued discharge of foundation drains to sanitary sewers.*

[g]*Exclusive of new house connections.*

[h]*Assumes that 25 per cent of all roof leaders will be reconnected to storm drainage system.*

storm flow from roof leaders.

The existing storm drain on McGilvra Boulevard has only enough capacity for street drainage. Some relief will be required in order to accommodate the total flow of storm water.

Locations of relief sanitary sewers and of storm drainage facilities required to effect partial separation are shown in Fig. 18-6 together with a layout of the existing system. Descriptions and estimated construction costs are given in Table 18-6.

Locations of storm drainage facilities required for complete separation would be substantially the same as those shown in Fig. 18-6. Complete separation would necessitate the addition of relief lines for the existing storm drains and the extension of local storm drains to the extremity of each existing sewer line. Descriptions and estimated construction costs of the required facilities are also given in Table 18-6.

For partial separation of the Madison Park district, the estimated cost of construction is $173,900, or about $1,740 per acre. Of this total, approximately 49 per cent is for relief sanitary sewers, 24 per cent for trunk drains and 27 per cent for local storm drains. For complete separation, the estimated cost is $369,100, of which about 27 per cent is for trunk storm drains and 73 per cent is for local storm drains, including new house connections.

District 58 - East Madison

The East Madison district is situated on the east slopes of Capitol Hill south of the University of Washington Arboretum. It comprises an area of 640 acres and has a predicted ultimate population of 20,000. Drainage in this district is northward to Union Bay, with steep surface slopes perpendicular to the main drainage axis.

The existing combined system, which is a part of the Lake Washington district on the North Trunk sys-

Table 18-4 Footnotes (continued).

[g]*Plan II provides for rerouting of storm drains with 3 points of disposal.*

[h]*See Fig. 18-5 for location of trunk storm drains.*

[i]*Relief sanitary sewers same as for Plan I.*

[j]*Complete separation under Alternative 1 provides for removal of all street drainage and roof leaders from sanitary sewers, but permits continued discharge of foundation drains to sanitary sewers. Assumes that storm drains will be laid at minimum depth. See Fig. 18-5 for location of facilities.*

[k]*Assumes that 25 per cent of all roof leaders will be reconnected to storm drainage system.*

[l]*Design flow, length, size and slope of all storm drains same as for Alternative 1. Increased cost due to added depth required for interception of foundation drains.*

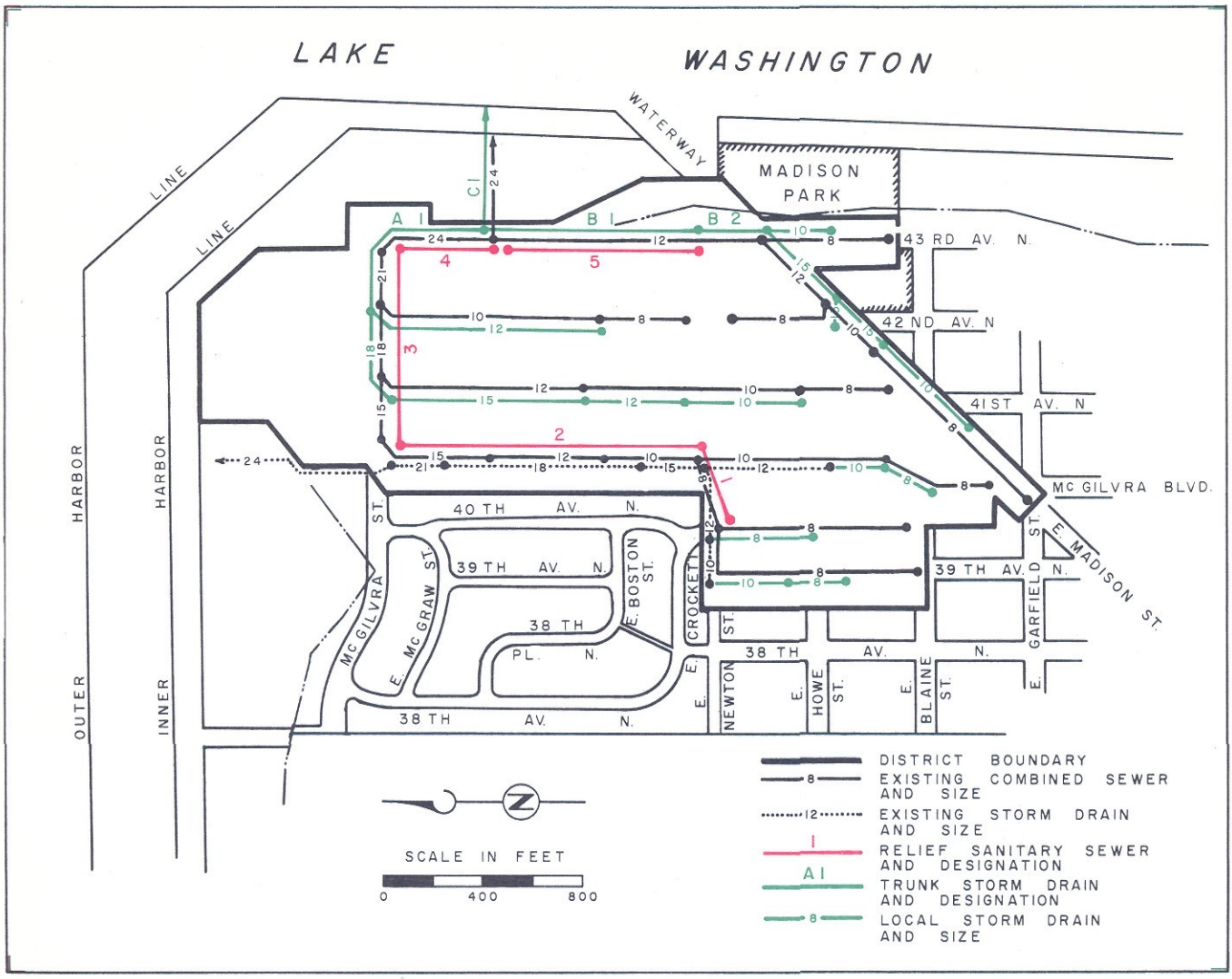

Fig. 18-6. Layout of Facilities for Partial Separation, District 33 - Madison Park

Table 18-6. Description and Estimated Construction Costs for Separation of District 33 – Madison Park

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	0.2, 1.8[b]	290 ft of 8-in. conc at 1.7%	2,000
2	0 – 2.6, 3.3 – 3.7[b]	1,230 ft of 10-in. conc at 0.7 – 1.2%	10,400
3	4.8 – 7.0, 8.9 – 13[b]	920 ft of 21-in. RC at 0.12 – 0.3%	20,000
4	5.5, 13[b]	400 ft of 24-in. RC at 0.11%	15,000
5	0.2, 2.2 – 2.8[b]	830 ft of 8-in. conc at 0.3 – 0.6%	17,400
–	–	Reconnect 36 house connections	1,400
–	–	Reconnect 15 manholes	1,500
Subtotal, relief sanitary sewers			67,700

Continued on next page

Table 18-6. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Trunk storm drains			
A-1	7.3 – 7.5	750 ft of 24-in. RC at 0.11 – 0.19%	12,000
B-1	4.8 – 6.3	830 ft of 18-in. RC at 0.33 – 0.6%	10,000
B-2	3.6	270 ft of 15-in. RC at 0.33%	2,900
C-1	12.5	500 ft of 24-in. RC outfall	8,700
Subtotal, trunk storm drains			33,600
Local storm drains			
–	–	870 ft of 8-in.	4,900
–	–	1,750 ft of 10-in.	11,700
–	–	1,360 ft of 12-in.	12,500
–	–	1,480 ft of 15-in.	17,200
–	–	350 ft of 18-in.	4,300
–	–	26 intersection crossings, includes catch basin reconnections	7,200
Subtotal, local storm drains			37,800
Total contract cost, partial separation			139,100
Engineering and contingencies, 25 per cent			34,800
Total construction cost, partial separation			173,900
Complete separation[c]			
Trunk storm drains			
A-1	15 – 17	400 ft of 33-in. RC at 0.11%	17,300
A-1	15	350 ft of 30-in. RC at 0.12 – 0.19%	13,500
B-1, B-2	6 – 10	1,100 ft of 27-in. RC at 0.33 – 0.6%	29,500
C-1	25	500 ft of 36-in. RC outfall	12,100
–	3.0, 12[b]	350 ft of 12-in. at 0.75% to parallel existing 18-in.	3,200
–	4.2, 11[b]	270 ft of 12-in. at 1.0% to parallel existing 15-in.	2,500
–	–	Reconnect 4 manholes	400
Subtotal, trunk storm drains			78,500
Local storm drains			
–	–	1,520 ft of 8-in.	9,800
–	–	2,180 ft of 10-in.	17,700
–	–	2,150 ft of 12-in.	22,100
–	–	1,010 ft of 15-in.	13,500
–	–	780 ft of 18-in.	11,000
–	–	440 ft of 21-in.	9,100
–	–	300 ft of 24-in.	6,400
–	–	26 intersection crossings, includes catch basin reconnections	7,200
–	–	400 new house connections	120,000
Subtotal, local storm drains			216,800
Total contract cost, complete separation			295,300
Engineering and contingencies, 25 per cent			73,800
Total construction cost, complete separation			369,100

(See page 515 for footnotes.)

tem, presently discharges through a 60-inch trunk to an inverted siphon at the Montlake Bridge. Storm flows in excess of the capacity of the siphon overflow to Montlake Canal.

Originally, the portion of East Madison district west of 23rd Avenue North and north of East Madison Street drained south to the Hanford-Rainier Valley system. Sewage from this area, however, is now diverted eastward along East Denny Way to the principal trunk serving the district.

Although the existing system has ample capacity for the peak sanitary flow, it is unable generally to accommodate the flow from a 10-year storm. In fact, a considerable amount of relief sanitary sewer construction will be required to handle the sanitary peak plus storm runoff from roof leaders. Capacity for a 10-year storm is available, however, along the downstream sections of the principal trunk.

Locations of facilities required for partial separation of the East Madison district are shown in Fig. 18-7 together with a layout of the existing sewers. Descriptions and estimated construction costs are given in Table 18-7. As shown in both the figure and the tabulated data, the existing trunk leaving the district would be utilized as a storm water line to the fullest possible extent. This, of course, would require construction of a new sanitary sewer along the section so utilized.

Locations of facilities required for complete separation are substantially the same as those shown in Fig. 18-7. Descriptions and estimated construction costs are also given in Table 18-7. As in partial separation, the existing main trunk would be utilized to the fullest possible extent as a storm drain. Under design flow conditions, the existing trunk would be surcharged to 30th Avenue North and East John Street.

For partial separation of the East Madison district, the estimated construction cost amounts to a total of $1,390,200, or about $2,170 per acre. Of this total, approximately 24 per cent is for relief sanitary sewers, 12 per cent for trunk storm drains, and 64 per cent for local storm drains. For complete separation, the estimated cost is $2,690,800, or about $4,200 per acre. Of this total, approximately 3 per cent is for new sanitary sewers, 12 per cent for trunk storm drains, and 85 per cent for local storm drains, including new house connections.

Under both partial and complete separation, part of the proposed storm drain (A 9) would be laid along 28th Avenue North. Since this street will shortly become an arterial highway under the Empire Way extension program, it is believed that the 28th Avenue section of the storm drain should be constructed immediately.

Although the study here reported was concerned only with the collection of storm water within the district and not with the problem of disposal, it is evident that disposal could be achieved satisfactorily by discharge to an open channel following an old creek bed through Washington Park. This channel, which would be about 4,200 feet in length and would terminate at Union Bay, would be considerably cheaper than closed conduit of the size here required.

RECOMMENDED SEPARATION PROGRAM

It is evident from the information here presented, as well as that developed in past studies, that the city of Seattle is faced with a continuing program of storm water separation. This program is necessary to prevent the periodic overloading of trunk and local collection systems. In addition, and as set forth in Chapter 15, many of the existing major trunk sewers which are to be incorporated in the central sewerage project lack the capacity to accommodate the flows resulting from a storm with a 10-year recurrence interval. As a consequence, separation will be required in areas tributary to those trunks.

With all of the various factors taken into account, it appears that separation of storm water from sanitary sewage will be required to some extent in all areas presently served by combined sewers. Analysis will have to be made, of course, to determine local requirements. It appears likely, however, that separation in all areas will be required at least to the extent of removing street drainage from existing sewers.

Summary of Separation Costs

A summary of the estimated construction costs for storm water separation in the six areas herein considered is presented in Table 18-8. For partial separation, the estimated cost varies from $1,690 to $2,170 per acre and averages $1,860 per acre. For complete separation, the cost varies from $3,180 to $4,570 per acre and averages $3,890 per acre. Since the deviation from the average is less than 20 per cent, use of the average figures appears justifiable in attempting to assess the total cost to the city of storm water separation.

Table 18-6 Footnotes.

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-6 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Routes of trunk storm drains are generally as shown in Fig. 18-6.*

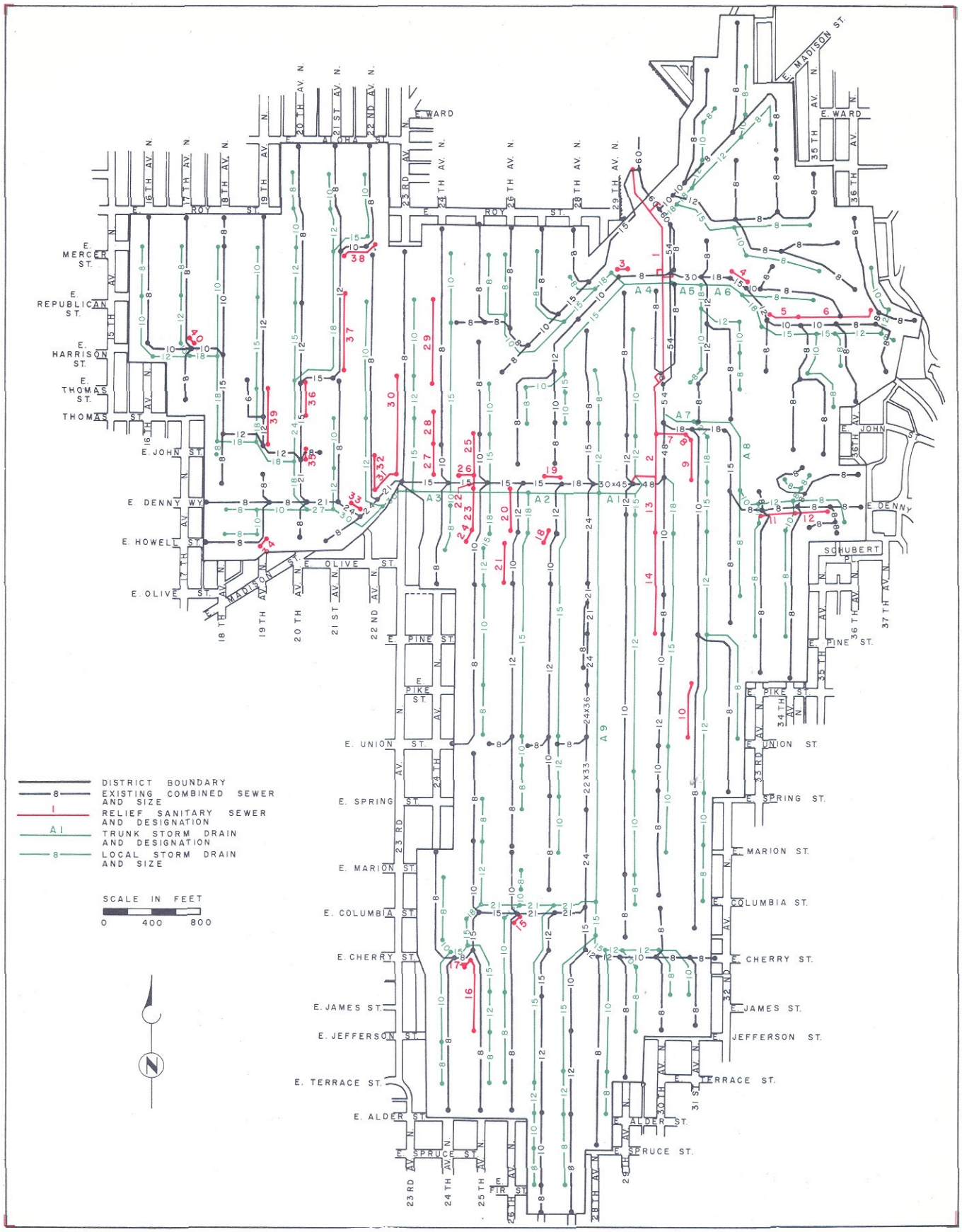

Fig. 18-7. Layout of Facilities for Partial Separation, District 58 - East Madison

Table 18-7. Description and Estimated Construction Costs for Separation of District 58 – East Madison

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	64 – 65	2,120 ft of 39-in. RC at 0.50 – 0.70%, to replace existing 54 – 60-in. which is to be used as storm drain	119,000
2	44 – 49	880 ft of 30-in. RC at 1.4 – 2.3%, to replace existing 48-in. which is to be used as storm drain, includes 880 ft of 8-in. lateral parallel to trunk to avoid making house connections to trunk	31,300
3	1.2, 2.7[b]	330 ft of 8-in. conc at 1.4%	2,000
4	1.7, 9.0[b]	210 ft of 10-in. conc at 1.3%	1,500
5	2.4, 7.0[b]	310 ft of 8-in. conc at 4.4 – 8.1%	1,900
6	0.4 – 1.0, 2.0 – 4.0[b]	720 ft of 8-in. conc at 1.0 – 4.0%	4,300
7	3.0, 13[b]	210 ft of 12-in. RC at 0.8%	3,900
8	5.3, 7.4[b]	60 ft of 15-in. RC at 0.9%	1,300
9	2.7, 6.0[b]	470 ft of 10-in. conc at 2.4%	6,200
10	0.7, 2.5[b]	320 ft of 8-in. conc at 2.1 – 3.3%	1,900
11	1.1, 4.0 – 5.3[b]	510 ft of 8-in. conc at 5.4 – 11.8%	3,100
12	2.8, 4.0[b]	270 ft of 12-in. RC at 1.0%	2,900
13	1.3, 3.4[b]	470 ft of 10-in. conc at 0.9%	6,200
14	0.7 – 1.3, 2.7 – 2.9[b]	850 ft of 8-in. conc at 1.0 – 1.4%	5,100
15	0.4, 2.0[b]	60 ft of 8-in. conc at 1.6%	400
16	0.3 – 0.6, 1.0 – 1.5[b]	690 ft of 8-in. conc at 0.4 – 1.1%	4,100
17	2.4, 3.7[b]	60 ft of 10-in. conc at 1.1%	400
18	2.4, 3.9[b]	80 ft of 12-in. conc at 0.4%	900
19	3.6, 28[b]	80 ft of 8-in. conc at 11.6%	500
20	1.1, 3.1[b]	390 ft of 10-in. conc at 0.30%	2,800
21	0.8, 2.6[b]	350 ft of 8-in. conc at 0.70%	2,100
22	0.4, 2.4[b]	60 ft of 8-in. conc at 0.80%	400
23	1.0, 2.2[b]	390 ft of 10-in. conc at 0.30%	2,800
24	0.4, 2.1[b]	90 ft of 8-in. conc at 0.60%	500
25	0.5 – 0.7, 1.7 – 2.0[b]	450 ft of 8-in. conc at 0.60 – 1.45%	2,700
26	3.5, 22[b]	100 ft of 8-in. conc at 7.7%	600
27	0.6, 2.2[b]	330 ft of 8-in. conc at 0.5%	2,300
28	1.2, 2.1[b]	230 ft of 10-in. conc at 0.6%	1,900
29	0.3 – 0.8, 1.0 – 1.6[b]	640 ft of 8-in. conc at 0.3 – 0.4%	4,500
30	0.6 – 0.8, 1.5 – 2.3[b]	890 ft of 8-in. conc at 0.8 – 1.5%	6,200
31	2.7, 18[b]	410 ft of 12-in. RC at 0.9%	5,000
32	0.4, 2.0[b]	230 ft of 8-in. conc at 1.9%	1,400

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Table 18-7. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
33	11, 17[b]	170 ft of 30-in. RC at 0.06%, includes 170 ft of 8-in. lateral parallel to trunk to avoid making house connections to trunk	5,100
34	0.7, 1.1[b]	50 ft of 10-in. conc at 0.11%	400
35	6.9, 8.6[b]	60 ft of 27-in. RC at 0.07%	1,300
36	4.2, 8.6[b]	330 ft of 15-in. RC at 0.45%	4,600
37	1.2 – 1.6, 4.7 – 5.2[b]	660 ft of 8-in. conc at 1.2 – 1.6%	4,000
38	0.8, 2.0[b]	310 ft of 8-in. conc at 0.5 – 0.7%	1,900
39	1.7, 2.1[b]	600 ft of 12-in. RC at 0.25%	7,300
40	0.7, 1.7[b]	50 ft of 8-in. conc at 0.7%	300
–	–	Reconnect 132 house connections	6,500
–	–	Reconnect 89 manholes	8,900
Subtotal, relief sanitary sewers			270,400
Trunk storm drains			
A-1	100	320 ft of 36-in. RC at 2.2%	6,900
A-2	59 – 66	600 ft of 30-in. RC at 8.8 – 16.7%	10,500
A-3	42 – 53	1,020 ft of 30-in. RC at 7.7 – 16.7%	17,800
A-4	11 – 16	560 ft of 15-in. RC at 2.9 – 6.7%	5,700
A-5	22	250 ft of 24-in. RC at 0.80%	4,100
A-6	17	420 ft of 21-in. RC at 1.3 – 1.4%	5,700
A-7	26	270 ft of 27-in. RC at 0.8 – 5.8%	4,900
A-8	15	890 ft of 24-in. RC at 0.6 – 15.3%	13,000
A-9	34 – 37	3,600 ft of 24-in. RC at 2.3 – 10.0%	59,400
–	–	Connections to existing trunk to be utilized as storm drain	1,500
Subtotal, trunk storm drains			129,500
Local storm drains			
–	–	22,000 ft of 8-in.	102,100
–	–	19,000 ft of 10-in.	112,700
–	–	13,000 ft of 12-in.	106,300
–	–	17,000 ft of 15-in.	169,900
–	–	8,000 ft of 18-in.	91,300
–	–	1,000 ft of 21-in.	12,800
–	–	800 ft of 24-in.	13,100
–	–	700 ft of 27-in.	12,000
–	–	900 ft of 30-in.	17,400
–	–	212 intersection crossings, includes catch basin reconnections	74,700
Subtotal, local storm drains			712,300
Total contract cost, partial separation			1,112,200
Engineering and contingencies, 25 per cent			278,000
Total construction cost, partial separation			1,390,200

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Table 18-7. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Complete separation[c]			
Relief sanitary sewers			
–	3.5 – 4.5	1,940 ft of 15-in. at 0.48 – 0.70%, to replace existing 54 – 60-in. which is to be used as storm drain	62,300
Trunk storm drains			
–	180 – 210	490 ft of 54-in. RC at 0.8 – 1.45%	35,500
–	179	350 ft of 51-in. RC at 0.8%	23,400
A-1	163	320 ft of 42-in. RC at 2.2%	13,000
A-2, A-3	62 – 100	1,620 ft of 30-in. RC at 7.7 – 16.7%	32,200
A-4	19 – 28	560 ft of 18-in. RC at 2.9 – 6.7%	8,300
A-5, A-6	32	460 ft of 30-in. RC at 0.8 – 1.3%	10,500
A-6	26	210 ft of 24-in. RC at 1.4%	3,300
A-7	44	270 ft of 33-in. RC at 0.8 – 5.8%	6,700
A-8	22	890 ft of 24-in. RC at 0.6 – 15.3%	14,100
A-9	54 – 58	3,600 ft of 30-in. RC at 2.3 – 10.0%	100,300
–	–	Connections to existing trunk which is to be utilized as storm drain	1,500
Subtotal, trunk storm drains			248,800
Local storm drains			
–	–	34,100 ft of 8-in.	182,400
–	–	15,600 ft of 10-in.	97,300
–	–	15,600 ft of 12-in.	139,600
–	–	19,600 ft of 15-in.	210,400
–	–	10,100 ft of 18-in.	126,700
–	–	4,200 ft of 21-in.	57,000
–	–	1,600 ft of 24-in.	24,600
–	–	620 ft of 27-in.	11,000
–	–	1,000 ft of 30-in.	19,500
–	–	600 ft of 33-in.	13,300
–	–	170 ft of 48-in.	6,000
–	–	212 intersection crossings, includes catch basin reconnections	74,700
–	–	2,930 new house connections	879,000
Subtotal, local storm drains			1,841,500
Total contract cost, complete separation			2,152,600
Engineering and contingencies, 25 per cent			538,200
Total construction cost, complete separation			2,690,800

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-7 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Routes of trunk drains are generally as shown in Fig. 18-7.*

At present, the sewered area within the city of Seattle comprises about 67.5 square miles, of which approximately 58.5 square miles, or 37,000 acres, are served by combined sewers. Using this area and an average cost for partial separation of $1,860 per acre, the total cost for the entire city amounts to about $69 million. While it is true that this method of calculation results in no more than an approxima-

Table 18-8. Summary of Separation Costs

District	Area, acres	Partial separation		Complete separation	
		Total cost,[a] dollars	Unit cost,[a] dollars per acre	Total cost,[a] dollars	Unit cost,[a] dollars per acre
6 – Briarcliff	150	253,600	1,690	477,000	3,180
17 – Wedgewood	480	873,000	1,820	2,196,000	4,570
23 – Southwest Seattle	960	1,735,000	1,810	3,495,800	3,640
29 – South Magnolia	530	893,500[b]	1,690	1,890,000[c]	3,570
33 – Madison Park	100	173,900	1,740	369,100	3,690
58 – East Madison	640	1,390,200	2,170	2,690,800	4,200
Total or average	2,860	5,319,200	1,860	11,118,700	3,890

[a]*Includes engineering and contingencies.*

[b]*Plan II, see Table 18-6.*

[c]*Plan II, Alternative 2, see Table 18-6.*

tion of the total cost which would thus be incurred, the resulting figure is nevertheless considered to be realistic and should be of value in planning for any separation program that might be undertaken in the future.

Because of the magnitude of the work involved in such a project, it is evident that separation will have to be undertaken as a long-range program. To that end, a program should be planned and initiated under which the first step would involve correction of the most serious flooding conditions and elimination of overflows to Lake Washington. While the latter will be alleviated by construction of storm water holding tanks as recommended in Chapter 15, the size of these tanks could be reduced materially by an effective program of storm water separation. Obviously, therefore, separation in the Lake Washington drainage basin is of paramount importance not only because of the troubles presently being experienced throughout most of the basin with sewage backups (Chapter 8) but because of the saving which could be realized through the construction of smaller holding tanks.

Based on an area of 9,200 acres in the Lake Washington drainage basin within the city of Seattle and an area of approximately 5,000 acres in other locations where serious conditions have developed, the total area to be considered in first-stage construction of separation facilities amounts to approximately 14,000 acres. On that basis, the total cost of first-stage construction would amount to about $26 million. By constructing only those facilities which are required to obtain immediately relief, this cost probably could be reduced to about $18 million. Following the first-stage program, separation should be undertaken throughout the city on a planned yearly basis until all deficiencies associated with the operation of a combined system have been adequately corrected.

Construction of New Sanitary Sewers

Although only minor problems have been encountered in the operation of combined sewers in districts which are not presently fully developed and therefore not completely sewered, it is evident from the information presented herein that the trunks serving many of these districts lack the capacity required to accommodate the flow from a 10-year storm. Further, it is no doubt true that most of the other sewers within these districts are similarly lacking in capacity. With continued development and with further installation of combined sewers, the problems attendant thereto will increase proportionately. Such a situation is bound to result eventually in relatively large expenditures for storm water separation. To reduce future costs to the greatest possible extent, it is essential that all new sewer construction in the city of Seattle be undertaken on the basis of providing separate sewers for sanitary sewage and storm drainage.

In addition to the construction of separate systems, all new buildings, even when situated in areas served by combined sewers, should be required to install separate lines for sanitary sewage and storm water. Under such an arrangement, both lines would be connected initially to the combined system. Eventually, however, following construction of a storm drain, the storm water line would be reconnected. This procedure will eliminate the need for new roof leader connections and thus will serve to reduce the construction cost of new storm drainage facilities.

Construction of New Storm Drains

Only one study was made of a program under which storm drains would be designed to pick up all runoff from streets and roofs, and foundation drains would discharge to the existing system. Cost analyses for this one area (District 29 - South Magnolia) indicate that such a program is both feasible and logical. This type of separation would be particularly economical

in similar districts where extensive construction of relief sanitary sewers would otherwise be required.

In the six areas selected for study, the cost of relief sanitary sewers runs as high as 50 per cent of the total separation costs. Elimination of these relief sewers by the procedure noted above would offset to a large extent the cost of providing capacity for the total storm runoff plus the cost of installing new roof leader connections. It appears, therefore, that the most desirable program of separation would involve the provision of a storm drainage system having a capacity sufficient to handle the total storm runoff, including that contributed by roof leaders. Under such a program, storm drains could be constructed initially to intercept street drainage only. Connection of roof leaders could be undertaken subsequently to the extent necessary to relieve the existing combined sewers of storm flows in excess of their capacities.

New storm drains should be designed and constructed in accordance with standard practices. These practices were discussed briefly in Chapters 13 and 17 and cover such items as use of gutters for conveyance of storm water, street inlet design and spacing, and methods of laying pipe.

Chapter 19
FINANCING OF RECOMMENDED FACILITIES

One of the problems common to all major public works projects, including sewerage and storm drainage, is that of developing adequate and economical procedures for financing both their construction and their subsequent operation and maintenance. A closely associated problem, particularly in a metropolitan area, is that of obtaining appropriate enabling legislation under which a project can be administered on a sound and equitable basis. To avoid potential difficulties in financing and administration, it becomes necessary in some cases to enact new laws or to modify existing laws.

In general, financial and legal problems pertaining to public works projects should be assigned to experts in those fields. An engineering report can be of assistance, however, by providing essential preliminary information. In the case of a metropolitan undertaking, this can be achieved by reviewing:

1. The financial resources of the area and the powers and limitations of a metropolitan agency with respect to the use of those resources.
2. The bases for development of financing programs, including both fundamental concepts and specific costs.
3. The approximate magnitude of total annual costs, including those of the construction program and of administration, operation and maintenance of the required facilities.

This chapter contains basic information relating to the above listed items and outlines each of several alternative plans for financing the recommended sewerage and drainage projects. In submitting these plans, it should be emphasized that they are for exploratory and illustrative purposes only and are not to be regarded as specific recommendations.

In line with the balance of this report, this chapter presumes participation of the entire study area in the metropolitan sewerage and drainage programs. In so doing, it is recognized that a metropolitan agency comprising a smaller area may be formed initially. It is recognized also that the city of Seattle is financially capable of undertaking construction of the metropolitan facilities lying within its boundaries. But with problems to solve and conditions to correct which are area-wide in scope, it is obvious that remedial action will require not only a cooperative approach but a coordination of effort through some form of metropolitan sewerage agency. The extent of the area which thus might be served initially is a matter of political rather than engineering decision and obviously is beyond the province of the study here reported.

FINANCIAL RESOURCES OF THE METROPOLITAN AREA

Resources of the metropolitan Seattle area which can be made available for financing the recommended sewerage and drainage projects comprise those funds which could be raised by (1) ad valorem property taxes, (2) service charges and connection fees, and (3) special assessments against benefited properties. The amount of revenue which can be obtained in the future from these sources depends on (1) the assessed valuation of the property within the metropolitan area, (2) the number of residential, commercial and industrial sewer services, and (3) the powers and limitations of a metropolitan agency as authorized under present legislation, or as it may be amended. In turn, assessed valuations and the number of sewer services are related to future increases in population.

Application of Population Forecasts

In general, the planning of sewerage and drainage works should be based on a forecast of the greatest population growth which might reasonably occur (Chapter 5). For financing purposes, however, the forecast of the least growth likely to occur must be considered. In the present study, a low projection was derived for the state as a whole and its relation to the high projection was expressed as a percentage of the high projection for each tenth year (Table 5-8). These percentage values are applicable also to the metropolitan Seattle area. When so applied, the low population projections are found to be 900,000 for 1960, 1,015,000 for 1970, and 1,100,000 for 1980.

Projected Growth of Assessed Valuation

The extent to which revenue can be obtained through the medium of an ad valorem tax will depend on the assessed valuation of property within the metropolitan area. It is necessary, therefore, to determine what increases can be expected during the period allowed for construction of the principal facilities. This requires a determination of present values and an application thereto of ratios based on the predicted lowest rate of population growth.

No information is available concerning the present assessed valuation of real and personal property within

the study area. Nevertheless, a reasonably accurate figure can be developed from county assessors' records. In King County, the assessed valuation for 1956, upon which 1957 taxes are levied, was $909,637,000 and amounted to $1,080 per capita. Within the city of Seattle, as would be expected, the per capita value was somewhat higher and amounted to $1,138 in 1956.

In 1956, the assessed valuation of all King County school districts lying either entirely within or largely within the study area was 96.5 per cent of that of the county valuation. At the same time, 95.2 per cent of the county population resided within the study area. On that basis, the per capita assessed valuation within the King County portion of the study area was close to $1,100 per capita.

In Snohomish County, the total assessed valuation in 1956 was $108,209,000 and amounted to $812 per capita. Although the city of Everett, which is outside the study area, and the Everett School District, which is partly in the area, both had per capita valuations of $1,100, values in other portions of the area within Snohomish County apparently were much lower. In the Edmonds School District, which is in the western part of the study area, the assessed value was approximately $640 per capita. In the city of Mountlake Terrace, it was only $360 per capita. At present, however, the Snohomish County portion contains less than 4 per cent of the study area population.

A threefold increase in assessed valuation in King County since 1940 (Fig. 19-1) represents not only the increase in real wealth but the effect of inflation. A similar situation is reflected by the rise in per capita values from $600 in 1940-42 to $1,080 in 1956.

A constant per capita value of $1,100 was assumed in projecting the future assessed valuation. Based on the low population projection, it is estimated that assessed valuations of the study area will be $990 million in 1960, $1,116.5 million in 1970, and $1,210 million in 1980. Values for intermediate years are given in Table 19-1.

Projected Increases in Service Connections

Based on data obtained from the various sewerage agencies (Chapter 6), the total number of service connections in the study area outside Seattle amounts to about 16,000. Specific figures for Seattle are not available. This is because every water service connection, including all those to premises utilizing septic tanks, pays a sewerage service charge. It is estimated, however, that approximately 13,700 private disposal systems are presently in use within the city.

The situation in Seattle is further complicated by the large number both of commercial and industrial establishments and of multiple unit dwellings. Seattle Water Department records for 1956 show 141,000 accounts within the city. Based on the estimated population of 561,000 for that year, the number of persons per account is 4.0 as compared to a range of 3.0 to 3.5 in suburban areas outside the city.

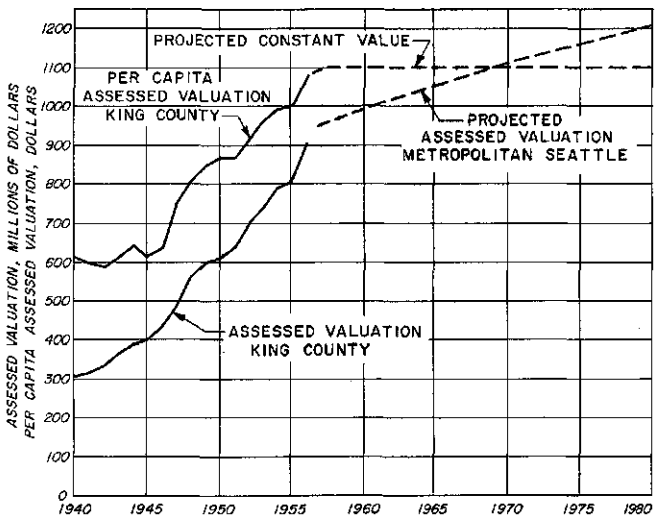

Fig. 19-1. Past and Projected Assessed Valuations, 1940-1980

Past increases in assessed valuation result from increase in real wealth and the effects of inflation. The projected valuation for the metropolitan area is based on a constant per capita value of $1,100 and on the low population projection.

Due to the large proportion of multiple housing, commercial and industrial accounts within the city, the average revenue per account is much higher than that indicated by the present residential service charge of $12.00 per year. In 1956, for example, the total revenue was $2.75 million, or an average of $19.50 per account. Expressed in another way, the total revenue is equivalent to that which would be obtained from 239,000 single family services. On a population basis, the latter number is equal to one such service for each 2.35 persons in the city.

To estimate the future number of service connections, expressed as equivalent single-family services, a figure of 2.8 persons per sewer service was assumed for the area within the present city limit. This value is somewhat higher than that developed above but was used in order to avoid possible over-estimation upon expansion of the system in residential areas. For all other parts of the study area, a value of 3.5 persons per service was assumed and is the highest of the range reported by the suburban sewerage agencies.

To utilize the foregoing values, separate low projections of population and estimates of the number of persons served by public sewerage agencies were developed both for Seattle and for the balance of the study area (Table 19-1 and Fig. 19-2). In estimating the population served within Seattle, it was assumed that the present program of sewer system extension will continue beyond 1960 and that virtually the entire population will be served by 1970.

In the remainder of the study area, the number of persons served can be expected to increase rapidly after 1960 as trunk facilities become available. On the other hand, as private disposal systems are abandoned in the more densely settled areas, new residential construction in fringe areas can be expected to continue and the number of persons dependent on private systems will, by comparison, decline quite slowly. Outside Seattle, the number served is expected to increase from 70,000 in 1960 to 385,000 by 1980. In the same period, the number of persons not served is expected to decline only from 236,000 to 115,000. This outlook could be improved by more effective control of the location of residential development than has heretofore been practiced. It could be improved also by strengthening state laws to provide for abatement of health hazards through the installation of sewers under local improvement district proceedings.

For the study area as a whole, the number of equivalent single-family services is expected to increase from 210,000 in 1960 to 324,000 in 1980. Estimated numbers for intermediate years are given in Table 19-1.

Financing Powers and Limitations of a Metropolitan Agency

General powers and limitations of a metropolitan agency in the State of Washington are defined by the Metropolitan Municipal Corporation Act of 1957 and were described briefly in Chapter 9. With respect to financing sewerage and drainage functions, such a corporation has power:

1. To levy a one-mill one-year tax. Upon appropriate approval of the electorate at the formation election, a general tax levy of one mill upon all taxable property may be authorized for one year only.
2. To assess component cities and counties for supplemental income.

Table 19-1. Projection of Assessed Valuation and Number of Sewerage Services

Year	Study area		Within Seattle[c]			Balance of study area			Study area	
	Low population estimate,[a] 1,000	Assessed valuation,[b] million dollars	Low population estimate, 1,000	Population served,[d] 1,000	Number of services,[e] 1,000	Low population estimate, 1,000	Population served,[d] 1,000	Number of services,[e] 1,000	Population served,[d] 1,000	Number of services,[c] 1,000
1957	864	950	572	522	186	292	56	16	578	202
1960	900	990	575	535	190	325	70	20	605	210
1961	912	1,003	577	540	192	335	90	26	630	218
1962	924	1,016	578	545	194	346	110	31	655	225
1963	936	1,029	579	550	196	357	130	37	680	233
1964	948	1,042	581	555	198	367	150	43	705	241
1965	960	1,055	583	560	200	377	170	49	730	249
1966	971	1,068	584	565	202	387	193	55	758	257
1967	982	1,080	586	570	204	396	216	62	786	266
1968	993	1,092	587	575	205	406	239	68	814	273
1969	1,004	1,104	589	580	207	415	262	75	842	282
1970	1,015	1,116	590	585	209	425	285	81	870	290
1971	1,024	1,126	591	587	210	433	295	84	882	294
1972	1,033	1,136	592	588	210	441	305	87	893	297
1973	1,045	1,146	593	590	211	452	315	90	905	301
1974	1,051	1,156	594	591	212	457	325	93	916	305
1975	1,060	1,165	595	593	212	465	335	96	928	308
1976	1,068	1,174	596	594	212	472	345	99	939	311
1977	1,076	1,183	597	596	213	479	355	102	951	315
1978	1,084	1,192	598	597	213	486	365	104	962	317
1979	1,092	1,201	599	599	214	493	375	107	974	321
1980	1,100	1,210	600	600	214	500	385	110	985	324

[a] *High projection for study area multiplied by percentage ratio of high to low projections for State of Washington, from Table 5-8.*

[b] *At $1,100 per capita.*

[c] *As defined by 1957 limit.*

[d] *By any public sewerage agency.*

[e] *Equivalent single family swelling services, at 2.8 persons per service within Seattle and 3.5 persons per service in balance of study area.*

3. To fix rates and to charge for the use of metropolitan sewerage and drainage facilities.

4. To issue general obligation bonds, upon appropriate vote of the electorate, and to levy an ad valorem property tax to cover principal and interest payments thereon.

5. To issue revenue bonds, without vote of the electorate, payable solely from revenue and special assessments of the utility which they finance.

6. To levy special assessments against property specially benefited, to form local improvement districts therefor, and to issue warrants or bonds payable entirely from such special assessments.

7. To apply for and receive grants-in-aid from federal or other sources.

The foregoing powers are limited by state constitutional amendments and statutes applicable to all municipal corporations. Limitations thus imposed include:

1. Approval of the electorate for the one-mill one-year tax levy and for authorization of general obligation bonds requires a three-fifths majority vote.

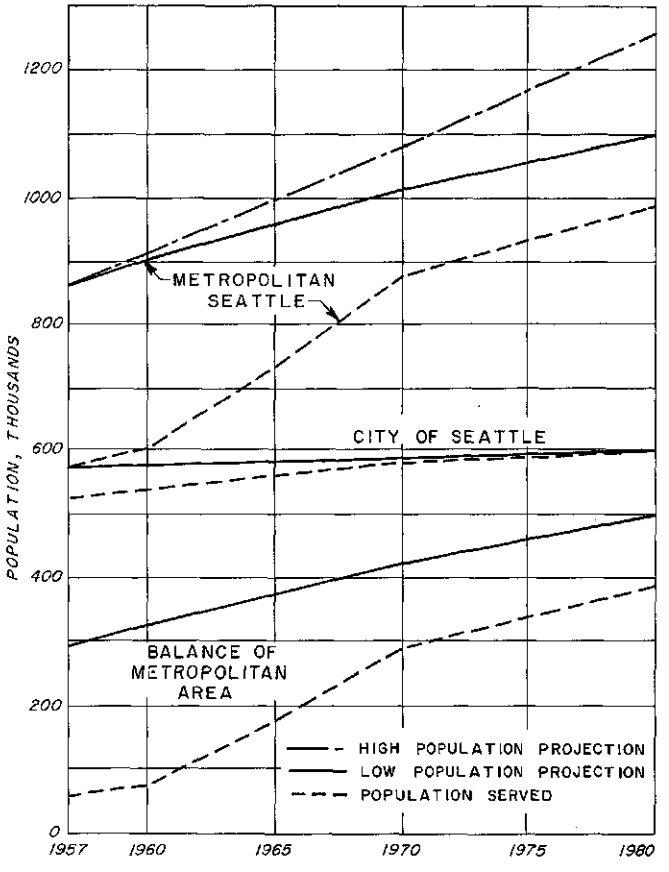

Fig. 19-2. Low Population Projections and Estimate of Population Served by Sewerage System, 1957-1980

Differences in growth rate and percentage of population served by public sewers requires separate projections for Seattle (within present city limit) and for balance of study area.

Further, the number voting must be not less than 40 per cent of the number of votes cast in the last preceding general election.

2. General obligation debt cannot exceed 5 per cent of assessed valuation. Although the Metropolitan Municipal Corporation Act permits deferment of principal repayment on a general obligation bond issue for five years, it requires, in common with other municipal statutes, that the sum of annual principal and interest payments be such that it will be met by equal annual tax levies. While the act does not limit the maximum term of bonds, the 30-year limit for other municipal corporations may be presumed to apply also to a metropolitan agency.

3. The cost of any project to be constructed under special assessment proceedings cannot exceed an amount equal to the true value of the land plus one-fourth the value of improvements on the land. By law, the true value is defined as twice the assessed value, even though the latter in recent years has been only about 20 per cent of actual value. This limit can be exceeded if the property owners involved deposit cash in the amount the project exceeds the limit. In the case of a sanitary sewerage project, the limit can be exceeded if the governing board, by unanimous vote, finds the project necessary to the public health. In any event, assessment proceedings can be nullified by written protest of property owners representing 60 per cent of the value of the assessments. Although such bonds are ordinarily limited to a maximum term of 12 years, the maximum may, under certain conditions, be extended to 22 years.

4. No ad valorem property tax, other than the one-year one-mill initial levy and that required for general obligation bond debt service, is permitted.

Under the above limitations, costs of operation, maintenance and administration of any facilities or services can be met only from service revenue or from supplemental assessments against the component cities and counties. To provide income from a property tax would require either (1) the repeal of a 1934 amendment to the state constitution, which limits ordinary property tax levies to a total of 40 mills, or (2) the raising of assessed valuations to 50 per cent of actual value (as stipulated by the state constitution) and reallocation of allowable levies within the 40-mill limit.

Supplemental Income. Resources available as supplemental income through assessments upon constituent cities and unincorporated portions of county areas are not restricted by the act. Such assessments, nevertheless, are under the control of the governing council, whose members represent the several local agencies. In general, assessments would be limited normally to amounts required for (1) administrative

costs not chargeable to specific functions, (2) minor budgetary deficits, and (3) services which supplant a municipal service and for which other means of reimbursement are not available.

Property Taxes and General Obligation Bonds. An initial one-mill one-year levy would, in 1960, raise $990,000 if applied over the entire study area. Maximum general obligation bonding capacity would be $49.5 million in 1960 and would increase to $60.5 million by 1980.

Service Charges and Revenue Bonds. Resources with respect to revenue bonds are limited only by the amount of service charge which the public will accept as necessary to correct present deficiencies and to provide adequately for the future. This in turn depends first upon public recognition of the sewerage and drainage problems, and second upon public assumption of the responsibility for corrective action. While a determination of the upper limit of the resources which may thus be made available is beyond the scope of an engineering report, the general magnitude can be indicated for illustrative purposes.

In considering the use of service charges, it is important to bear in mind the fact that the revenue therefrom must also support operation, maintenance and administrative costs of the central sewerage facilities. After allowing for those costs, each additional dollar per month per equivalent single-family residential service would, in total, support capital improvements of about $2.5 million per year by 1960, and of $3.9 million per year by 1980. For 30-year revenue bonds at 5 per cent interest and equal annual debt service payments, the revenue derived from one additional dollar per month per service would support bond issues in the amount of $39 million in 1960 and of $60 million by 1980.

Special Assessments. In general, resources available through special assessments under local improvement district proceedings are not applicable to the financing of metropolitan sewerage works or major drainage facilities.

Federal Aid. Grants may be available for construction of interceptor sewers, treatment works, and outfalls under Public Law 660 of the 84th Congress, entitled "Water Pollution Control Act Amendments of 1956". These grants are limited to 30 per cent of the cost of a project, or to $250,000, whichever is the lesser, and are to be obtained from appropriations of $50 million per year over a 10-year period. Within each state, not more than one-half the annual allocation to the state can be approved for cities having a population in excess of 125,000.

Since the annual allocation to the State of Washington has been approximately $750,000, only one of the three cities or metropolitan areas having a population in excess of 125,000 is eligible for a maximum grant each year. It is not likely, therefore, than any one of the three would receive such a grant in two successive years.

Actually, the federal aid program is geared primarily to the needs of small cities undertaking necessary construction of a single project. How it will be administered in the case of a metropolitan project with construction undertaken as a continuing program has not yet been revealed. In any event, since the awarding of such grants depends upon annual appropriations by Congress, their availability cannot be assumed in developing a financing program for the central sewerage project.

BASIS FOR DEVELOPMENT OF FINANCING PROGRAMS

Before undertaking the development of alternative programs for financing the sewerage and drainage projects, it is desirable to set forth the basic considerations and data which govern such programs. These include: (1) fundamental concepts of a metropolitan financing program; (2) characteristics of various types of bond issues and income sources as related to these concepts; (3) procedures with respect to reimbursement of existing sewerage agencies for facilities to be abandoned or incorporated in the metropolitan projects; (4) procedures with respect to other administrative operations which may affect financing programs; and (5) annual requirements for capital and for operation and maintenance revenue as affected by the timing of the construction program.

Concepts of Metropolitan Financing

In undertaking any major public works project, the financing program must not only provide adequate funds, but should accomplish this in a manner which, insofar as practicable, is equitable to those bearing the burden. This is particularly important in the case of metropolitan sewerage and drainage projects where the costs are borne by a number of different cities and local government agencies. Three objectives must be considered. These are:

1. Costs of the projects should, in general, be borne by those benefited.
2. Total costs of financing should be kept as low as possible.
3. Interference with other tax-supported needs should be held to a minimum.

Unfortunately, these objectives are not all compatible. For example, the least total outlay for interest charges would obtain with a short-term bond issue. Under such a program, the costs would be borne largely by present users of the system, whereas the

facilities to be constructed would serve for many years in the future. Further, high amortization costs of short-term general obligation bonds would interfere with other critical community needs supported by taxation. For these reasons, none of the three objectives can be met fully and compromises have to be made.

Benefits of a Metropolitan Sewerage and Drainage Program

If the cost of a central sewerage system is to be borne by those benefited, it is well to consider what benefits are derived and who gains thereby.

As an approach to the question of benefits, it is desirable to list the three principal functions of a public sewerage system. These functions are (1) to remove sewage and wastes from individual premises, (2) to transport it to a suitable location for treatment and disposal, and (3) to treat it for final disposal. In a metropolitan system, the first function is a local responsibility while the second and third are the responsibility of the metropolitan agency. Insofar as the individual householder is concerned, his direct benefit is the same regardless of whether the transmission, treatment, and disposal facilities are adequate or non-existent. In that respect, a sewerage system is unlike a water, electric or gas utility wherein the adequacy of supply and transmission facilities directly affect the quality of the service.

The benefits of sewerage facilities to be provided under a metropolitan agency, as distinct from those provided by local facilities, pertain to the protection both of public health and of the natural resources of the entire area. They accrue equally to all residents, whether or not served by public sewers. Additionally, metropolitan facilities are designed not only to serve the present population but to provide necessary capacity for future generations. It is obvious, therefore, that it is entirely equitable and proper to spread the cost of a metropolitan system equally over the widest possible base in terms of area and time.

Equal areal distribution of the cost implies that the property tax or service charge be identical throughout the metropolitan area regardless of the cost of serving the individual sewerage areas. For example, the principal trunk sewer, treatment, and disposal facilities of several of the small independent systems discharging to Puget Sound cost more on a per capita or acreage-served basis than those of the central system. Nevertheless, such facilities would be provided without a differential in cost to residents of the independent areas. Since the benefits so derived, namely protection of public waters and beaches, accrue to the public at large, this procedure is amply justified.

Trunk drainage facilities, while not necessary to protect public health or the esthetic and economic value of surface waters, are nevertheless essential to urban development of their tributary areas. Such facilities not only prevent property damage and inconvenience in the lower parts of a watershed but provide in the upland parts for installation of local drainage on a scale necessary to obtain complete development. In those upland areas where the topography, as a result of glaciation, hampers natural drainage, urban development either may be extremely limited or may be halted until trunk drains are made available.

Obviously, direct benefits of trunk drainage accrue primarily to residents and property owners, both present and future, in an entire watershed. But indirect benefits of considerable significance accrue to the metropolitan area as a whole. Of these, the protection of main thoroughfares against flooding and damage is the most obvious. Less tangible but of greater significance is the fact that provision of trunk drainage facilities allows orderly development of areas which otherwise would remain dormant or become substandard or slum neighborhoods. Orderly urban development results in new residential, commercial, and industrial areas which become an asset rather than a liability to the rest of the metropolitan community. It is entirely proper, therefore, to recognize the metropolitan interest in establishing a financing pattern for trunk drainage facilities.

Metropolitan Financing by Bond Issues

Financing by means of three types of bond issues, namely general obligation, revenue, and special assessment, is authorized under the Metropolitan Municipal Corporation Act. Each of these differs with respect to its suitability for metropolitan use.

General Obligation Bonds. Because they are backed by the total assets of a community and, in common with other local government bonds, are exempt from federal income tax, general obligation bonds bear the lowest interest rate of any type of long-term security. When used to finance projects of general public benefit, they have the further advantage of spreading the cost over the entire community on a generally equitable basis through an ad valorem property tax. In addition, debt service costs can be met in part or in whole from other sources of revenue, including service charges. General obligation bonds, therefore, meet two principal objectives, namely, equitable cost distribution and minimum cost financing.

Disadvantages of general obligation bonds include the three-fifths majority vote required for authorization and the indebtedness limitation of 5 per cent of assessed valuation. In the latter case, use of such bonds for sewerage purposes may seriously deplete borrowing capacity for projects, such as metropolitan

storm drainage and parks, which cannot be financed by other means.

During the past year or two, interest rates on general obligation issues exhibited a general increase but began to taper off somewhat in the spring and early summer of 1958. Although a rate of 4 per cent per year was assumed for the purpose of the present study, the current trend indicates that lower rates may again prevail in the near future.

Revenue Bonds. Revenue bonds offer the advantages of ease of authorization, unlimited bonding capacity, and noninterference with tax-supported governmental functions. In addition, since bond terms are governed only by their effect on the marketability of an issue, considerable flexibility with respect to repayment terms can be arranged.

Interest rates on revenue bond issues vary over a considerable range and depend upon many factors. These include the financial stability and reserves of the issuing agency, its past and anticipated revenues and expenditures, and, of course, market conditions at the time of sale. In general, the interest rate is three-fourths to one percentage point higher than that for general obligation bonds of the same agency. For the purpose of this report, a rate of 5 per cent was assumed, although present conditions indicate a probable return to somewhat cheaper money.

In addition to the higher interest rate, revenue bonds have an inherent disadvantage in the limited scope of the income which can be applied to the financing of central sewerage and other projects of general public benefit. Under present law, except as noted below, revenue can be derived only from those directly served. Hence, those who are served initially must bear the cost of the capacity which must be provided both to accommodate new areas and to allow adequately for future growth in population and industry.

A second factor leading to inequitable cost distribution in revenue bond financing is the necessity, in order to market such bonds, of maintaining revenues high enough to produce surplus income in an amount equal to fifty per cent of the annual debt service. If not siphoned off to support other governmental activities, the surplus can be used directly for construction, for retirement of callable bonds, or for deposit in a sinking fund established for bond redemption. In all of these, the end result is the same, that is, the capital cost is paid off in a much shorter time than called for by the bond terms and those initially served assume a disproportionate share of the total financial burden.

Under existing state law, the scope of sewerage revenue bond support can be broadened if water supply and sewerage functions are merged in a single utility. In that case, those served by the water system also can be charged for sewerage service regardless of whether they are actually connected to a sewerage system. No consideration is given herein to such a possibility.

As provided for under the enabling legislation, a metropolitan municipal corporation may establish rates and charges for the use of metropolitan drainage facilities. Such a corporation, therefore, could charge cities and other local agencies for drainage service. But there is no practicable way whereby this cost can be passed along to individual property owners on a service charge or tax basis. Moreover, the boundaries of political entities do not coincide with topographic features which define watersheds for drainage purposes. Fundamentally, then, revenue bonds are not appropriate for financing construction of trunk storm drainage facilities. Although the financing of facilities required for storm water separation is a possible exception, separation is primarily a local rather than a metropolitan problem.

Special Assessment Bonds. Local sewerage and drainage collection facilities are financed normally by special assessment bonds of a local improvement district. This procedure, in conjunction with funds from other sources, has been generally employed in Seattle to finance construction of combined trunk sewers.

For separate trunk sewers or drains of the magnitude proposed for the metropolitan system, the special assessment procedure has distinct disadvantages. Experience in general indicates extreme difficulty in obtaining sufficient unanimity among the property owners in a large service area or watershed to permit such a procedure.

Administrative, legal and engineering costs involved in special assessment proceedings are high and interest rates on the bonds are higher than those on general obligation or revenue bonds. And finally, although the proceedings provide for a presumably fair distribution of costs among those benefited, the short term allowed for the life of the bonds prevents an equitable spread of the costs to future residents. For these reasons, no consideration is given to the use of special assessment bonds for financing any of the metropolitan sewerage or drainage facilities.

Reimbursement for Existing Sewerage Facilities

Under the enabling act, a metropolitan municipal corporation is authorized to acquire or use existing facilities and properties of component cities and districts. Acquisition, lease, or contracts for joint use are to be made on such terms as may be fixed by agreement between the legislative body of the local agency and the metropolitan council.

In developing the metropolitan sewerage system

recommended in Chapter 16, certain existing facilities will be incorporated therein while others, including some recently constructed, will be abandoned. It is necessary, therefore, to consider (1) what general policy of reimbursement to the local agencies would be equitable, and (2) what effect this would have on the financing program.

Policy with Respect to Reimbursement. The value of a public sewerage facility lies in its capacity both to provide service to those tributary to it and to protect public health and natural resources. It cannot be moved to another location to serve others, nor is it a source of monetary profit. Transfer of ownership from one public body to another, therefore, is simply a paper transaction and, as in the case of local sewers and streets in an annexation proceeding, normally is effected without payment. Problems of reimbursement arise only where the burden of financing prior obligations, such as debt service on outstanding bonds, would result in inequities. For example, if the metropolitan sewerage project included a trunk sewer through each of two communities and one of these had recently constructed a trunk suitable for incorporation in the metropolitan system, it would be inequitable to require that community to carry both the burden of debt service costs for that trunk plus its full share of the metropolitan costs.

A similar situation prevails in the case of the treatment plants which now discharge to Lake Washington and are scheduled to be abandoned upon construction of central facilities. Policies with respect to reimbursement, therefore, should be based upon financial considerations rather than on an appraised value of the existing facility. In other words, the metropolitan agency, when assuming responsibility for operation and maintenance of a facility, should also assume responsibility for its capital costs.

Reimbursement of applicable capital costs may be made in a variety of ways, including:

1. A cash payment in an amount which, with accumulated interest, will pay annual debt service on outstanding bonds.
2. Issuance of refunding bonds to replace original bonds.
3. Assumption of responsibility for outstanding bonds.
4. A lease-purchase agreement under which annual payments are made in an amount equal to debt service requirements.

Choice of the method by which reimbursement is made will depend upon current interest rates, bond terms, approval of bond holders and similar factors which vary with time and place. For the purpose of this report, it is assumed that new capital will be required by the metropolitan agency in an amount equaling the approximate outstanding indebtedness for the principal sewerage works either to be incorporated in the metropolitan system or to be abandoned because of it. Bonds which have been issued to finance such works may vary as to term, but the maximum is thirty years. A 30-year life is therefore assumed equitable for the purpose of determining reimbursements value.

Some facilities, such as Seattle's Alki Point sewage treatment plant and the additions to the Lake City plant, have been financed on a cash basis, rather than through bonds. These cases, as well as others involving significant capital improvements financed from income, are treated the same as those financed by bonds.

Effect of Reimbursement. Reimbursement of local agencies for sewerage works to be incorporated or abandoned will tend to equalize sewerage costs throughout the metropolitan area. While reimbursement will not add to the total indebtedness of the metropolitan area, funds required for that purpose must be available to the metropolitan agency.

To estimate the amount required for reimbursement, information with respect to construction costs and dates was obtained from the participating cities and sewer districts (Table 19-2). Since most of the figures so obtained were actual contract costs, an arbitrary allowance of 10 per cent has been added for engineering.

Reimbursement values given in the table are equivalent to a depreciated value based on a 30-year life and on the straight-line method applied as of 1960. For that reason, facilities constructed prior to 1931, such as Seattle's North Trunk, would be transferred without reimbursement and are not listed.

The initial cost of all sewerage works listed amounts to $12,402,000. Their total reimbursement value is $9,419,000, of which $6,276,000 is for facilities to be incorporated and $3,143,000 is for facilities, mostly treatment plants, to be abandoned. It is of interest to note that 73 per cent of the reimbursement value, or $6,867,000, is for Seattle sewerage works, including Lake City Sewer District. On a per capita basis, reimbursement values are roughly the same both for Seattle and for sewered portions of the suburban area.

Financial Requirements

Before any programs of financing sewerage and drainage facilities can be considered, it is necessary to ascertain all costs which will be incurred and when they will accrue. In addition to capital requirements, these costs include those of operation, maintenance and administration. When they will accrue will depend both on the rate of progress in stage construction and on the growth of population and industry.

Two possible rates of stage construction of sewer-

Table 19-2. Reimbursement Value of Principal Sewerage Works to be Abandoned or Incorporated in Metropolitan System

Sewerage agency	Type of works[a]	Dates of construction	Initial cost,[b] $1,000	Reimbursement value, 1960,[c] $1,000		
				Incorporate[d]	Abandon[d]	Total
Bellevue Sewer District	STP	1954, 1957	469		391	391
Bryn Mawr-Lake Ridge Sewer District	STP	1952	218		160	160
East Mercer Sewer District		1954	73		58	58
Mercer Island Sewer District	PS(2), FM(3)	1957	195	157	18	175
Lake Hills Sewer District	STP	1955	33		28[e]	28
Val-Vue Sewer District	STP	1955	88		73	73
Sewerage and Drainage District No. 4	STP	1943	88		38	38
Southwest Suburban Sewer District	TS, PS(2), STP, O	1955	1,112	927		927
Auburn	STP	1950	74		49[f]	49
Issaquah	STP	1940	22		7	7
Kent	STP	1954	297		234	234
Kirkland	STP	1943-51	375		232	232
Renton	STP	1943-53	335		180	180
Seattle[g]						
Lake City Sewer District	TS, STP, O	1949-57	2,621	594	1,489	2,083
Southwest Lake Washington	TS, PS, STP	1932-52	1,755	703	106	809
Lake Union	TS	1935-58	524	371		371
Laurelhurst	PS(2), TS	1935	159	27		27
Southwest (Alki Point)	TS, PS(3), FM, STP, O	1953-59	3,709[h]	3,416		3,416
Greenwood	TS, STP, O	1949	255	81	80	161
Total			12,402	6,276	3,143	9,419

[a]*TS—trunk sewers; PS—pumping stations; FM—force main; STP—sewage treatment plant; O—outfall.*

[b]*Contract cost plus 10 per cent for engineering.*

[c]*Based on straight-line depreciation, 30-year life.*

[d]*During Stage I, except as noted.*

[e]*Stage II.*

[f]*Stage III.*

[g]*North Beach and Roxbury Heights sewage treatment plants also to be abandoned. Construction costs not available.*

[h]*Including estimated cost of works to be completed before 1960.*

age works are considered, both beginning in 1960. The first assumes that Stage I construction can be completed in five years, or by 1965, and that Stage II construction would begin immediately thereafter and would be completed by 1980. This program is hereinafter termed the 5-year Stage I construction program. The second program assumes that Stage I construction would require ten years for completion and that Stage II construction would be completed by 1980. This second program is hereinafter termed the 10-year Stage I construction program.

If the higher rate of population growth utilized for design purposes is assumed to prevail, certain facilities scheduled for early Stage II construction will be required before 1970 and possibly before 1965. It is obvious, however, that a financing program based on a low rate of population growth will, if a higher rate prevails, yield greater revenue than anticipated. Additional revenue thus obtained would be available for the construction of facilities needed earlier than herein programmed.

For reasons stated previously in Chapter 17, a stage construction program has not been outlined for trunk drainage facilities. As a consequence, discussions involving the financing of these works are limited to general principles.

Capital Funds. Capital funds will be required for the stage construction program, for reimbursement of local agencies for existing facilities, and, as discussed in Chapter 16, for construction of temporary treatment facilities. These requirements are summarized in Tables 19-3 and 19-4 for the two programs of stage construction.

It is assumed that the sum of $9,419,000 for reimbursement purposes will be required in 1960 upon acquisition of all existing facilities. It is assumed also that, if reimbursement is made in annual rather than lump sum payments, these payments would be approximately equal in total to the annual debt service on an equivalent issue of bonds. It is further assumed that the sum of $2 million allowed for construction of

Table 19-3. Summary of Capital Fund Requirements and Operating Costs for Five-Year Stage I Sewerage Program

	Stage I 1960-64	Stage II 1965-69	Stage II 1970-74	Stage II 1975-79
Capital funds required, total for period, $1,000				
Construction, permanent[a]				
Sewers	44,433	9,240	9,271	9,270
Pumping stations	4,609	445	1,135	1,271
Sewage treatment plants	26,993	550	2,697	800
Outfalls	5,180	467	271	
	81,215	10,702	13,374	11,341
Construction of temporary plants[a]	2,000			
Acquisition of existing facilities[b]	9,419			
Total	92,634	10,702	13,374	11,341
Average annual cost of operation and maintenance, $1,000				
Permanent facilities	464	1,010	1,186	1,324
Temporary facilities[c]	355	161	151	119
Administration[d]	110	130	150	150
Total	929	1,301	1,487	1,593

[a]*From Tables 16-1 and 16-12.*

[b]*From Table 19-2.*

[c]*Including existing works to be abandoned.*

[d]*Administrative, legal, and engineering costs not chargeable to specific operations.*

Table 19-4. Summary of Capital Fund Requirements and Operating Costs for Ten-Year Stage I Sewerage Program

	Stage I, thousand dollars					Stage II, thousand dollars				
	1960-61	1962-63	1964-65	1966-67	1968-69	1970-71	1972-73	1974-75	1976-77	1978-79
Capital funds required, total for period										
Construction, permanent[a]										
Sewers	9,126	8,716	8,716	8,760	8,714	5,915	5,301	5,504	5,761	5,300
Pumping stations	–	304	1,063	2,336	1,307	758	304	333	1,192	264
Sewage treatment plants	2,901	7,899	3,949	8,403	3,841	550	350	709		2,438
Outfalls	781		318	235	3,846	467		271		
	12,808	16,919	14,046	19,734	17,708	7,690	5,955	6,817	6,953	8,002
Construction of temporary plants[a]	2,000									
Acquisition of existing facilities[b]	9,419									
Total	24,227	16,919	14,046	19,734	17,708	7,690	5,955	6,817	6,953	8,002
Average annual cost of operation, maintenance and administration										
Permanent facilities	326	350	485	570	1,022	1,101	1,164	1,238	1,318	1,369
Temporary facilities[c]	406	417	312	315	178	204	119	118	115	126
Administration[d]	100	110	120	130	140	150	150	150	150	150
Total	832	877	917	1,015	1,340	1,455	1,433	1,506	1,583	1,645

[a]*From Tables 16-1 and 16-12.*

[b]*From Table 19-2.*

[c]*Including existing works to be abandoned.*

[d]*Administrative, legal, and engineering costs not chargeable to specific operations.*

temporary treatment facilities will be needed in 1960.

Funds for the 5-year Stage I construction program (Table 19-3) are assumed to be needed in more or less equal amounts per year within each 5-year increment. For the 10-year Stage I program, however, the variation in annual requirements, particularly during the first stage, necessitates the more detailed breakdown in 2-year increments (Table 19-4).

Operation and Maintenance. Annual costs of operation and maintenance include those of (1) new facilities as constructed and placed in operation, (2) existing facilities incorporated in the metropolitan system, and (3) treatment works, both existing and temporary, which must be continued in operation until replaced by permanent facilities. These costs (Tables 19-5 and 19-6) are based on the pertinent cost curves and factors given in Chapter 13 and take into account the average loading on pumping stations and treatment facilities during each incremental period. They do not include operation and maintenance of local collection facilities, either existing or to be constructed hereafter.

Fiscal and Administrative Costs. Regardless of whether capital requirements are obtained from revenue or general obligation bonds, service charges will be necessary to meet operation and maintenance costs of the metropolitan system. Practically all the local sewerage agencies now collect service charges and would continue to do so for maintenance of the local systems. To avoid duplication in billing expense, it is assumed that the metropolitan costs would be charged to the local agencies which, in turn, would add that cost to their local billings. No allowance is made, therefore, for service charge collection expense to the metropolitan agency.

Sewerage service may, in the future, be extended in unincorporated areas by local improvement district proceedings under sponsorship of the metropolitan

Table 19-5. Revenue Bond Financing of Five-Year Stage I Sewerage Program

	Stage I	Stage II		
	1960-64	1965-69	1970-74	1975-79
For period:				
Capital funds required,[a] $1,000	92,634	10,702	13,374	11,341
Revenue bonds issued, $1,000	80,000[b]			
Average number of services,[c] 1,000	225	265	297	315
Monthly service charge, dollars	3.00	2.90	2.70	2.50
Average annual amounts, $1,000				
Operating income	8,100	9,222	9,623	9,450
Debt service	3,643	5,204	5,204	5,204
Operation and maintenance[a]	929	1,301	1,487	1,552
State tax[d]	65	74	77	76
Total costs	4,637	6,579	6,768	6,832
Debt service coverage[e]	1,822	2,602	2,602	2,602
Net income	1,641	41	253	16
Total surplus	3,463	2,643	2,855	2,618
Construction from surplus	2,523	2,140	2,675	2,268
Net to reserve fund	940	503	180	350
Cumulative amounts, $1,000				
Capital fund requirements	92,634	103,336	116,710	128,051
Capital from income	12,634	23,336	36,710	48,051
Capital from revenue bonds	80,000	80,000	80,000	80,000
Bonds outstanding[f]	76,840	69,320	61,780	50,120
Reserve fund[f, g]	4,700	7,215	8,115	9,865

[a] *From Table 19-3.*

[b] *Four $20,000,000 issues of 30-year bonds at 5 per cent, issued in 1960, 1961, 1962, and 1963, with uniform annual interest and redemption payments.*

[c] *All classes, expressed as equivalent single-family residences, from Table 19-1.*

[d] *At 0.8 per cent of gross income.*

[e] *Fifty per cent of bond interest and redemption.*

[f] *At end of period.*

[g] *Exclusive of interest earned.*

Table 19-6. Revenue Bond Financing of Ten-Year Stage I Sewerage Program

	Stage I				
	1960-61	1962-63	1964-65	1966-67	1968-69
For period:					
Capital funds required,[a] $1,000	24,227	16,919	14,046	19,734	17,708
Revenue bonds issued,[b] $1,000	18,000	10,000	7,000	15,000	14,000
Average number of services,[c] 1,000	214	229	245	260	277
Monthly service charge, dollars	2.50	2.50	2.50	2.50	2.40
Average annual amount, $1,000					
Operating income	6,420	6,870	7,350	7,800	7,868
Debt service	1,171	1,821	2,277	3,253	4,163
Operation and maintenance[a]	832	877	917	1,015	1,340
State tax[d]	52	55	59	63	63
Total cost	2,055	2,753	3,253	4,331	5,566
Debt service coverage[e]	586	911	1,139	1,627	2,082
Net income	3,779	3,206	2,958	1,842	220
Total surplus	4,365	4,117	4,097	3,465	2,302
Construction from surplus	3,114	3,460	3,523	2,367	1,854
Net to reserve fund	1,251	657	574	1,098	448
Cumulative amounts, $1,000					
Capital fund requirements	24,227	41,146	55,192	74,926	92,634
Capital from income	6,227	13,146	20,192	24,926	28,634
Capital from revenue bonds	18,000	28,000	35,000	50,000	64,000
Bonds outstanding	17,700	27,000	32,930	46,420	58,370
Reserve fund[f, g]	2,502	3,816	4,964	7,160	8,056

	Stage II				
	1970-71	1972-73	1974-75	1976-77	1978-79
For period:					
Capital funds required,[a] $1,000	7,690	5,955	6,817	6,953	8,002
Revenue bonds issued,[b] $1,000	4,000	1,000	2,000	2,000	3,000
Average number of services,[c] 1,000	292	299	306	313	319
Monthly services charge, dollars	2.40	2.40	2.40	2.40	2.40
Average annual amount, $1,000					
Operating income	8,410	8,611	8,813	9,014	9,187
Debt service	4,423	4,488	4,619	4,749	4,944
Operation and maintenance[a]	1,455	1,433	1,506	1,583	1,645
State tax[d]	67	69	70	72	74
Total cost	5,945	5,990	6,195	6,404	6,563
Debt service coverage[e]	2,212	2,244	2,310	2,375	2,472
Net income	243	377	308	235	152
Total surplus	2,455	2,621	2,618	2,610	2,624
Construction from surplus	1,845	2,478	2,409	2,477	2,501
Net to reserve fund	610	143	209	133	123
Cumulative amounts, $1,000					
Capital fund requirements	100,324	106,279	113,096	120,049	128,051
Capital from income	32,324	37,279	42,096	47,049	52,051
Capital from revenue bonds	68,000	69,000	71,000	73,000	76,000
Bonds outstanding[f]	59,700	57,570	56,380	54,700	52,750
Reserve fund[f, g]	9,276	9,562	9,980	10,246	10,492

[a] *From Table 19-4.*

[b] *Serial bonds, 30-year term at 5.0 per cent interest, with uniform annual interest and redemption payments.*

[c] *All classes, expressed as equivalent single-family residences, from Table 19-1.*

[d] *At 0.8 per cent of gross income.*

[e] *Fifty per cent of bond interest and redemption.*

[f] *At end of period.*

[g] *Exclusive of interest earned.*

agency and without formation of sewer districts. For such areas, direct billing by the metropolitan agency would be necessary and the cost thereof, along with that of local sewer maintenance, would have to be added to the metropolitan charge.

Another expense in connection with service charges is a utility gross earnings tax of 0.8 per cent imposed by the state. The amount thereof will vary with the type of financing and is included later in each alternative program.

Parenthetically, the imposition in Washington of state and city taxes on the gross earnings of a sewerage utility is an application of the principle that a publicly owned utility should be taxed an amount equivalent to what it would pay if privately owned. Because privately owned sewerage utilities are practically nonexistent, there is no real basis for a tax on this type of utility. Further, few sewerage systems are actually administered as true revenue-financed utilities. Subsidies in the form of property tax support and state and federal grants have been necessary in many instances to bring about the construction of treatment facilities which are required both to protect public waters and to maintain environmental sanitation at a safe level. In view of the expressed intention of the state to encourage construction of sewerage facilities as a matter of general welfare, the imposition of a gross earnings tax on sewerage utilities is, to say the least, contradictory. Amendment of pertinent state legislation is indicated. Furthermore, if present legislation would permit component cities or counties to tax the sewerage service revenue of a metropolitan agency, the metropolitan municipal corporation act should be amended to prohibit such taxation.

Finally, allowance must be made for fiscal and administrative costs not included in the foregoing costs of construction, operation, and maintenance. While the latter include engineering, supervision, and overhead costs directly attributable to these functions, provision must be made for:

1. Expenses of the governing board of the metropolitan agency.
2. General expenses, such as for audits, annual reports, and fiscal planning.
3. Legal expenses with respect to amendments of legislation affecting the metropolitan corporation as a sewerage agency.
4. Expenses of bond elections and sales of bond issues.
5. Preliminary engineering planning, including such revisions of the plans herein recommended as are made necessary by changing conditions.
6. Monitoring studies to determine the effectiveness of the sewage treatment and disposal program.
7. Other miscellaneous expenses.

To meet the expenses listed above, an allowance of $120,000 is provided for the first two years. This allowance is increased to $150,000 per year by the seventh year.

FINANCING THE SEWERAGE PROGRAM

Two general plans are presented for financing the sewerage program, one utilizing revenue bonds and the other general obligation bonds. For each of them, two rates of progress are considered for stage construction of the metropolitan system. In presenting these plans, the purpose is to indicate the general magnitude of debt service and total annual costs which are thus incurred and the approximate service charges and tax levies needed to meet these costs.

No attempt has been made to refine bond terms and the reserve fund arrangements which will make it possible to arrive at the lowest possible costs. This is the function of financial consultants who are experienced in the municipal field and are properly familiar with bond market conditions. Costs given for the two plans, therefore, represent the upper limits of an undetermined range within which the sewerage program can be financed. These costs are expressed in terms of 1958 price levels and are subject to adjustment on the basis of further changes in the value of the dollar.

Revenue Bond Financing

The procedures involved in revenue bond financing of the recommended sewerage program can be illustrated by means of two specific examples. Of these, the first provides for construction of Stage I facilities over a period of five years beginning in 1960. This is considered to be the minimum period in which the necessary engineering and construction can be undertaken. The second example calls for construction of Stage I facilities over a period of ten years from 1960 to 1969 inclusive. In both examples, Stage II construction would follow Stage I immediately and would be completed by 1979.

Five-Year Example. In the 5-year Stage I example (Table 19-5), initial capital requirements would be met by four issues of serial revenue bonds, each for $20 million and each having a term of thirty years. Under such a program, the balance of the first-stage capital requirements, along with those of Stage II, would come from the surplus revenue which must be collected to meet bond requirements. In addition, a reserve fund would be accumulated from the surplus. This fund would amount to about 5 per cent of the total capital expenditure by the end of the first 5 years and to almost 8 per cent by the end of Stage II construction in 20 years.

Although a total reserve of almost $10 million might appear to be excessive, it actually would amount to only 5.5 per cent of the total revenue in the 20-year period. In any case, an adequate reserve would be required to protect the revenue bonds and thus to assure a minimum rate of interest. A reserve would be useful also in that it would provide working capital for the construction program as well as funds for replacements, minor improvements, and contingencies.

To support the construction program and to provide necessary funds for operation and maintenance of the sewerage system, the average annual revenue requirement will range from $8.1 million in the first five years to $9.45 million in the 16th to 20th years. Unless state laws are amended, the gross earnings tax will average $73,000 per year and will total $1.47 million for the 20-year period. In other words, the total revenue requirement for the 20 years will amount to $181,975,000. Monthly service charges indicated as necessary to produce that income range from $3.00 for the first period to $2.50 for the last period.

Fig. 19-3 illustrates the status of capital funds throughout the 20-year period. A total of $128 million will be expended for construction, of which $48 million will be derived from service charges. Of the $80 million in revenue bonds, about $30 million will be retired by 1980 and another $10 million will be offset by the reserve fund. On that basis, the net debt will be approximately $40 million. Due to the large proportion of construction financed from income, almost $88 million of the total construction cost will be paid off in the 20-year period.

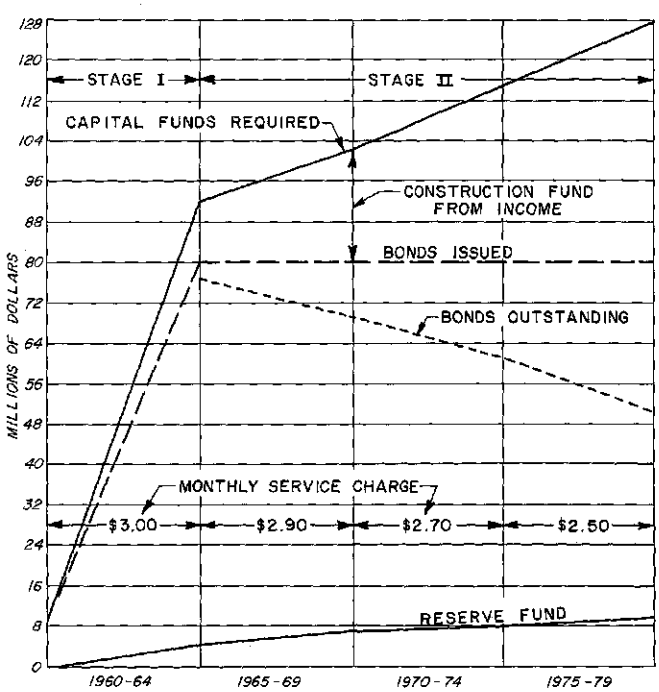

Fig. 19-3. Cumulative Funds, Revenue Bond Financing of Five-Year Stage I Sewerage Program

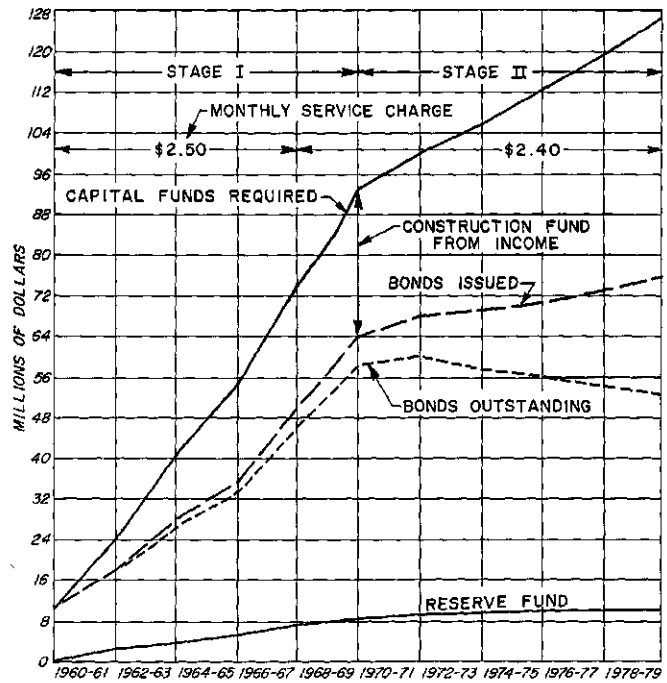

Fig. 19-4. Cumulative Funds, Revenue Bond Financing of Ten-Year Stage I Sewerage Program

Total interest charges to maturity of the four bond issues will amount to about $76 million. That amount, if applied to a 30-year bond issue of $128 million, would represent an interest rate of 3.4 per cent. In all, therefore, the 5-year program would enable relatively low cost financing but would entail a disadvantage in the spread of costs with respect to time.

Ten-Year Example. Under a 10-year program of revenue bond financing (Table 19-6), surplus income again would be used for construction purposes and would thus reduce the total bond issue requirement. During the first stage construction program, five issues totaling $64 million would be sold. During the second stage, annual requirements for construction would be greater than under the 5-year program and would have to be met by the issuance of an additional $12 million in revenue bonds. As in the first example, a reserve fund of approximately $10 million would be accumulated during the 20-year period.

Annual revenue required both to support the construction program and to operate and maintain the system would range from about $6.4 million in 1960 to about $9.2 million in 1979. Reduced costs in the first ten years, as compared to the first example, would result from the deferred construction schedule, which affects not only debt service but operation and maintenance costs as well.

As illustrated in Fig. 19-4, the value of outstanding bonds would reach a peak of slightly less than $60 million at the end of the twelfth year and thereafter would decline to $52.75 million by the end of the 20-

year period. With the accumulated reserve taken into account, the net debt at the end of the second stage of construction would amount to $42 million, or only $2 million more than that estimated under the first example.

Although it would delay construction of facilities needed now to protect Lake Washington and the Puget Sound beaches, the 10-year Stage I program would require a significantly lower revenue during the first ten years than the 5-year program. In other respects, this second example is similar to the first and results in a poor spread of costs with respect to time.

Sinking Fund Alternative. As an alternative to financing part of the construction program from surplus income, revenue bonds could be issued in amounts sufficient to cover all construction costs. Under this alternative, the surplus income could be placed in a sinking fund to retire callable bonds in advance of maturity. Provided the interest earned on the sinking fund is equivalent, or nearly so, to that paid on the bond issues, the net result with respect to financing cost would be practically identical to the examples involving construction from surplus funds. The rate of debt retirement, however, would be more rapid and, as a consequence, annual costs would be higher.

Sewer Service Charge. Present costs of local sewerage agencies for debt service, as well as those for construction, operation and maintenance of sewage works and certain trunk sewers would be assumed by the metropolitan agency and are included in the total annual costs estimated for that agency. In such an event, neither the assumed costs nor the metropolitan service charges would necessarily be added to the costs already being borne by and the service charges being paid to the local agencies. In cities and districts outside Seattle, local sewerage costs would be reduced essentially to those involved in operation, maintenance, and debt service of the collection system. While a detailed analysis of the effect on costs within these individual agencies is premature at this time, a cursory examination indicates that the sum of metropolitan and local charges would range from a nominal increase in present costs to an actual reduction in a few cases.

Within the city of Seattle, financing of metropolitan facilities is complicated by the need for extensive and correspondingly costly separation of combined sewers. In fact, all presently available revenue could be expended on separation and relief sewer construction for many years to come. If that were the case, the cost of metropolitan sewerage facilities would then have to be financed entirely from newly acquired revenue. However, by confining the separation program to the areas most seriously affected by storm flow conditions, a portion of the present revenue from sewer service charges could be utilized for participation in the metropolitan program.

Pending formation of a metropolitan agency, Seattle could finance a substantial portion of the metropolitan facilities lying within its boundaries, plus some of the most urgent separation projects. Although the first phases of these projects could be financed from present revenue, a substantial increase in sewer service charges will be required in Seattle within a few years regardless of whether a metropolitan agency is established.

General Obligation Bond Financing

At present, financing of the entire sewerage program by general obligation bonds is not legally possible. This is because bonded indebtedness cannot exceed 5 per cent of assessment valuation and also because assessed values are considerably below the legal limit of 50 per cent of true value. Nevertheless, examples of such financing are given below to illustrate the differences with respect to revenue financing.

An example of general obligation bond financing of the 5-year Stage I sewerage program is given in Table 19-7. A series of bonds, all having terms of 30 years and assumed to bear a 4.0 per cent interest rate, would be issued as needed to meet all construction requirements. Since no surplus income would be required, no construction from income would be undertaken.

Total annual costs would range from an average of $4,325,000 during the first five years to $8,777,000 during the 16th to 20th years. Total costs for the 20-year period would amount to $140.13 million, or about 77 per cent of that required for revenue bond financing.

Debt service costs could be met by a tax levy averaging 3.34 mills per dollar of assessed valuation during the first five years and 6.06 mills during the last five years of the 20-year period. Because no provision is made in the metropolitan corporation act for a property tax to cover operation and maintenance, a service charge ranging from an average of $0.35 to $0.42 per month would be necessary.

As an alternative to the tax levy and service charge shown in Table 19-7, part or all of the debt service could be included in the service charge. Total annual costs would be increased to the extent of the additional state tax thus incurred. If all costs were raised by service charges, the charge would range from an average of $1.62 to $2.33 per month respectively for the first and last 5-year periods of the twenty years.

Outstanding bonds (Table 19-7 and Fig. 19-5) during the first fifteen years would remain relatively constant near $87 million and would drop to $79 million by the end of the 20-year period. Expressed as a percentage

Table 19-7. General Obligation Bond Financing of Five-Year Stage I Sewerage Program

	Stage I	Stage II		
	1960-64	1965-69	1970-74	1975-79
For period:				
Capital funds required,[a] $1,000	92,634	10,702	13,374	11,341
General Obligation bond issues,[b] $1,000	93,000	10,000	14,000	11,000
Average assessed valuation,[c] million dollars	1,016	1,080	1,136	1,183
Average number of services,[c] 1,000	225	265	297	315
Average annual amount, $1,000				
Debt service	3,389	5,725	6,388	7,171
Operation and maintenance[a]	929	1,301	1,487	1,593
State tax[d]	7	11	12	13
Total cost	4,325	7,037	7,887	8,777
Average annual levy for debt service, mills	3.34	4.80	5.62	6.06
Average monthly service charge for operation and maintenance, dollars	0.35	0.41	0.42	0.42
Cumulative amounts, $1,000				
Capital fund requirements	92,634	103,336	116,710	128,051
Bonds issued	93,000	103,000	117,000	128,000
Bonds outstanding[e]	87,600	86,800	86,700	79,000
Estimated debt limit,[e] $1,000	52,100	55,200	57,800	60,000
Outstanding bonds, per cent of assessed valuation[e]	8.40	7.87	7.50	6.58

[a]*From Table 19-3.*

[b]*General obligation 30-year serial bonds at 4.0 per cent interest, with uniform annual interest and redemption payments.*

[c]*From Table 19-1.*

[d]*At 0.8 per cent of gross revenue from service charges.*

[e]*At end of period.*

of assessed valuation, outstanding bonds would equal 8.4 per cent at the end of the first five years and would drop to 6.6 per cent after twenty years. Total interest costs throughout the life of the bonds would amount to $95 million.

Stretching the first stage construction program to ten years (Table 19-8) would afford no significant advantage under general obligation bond financing. Annual costs during the first six years would be lower than those for the 5-year Stage I program. In both cases, the costs for the six years would be well below the average for the 20 year period.

Outstanding bonds under the 10-year first stage program would reach a maximum of $87 million by the end of the 18th year and thereafter would decline. Expressed as a percentage of assessed valuation, outstanding bonds would exceed the 5 per cent debt limit after the sixth year and would reach a maximum of 7.45 per cent in the 16th through 18th years.

Comparison of Revenue Bond and General Obligation Bond Financing

Use of revenue bonds for financing the metropolitan sewerage program would permit the following advantages:

1. Noninterference with other governmental functions which must be supported by property taxes.
2. Bonds may be authorized without a vote of the electorate. This means that the long-range construction program could be planned and undertaken in an orderly manner without the delays inherent in recurrent bond elections.
3. Lower cost of financing despite a higher interest rate. This is because less indebtedness would be incurred.

As stated earlier, revenue bond financing is subject to a disadvantage in that it would fail to provide an equitable distribution of the costs to all those who would be benefited by the sewerage program. There are two reasons for this disadvantage. First, only those connected to the sewerage system would bear the cost. Second, the surplus income requirement would result in a rapid write-off of capital costs. This in turn means that those served initially would pay also for the capacities then provided to meet future needs. As a consequence, service charges would be higher than otherwise necessary.

General obligation bonds would provide for a more equitable distribution of costs in relation to scope and time. Resulting tax levies and service charges would

Table 19-8. General Obligation Bond Financing of Ten-Year Stage I Sewerage Program

	Stage I				
	1960-61	1962-63	1964-65	1966-67	1968-69
For period:					
Capital funds required,[a] $1,000	24,227	16,919	14,046	19,734	17,708
General obligation bond issues,[b] $1,000	25,000	17,000	14,000	19,000	18,000
Average assessed valuation,[c] million dollars	996	1,022	1,048	1,074	1,098
Average number of services,[c] 1,000	214	229	245	260	277
Average annual amount, $1,000					
Debt service	1,445	2,425	3,240	4,450	5,380
Operation and maintenance[a]	832	877	917	1,015	1,340
State tax[d]	7	7	7	8	11
Total cost	2,284	3,309	4,164	5,473	6,731
Annual levy, mills	1.45	2.37	3.10	4.15	4.90
Monthly service charge, dollars	0.33	0.32	0.31	0.33	0.41
Cumulative amounts, $1,000					
Capital fund requirements	24,227	41,146	55,192	74,926	92,634
Bonds issued	25,000	42,000	56,000	75,000	93,000
Bonds outstanding[e]	24,200	39,600	51,300	67,200	81,300
Estimated debt limit,[e,f] $1,000	50,015	51,400	52,700	54,000	55,300
Outstanding bonds, per cent of assessed valuation[e]	2.42	3.86	4.87	6.22	7.37
	Stage II				
	1970-71	1972-73	1974-75	1976-77	1978-79
For period:					
Capital funds required,[a] $1,000	7,690	5,955	6,817	6,953	8,002
General obligation bond issues,[b] $1,000	7,000	7,000	7,000	7,000	7,000
Average assessed valuation,[c] million dollars	1,121	1,141	1,160	1,178	1,196
Average number of services,[c] 1,000	292	299	306	313	319
Average annual amount, $1,000					
Debt service	5,850	6,200	6,600	7,000	7,402
Operation and maintenance[a]	1,455	1,433	1,506	1,583	1,645
State tax[d]	12	12	12	13	13
Total cost	7,317	7,645	8,118	8,596	9,060
Annual levy, mills	5.20	5.43	5.68	5.95	6.18
Monthly service charge, dollars	0.42	0.40	0.41	0.42	0.43
Cumulative amounts, $1,000					
Capital fund requirements	100,324	106,279	113,096	120,049	128,051
Bonds issued	100,000	107,000	114,000	121,000	128,000
Bonds outstanding[e]	83,800	85,400	86,700	87,000	86,100
Estimated debt limit,[e,f] $1,000	56,300	57,300	58,300	59,200	60,000
Outstanding bonds, per cent of assessed valuation[e]	7.44	7.45	7.45	7.35	7.17

[a] *From Table 19-4.*

[b] *Serial bonds, 30-year term at 4.0 per cent interest, with uniform annual interest and redemption payments.*

[c] *From Table 19-1.*

[d] *At 0.8 per cent of gross revenue from service charges.*

[e] *At end of period.*

[f] *At 5.0 per cent of projected assessed valuation.*

be relatively moderate considering the magnitude of the project.

In order to use general obligation bonds for financing the entire project, the debt limit would have to be raised either by increasing the allowable percentage of assessed valuation or by obtaining a substantial increase in assessed valuation. In the absence of such changes, partial financing by general obligation bonds would be possible but would interfere with the financing of drainage improvements. For the latter, as pointed out previously, general obligation bonds offer the only feasible means of obtaining the necessary funds. Under present conditions, therefore, it is obvious that revenue bonds would provide the most practicable solution to the problem of financing a metropolitan sewerage project.

Independent Financing of Separate Sewerage Systems

While the necessity for financing and administering the central sewerage system through a metropolitan agency is apparent, there may be some question as to the advisability of such an arrangement in the case of several of the independent systems discharging to Puget Sound. Since separate financing and administration of these systems would be practicable, two questions may be raised. These are:

1. What will metropolitan Seattle gain by inclusion of the independent systems in a metropolitan plan of financing and administration?
2. What will the residents of the topographically separate sewerage areas gain by such inclusion?

An answer to the first question may be found in the discussion earlier in this chapter relating to the function or purpose of metropolitan sewerage facilities and the benefits thereby derived. Provision of sewerage service by a metropolitan agency will assure a prompt recognition of local problems, as well as prompt and effective action in providing necessary facilities for sewage collection, treatment and disposal. Under such a program, sewerage construction would be undertaken on a systematic long-range basis and future additions would be made as needed regardless of any apathy on the part of local residents or officials. Technical supervision would be made available to the independent systems, as would a monitoring program to determine the effectiveness of the treatment and disposal operations.

It is not intended to imply that it is either overly difficult or impossible for a small sewerage agency to build suitable facilities and achieve satisfactory results. With efficient management, adequate financing, and a minimum of political interference, sewerage service can be furnished just about as well by a small agency as it can be a large agency. Experience in general, however, indicates that the probability of equal performance is relatively remote.

As an answer to the second question, it is necessary to consider the problems of financing sewerage works in a partially developed but growing area. In such areas, initial financing limitations, rather than sound engineering and long-range economy, tend to govern the design of sewerage facilities. Faced with excessive costs in relation to the initial contributory population, an engineer is compelled to limit his planning to facilities which later must be duplicated or abandoned.

For the purpose of illustration, it can be assumed that facilities equal to those obtainable through a metropolitan agency would be financed locally and that equal performance could be attained at no difference in operating costs. On that basis, an analysis can be made of total annual costs and of resulting service charges for a typical independent sewerage area. Using the Des Moines sewerage area as a typical example, an analysis was made of the total annual costs which would accrue under both metropolitan financing and completely independent financing.

In 1957, the population of the area topographically tributary to the proposed Des Moines treatment plant was estimated to be 13,000. Application of the low population projection results in estimates of 14,000 by 1960, 19,000 by 1970, and 22,000 by 1980. In contrast, application of the high projection used for design purposes gives a 1980 population of 30,000.

For the purpose of revenue calculations, the number of persons served was assumed to be 7,000 by 1960, 14,000 by 1970, and 19,250 by 1980. It was assumed also that first stage construction would be undertaken in 1960 at a cost of $1,618,000 (Chapter 16). Under that program, the initial facilities would consist of trunk sewers, a primary type treatment plant, and a submarine outfall. Additional trunk sewers would be constructed in 1970-71 and 1976-77 at estimated costs of $168,000 and $253,000 respectively. In 1978, the treatment plant would be enlarged at an estimated cost of $800,000.

Estimated costs and revenue requirements for separate financing of the Des Moines sewerage program are given in Table 19-9. Debt service costs are based on 30-year revenue bonds at 5 per cent interest. After the second year, it is assumed that part of the surplus income of each preceding year would be utilized for operation and maintenance, and that the balance would be placed in a reserve fund. Construction of the trunk sewers scheduled for 1970-71 would be financed from the reserve, but additional bonds would have to be issued to finance the sewer and plant additions scheduled to start in 1976.

Operation and maintenance costs are assumed to be identical to those under the metropolitan agency. In this case, however, Des Moines' share of undistributed overhead cost of the metropolitan agency is excluded.

Annual income necessary for separate financing

would amount to $185,000 during the first two years and to $160,000 in the third through sixteenth years. It would increase thereafter and would reach $263,000 in the nineteenth and twentieth years. During the 20-year period, the estimated number of equivalent single-family service connections would increase from 2,200 to 5,350.

Service charges would be excessive initially and would amount to $7.00 per month. They would drop to a minimum of $2.80 by 1974-75 and thereafter would increase again to more than $4.00 per month. Under suitable bond terms, it would be possible to obtain an arrangement under which the annual debt service would be the same each year and the sewer service charge would be held at a constant level. In such an event, the latter would still be in excess of $4.00 per month, not including the costs of either local sewer maintenance or revenue collection.

In contrast, the costs to local residents of financing sewerage facilities through a metropolitan agency would be about 60 per cent of those incurred through separate financing. This comparison does not take into account the lower interest rates which metropolitan revenue bonds would command, nor does it include the legal, fiscal and engineering services which would be provided by such an agency.

Part of the difference between metropolitan and local financing lies in the scheduling of construction. At Des Moines, all Stage I facilities would be constructed during the first year or two, as opposed to a five to ten year spread for the metropolitan area as a whole. This means that, by the end of 20 years, a larger share of the indebtedness for first stage facilities at Des Moines would be repaid than would be the case for first stage facilities of the metropolitan area as a whole. In other words, local financing would be more costly for the period under consideration.

A second factor is that facilities at Des Moines require provision for a greater rate of growth than the average for the entire metropolitan area. For that reason, the initial costs per service connection would be higher under local financing.

Table 19-9. Independent Financing of Separate Sewerage System at Des Moines

	Stage I					Stage II				
	1960 -61	1962 -63	1964 -65	1966 -67	1968 -69	1970 -71	1972 -73	1974 -75	1976 -77	1978 -79
For period:										
Construction funds required[a]	1,618					168			253	800
Revenue bonds issued,[b] $1,000	1,618								253	800
Average number of services[c]	2,200	2,600	3,000	3,400	3,800	4,150	4,450	4,750	5,050	5,350
Monthly service charge, dollars	7.00	5.12	4.45	3.92	3.51	3.22	3.00	2.80	3.05	4.10
Average annual amount, $1,000										
Operating income	185	160	160	160	160	160	160	160	185	263
Debt service	106	106	106	106	106	106	106	106	122	174
Operation and maintenance	24	24	27	29	32	36	41	44	47	59
State tax	2	1	1	1	1	1	1	1	2	2
Total cost	132	131	134	136	139	143	148	151	171	235
Debt service coverage[d]	53	53	53	53	53	53	53	53	61	87
Net income[e]	0	(24)	(27)	(29)	(32)	(36)	(41)	(44)	(47)	(59)
Total surplus	53	29	26	24	21	17	12	9	14	28
Construction from surplus	0	0	0	0	0	168	0	0	0	0
Net to reserve fund	53	29	26	24	21	(151)	12	9	14	28
Cumulative amount, $1,000										
Construction fund requirements	1,618	1,618	1,618	1,618	1,618	1,786	1,786	1,786	2,039	2,839
Construction from income	0	0	0	0	0	168	168	168	168	168
Construction from bonds	1,618	1,618	1,618	1,618	1,618	1,618	1,618	1,618	1,871	2,671
Bonds outstanding[f]	1,590	1,530	1,480	1,420	1,350	1,270	1,190	1,080	1,240	1,900
Reserve fund[f, g]	106	164	216	264	306	155	179	197	225	281

[a] *From Table 16-1 and 16-12.*

[b] *Thirty-year serial revenue bonds at 5 per cent interest with uniform annual interest and redemption payments.*

[c] *Based on low population estimate of 14,000 by 1960 and 22,000 by 1980.*

[d] *Fifty per cent of debt service cost.*

[e] *Operation and maintenance costs, in effect, supplied from surplus of preceding year.*

[f] *At end of period.*

[g] *Exclusive of interest earned.*

Granting that it is to the advantage of the independent sewerage areas to participate in the metropolitan plan, a question arises as to whether their inclusion is a disadvantage to areas served by the central system. In that connection, an analysis of Stage I costs shows that the capital outlay per service connection would amount, by 1970, to $332 for the independent systems as compared to $326 for the central system. A similar situation would prevail in 1980. In the long run, therefore, the independent areas as a group would not be subsidized by the central area.

FINANCING THE STORM DRAINAGE AND SEPARATION PROGRAMS

It is not possible at this time to develop a program for stage construction of the proposed drainage improvements. Consideration can be given nevertheless to the rate of progress which could be achieved within financial limitations. Consideration can be given also to the program of separating storm water and sanitary sewage which is to be undertaken by the city of Seattle.

Storm Drainage

In general, two procedures have been found practicable in other metropolitan areas for financing major drainage projects by means of general obligation bonds. Under the first procedure, a schedule of construction is established for the metropolitan area as a whole, with priority based on immediate need and on the value of property to be protected. General obligation bonds in an amount sufficient to carry the program for a considerable number of years are then authorized by a vote of the electorate of the entire area and bonds are issued as required to maintain the construction schedule.

Where taxing powers permit, construction of drainage facilities may be financed entirely on an area-wide basis by means of a fixed annual levy. As an example, this method of financing has been utilized successfully by the Los Angeles County Flood Control District.

Under the second procedure, the metropolitan area is divided into zones or special districts, each of which comprises an entire drainage area. Construction projects therein are financed by bond issues which are independently voted and supported in each zone. This procedure has been used successfully in California by the Alameda County Flood Control District.

Of the two procedures, the first is particularly applicable where drainage problems are wide-spread and general public support can be obtained. The second, while relatively easy to initiate in developed zones subject to flooding, may be impracticable in others because of limited development and low assessed valuation. In the latter case, land development of value to the metropolitan area, such as that for industrial purposes, may be unduly delayed.

Because of the wide-spread need for major drainage improvements throughout the metropolitan area, including those involving separation of combined sewers in Seattle, it is assumed herein that construction will be financed on an area-wide basis in accordance with the first procedure. Further consideration of the second procedure would require at least a preliminary determination of the drainage zones and their assessed valuations. In any case, use of this procedure would necessitate an amendment of the Metropolitan Municipal Corporation Act.

Exclusive of that portion of Seattle served by combined sewers, the estimated eventual cost of constructing major storm drainage facilities in the metropolitan area amounts to a total of $145 million (Chapter 17). Of this total, roughly 25 to 30 per cent, or $35 to $45 million, represents facilities which are needed now or will be needed in the near future.

Major drainage works required to relieve overloaded combined sewers in Seattle are estimated to cost approximately $19 million, which amount is about 27 per cent of the total cost of partial separation. Most of these facilities are needed now and all of them should be constructed as rapidly as the necessary funds can be made available. In all, therefore, the total cost of storm drainage facilities requiring early construction is not less than $54 million and may be as high as $64 million.

Based on the low population projection for the metropolitan area, the general obligation debt limit is estimated to be $49.5 million by 1960 and $55.8 million by 1970 (Table 19-8 and Fig. 19-5). At those levels, a storm drainage construction program involving an expenditure of $6 million per year for 10 years would be feasible.

As an example, assume an issue of 30-year serial bonds at 4 per cent interest, with combined interest and debt retirement charges paid in equal annual installments. Under such a program, outstanding bonds at the end of the tenth year would amount to $53.5 million. Annual debt service would increase from $347,000 the first year to $3.47 million in the tenth year. Tax levies would range from 0.35 mills to 3.15 mills over the 10-year period. Beyond that time, drainage construction would depend largely on population growth and increased assessed valuation. As a minimum, an expenditure of $1.5 million to $2.0 million per year could be supported thereafter based on increased bonding capacity estimated from the low rate of population growth.

No estimates were prepared of operation and maintenance costs for the completed drainage facilities. Normally derived from general fund sources, including tax levies, the amount required for this purpose is small and generally can be raised by a tax levy of

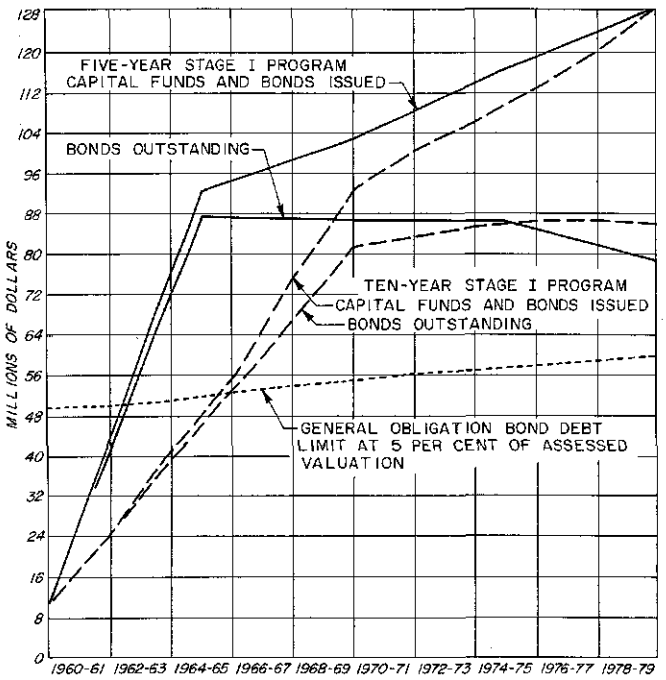

Fig. 19-5. Cumulative Funds, General Obligation Bond Financing of Sewerage Program

about 0.1 mill. Under present limitations, however, a metropolitan agency is not empowered to levy a tax for operation and maintenance purposes. Funds thus required would have to be collected from constituent cities and counties as supplemental income.

Separation of Combined Sewers

Financing the separation of combined sewerage systems in Seattle is primarily the responsibility of that city. Partial separation is estimated to cost a total of $69 million (Chapter 18), of which $19 million is for major storm drains and could be financed as part of a general drainage program undertaken by a metropolitan agency. This would leave a balance of $50 million to be financed by the city.

Particular urgency is attached to the separation of systems within the Lake Washington watershed. For these systems, the estimated initial cost of partial separation is $18 million, or $13 million exclusive of the trunk drains which could be financed by a metropolitan agency.

Financial resources which are now or may become available in the future for the separation program include:

1. Approximately $5.5 million as reimbursement for existing facilities by the metropolitan agency. This amount (Table 19-2) excludes $1,333,000 due Lake City Sewer District for facilities still being financed by that district.
2. Up to $2 million per year from the city sewer service fund. Upon inauguration of the metropolitan sewerage program, normal expenses to be met from this fund will be reduced largely to those of collection system maintenance. The foregoing income will become available each year in the event that the local charge is not reduced.
3. Existing general fund revenues.
4. General obligation bonding capacity of the city available only for water, sewerage, and electric utilities.

If income available under items 1 and 2 were utilized directly for construction, the Lake Washington watershed separation program could be achieved in four to seven years, depending on whether the major storm drainage works were financed by the metropolitan agency. Continuing on the same basis, the balance of the separation work could be completed in an additional 18 to 25 years.

PART VI

FINDINGS AND RECOMMENDATIONS

Chapter 20. Summary of Findings
Chapter 21. Recommendations

Chapter 20
SUMMARY OF FINDINGS

In the preceding chapters we have presented what has become a somewhat massive report, weighted down not only by a text containing many thousands of words but by numerous tables, charts and diagrams. As such, it can hardly be regarded as suitable for cover to cover reading and study. We are confident, however, that the information here recorded will be utilized for many years in solving sewerage and drainage problems of the metropolitan Seattle area.

Many of those who find it necessary to refer to this report will have neither the time nor the inclination to study it in detail. For their purposes, a summary is required which covers the essential facts and presents them in a manner which is both useful and readily understandable. This chapter has been prepared accordingly and reviews, as briefly as possible, the nature, magnitude, and scope of the recommended sewerage and drainage projects.

PART I. INTRODUCTION AND HISTORY

Residents of Seattle and surrounding communities are faced with long-standing sewerage and drainage problems which are becoming increasingly serious and which, if not resolved, will emerge as major obstacles to continued growth and development of the metropolitan area. Recognition of these problems and of the need for effective long-term solutions prompted city, county and state officials to join in sponsorship of the survey here reported. Part I summarizes major sewerage and drainage problems now confronting the metropolitan Seattle area. In addition, it sets forth the objectives and scope and the general procedures employed in making the survey, and presents a chronological history of sewerage developments and events.

Chapter 1. Introduction

1. The metropolitan Seattle area enjoys an abundance of diversified water resources. These range from fresh water inland lakes and streams to the saline waters of Puget Sound and are utilized intensively for a wide variety of recreational, industrial, commercial, agricultural and other purposes. They represent a community asset of inestimable value and as such must be protected against unreasonable impairment by sewage and industrial waste discharges.

2. Among the factors contributing to the sewerage and drainage problems of the area are:

a. Increased population. Long-term population estimates indicate that the metropolitan population will increase from the present total of 860,000 to about 1,250,000 by 1980 and to about 2,250,000 by 2030.

b. Discharges of untreated sewage. Sewage from 53 per cent of the present population is being discharged into environmental waters without treatment. As a result, recreational shore waters are subject to contamination, water quality is being impaired with respect to other beneficial uses, and nuisance conditions prevail at many locations.

c. Sewage overflows into recreational waters. In sewerage systems carrying both sanitary sewage and storm water, overflows occur at some locations nearly every time it rains.

d. Algal growths in Lake Washington. Lake Washington is being degraded as a result of algal growths, caused in part by the nutrient substances contained in sewage and sewage effluent discharges.

e. Sewage receiving capacity of Duwamish River. This river is approaching the limit of its capacity to receive putrescible substances.

f. Use of individual sewage disposal systems. About one-third of the metropolitan population is without public sewerage service. As a consequence, these people are compelled to rely on individual household sewage disposal systems, construction of which is being maintained at an average rate of about 6,000 per year. Many such systems are doomed to early failure because of the unfavorable soil drainage conditions which prevail in most of the area.

g. Overloading of combined sewers. Many of the combined sewers which serve the city of Seattle become grossly overloaded during periods of rainfall. Overloading causes frequent back-ups of sewage into basements and streets.

3. Physical and environmental factors require planning of sewerage and drainage facilities on a watershed rather than political boundary basis.

4. Present conditions in the metropolitan Seattle area reflect an urgent need for the development of a comprehensive, long-range program of sewerage and drainage improvements. Such a program must incorporate existing facilities to the fullest practicable extent and must have as its principal aim the protection of shores and shore waters from contamination, pollution and nuisance.

Chapter 2. History of Sewerage Problem

1. Sewers consisting of wood troughs were constructed in Seattle as early as 1865. The first permanent sewers were constructed in 1883.

2. In 1889, Colonel George E. Waring Jr., submitted a comprehensive plan calling for the construction of a separate system of sanitary sewers. After considerable controversy, the Waring plan was rejected in favor of a proposal submitted by Benezette Williams which recommended construction of combined sewers carrying both sanitary sewage and storm water.

3. Extensive sewerage works constructed in accordance with the Williams plan during the period from 1890 to 1914 still serve as principal elements of the Seattle system. One of the primary functions of these early sewers was to intercept dry weather flow from the Lake Union, Green Lake and Lake Washington drainage basins and to convey it through and around intervening ridges to the saline waters of Puget Sound.

4. As the city grew, the older systems were extended and numerous independent systems were constructed. By 1920, developed sections of the city were moderately well covered by a network of sewers.

5. Because no treatment facilities were provided and sewage outfalls usually terminated at the water's edge, hazards to health and nuisance conditions soon developed in many parts of the city.

6. Since 1920 repeated recommendations have been made to the effect that treatment should be provided and that the combined sewer systems should be either partially or completely separated.

7. Construction of the Lake Washington interceptor system between 1926 and 1936 eliminated raw sewage outfalls into the lake. Nevertheless, overflows of combined sewage still occur at some 30 points nearly every time it rains.

8. The Diagonal Avenue sewage treatment plant was constructed in 1940 and serves about one-sixth of the area connected to combined sewers. Sewage from the remainder of the combined system is still discharged without treatment.

9. Areas annexed to the city since 1954 included five sewer districts, each of which was served by separate sanitary sewer systems prior to annexation. These systems, which include four sewage treatment plants, have since been operated by the city.

10. In 1956, the Seattle electorate approved a $6.25 million bond issue for sewerage improvements. Adoption of a monthly sewer service charge was also approved at the same time. Funds thus made available resulted in the design and present construction of (1) an interceptor system and treatment works to serve West Seattle; (2) enlargements to the Lake City sewage treatment works; and (3) an interceptor along Shilshole Bay north of the ship canal.

11. Despite the fact that there now are 26 independent sewerage systems in operation within the metropolitan Seattle area, nearly one-third of the population is without public sewerage service.

PART II. CHARACTERISTICS OF THE METROPOLITAN SEATTLE AREA

The planning of comprehensive sewerage and drainage facilities requires as a background an adequate knowledge of environmental factors, both physical and economic. Part II deals with these factors, particularly as they relate to future growth and development of the metropolitan area.

Chapter 3. Physical Geography

1. Local geography, as defined by topography, geology, climate, natural and political boundaries, and water areas, influences in many ways the design, construction, cost and operation of sewerage and drainage works.

2. As used in this report, the metropolitan Seattle area extends from the vicinity of Silver Lake on the north to the city of Auburn on the south and from Puget Sound inland for a distance of 18 miles at the widest point. Containing 19 incorporated cities, it encompasses an area of about 575 square miles and includes, in addition to western King County, portions of both Snohomish and Pierce counties. Within the area are about 27,000 acres of inland water and some 140 miles of fresh water shoreline.

3. Topographically, the metropolitan area is comprised of striated hills, rolling glaciated uplands, and deeply incised adjoining troughs. Except for a prominent east-west ridge between the south end of Lake Washington and the foothills of the Cascade Range to the east, the principal geographic features have a general north-south trend.

4. Although the entire area drains ultimately to Puget Sound, it is divided naturally into four major drainage basins, namely, Lake Washington, Green-Duwamish River, Lake Union-Ship Canal and Puget Sound. Of these, the Lake Washington drainage basin is by far the largest, comprising about 350 square miles or nearly 60 per cent of the entire metropolitan area. Only 10 per cent of the area drains directly to Puget Sound.

5. Lake Washington, the largest of numerous fresh water lakes, is about 25 miles in length and has an average depth of 150 feet and a maximum depth of 220 feet. Puget Sound, which comprises the westerly boundary of the metropolitan area, is one of the deepest salt water basins in the United States and has an average depth of 600 feet.

6. Geologically, considerable horizontal and vertical variations can be expected in subsurface condi-

tions. Upland valleys and hills generally have a shallow weathered soil underlain extensively by hard cemented till. Upland soils generally provide good foundation conditions. Lowland valleys consist of alluvial deposits of soft clay, silts, and fine to coarse sands intermixed with gravel. High ground water tables, reaching the surface during the winter, are encountered in the lowlands. Piling and special bedding will be required frequently in the valley areas.

7. Marine air from the Pacific Ocean and the large water area of Puget Sound account for an equable year-round climate. Average daytime temperatures during the winter range from 40° to 50° F, while those at night are in the 30° to 40° range. Summer temperatures range generally from 70° to 80° F in the afternoon and from 50° to 60° during the night. Average maximum monthly temperatures range from 75.1° F in July, the warmest month, to 45.2° F in January, the coldest month. During 64 years of record at the Seattle weather station, the highest temperature was 100° F and the lowest was 3° F.

8. Rainfall occurs during a pronounced though not sharply defined rainy season and totals between 30 and 40 inches per year. In downtown Seattle, a total of 5.81 inches, or 18 per cent of the annual normal precipitation usually occurs during the five-month period, May through September. From 1951 through 1955, summer rainfall of 0.01 inch or more occurred during 173 hours, or only 4.6 per cent of the total hours in the season.

Chapter 4. Economic Development

1. Land uses in the metropolitan area range from intense residential, commercial and industrial in the city of Seattle to undeveloped cut-over lands and second growth timber around the periphery of the area. At present, over 50 per cent of the residential development is taking place outside Seattle.

2. Industry has been a dominant force in economic development. Centered initially on extractive industries such as forestry, fishing, and mining, operations have expanded and now include a variety of new products, among which are transportation equipment, building materials, textiles, and sporting goods. Aircraft manufacturing, by far the largest industry, accounts for three out of every five workers presently employed. Water, air and rail transportation, trade and commerce, fisheries, and military installations comprise other important segments of the metropolitan economy. Industrial expansion is expected to continue by the addition of such new industries as oil refining and the manufacture of petrochemical and synthetic chemical products.

3. Water supply and power facilities constitute two of the most valuable economic resources of the area. Natural gas is now available and provides a further inducement to industrial development.

4. Outstanding opportunities for recreation, especially boating and fishing, coupled with the area's scenic beauty, serve to attract tourists and to stimulate immigration.

5. Potential industrial employment is estimated to be 200,000 at ultimate development. At a gross density of 8.0 persons per acre, this means that a total of about 25,000 acres will required for industry.

Chapter 5. Population

1. For sewerage and drainage planning purposes, forecasts of population growth and distribution to the year 2030 were made with the assistance of the planning commission staffs of King and Snohomish counties and of the city of Seattle. These are shown below. For financial planning purposes, somewhat lower figures, representing the least growth likely to occur, were used.

	Population in thousands			
	1957	1980	2000	2030
State of Washington	2,670	4,320	6,500	10,600
Central Puget Sound Region[a]	1,370	2,240	3,380	5,500
King County	859	1,290	1,690	2,400
Snohomish County	134	264	592	1,100
Survey study area	864	1,260	1,740	2,240

[a]*Comprising King, Snohomish, Kitsap and Pierce counties.*

2. Population densities within the city of Seattle now range from 6.9 to 30.0 per acre, while those outside the city range from 0.5 to 4.7 per acre. Within the city, densities 70 years hence are estimated to range from 8.3 to 23.1 per acre, with an average of 12.3 per acre. Outside the city, the estimates range from 2.4 to 10.5 per acre and average 6.0 per acre.

PART III. EXISTING SEWERAGE, SEWAGE DISPOSAL AND DRAINAGE

Competent planning and design of sewerage and drainage facilities require a thorough knowledge of existing facilities, of the characteristics of the sewage to be dealt with, and of the environmental and economic effects of present deficiencies. Part III presents a description of the facilities now in use, a discussion and interpretation of sewage analyses, and a summary of conditions and problems which are in need of correction.

Chapter 6. Existing Sewerage and Drainage Facilities

1. Responsibility for providing sewerage service within the study area is presently divided among 41 separate agencies, including 19 cities and 22 sewerage districts. Of these, only 15 now operate sewerage systems. In addition, however, there are eight

semi-public and private systems which serve limited areas.

2. Existing sewerage facilities consist of about 1,550 miles of sewers, 75 sewage pumping stations and 25 sewage treatment works. About 1,040 miles of combined sewers are presently in use, essentially all of which are in the city of Seattle.

3. Existing treatment plants have an aggregate capacity of about 28 mgd, or only enough to treat about one-third of the present average daily dry weather output. The remaining output, amounting to about 50 mgd, is discharged without treatment through about 60 outfalls scattered along Puget Sound, Elliott Bay, Duwamish River, Lake Union and the Lake Washington Ship Canal system.

4. Only 70 per cent of the metropolitan population is now served by public sewers. The remainder must provide and maintain private septic tank systems. Private systems are being constructed at an average rate of about 6,000 per year.

5. Essentially all sewers outside the city of Seattle are of the separate sanitary type. Within the city, there are 1,029 miles of combined sewers and 204 miles of separate sanitary sewers. Outside Seattle, only four cities and one sewer district have storm drainage facilities worthy of note.

6. The Seattle sewerage system is composed of four major and numerous small independent systems. About one-fifth of the dry weather flow is treated at the five plants operated by the city, while the remainder is discharged without treatment. In addition to numerous raw sewage outfalls, there are about 60 bypass stations where combined sewage is discharged to adjacent waters nearly every time it rains.

7. Design capacities of most of the combined sewers are sufficient to carry no more than the flow resulting from a two-year storm. For that reason, and also because many of the systems have been overextended, overloads are frequent and cause sewage to back up into basements and to overflow from manholes.

8. While relief sewers have been provided from time to time, remedial measures have consisted mostly of interconnections between systems which were originally intended to function independently.

Chapter 7. Sewage Characteristics

1. Field and office studies were undertaken to develop design factors for (1) sewage volume, sewage strength and composition, (2) ground water infiltration, and (3) direct storm water entrance to sanitary sewers.

2. Continuous flow measurement devices were installed at 7 locations and additional flow data were obtained from records of 11 treatment plants. Minimum flow gagings were made at 50 points in the Lake City and Southwest Suburban systems to determine rates of infiltration. Composite samples for laboratory analysis were collected at 2 metering stations and 5 treatment plants.

3. A sanitary sewage contribution of 60 gpcd was decided upon for design purposes and represents a moderate increase compared with the present contribution. For design of trunk sewers within a major tributary area, the selected peak flow rate is 175 per cent of average. For trunks serving more than one major sewerage area and for treatment works, a peak of 150 per cent of the average is considered adequate. For presently developed industrial areas, an average waste contribution of 4,000 gpad and a peak of 8,000 gpad are in line with present experience. For new light industrial zones occupying an area of 1,000 acres or more, the allowance is 2,000 gpad. For heavy industrial zones smaller than 1,000 acres, a value of 4,000 gpad is appropriate.

4. Infiltration allowances applicable to existing sewers are 1,200 gpad for the wet season and 300 gpad for the dry season. For new sewers, similar allowances are 600 gpad and 300 gpad respectively. Storm inflow allowances are 2,000 gpad for existing sewers and 500 gpad for new sewers.

5. Values of 0.20 ppcd for BOD and 0.25 ppcd for suspended solids were determined to be applicable for sewage treatment plants in the metropolitan area.

6. Rates of infiltration and storm inflow in the Lake City system vary over a wide range and are excessive in certain local areas. Excessive infiltration and storm inflow rates were found also in many of the existing systems.

Chapter 8. Environmental and Economic Effects of Sewerage and Drainage Deficiencies

1. Environmental effects stemming from sewerage and drainage deficiencies in the metropolitan Seattle area range from minor nuisances to conditions involving a significant hazard to community health and well being. Economic effects include damage to property and impairment of water uses.

2. On the basis of design criteria employed until recently, overloading of combined sewers in Seattle can be expected to occur on an average of at least once every two years. Over 50 per cent of these sewers are overloaded and many of them have been extended to serve areas far in excess of those originally intended.

3. During the past six years, 692 complaints were received by the engineering department in regard to the backup of sewage into basements. Sewage flooding of streets and washouts in the vicinity of overflowing manholes are frequent occurrences.

4. While essentially all of the sewerage systems outside Seattle are composed of separate sanitary

sewers, high rates of infiltration and storm inflow occur in many cases and overtax both sewer and treatment plant capacities.

5. Approximately 100 square miles of the metropolitan area, with a population of 260,000 persons, is presently in need of public sewerage service. Close to 85,000 individual household disposal systems are now in use.

6. Septic tank installations depend on leaching systems for effluent disposal. Due to poor drainage conditions generally prevailing in the metropolitan area, these installations are failing at a rate of about 6,000 per year.

7. Septic tank overflows are common occurrences in unsewered areas which are heavily developed. In some sections, conditions have become so intolerable that subdividing and building have been greatly curtailed.

8. Construction, maintenance and repair of private sewage disposal systems are estimated to cost in excess of $3.2 million per year. An annual outlay of that magnitude would finance nearly $50 million worth of public sewers and would be sufficient to provide public sewers for essentially all of the area presently in need of such facilities.

9. Lack of trunk sewerage facilities for conveying sewage from inland areas to suitable points of treatment and disposal is one of the principal deterrents to local sewerage construction. Failure to provide these facilities is resulting in (1) exhaustion of financial resources in construction of temporary treatment works, (2) curtailment of desireable land developments, and (3) continued degradation of Lake Washington by discharges of both raw and treated sewage.

10. Numerous studies have shown that water resources of the area are being seriously impaired as a result of indescriminate waste discharge practices. Together with present findings, these studies show (1) that most of the public recreational waters fail to meet acceptable bacteriological standards, (2) that visible and odor nuisances prevail in the vicinity of points of raw sewage discharge, and (3) that waste discharges are exerting degrading chemical and biological effects on the waters of Duwamish River and Lake Washington.

11. Storm drainage facilities are generally lacking in the suburban areas. As a consequence, localized flooding occurs during periods of moderate to heavy rainfall.

PART IV. BASIS FOR PLAN DEVELOPMENT

Before proceding with the development of sewerage and drainage plans, it is necessary to establish specific bases for design. This in turn involves a determination, evaluation and analysis of a series of factors, among which are (1) legal requirements governing the discharge of sewage and industrial wastes, (2) beneficial water uses to be protected, (3) characteristics of potential receiving waters, and (4) potential waste disposal sites. It also involves development of design criteria and of bases for estimating costs, and division of the study area into units for detailed planning.

Chapter 9. Principles and Functions of Sewerage and Drainage.

1. For nearly thirty centuries sewerage has been allied with the growth of urban centers. The average citizen, nevertheless, has little knowledge regarding the basic concepts and processes involved.

2. All sewage, regardless of its origin, is a potential hazard to public health and to community comfort and well being. As such, it must be removed promptly from all premises and must be disposed of in a manner which is safe and innocuous both from a public health standpoint and from a water pollution standpoint.

3. Requirements relating to sewage disposal, as established under state and other legislation, are administered in the state of Washington by the Pollution Control Commission.

4. Collection systems for sewage and storm water may be of either the combined or separate type. In combined systems, both sewage and storm water are conveyed in a single conduit. In separate systems, they are conveyed in separate conduits.

5. Construction of combined sewers has declined in the United States during the last 75 years. This is due largely to the high cost both of providing treatment capacity for combined sewage and storm flows and of constructing interceptors capable of carrying such flows without indescriminate bypassing.

6. Sewage treatment comprises the removal of relatively small amounts of mineral and organic material from the transporting water. Relative proportions thus removed define the degree of treatment.

7. Disposal conditions, particularly those relating to the ability of a receiving water to dilute and disperse sewage effluent, determine the degree of treatment required prior to disposal at a given site.

8. Primary treatment generally includes screening, grit separation, sedimentation, and separate digestion and disposal of the putrescible matter removed by sedimentation.

9. Secondary treatment utilizes biologic oxidation processes to remove dissolved and colloidal substances not affected by primary sedimentation.

10. Disinfection, meaning the destruction of disease-producing organisms contained in sewage, may be accomplished prior to disposal of either primary or secondary effluent.

11. Experience elsewhere demonstrates the economic desirability of establishing a metropolitan agency for the provision of sewerage service to a metropolitan area.

12. Of the many different types of sewerage agencies which can be formed in Washington, only one, a metropolitan municipal corporation, is legally capable of undertaking area-wide projects of the magnitude and scope required in the metropolitan Seattle area.

Chapter 10. Sewage Disposal in Lake Washington

1. Inland lakes and streams of the metropolitan Seattle area must be preserved and their waters must be protected from contamination and pollution by sewage discharges.

2. At present, approximately 20 per cent of the population of the metropolitan area resides in the immediate vicinity of Lake Washington. Over 50 per cent of the ultimate population is expected to reside within its drainage basin.

3. Lake Washington is the natural drainage basin for an area of approximately 182 square miles. In addition, it receives runoff from 402 square miles of adjacent basins. Outflow from the lake is discharged to Shilshole Bay through a series of ship canals, Lake Union and the Government Locks.

4. Water uses of Lake Washington and its tributaries are mainly recreational, including swimming, boating and fishing. Other uses are for private and public water supply purposes, and for fish propogation. Waters of Lake Union and the ship canal are used primarily for shipping and navigation.

5. Physical and bacteriological impairment can be prevented by adequate sewage treatment and disposal. Chemical impairment through mineral enrichment cannot be prevented by any economical treatment process. The capacity of waters in the Lake Washington drainage basin to receive sewage, therefore, is based on the ability of the lake to tolerate inflows of fertilizing substances, principally nitrogen and phosphorus.

6. Major tributary streams and sewage discharges are the principal sources of nitrogen and phosphorus entering Lake Washington. More than 98 per cent of the natural surface runoff into the lake occurs from nine principal tributary streams. These streams also contribute the major portion of nutrient inflow from natural sources. Treated sewage from a contributory population of 64,000, coupled with septic tank seepage from an unsewered population of 13,500 constitutes the principal source of nutrient inflow from sewage. In addition, sewage reaches the lake from combined sewer overflows in the city of Seattle. Untreated industrial wastes are discharged by the Boeing aircraft plant at Renton.

7. During winter months when runoff in the tributary streams is at a maximum, natural sources account for the major portion of nutrient materials. During summer months sewage sources gain in relative importance.

8. The estimated amount of nitrogen entering Lake Washington each year from sewage sources has more than doubled in the past 40 years. In the same period, the amount of phosphorus has increased by almost 300 per cent.

9. Expressed in terms of total input, sewage contributions of nitrogen are expected to increase from 6.5 per cent at present to an ultimate maximum of 35.2 per cent. Similarly, sewage contributions of phosphorus are expected to increase from 43 per cent at present to 92 per cent.

10. The biological response of a lake to fertilization may be manifested in several ways. Increasing fertilization is accompanied by an increased population of microscopic plant and animal life (plankton). Waters low in nutrient materials usually contain plankton of the diatom species, whereas those high in such materials usually contain large numbers of blue-green algae. Growths of the latter are frequently manifested by turbid water and disagreeable odors.

11. Plankton counts in Lake Washington almost doubled during a 5-year period, 1950 to 1955. In 1933, the organisms were predominantly diatoms. In 1957, the predominant forms were blue-green algae.

12. Studies of lakes in southeastern Wisconsin by C. N. Sawyer showed that nitrogen and phosphorus were the critical elements in relation to plankton productivity. From data then obtained, it was concluded that nuisance conditions resulting from excessive biological activity could be expected when the concentration of inorganic phosphorus in the water equals or exceeds 0.01 ppm.

13. Based on nutrient contributions and physical conditions in Lake Washington it appears that the inorganic phosphorus concentration that can be tolerated approaches 0.02 ppm as a limit. This concentration would require a total input of phosphorus of between 10 and 17 pounds per acre per year.

14. Since the diversion of Cedar River to Lake Washington in 1916, the quantity of phosphorus supplied to the lake from natural sources amounts to 9 pounds per acre per year, or about the lower limit of the tolerance range. Phosphorus supplied from sewage sources has increased steadily from about 2 pounds per acre per year in the 1916-1930 period to a present input of 6 pounds per acre per year. In other words, the total input from natural and sewage sources now approaches the upper limit of the tolerance range. Failure to reduce the amount of phosphorous entering the lake will lead to its eventual destruction as a recreational and esthetic asset.

15. The only practical method by which the phosphorus supply to Lake Washington can be reduced is through the elimination of all discharged thereto both of raw sewage and of sewage treatment plant effluents.

16. Bacteriological contamination of the waters of Lake Washington by storm water overflows from the combined sewer system of the city of Seattle can be brought under practical control by reducing the frequency of overflows to an average of once per recreational season. Contamination by miscellaneous raw sewage discharges, principally from house boats and yachts, should be controlled by the local agency in the affected area.

Chapter 11. Sewage Disposal in Puget Sound

1. Disposal of sewage in tidal waters must not impair present beneficial uses of the waters of Puget Sound or endanger anticipated future uses. Beneficial uses adjacent to the metropolitan area include recreation, fishing and fisheries, navigation, and industrial and commercial operations.

2. As related to possible points of sewage disposal, water quality criteria established by the Pollution Control Commission indicate that the bacterial concentration resulting from a sewage discharge is the controlling factor. If applicable bacteriological criteria are met, conditions with respect to other criteria also will be satisfactory.

3. Characteristics of Puget Sound which affect sewage disposal include those of temperature, dissolved oxygen, density, and current movement. Because of the relatively large tidal variations in the sound, mainstream currents are due predominantly to tidal rather than wind action.

4. Results of studies made elsewhere were utilized in developing graphic methods for estimating initial dilution and subsequent dilution and disappearance of coliform organisms.

5. Results of studies made elsewhere, together with results of a biological survey in the vicinity of the raw sewage discharge from the North Trunk sewer, indicate that submarine disposal of digested sludge would be feasible.

6. Disposal of digested sludge through an independent outfall should be undertaken on a trial basis at the new Alki Point treatment plant. Controlled disposal at that point, coupled with a comprehensive monitoring program for a period of about five years, would provide conclusive information needed for the design of future installations.

7. Studies were made in Puget Sound to determine the action of local eddy currents and longshore currents which prevail at possible disposal sites. Data from previous studies, and from investigations made with the hydraulic model of the sound at the University of Washington, were utilized in planning the current studies and evaluating their results.

8. Currents were measured at depths up to 400 feet, using a number of free-floating biplane drags suspended at fixed depths from small surface floats. Positions of the floats were determined at approximately hourly intervals.

9. Each of the possible disposal sites was analyzed to determine both the degree of treatment required and the length and depth of the submarine outfall. In these analyses, the controlling criterion was that of keeping the maximum concentration of coliform organisms at critical shore locations below the limit established by the Pollution Control Commission.

Chapter 12. Sewage Disposal in Green-Duwamish River

1. In its downstream reaches, Green-Duwamish River flows through areas in which future industrial and residential developments can be expected. Use of its waters for waste receiving purposes is of paramount importance to the future of the metropolitan Seattle area.

2. Green River originates in the Cascade Mountains east of Auburn. Duwamish River is formed by the confluence of the Green and Black rivers in the Renton-Tukwila area and discharges to Elliott Bay. Tidal effects presently extend about 90,000 feet upstream from the mouth of the river. Following construction of the proposed waterway extension, these effects will not extend beyond Orillia, a distance of about 58,000 feet upstream. Minimum flow in Green River during a 10-year period from 1944 through 1953 was 110 cfs. This minimum will be increased to 180 cfs on completion of the proposed Eagle Gorge Dam.

3. Beneficial water uses in the Green-Duwamish river system are diversified and include fish propagation and migration, shipping and navigation, and irrigation. Other uses are for industrial and waste disposal purposes, and for recreational fishing and boating.

4. Sewage and industrial waste disposal practices which satisfy the requirements for fish propagation and migration and for irrigation of crops will be sufficient to satisfy all other requirements with respect to beneficial uses. For the protection of fish life, the required condition is a dissolved oxygen concentration of not less than 5.0 ppm. For crop irrigation, the requirement prescribes a median coliform count not to exceed 50 per 100 ml, providing the contaminating organisms are of human origin.

5. Dissolved oxygen concentrations in Green-Duwamish River are presently approaching the minimum prescribed by the Pollution Control Commission, indicating that the river has about reached its limit for the safe disposal of sewage.

6. Based on conditions of minimum river flow and maximum water temperature, 20,000 pounds of 5-day BOD per day can be discharged at Auburn without reducing the dissolved oxygen concentration of Green River below 5.0 ppm. Of this load, 18,000 pounds will still remain in the river as it enters the Duwamish estuary. Downstream significance of this residual is dependent on the self-purification capacity of the estuary.

7. In terms of 5-day BOD, the maximum load which can be discharged to the head end of the existing Duwamish estuary is slightly in excess of 5,000 pounds per day. This can be increased to 10,000 pounds per day by moving the point of discharge downstream to Black River junction. In the proposed Duwamish estuary, the maximum 5-day BOD loads which can be discharged at Orillia and at Black River junction amount to 10,000 and 15,000 pounds per day respectively.

8. Maximum BOD loads which can be safely discharged to Green River at Auburn without reducing the dissolved oxygen concentration in the estuarial portion of the river system below 5.0 ppm amount to 7,000 pounds per day in the existing Duwamish estuary system and to 12,000 pounds per day in the proposed estuary system.

Chapter 13. Design Criteria and Basis of Cost Estimates

1. All projects discussed herein are laid out to serve ultimate development of the tributary area. As here used, "ultimate" refers to conditions expected about 70 years in the future, or about the year 2030.

2. Design criteria pertaining to trunk sewers, storm drains, intercepting sewers, pumping stations and treatment plants are based on the assumption that all new areas will be served by separate sanitary sewerage systems.

3. Unit design quantities for sanitary sewerage systems are based on studies of local sewage characteristics and take into account expected future variations. They also anticipate establishment of appropriate regulations by responsible agencies to set limits on physical and chemical characteristics which would produce undue loadings or detrimental effects on processes or structures

4. Design of storm drainage systems is based on use of the rational method, represented by the formula $Q = ciA$, wherein Q is the runoff rate in cubic feet per second, c is a selected coefficient of runoff, i is the mean intensity of rainfall of a specific duration and frequency in inches per hour, and A is the tributary area in acres.

5. Design of interceptor systems in areas served by combined sewers must make provision for certain quantities of storm water to minimize overflows to recreational waters.

6. Detailed analyses of rainfall records, overflow frequencies and interceptor capacities indicate that use of storm water holding tanks represents the most feasible method of providing capacity for storm flows originating in combined systems. Installation of such tanks at appropriate locations will reduce overflow frequencies to a safe level.

7. All construction costs used herein are based on an Engineering News-Record construction cost index of 800. This value represents bidding conditions in the spring of 1958.

8. Unit costs for construction of gravity sewers and storm drains, as well as those for force mains, inverted siphons and outfalls, are based on actual construction costs in the Puget Sound area.

9. Construction costs for pumping stations and treatment plants were developed from known costs of comparable facilities elsewhere and are based on the provision initially of basic structural units having a capacity sufficient to meet ultimate needs.

10. Annual costs include charges for interest, depreciation, and operation and maintenance. Interest was assumed to average 5 per cent per annum. Depreciation was computed by the sinking fund method, with interest at 5 per cent. Computations were based on a 50-year life for sewers and other conduits, including outfalls, and on a 30-year life for pumping stations and treatment plants. Annual charges for operation and maintenance were developed from records of various agencies operating facilities similar to those considered for the metropolitan Seattle area.

Chapter 14. Sewerage and Drainage Areas

1. One of the basic requirements in planning comprehensive sewerage and drainage facilities for a large area, such as metropolitan Seattle, is the division of that area into more or less independent units. Units for drainage planning are limited almost exclusively by topography, while those for sewerage planning are limited and defined not only by topography but by economic and political considerations as well.

2. Except for the southeast border, the metropolitan Seattle sewerage study area is defined by watershed boundaries. An arbitrary boundary was established along the southeast border because watersheds of the Cedar and Green rivers and of Issaquah Creek extend beyond any probable urban development.

3. The study area consists of four primary watersheds. For purposes of sewerage planning, these were divided into 12 major sewerage areas, which in turn were further subdivided into a number of smaller units designated as local service areas.

4. Due to inherent differences in the principles of design, drainage areas usually differ from sewerage areas. Drainage planning is usually confined to small, integral units having a suitable outlet to the

nearest available point of disposal. In any event, drainage planning for local areas within each unit must allow properly for storm flows tributary to it from upstream sources.

PART V. SEWERAGE AND DRAINAGE PLANS S

As a final step in the development of long-range sewerage and drainage projects, it is necessary not only to determine what alternatives are reasonably available but to evaluate and compare such alternatives on the basis of cost and other pertinent factors. It is necessary also to formulate programs of stage or incremental construction for all the recommended facilities.

Chapter 15. Development of Sewerage Plans

1. Problems of metropolitan sewerage are generally solved with greatest satisfaction and economy when the sewage of the entire area is concentrated either at a single point or at a relatively few points for treatment and disposal.

2. Due to the requirement that all sewage and treatment plant effluents be removed from the Lake Washington watershed, final disposal of the metropolitan area sewage is limited to Puget Sound and the Green-Duwamish River.

3. Four sites were selected for possible construction of sewage treatment plants. These are designated as West Point, Government Locks, Elliott Bay, and Renton. Of the four, West Point is the most satisfactory in terms of meeting basic requirements for a treatment plant location.

4. Four central sewerage projects, designated herein as core plans, were developed for a detailed analysis and for comparison on the basis of construction cost and total annual cost. These projects were as follows:

Core Plan A. Conveyance of all sewage from the area to a single primary type treatment plant at the Government Locks site with effluent disposal to Puget Sound off West Point.

Core Plan B. Conveyance of sewage from the area to two treatment plants, one a primary type at the West Point site with effluent disposal to Puget Sound, and the second a complete type at the Renton site with effluent disposal to Duwamish River.

Core Plan C. Conveyance of sewage from the area to two treatment plants, both primary type. Of these, one would be at the West Point site with effluent disposal to Puget Sound, and the second at the Elliott Bay site with effluent disposal to Elliott Bay.

Core Plan D. Conveyance of sewage from the area to three treatment plants. One would be a primary type at the West Point site with effluent disposal to Puget Sound, the second a primary type at the Elliott Bay site with effluent disposal to Elliott Bay, and the third a complete type at the Renton site with effluent disposal to Duwamish River.

5. Estimated construction costs for the four core plans range from a total of $51,029,000 for Plan D to $73,932,000 for Plan A. For Plan B, which is the second lowest, the estimated construction cost is $54,346,000, or 6.5 per cent higher than Plan D.

6. Total annual costs for the four core plans range from $4,203,000 for Plan D to $5,708,000 per year for Plan A. For Plan B, the annual cost is $4,233,000, or less than one per cent higher than Plan A.

7. Because of the small difference in annual cost between Core Plan B and Core Plan D, other factors had to be considered in determining which would be the more acceptable. These factors include such items as duplication of operation; interference with business activity during construction; possible future upgrading of disposal requirements; ability to expand facilities in the event that the estimated growth of the tributary area is exceeded; simplicity and flexibility of the treatment process; and esthetic impression.

8. Based on a consideration both of the foregoing factors and of first cost and total annual cost, it is concluded that the most satisfactory plan for central sewerage of the metropolitan area is represented by Core Plan B.

9. Three possible modifications of Core Plan B were investigated. These involve (1) conveyance of sewage, including that from the North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, from the east to the west side of Lake Washington across the lake rather than south to the Renton treatment plant; (2) construction of a primary type treatment plant at the Renton site, with effluent disposal to Puget Sound rather than a complete type treatment plant with effluent disposal to the Duwamish River; and (3) construction of a complete type treatment plant at the Government Locks site with effluent disposal to the Lake Washington Ship Canal in lieu of a primary type plant at the West Point site. For all of these, the estimated construction costs and total annual costs are substantially higher than those for the proposed project under Core Plan B.

10. Due to problems involved in crossing Lake Washington from an independent treatment plant on the east side of the lake, no economical alternative to the central sewerage project is available to the East Lake Washington and South Lake Sammamish sewerage areas.

11. Five plans for independent sewerage of the North Lake Sammamish, North Lake Washington and Northwest Lake Washington sewerage areas were laid out and their costs were estimated. From the information thus developed, it is evident that the most eco-

nomical means of sewage collection, treatment and disposal for the three areas is through the central sewerage project.

12. Two plans for independent sewerage of the South Lake Washington and Green River sewerage areas were investigated. Although the estimated costs favor a plan calling for construction of a treatment plant north of Auburn in addition to the plant at the Renton site, receiving water and other conditions are such that these two areas can best be served under Core Plan B.

13. No reasonable alternatives to the central sewerage project are available to the Southwest Lake Washington, Elliott Bay and Lake Union sewerage areas.

14. In the South Puget Sound sewerage area, the most economical plan is one under which facilities for sewage collection, treatment and disposal would be provided independently in each of five individual subareas. New primary type treatment plants would be constructed in two of the subareas, Redondo Beach and Miller Creek, and a new complete type plant would be constructed in the Des Moines subarea. Existing primary type treatment plants in the Southwestern Suburban and West Seattle subareas would be fully utilized and secondary treatment facilities would be added at the Southwest Suburban plant. All plants would discharge effluent to Puget Sound.

15. In the North Puget Sound sewerage area, which comprises one small subarea, Seaview, and two principal subareas, Piper Creek and Boeing Creek, the most economical sewerage plan again is that of constructing independent facilities in each individual subarea. Under this program, primary type treatment plants are to be constructed in the Piper Creek and Boeing Creek subareas, with both discharging effluent to Puget Sound.

16. Because of the magnitude of the work which will have to be undertaken in developing the sewerage system recommended for the metropolitan Seattle area, it is unlikely that any single existing political agency could finance construction of the required facilities. It is not likely either that existing agencies acting individually could attain the desired objectives. It seems apparent, therefore, that such a project will require the formation of a central agency encompassing the entire metropolitan area. This agency would be responsible for all administrative and engineering duties relating to the financing, design, construction, maintenance and operation of the recommended sewerage facilities. In addition, it should take over and become responsible for operation and maintenance of all existing sewage treatment plants in the metropolitan area.

Chapter 16. Stage Construction of Sewerage Facilities

1. Development of a long-range program of sewerage improvements requires, as a final step, the scheduling of construction work in accordance with present need and expected growth. Additionally, consideration must be given to the effective utilization of available engineering and construction forces and to problems involved in financing the various projects.

2. Stage I construction, scheduled to start in 1960 and to be completed by 1970, provides for relief of the most urgent sewerage needs of the metropolitan area. In addition to treatment plants at two sites, West Point and Renton, this stage includes facilities to (1) intercept major sewage discharges to Lake Washington, (2) intercept raw sewage and industrial waste discharges to Duwamish River, Elliott Bay, and Puget Sound, and (3) provide service to highly developed areas presently without public sewerage. Estimated construction cost of facilities to be provided under Stage I amounts to a total of $83,215,000.

3. Upon completion of facilities for the Renton system under Stage I, central sewerage service will be available to the East Lake Washington, South Lake Washington, and Green River sewerage areas. Intercepting and trunk sewers will be constructed in these areas to effectuate the program of removal of sewage and sewage effluent discharges to Lake Washington. The Renton sewage treatment plant will be of the complete type employing the activated sludge process and will have a capacity of 24 mgd, average dry weather flow. This capacity will be sufficient for 15 to 20 years.

4. Stage I construction in the West Point system includes intercepting and trunk sewers to (1) intercept a major sewage discharge to Lake Washington at Lake City, (2) intercept raw sewage and industrial waste outfalls to Duwamish River and Elliott Bay, and (3) to provide service to developed areas presently without public sewerage. This stage also includes construction of a primary type treatment plant at West Point and a submarine outfall to Puget Sound. The West Point plant will have an initial capacity of 118 mgd, average dry weather flow.

5. Upon completion of Stage I construction, the West Point system will provide central sewerage service to the Northwest Lake Washington, Southwest Lake Washington, Elliott Bay and Lake Union sewerage areas.

6. Independent systems to be constructed under Stage I include those required to intercept raw sewage discharges to Puget Sound and to provide service to areas presently without public sewerage. This program comprises:

a. Construction of trunk sewers, primary type treatment plant, and submarine outfall in the Des Moines subarea, South Puget Sound sewerage area.

b. Construction of trunk sewers, primary type treatment plant, and submarine outfall in the Miller Creek subarea, South Puget Sound sewerage area.

c. Incorporation of existing sewers, existing sewage treatment plant and submarine outfall of the Southwest Suburban Sewer District, and construction of trunk sewers in the Southwest Suburban subarea, South Puget Sound sewerage area.

d. Incorporation of existing sewers, sewage treatment plant and submarine outfall of the city of Seattle in the West Seattle subarea, South Puget Sound sewerage area.

e. Construction of trunk sewers, primary type treatment plant and submarine outfall, and incorporation of existing sewers of the city of Seattle in the Piper Creek subarea, North Puget Sound sewerage area.

f. Construction of trunk sewers, primary type treatment plant and submarine outfall in the Boeing Creek subarea, North Puget Sound sewerage area.

7. Stage II construction, scheduled for the period 1970 to 1980, calls for extension of central sewerage facilities to the North Lake Washington and Green River sewerage areas. During this stage, the Renton treatment plant will be enlarged to accommodate an average dry weather flow of 48 mgd. Also called for are enlargement to ultimate capacity of plants serving the Des Moines and Miller Creek subareas; provision of secondary treatment facilities at plants serving the Des Moines and Southwest Suburban subareas; extension of submarine outfalls in the Southwest Suburban and West Seattle subareas; and construction of new trunk sewers, primary type treatment plant, and submarine outfall to serve the Redondo Beach subarea. Based on present day prices, the total cost of Stage II construction is estimated at $35,417,000.

8. Construction of the balance of the recommended facilities is provided for under Stage III. This work will be undertaken after 1980 as the need develops, and is estimated to cost a total of $45,366,000. Completion of Stage III construction will result in a sewerage system capable of serving the metropolitan area for many years.

Chapter 17. Development of Storm Drainage Plans

1. The extent to which storm drainage facilities are provided in urban areas depends on climate and topographic conditions, on the value of property to be protected, and on the ability and willingness of the public to meet the necessary costs.

2. For economic and other pertinent reasons, natural watercourses in the metropolitan Seattle area should be utilized to the fullest possible extent as storm drainage channels. Similarly, lakes of the area should be utilized as storm water storage or holding basins.

3. Since natural watershed boundaries generally transcend political boundaries, cooperation between political entities is required to protect watercourses and public and private property from damage due to storm water runoff. In the metropolitan Seattle area, such cooperation could be obtained if the administration of all major storm drainage facilities became the responsibility of the central agency recommended for administration of major sewerage facilities.

4. To demonstrate the method of runoff computation and conduit size selection and to provide a basis for a preliminary cost estimate of drainage, preliminary designs were developed for four typical drainage areas. These are designated as Kirkland-Houghton, Mountlake Terrace, Des Moines and Kent.

5. Topographic and other conditions in the Kirkland-Houghton area are conducive to effective and low cost drainage. Two plans were considered for this area, the first providing for use of open channels to the maximum possible extent, and the second for enclosed conduits throughout the system. Depending on the extent to which open channels can be used, provision of major storm drainage facilities in areas such as Kirkland-Houghton is estimated to cost between $270 and $440 per acre.

6. In the Mountlake Terrace drainage area, conditions are such that a portion of the runoff from one drainage basin could be diverted to another. Two alternatives were considered for this area. The first calls for diversion of the runoff from one drainage basin to another, and the second for disposal of runoff in each basin separately. Estimated construction costs for the two alternatives are virtually the same and amount to about $260 per acre.

7. Drainage in the Des Moines and similar areas can be achieved most economically by using natural watercourses to the fullest possible extent. Due to long distances to the final points of disposal, the cost of providing storm drainage facilities in areas of this type is relatively high and amounts to about $550 per acre.

8. Because extensive sections of the Kent drainage area lie below flood stage of the Green River, construction of storm drainage facilities is a difficult and expensive undertaking. Two alternatives were investigated for this area, each involving a different route for the major drains. Based on these findings, the cost of major storm drainage facilities in areas of this type is estimated to vary between $730 and $800 per acre.

9. Although it is impossible at present to estimate accurately the cost of providing major storm drainage facilities throughout the entire metropolitan Seattle area, an approximation thereof, based on unit costs for the four typical areas, can be developed for planning purposes. To that end, the metropolitan area was divided into five general categories and cost estimates were prepared, based on the applicable unit cost and on the total acreage within each such cate-

gory. As thus determined, the estimated cost of major storm drains required at ultimate development of the metropolitan area amounts to a total of $145 million. This total is exclusive of any improvements in the portion of Seattle served by combined sewers.

10. Based on the present stage of development in the metropolitan area, it is estimated that about one-fourth to one-third of the storm drainage facilities required for ultimate development should be constructed within the next 10 years.

Chapter 18. Separation of Combined Sewers in the City of Seattle

1. Seattle is faced with a diversity of problems brought about by the use of combined sewers for the conveyance of sewage and storm water. Correction of these problems can be achieved only by separation to some degree of both trunk and local collection systems.

2. Two basic factors relate to the planning of separation programs. First, construction of holding tanks in areas draining to Lake Washington provides the most economical means of preventing bacterial contamination of the lake due to storm water overflows. Second, unlimited storm water overflows are allowable in commercial waterways such as Lake Washington Ship Canal, Elliott Bay, and Duwamish River. As a result, separation need be undertaken only to the extent necessary to relieve overloaded trunk and local sewers.

3. Studies made in 1951 by the Seattle engineering department showed that the existing system is generally inadequate in terms of its ability to carry combined flows of sanitary sewage flows and storm water runoff. Studies made of six areas during the present survey showed in all cases that these portions of the system do not have sufficient capacity in their entirety to carry the flow resulting from a storm with a recurrence interval of 10 years. All of them, however, have a capacity sufficient to accommodate the peak flow of sanitary sewage.

4. Two degrees of separation were investigated. The first, partial separation, provides for a new storm drainage system of sufficient capacity to collect storm water runoff from streets and yards. The second, complete separation, provides for a new storm drainage system of sufficient capacity to collect all storm water runoff, including that from roofs and foundation drains.

5. The six areas selected for separation studies occupy a total of 2,860 acres, or about 8 per cent of the total area served by combined sewers in the city of Seattle.

6. Detailed studies of the six areas show that the estimated cost for partial separation varies from $1,690 to $2,170 per acre and averages $1,860 per acre. For complete separation, the estimated cost varies from $3,180 per acre to $4,570 per acre and averages $3,890 per acre. In both cases, the deviation from the average is less than 20 per cent.

7. Based on the average cost and on a total area of 37,000 acres sewered on a combined basis, the total cost of partial separation within the city of Seattle approximates $69 million.

8. Although separation is required throughout the city, steps should be taken immediately to provide separation in areas draining to Lake Washington and in other areas experiencing major problems caused by sewage overflows from combined sewers. This immediate program is estimated to cost $18 million.

9. To avoid future expenditures for separation, all new sewer construction in the city of Seattle should be on a separate basis. Sanitary sewers should be designed for the peak sanitary sewage flow only and separate storm drains should be installed as required.

10. New drainage systems in areas requiring separation should be designed with a capacity sufficient to accommodate the runoff from a 10-year storm, including that from roofs but not from foundation drains. These systems could be constructed initially to intercept street drainage only and could be extended later to critical areas to intercept roof leaders as required.

Chapter 19. Financing of Recommended Facilities

1. Based on a lower estimate of population growth than utilized for design purposes, the assessed valuation of metropolitan Seattle is expected to rise from $990 million in 1960 to $1,210 million by 1980. In the same period, the number of sewer service connections is expected to increase from 210,000 to 324,000.

2. Existing sewerage facilities, which either would be incorporated in a metropolitan system or abandoned because of the provision of such a system, have an approximate value of $9.4 million. It is proposed that local agencies be reimbursed for these facilities through assumption of their outstanding indebtedness by a metropolitan sewerage agency.

3. Annual outlays both for capital expenditures and for operation and maintenance will depend upon the rate at which the proposed sewerage works are constructed. Under a program requiring 10 years each for construction of Stage I and Stage II facilities, capital outlay will range from $7 million to $12 million per year during Stage I, and from $3 million to $4 million per year during Stage II. Annual costs of administration, operation and maintenance, both of permanent and temporary facilities (new and existing), are estimated to range from $832,000 in 1960-61 to $1,645,000 in 1978-79.

4. Use of general obligation bonds for financing the sewerage program would interfere with the financing of storm drainage improvements. Revenue bond financing of sewerage is the only practicable alternative. Total annual revenue requirements, including debt service coverage and operational costs, will range, using the 10-year Stage I program, from $6.4 million during the first year in 1960 to $9.2 million by 1979. As an upper limit, the equivalent residential service charge necessary to support this program will amount to $2.50 per month for the first 8 years, and to $2.40 for the next 12 years.

5. Since the estimated costs include those of operation, maintenance and debt service for many facilities now supported by service charges of local agencies, the metropolitan service charge would not necessarily be an addition to present charges. Locally, total service charges will vary with local debt service for collection sewers and the need for their extension or replacement.

6. Trunk storm drains needed now or in the near future, including those within Seattle required in conjunction with the separation program can be financed by general obligation bonds of a metropolitan agency. Resulting property tax rates would range from 0.35 mills in the first year to 3.15 mills in the tenth year of a 10-year construction program.

7. Partial separation of combined sewers in Seattle, other than major drains, is the responsibility of that city. Funds which can be made available for this purpose include reimbursement for existing sewerage facilities plus revenue from the present sewer service charge. Amounts thus obtained would enable separation within the Lake Washington watershed in four to seven years without resorting to financing by general obligation bonds.

Chapter 21
RECOMMENDATIONS

For convenience in reference, it is desirable in this final chapter, to consolidate all recommendations pertaining to the design, construction, and financing of the proposed sewerage and drainage facilities. In brief, it is recommended:

1. That the sewerage projects proposed herein be adopted as a long-range plan for the metropolitan Seattle area. These projects consist of (1) a central system designated as Core Plan B, (2) tributary feeder sewers and service sewers for the core plan system, and (3) independent systems in each of seven service areas immediately contiguous to Puget Sound.

2. That a central agency be established for financing, constructing, operating and maintaining, and administering the proposed sewerage projects.

3. That construction of sewerage projects be undertaken in accordance with the schedule herein set forth.

4. That construction of the required sewerage facilities be financed by means of revenue bonds.

5. That monthly service charges be established on a basis which will assure both sound financing and an equitable distribution of costs among all users of the metropolitan facilities.

6. That existing sewerage agencies be reimbursed for those facilities which are to be abandoned or are to be integrated into and made a part of the metropolitan system. In general, reimbursement should be in accordance with the schedule herein set forth and should be made in such manner as may later be determined to be feasible and equitable.

7. That, pending establishment of a central agency, the city of Seattle proceed with construction of those elements of the proposed projects which lie wholly within its boundaries.

8. That, pending establishment of a central agency and the provision of service by that agency, sewerage planning in areas outside the city of Seattle be directed toward the eventual connection of all local sewerage systems to the metropolitan system.

9. That, pending provision for removal of all sewage from the Lake Washington drainage basin, land disposal or any other practicable means be utilized to prevent new or additional discharges of sewage effluent into the surface waters of that basin.

10. That the city of Seattle undertake a continuing program for separation of combined sewers and that the practice of constructing combined sewers be discontinued.

11. That specifications for sanitary sewers, including house connections, be revised to require the use of pipe and pipe jointing materials and methods which will assure adequate tightness, and that acceptance of construction be predicated on meeting the requirements of a standard leakage test.

12. That a policy of prohibiting the connection of roof, yard, foundation and similar drains to sanitary sewers be established and vigorously enforced.

13. That sanitary sewers subject to excessive infiltration or storm water inflow be further investigated to determine the sources thereof, and that such steps as are necessary be taken to reduce infiltration and inflow to permissible limits.

14. That the city of Seattle undertake, on a trial basis, the disposal to Puget Sound of washed digested sludge from the Alki Point treatment plant, and that a monitoring program be conducted in accordance with specifications to be prescribed by the Pollution Control Commission.

15. That a program of monitoring receiving water conditions within the zone of influence of waste discharges be established and continuously maintained.

16. That a continuing program of research be conducted relative to all phases of the nutrient enrichment problem in Lake Washington.

17. That trunk storm drains be designed in accordance with the principles herein set forth.

18. That the provision of trunk storm drains be made the responsibility of the central agency.

19. That trunk storm drains be financed by means of general obligation bonds supported by an ad valorem tax spread over the entire area ultimately to be benefited by such drains.

20. That a policy be adopted aimed at (1) obtaining permanent rights-of-way for trunk drainage purposes, and (2) preserving natural drainage channels which are presently or may in the future be required for trunk drain routes.

21. That, as a matter of policy, metropolitan sewerage facilities be defined as those necessary to serve a minimum area of approximately 1,000 acres and that trunk storm drains be defined as those necessary to serve a minimum area of 160 acres.

22. That uniform industrial waste ordinances, designated for the protection of sewerage facilities and the prevention of water pollution, be adopted by all agencies contributing waste to any metropolitan facility, and that enforcement of these ordinances be made a requirement for participation in the metropolitan project.

APPENDIX A

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ACKNOWLEDGEMENTS

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Planning Department
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Paul A. Wiatrak

Planning Department

John D. Spaeth, Planning Director
Albert H. Crosetti
Robert F. Hintz

Water Department

Henry R. Herold
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APPENDIX B

TOTAL COUNT METHOD

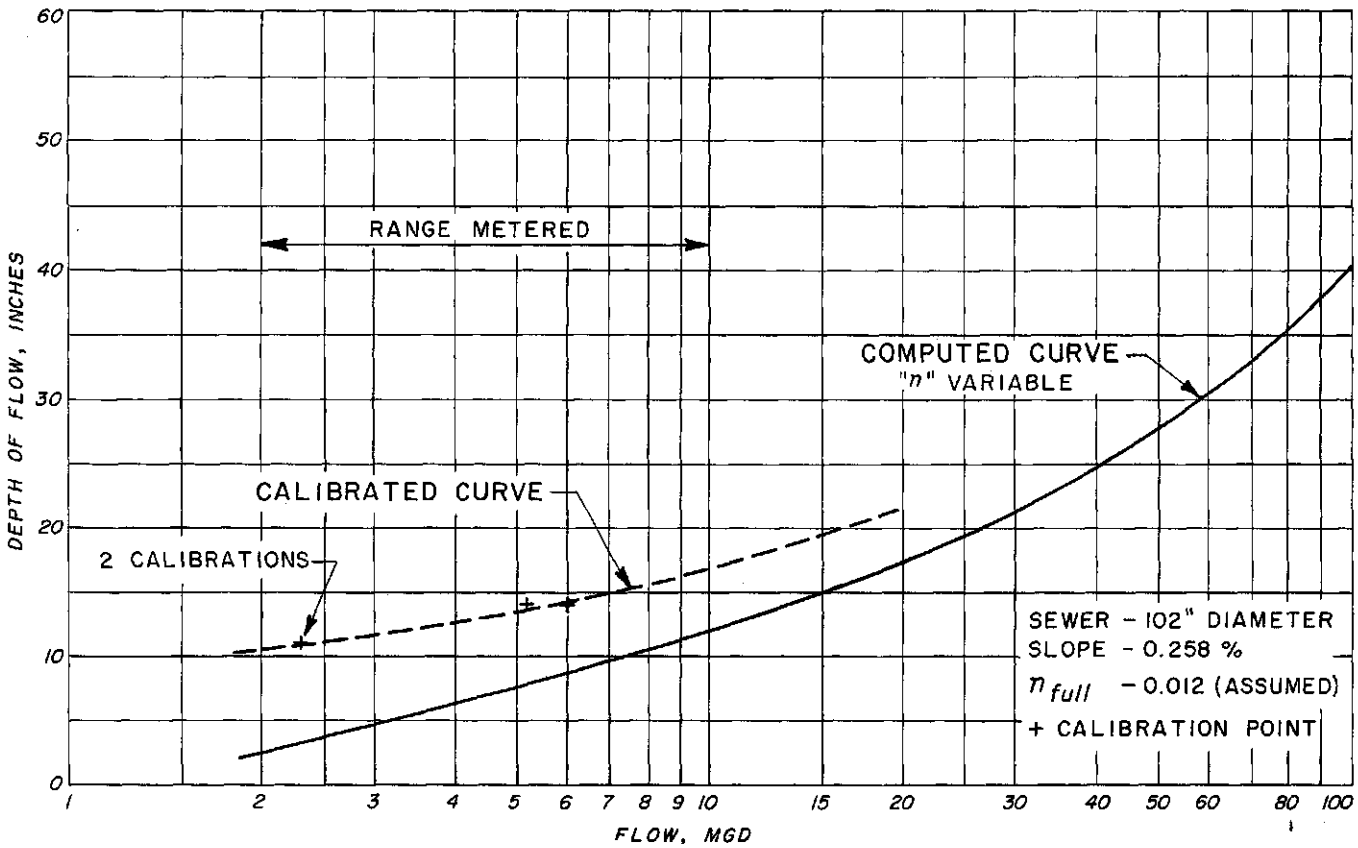
MEASUREMENT OF FLOWS BY TOTAL COUNT METHOD

As set forth in Chapter 7 of this report, a radioactive isotope was used for flow calibration in large diameter sewers wherein depth of flow and other conditions preclude any possible use of standard metering devices. In brief, the isotope method provides for flow calibration at various depths in the sewers and enables subsequent flow measurements by means of depth recording meters.

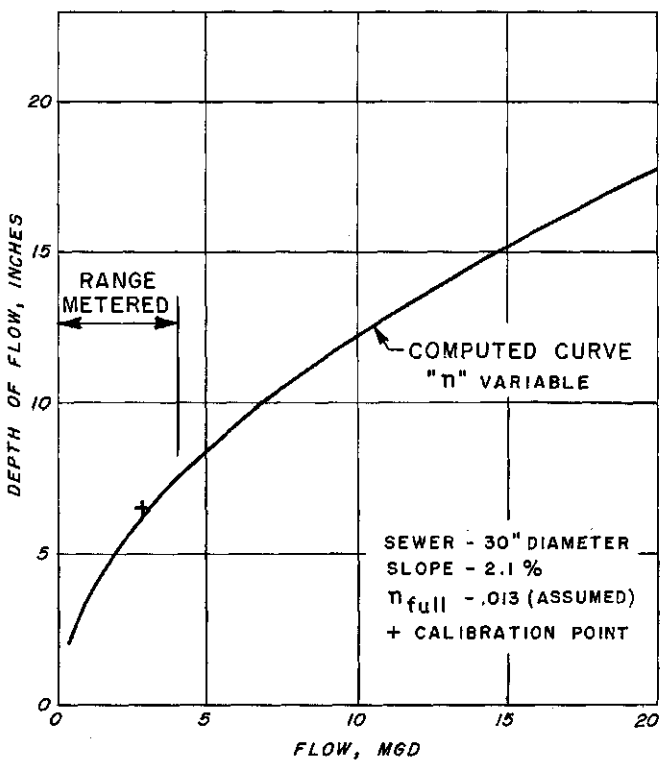
In the calibration operation, use is made of the total count method[1] wherein a small quantity of radioisotope is added into a flowing stream and resulting gamma ray emissions are measured quantitatively at a downstream point. Originated by D. E. Hull of California Research Corporation, this method is based on the principle that the integral or total count of gamma rays registered by a Geiger-Mueller (G-M) counter immersed in the flow is inversely proportional to the flow rate and directly proportional to the millicuries (mc) of isotope used to measure the flow. This con-

[1] *Nucleonics, April 1955, Volume 13, No. 4, Industrial and Engineering Chemistry, 50, 199 (1958) (U.S. Patents 2,826,699 and 2,826,700 - California Research Corporation).*

INDEPENDENT CHECK on average velocity computed by the total count method and measured depth of flow was obtained by calculating velocity of the radioactive isotope "slug" between upstream and downstream manholes. Synchronization of stop watches (photo) was necessary to determine the elapsed time between addition of isotope a known distance upstream and its recorded passage at the metering station.



RAINIER CALIBRATION CURVE was obtained at the metering manhole located at Rainier Avenue and Bayview Streets in Seattle. This reinforced monolithic concrete sewer with paved brick invert had heavy slime growths. These latter conditions resulted in poor agreement between the computed curve and the calibrated values for the low flow values encountered. Calibration during storm flows was attempted but the bubbler tube was torn loose when estimated velocities were 8 to 10 feet per second.

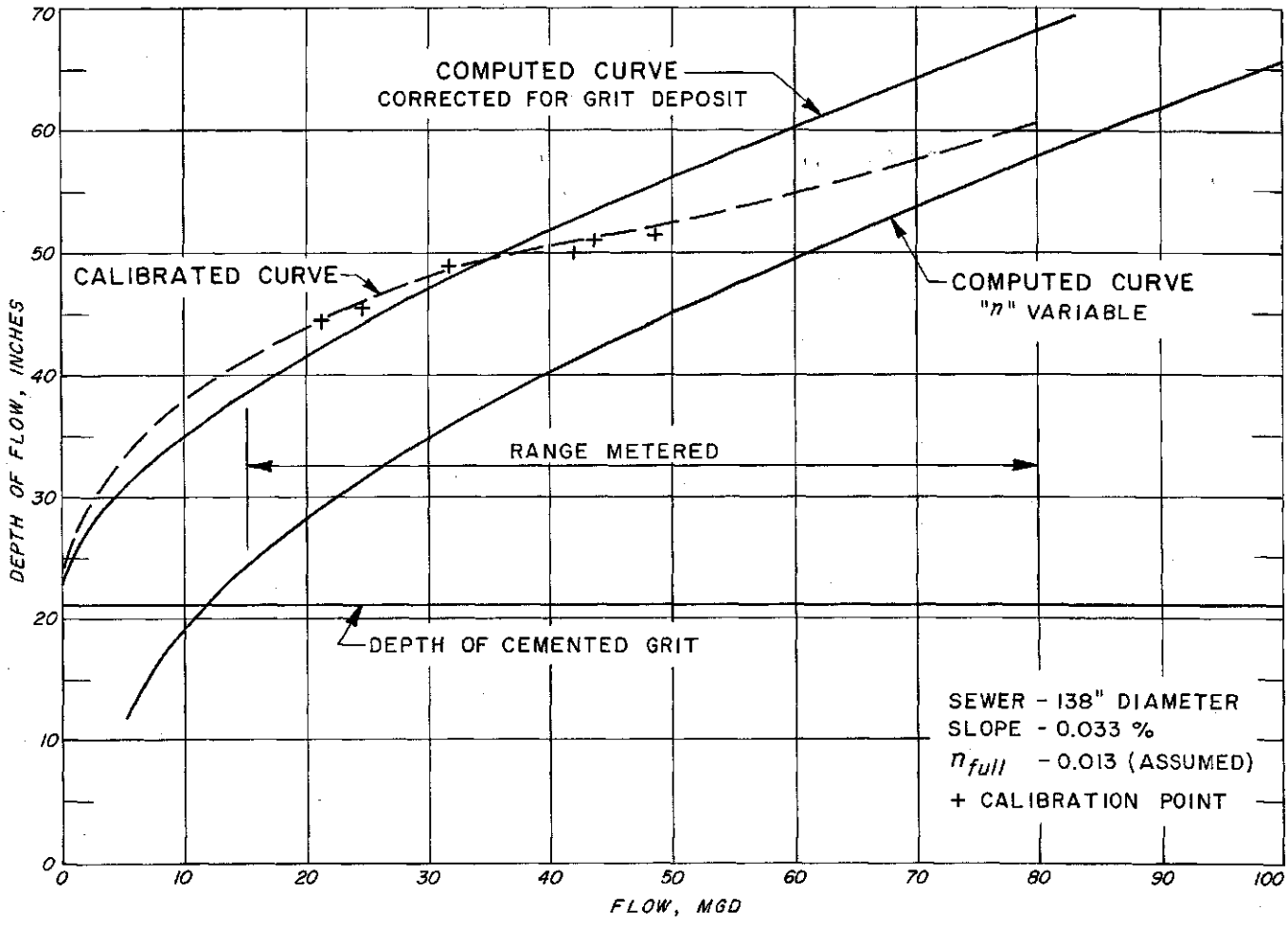


dition may be expressed as $Q \sim A/N$, wherein Q is the flow rate, A is the isotope activity, and N is the total count.

The total count technique was first discovered when it was noted that an isotope used to mark a liquid interface in a pipeline would always give the same total number of counts when monitored at several points on the pipeline. In other words, the integrated counting rate at several points was identical, even though the peak count rate differed because of longitudinal diffusion or spreading of the isotope in transit.

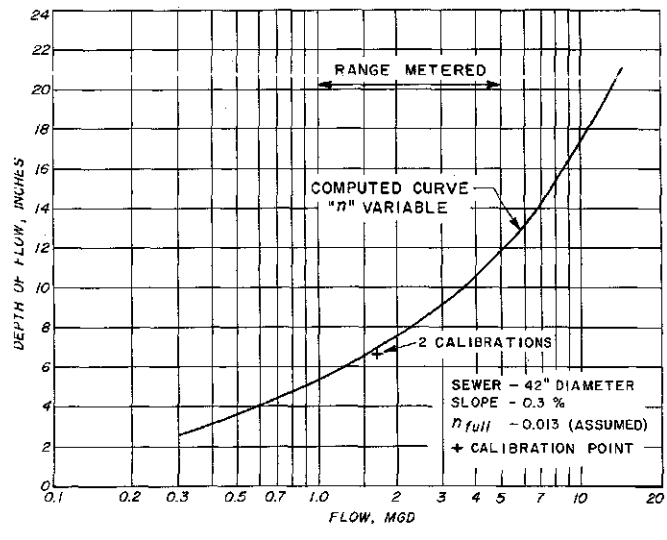
To convert the expression $Q \sim A/N$ to an absolute determination of flow, a proportionality constant based on the count rate from a known concentration of isotope solution has to be used. In that manner, the flow equation becomes $Q \sim AF/N$, where Q equals flow in cubic feet per second (cfs), A equals activity in mc, F is the proportionality constant expressed in counts

LAKE CITY NORTHERLY TRUNK calibration curve at Manhole 6N. Considerable difficulty was experienced in metering at this station because of supercritical velocities which adversely affected the pen recorder. Subsequently a weir was installed which reduced velocities and further calibrations were not undertaken.



NORTH TRUNK CALIBRATION CURVE was obtained at the metering manhole at Thorndyke Avenue and Emerson Streets in Seattle. This section on monolithic concrete sewer pipe was not affected by tidal variations but the invert was covered with approximately 22 inches of asphalt cemented grit. Because of the latter, it was necessary to correct the computed curve. At shallow depths of flow the cemented grit particles resulted in turbulence which was reflected in lower flow rates than actually occur. With increasing depths, however, this effect was materially lessened. In fact, extrapolation of the calibrated curve would indicate that the "n" value may approach 0.012 rather than the value 0.013 which was assumed for this conduit flowing full.

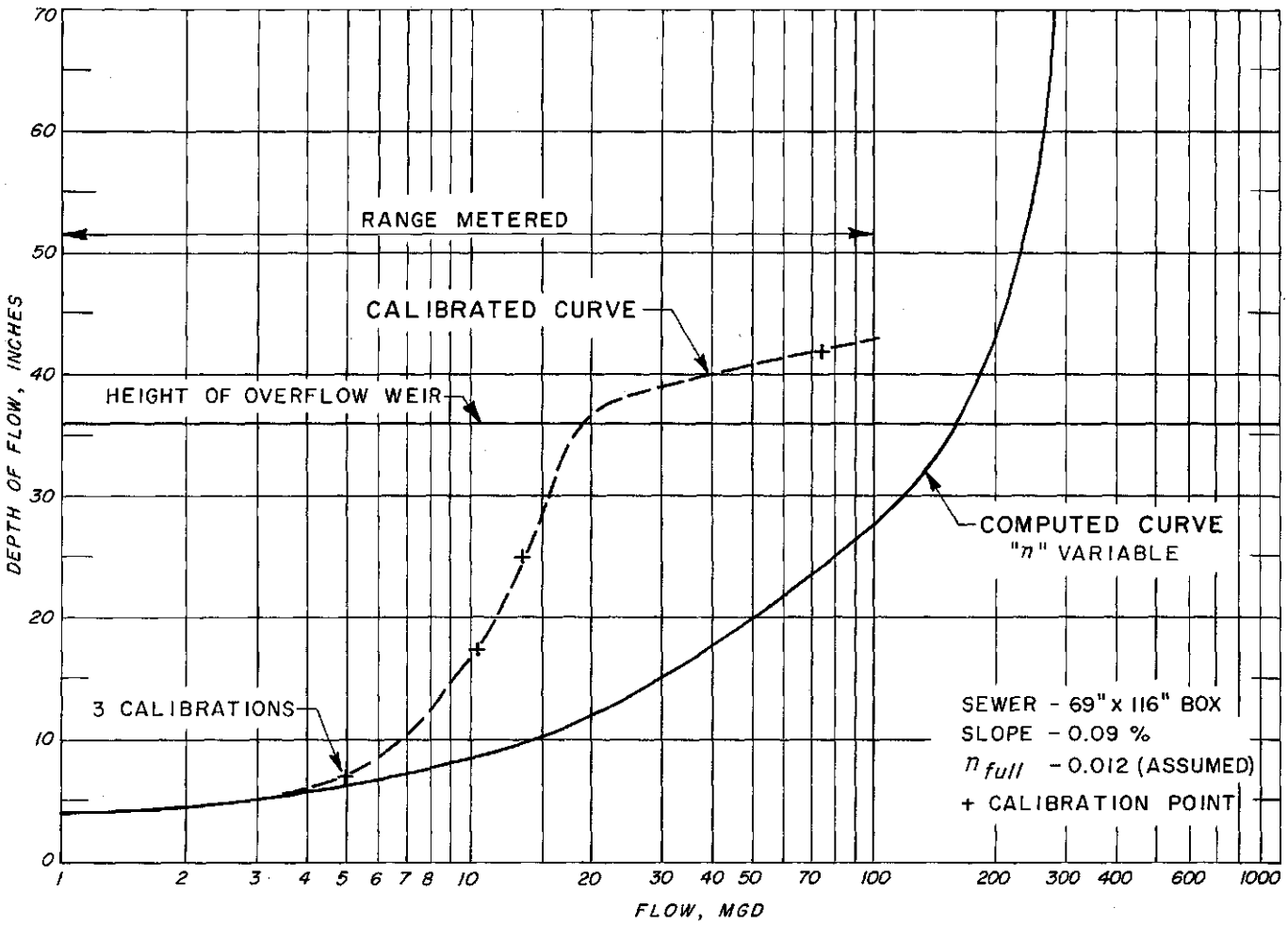
LAKE CITY NORTHERLY TRUNK calibration curve was obtained at Manhole 1N, just north of the Lake City treatment plant. Two calibration points obtained late in 1957 fell within 0.1 mgd of the computed curve for "n" variable. This error was within the standard deviation of the average counting rate. The computed curve was used directly. This sanitary sewer, constructed of 4 foot lengths of reinforced concrete pipe, had a noticeable slime growth along the normal water surface.



per second per mc per cubic foot, and N equals the total count.

An interesting aspect of the total count method which makes it particularly applicable to sewage metering in large conduits is that only part of the flow needs to be monitored. Providing thorough mixing has been achieved, the total count is independent of the fraction of the stream monitored by the counters.

Flow calibrations at Seattle were made by means of Cesium 134, an isotope which has a half-life of 2.3 years. Prior to actual flow measurements, the activ-



BALLARD CALIBRATION CURVE was obtained for this monolithic concrete sewer at a side overflow weir at 11th Avenue NW and W 45th Street in Seattle. The depth of flow recorder was installed just upstream from the location shown in the photograph on page 151. Marked deviation from the computed curve results from three conditions in the sewer. Low flow is affected by gravel in the invert. At greater depths of flow, downstream hydraulic conditions limit the hydraulic capacity. Finally, when a depth of 36 inches is reached, flow over the bypass weir occurs.

ity of the radiotracer had to be determined. This procedure was performed in the laboratory, using the gamma bench standardization method.[2] Other laboratory work includes a determination of the "F" factor, which involves calibration of the field instruments. For that purpose, a stainless steel counting cylinder is filled with a solution of Cesium 134 of known concentration and activity, and the counting rate is registered on G-M tubes immersed in the solution.

To calibrate the sewer at the selected metering manhole, a small quantity of sewage is continuously pumped from the sewage flow through the cylinder containing the G-M counter, thereby determining the average background count rate of the sewage. Then, the previously standardized isotope is emptied into the sewage flow at a manhole sufficiently far upstream from the metering point to achieve thorough mixing. As the isotope flows toward the metering point, the background count rate is continuously integrated on a battery operated scaler. This procedure continues until the isotope passes the metering point and the count rate returns to the initial background rate. As the main "slug" of isotope passes the G-M counter, the depth of flow is read from a chart on a bubbler actuated pneumatic type of recorder. The entire field count operation takes approximately 30 minutes.

Subtraction of the average background for the period of record from the total count gives the count due to the isotope. Flow rate is then computed from the millicuries of isotope and total count, and plotted against the depth as recorded on the bubbler. By a series of such measurements over a range of flow, a calibration curve for depth of flow versus flow rate is obtained. Error of the total count method is mainly dependent on the amount of isotope used, and can be reduced to less than 5 per cent if required.

[2] *Nucleonics, August 1956, Volume 14, No. 8, pages 95-96.*

Sale of all isotopes is supervised and licensed by the Atomic Energy Commission and requires persons using them to have approved health physics training. In every case, the entire operation must be under the supervision of a radiological safety officer, who is generally a chemist or physicist. For work at Seattle, the radiological safety officer was Peter Wootton of the Tumor Institute of the Swedish Hospital.

All persons directly connected with any of the operations using the isotope wore a personal monitoring device, similar to a gold leaf electroscope, which indicated the total radiation exposure in milliroentgens. In addition, when using laboratory facilities where exposure to larger amounts of radiation was possible, a film badge was worn. When not in use, isotopes were stored in adequate shielded storage lockers at the Tumor Institute of the Swedish Hospital.

The inherent safety of the total count procedure is best shown by a simple examination of the radiation levels present after the isotope is emptied into the sewage flow. For example, approximately 1 mc (1,000 μc) is used when metering a flow of 7.5 cfs. Assuming a 20-second pouring time, there is an average concentration of:

$$\frac{1 \text{ mc}}{7.5} = \frac{1{,}000\,\mu\text{c}}{7.5 \times 20} = 6.7\,\mu\text{c per cubic foot}$$

According to Handbook 52 of the National Bureau of Standards, the recommended maximum concentration for drinking water is approximately 6 μc per gallon, or 45 μc per cubic foot for Cs 137, a similar gamma emitting isotope with a 33-year half-life.

The maximum concentration of radioisotope which can be used is limited by the maximum counting rate of the portable scaler. For Cesium 134, the maximum rate of 100 cps limits the concentration to approximately 11 μc per cubic foot, or about one-fourth the recommended maximum for drinking water.

For those who may be interested in the cost of the total count method, the required equipment involves an outlay of close to $1,000. Cesium 134 sells for $1.00 per mc, plus a moderate royalty payable to the holder of the patent. Less than 50 mc were used for the calibration measurements at the five metering stations. Service of the radiological safety officer and use of the radiation laboratory facilities for a period of approximately ten months cost a total of $800.

APPENDIX C

REPORT ON THE BIOLOGY OF THE BOTTOM SHELF OF PUGET SOUND WEST POINT, SEATTLE, WASHINGTON, JULY 1957

RICHARD N. WILSON
REGIONAL BIOLOGIST
U.S. PUBLIC HEALTH SERVICE

U.S. Department of Health, Education, and Welfare
Public Health Service

December 1957

Report on the Biology of the Bottom Shelf of Puget Sound
West Point, Seattle, Washington, July 1957

In the course of an engineering survey to develop a suitable Master Plan for the collection and disposal of sewage and industrial wastes, Brown and Caldwell, Consulting Engineers, and the City of Seattle were desirous of obtaining information regarding the existing physical and biological characteristics of the shelf off West Point, Puget Sound. The North Trunk sewer, which carries the untreated wastes from more than 200,000 people, presently discharges into the Sound a short distance north of West Point. Plans may call for the construction of the combined sewage treatment plant at this location.

The objectives of this investigation were, therefore, to determine existing conditions as to sludge deposits and bottom life and to predict, insofar as possible, the effect of discharging digested sludge after a new plant is in operation.

Mr. William Roman, engineer, Brown and Caldwell, Mr. John N. Wilson, Regional Biologist, U. S. Public Health Service, and Dr. Richard Bogan, University of Washington, were the principal investigators. A boat and crew were provided by the U. S. Coast Guard. Mr. Charles Ziebell, Biologist of the Washington Pollution Control Commission, assisted with the laboratory determinations.

Basic data on physical, chemical and biological conditions in Puget Sound around West Point are in short supply. However, some useful data were found in the report on the "Oceanographic Survey on Submarine Portion of Snohomish-Kitsay K. V. Line" by the Department of Oceanography which was performed under contract with the Bonneville Power Administration in 1953. The following cogent items have been taken from the latter report and from the literature survey:

1. Puget Sound has a tidal prism of 1.27 cubic nautical miles. Because of long inland estuaries and peculiarities of flow, tides are different from those in open ocean and water pollution is critical in places.

2. Depth is 720 feet maximum, not less than a mile out from West Point. The water deepens most rapidly in a southwesterly direction from West Point.

3. Puget Sound is divided into four general sections which are partially separated by vertical or lateral constrictions. The main basin extends from a 240-foot threshold sill at the confluence of Admiralty Inlet with the Strait of Juan de Fuca, southward to another 180-foot sill at the Tacoma Narrows.

4. These sills are critical in controlling the mixing and the nature of the water contributing to the inner basins of Puget Sound. Replacement of the deep water is affected rather abruptly during the autumn by addition of greater density, high salinity water, and, more or less, gradually during the remainder of the year by turbulent mixing with less dense water.

5. Winter salinities, based on the survey February, 1953, show around 29 0/00 at surface and 30 0/00 at bottom in vicinity of Seattle. Summer salinities, July, 1953, range from 28 0/00 to about 30.5 0/00. Out from Point Jefferson, a few miles west and north of West Point (site of proposed sewage treatment plant, Seattle), lies the deepest hole in the Sound - more than 900 feet.

6. With respect to tidal currents, there is normally a net daily outflow at the surface and a net daily inflow at depth. Exceptions occur where there is more general mixing during certain seasons of the year.

7. The deep water of the Sound does not tend to stagnate. Although the currents at depths, such as those at 700+ feet off West Point, are slow, they are, nevertheless, positive and move both horizontally and vertically, depending upon the season of the year.

8. As to aquatic animals which live on the bottom in the vicinity of West Point, information included in the "Oceanographic Survey on Submarine Portion of Snohomish Kitsap 250 K.V. Line" indicates an abundance of marine organisms on the upper slopes of the channel, particularly in the uppermost one-hundred feet. Dredging in the deepest portion of the cross section where the power line was to be laid showed that there was lesser variety of biological organisms down to depths of 700 or more feet and the total numbers of organisms per unit area was also less than in the shallower waters.

Materials and Methods

In the field, a standard Petersen dredge was used with ten pounds of weight attached. A hand operated winch was used to raise and lower the dredge. The material was hand screened with a brass screen of 40-mesh. Preservation of the catch was with neutralized formalin. No other chemical or physical tests were made at the time of the bottom sampling except for temperature.

Physical Features

In the areas of bottom free from sludge and wood waste deposits there was sand in the shoals out to average depth of 100 feet. The sand deposits gradually merged with a firm blue clay material to a depth of 550 feet.

(See map, Fig. 1, Station D-17), the deepest point sampled. Maximum depths off West Point were in excess of 800 feet but the available equipment did not permit sampling at such depth.

The slope is much more steep than is usually expected along the western continental shelf in that a maximum depth of more than 800 feet is reached off West Point less than one and one-half miles off shore. In contrast in Santa Monica Bay, California, the shelf extends out six or seven miles with a maximum depth of less than 300 feet. Reference is made to Tables 2 - 5 inclusive in conjunction with the map, Figure 1 for the physical and biological characteristics of the bottom at the sampling points. It will be noted on this map that there are four concentric lines drawn from a common center which is the so-called sewage "boil" or outfall of the North Trunk sewer located at D-38. These concentric lines are drawn on approximately 1,000-foot intervals leading out from this common point. Most of these bottom samples were collected from points less than a mile distant from the sewage"boil."

Tables are prepared and numbered in accordance with these concentric lines. For example, Table II refers to the description of the samples of the first concentric line, that is 1,000 feet or less from the outfall, Table III within 2,000 feet, etc. Included in these tables are data on physical condition of the bottom and summarized results of the analysis of the bottom fauna collected at each point.

The stations where definite deposits of sewage sludge and sludge-like material were found have been circled. Such was true of every station within the 1,000-foot zone. Similar conditions of heavy sludge deposits were found at D-40, D-31 and D-26 in the next zone that is within or just slightly over 2,000 feet. Within this zone Stations D-39 and D-21 which are located within a 1,000-feet of shore are a somewhat special case. They have been underlined on the map which indicates a light but recognizable deposit of sludge found at these stations. The "W" indicates presence of wood chips, sawdust, and other debris which like ordinary sewage sludge tend to blanket the bottom, but decomposes much more slowly than does sewage sludge.

There were no evidences of sludge or wood wastes products in the bottom samples in the 3,000-foot zone. At D-18, however, almost a mile from the sewage "boil" and in one of the deepest points sampled, 350 feet, the sample contained sewage sludge, newspaper particles and leaves. (Figure 2). Wood chips and sawdust were found in the sample at D-35, also at some distance in that latter zone. Station S-5, nearly a mile from the sewage "boil" and south of West Point,is noteworthy in that sewage solids were observed in suspension at this point. The bottom was clean sand, however, and there were patches of actively growing eelgrass. (Zostera marina).

In further regard to the occurrence of seaweed it will be noted on

the map that two patches of kelp are shown. By kelp is meant the giant kelp Nereocystis luetkeana. The smaller patch occurred near the outlet of Union Canal just northwest of Station D-20 almost a mile away from the sewage "boil". The larger patch is shown growing in close to shore within just a few hundred feet of the sewer outfall. Although recognizable sewage solids were in suspension throughout this patch of kelp, the plants seemed to be growing actively and appeared to be in healthy condition.

As to some of the physical conditions of the bottom at outlying stations not included within the concentric lines, S-6 which lies south of West Point in 20 feet of water has a sand bottom. A sharp drop-off occurs between Station S-6 and D-12 and D-13 as indicated by the depth of 380 feet and 210 feet respectively. The bottom at these stations was a hard blue clay with some sand intermixed. Sampling points D-11, D-14, D-15 and D-16 extend out from West Point in a northwesterly direction with D-16 at 800 feet plus. D-11, D-14 and D-15 had very hard bottom so that the dredge failed to close properly or, if closed, did not bring up any appreciable amount of bottom material. D-17, which is a little over a half mile north and east from D-16, was the deepest point in which a successful sample of bottom material was obtained. At 550 feet the bottom was composed of mud, blue clay and a little sand. Temperature of bottom material was 10½ degrees centigrade which is only 1.5 degrees centigrade colder than the surface water at that time.

In the course of this investigation a brief reconnaissance was made in the area north of Meadow Point to learn something of the bottom conditions in the vicinity of Piper Creek sewer outfall. This is in the area near Spring Beach and is about 2 miles north of the mouth of Union Canal. Sampling was confined to a zone less than a mile distant from the outfall. The bottom was mostly sand with patches of eelgrass and kelp. Sludge deposits occurred in patches of variable size and with thickness ranging from a thin film to several inches.

Discussion of Biological Findings

Reference is made again to Tables 2 - 5 which combine the physical and biological data in summary form. Tables 6 - 9 inclusive present the population counts of bottom fauna. There is a rough breakdown in classification of the animals to orders, families and occasionally to species (Table 1).

Referring once again to Tables 2 through 5, in the third column, entitled "Summarized Results of Bottom Faunal Analysis", there is a brief written description of the results followed by a fraction. In each instance the numerator of this fraction refers to the total numbers of organisms found per square foot of bottom and the denominator designates the numbers of kinds of organisms.

Stations D-22, D-32 and D-38 contain the highest populations of organisms found at any station under study. In each instance many of the organisms, such as oligochaete worms and round worms, or nematodes, which are primarily

of fresh-water preference, were introduced with the sewage itself. Many of the other organisms,such as the amphipod crustacea, are scavengers and are therefore greatly encouraged and caused to multiply by having many sewage solids and other food material available.

Because of the steep drop-off along this entire shelf area, along with known strong currents, deposits of sludge are probably dispersed as fast as they are formed and are in a continual state of flux. This is borne out by the fact that in a number of instances where heavy deposits were brought up in the dredge, sand bottom occurred immediately under the sludge deposits and in this sand were growing numerous tube worms. (See photo figure 2).

In general, it appears that heaviest productivity occurs in the immediate vicinity of the sewer outfall and that there is a general decrease outward in every direction. The exception to this is D-20, Table III, which has probably been enriched by the Union Canal. The population of amphipods was so great that no attempt was made to count them. Owing to smothering action, possible toxicity and/or localized lowering of the dissolved oxygen, there were no clams, snails or starfish found in areas closest to the sewer outfalls and where deposition of sludge was taking place.

The four samples from the area around Piper Creek outfall are shown in Table IX. Population of organisms was much more sparse than the most heavily populated areas around the North Trunk sewer outfall. This is despite the fact that the bottom appeared to be stable and had a number of patches of water vegetation such as eelgrass and kelp which are conducive to the growth of normal to large populations of bottom fauna.

General Discussion

The results of bottom sampling in the area around the North Trunk sewer outfall, where raw sewage waste from more than 200,000 persons is discharged, indicate that although deposits of sludge could be found at a number of points within one-half mile of the sewage "boil", at no point was any deposit found so thick but what the dredge could reach through and take some sand. This indicates deposits of six inches or less. Because of the raw state of the sewage, many fresh-water organisms, such as oligochaete and nematode worms were introduced with the sewage. Numerous salt-water scavenger organisms, such as amphipod crustacea, were attracted to the area around the outfall.

In order to predict the biological effect of discharge of digested sludge into Puget Sound in the vicinity of West Point, it should be understood at the outset that the situation will not be comparable to that which presently obtains. The digested sludge as discharged at considerable depth will be finely divided material which has undergone partial decomposition. There will be few if any fresh-water invertebrates as are now being discharged with the raw sewage from North Trunk. Moreover, because of the finely divided state of the material from the expected sludge outfall, there will be little

encouragement to the larger scavenger organisms among the salt-water bottom fauna. Bacteria in the sludge will be anaerobic or facultative anaerobic.

Of greater overall significance than the local biotic effect of sludge discharge are the physical implications. These may well have far-reaching effects on water quality and general water uses. If digested sludge is discharged through a separate outfall line into deep water off West Point, the specific gravity of the material will presumably be adjusted to conform with that of the receiving water. Insodoing, it is hoped by the engineers that, instead of dispersing toward the surface, the material will drop into the 800-foot trough and remain there.

In the event of deep water discharge of digested sludge to Puget Sound, can the sludge be expected to remain in deep water or will it disperse rapidly when water densities approach uniformity throughout this section of Puget Sound? As indicated above by oceanographic studies in connection with laying the Snohomish-Kitsap K.V. power line ten miles north of West Point, although currents at depths of 700 feet or more are slow, they are positive and move both vertically and horizontally, depending upon the season of the year. At certain times of the year water densities approach equilibrium, thereby creating a situation wherein dispersed materials in suspension in deep water tend to surge upward into surface waters.

The effects of such upwelling would be significant biologically and esthetically, should the suspended matter find its way into the shallow waters of beaches and shore installations. Among other effects, decreased penetration of light caused by the expected increase in turbidity could affect the productivity of green plants among the ocean plankton. A decrease of this kind in the vegetative part of the food chain would eventuate in similar decreases in the larger animal plankters and ultimately the fish.

Summary and Conclusions

1. To determine the physical and biotic characteristics of the bottom of Puget Sound in the vicinity of West Point, Seattle and Piper Creek mouth, a series of forty bottom samples were collected with a weighted Petersen dredge.

2. Particular emphasis was placed on the existing sewer outfalls discharging raw sewage wastes, viz. North Trunk north of West Point and Piper Creek outfall near Spring Beach north of Meadow Point.

3. Physical determinations indicated intermittent sludge deposits concentrated mostly within 1,000 feet of North Trunk with an occasional depression containing sludge out as far as 4,000 feet to one mile. At no point was any deposit thicker than one foot, while in most instances thickness of deposits was but a few inches.

4. There was little evidence of thermal stratification of waters. Even

at 550 feet the bottom material was only 1.5 degrees centigrade colder than the surface water.

5. Kelp and eelgrass beds were found close to the sewer outfalls in areas where sewage solids were in suspension and sludge deposits were forming.

6. In shallow areas the bottom was sandy. The sand slowly changed to blue clay and muck in the deeper places.

7. Although time did not permit specific identification of all bottom organisms and their classification based on known tolerance to pollution, nevertheless, certain general conclusions may be drawn.

a. The smothering effect of the sludge deposits did not appear to be serious insofar as certain animals, such as tube worms,are concerned.

b. The heaviest productivity occurred in proximity to the sewage "boils", but many of the organisms encountered had been introduced with the sewage. Most of the increased population was comprised of both annelid and flat worms.

c. Smothering action, possible toxicity and/or localized lowering of the dissolved oxygen prevented occurrence of clams, snails and starfish in areas closest to the sewer outfalls and where deposition of sludge was taking place.

8. Although there are expected to be differences in the physical, chemical and biochemical characteristics of digested sludge in comparison with the raw sewage wastes which are presently being discharged near West Point, the following prediction is deemed reasonable in the light of our present knowledge of currents and density patterns in Puget Sound: Upon discharge of digested sludge into Puget Sound in the vicinity of Seattle, there is little likelihood of the material remaining in deep water. There will be a tendency toward wide dispersion and at certain times of year upwelling of the material from deeper waters into shallow areas. Therefore, physical, chemical and biological impairment of water quality may be expected in varying degree, depending upon the characteristics of the sludge itself.

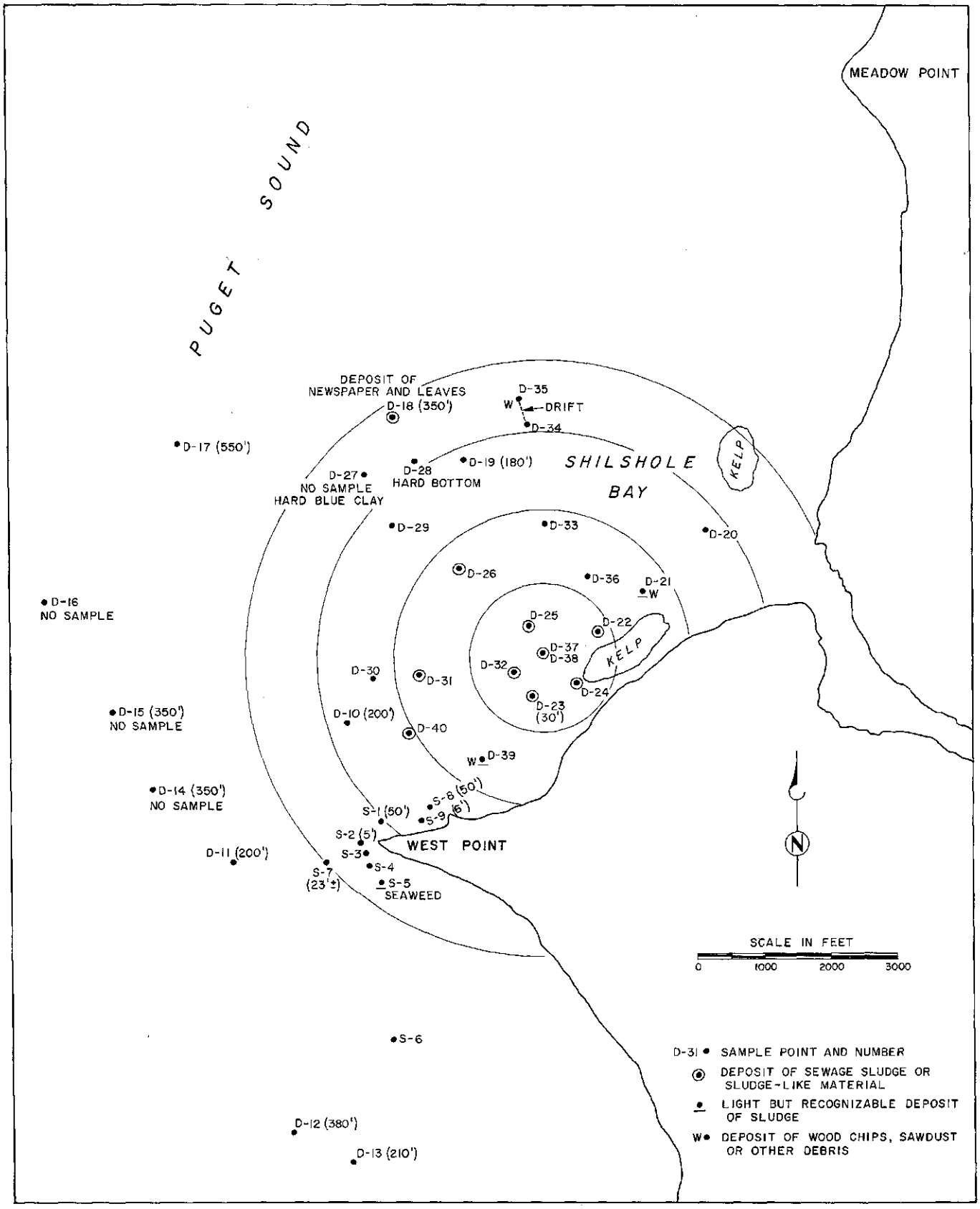

MEADOW POINT
PUGET SOUND
DEPOSIT OF NEWSPAPER AND LEAVES
D-18 (350')
D-35
W
DRIFT
D-34
D-17 (550')
D-28
HARD BOTTOM
D-19 (180')
SHILSHOLE BAY
KELP
D-27
NO SAMPLE
HARD BLUE CLAY
D-29
D-33
D-20
D-26
D-36
D-21
W
D-16
NO SAMPLE
D-25
D-22
D-37
D-38
KELP
D-32
D-30
D-31
D-24
D-23 (30')
D-15 (350')
NO SAMPLE
D-10 (200')
D-40
W
D-39
D-14 (350')
NO SAMPLE
S-8 (50')
S-1 (50')
S-9 (6')
S-2 (5')
WEST POINT
D-11 (200')
S-3
S-4
S-7 (23'±)
S-5
SEAWEED
N
SCALE IN FEET
0 1000 2000 3000
S-6
D-31 SAMPLE POINT AND NUMBER
DEPOSIT OF SEWAGE SLUDGE OR SLUDGE-LIKE MATERIAL
LIGHT BUT RECOGNIZABLE DEPOSIT OF SLUDGE
W DEPOSIT OF WOOD CHIPS, SAWDUST OR OTHER DEBRIS
D-12 (380')
D-13 (210')

FIGURE 1

TABLE 1. SCHEMATIC CLASSIFICATION OF BOTTOM FAUNA WITH EXPLANATORY NOTES

Group or Species Name	Common Name and Descriptive Material
Annelid Worms - Jointed worms	
Class - Polychaeta	
Order - Polychaeta Errantia	Free-living and predacious long-spined worms
Order - Polychaeta Sedentaria	Tube worms or burrowing worms
Class - Oligochaeta	Short-spined worms; most fresh water
Class - Hirudinea	Leeches or blood suckers
Class - Gephyrea	Peanut worm
Anthropods - Jointed animals	
Order - Amphipoda *Nebalia sp.*	"Water fleas" with body laterally compressed.
Caprella kennerlyi	
Order - Isopoda	"Water sow bugs" with body dorsoventrally flattened
Order - Ostracoda	Ostracods - like miniature clams
Order - Mysidacea	Shrimp-like
Order - Decapoda	Crabs
Order - Cirripedia	Barnacles
Platyhelminthes	Flat worms
Nemathelminthes	
Class - Nematoda	Round worms - no joints or segments
Mollusca	Snails, clams, chitons, squid, octopus
Class - Amphineura	Chitons
Class - Gastropoda	Snails
Class - Pelecypoda	Clams
Echinoderms	
Class - Ophiuroidea	Brittle stars
Class - Echinoidea	Sea urchins
Class - Holothuroidea	Sea cucumbers
Coelenterates	
Class - Anthozoa	Sea anemones

TABLE 2. SUMMARIZED PHYSICAL AND BIOLOGICAL DATA - Zone 1

Within one-thousand-foot radius of sewage "boil" from Seattle, North Trunk Sewer outfall

Station	Location and Description	Summarized Results of Bottom Faunal Analysis
D-22	Depth, 25 feet. Close to "boil" but in close to shore. Paper fragments, wood chips, gray sludge, sand bottom.	Scavengers among the Annelid worms and crustacea abundant. 3983+/6*
D-23	Depth, 30 feet. South of "boil". Pockets containing heavy sludge deposits.	Very few living organisms found. 128/2.
D-24	Depth, 15 feet. 200 feet from shore, near southwest edge of large kelp bed and directly shoreward from sewage "boil". Bottom sand covered lightly with sludge.	Abundant round worm population notably introduced with sewage. Otherwise sparse population of other fauna. 123+/5.
D-25	Depth, 75 feet. Wood chips, mud, odor of sewage.	Sparse population 142/6.
D-32	Depth, 60 feet. Blue-black sand-mud. Slight sewage odor.	Heavy production of tube worms, free-living polychaeta and scavenging crustacea. 3800+/5.
D-38	Depth, 50 feet. Directly over sewage "boil". Sewage solids in suspension all around.	Heavy production of scavengers including sewage sludge worms, round worms and crustacea. 3517/7.

*Numerator stands for total numbers of organisms per square foot of bottom; the denominator refers to the number of kinds of animals.

TABLE 3. SUMMARIZED PHYSICAL AND BIOLOGICAL DATA - Zone II

Within two-thousand foot radius of sewage "boil", Seattle North Trunk outfall

Station	Location and Description	Summarized Results of Bottom Faunal Analysis
D-21	Depth 35 feet. About 800 feet out from shore and nearly 2,000 feet northeast from sewage "boil". Fine sand bottom with some black mud, sticks and wood chips. Slight sewage odor.	Average productivity. Annelid worms are predominant. 633 - Total no/sq/ft; 6 - No. of kinds
D-26	Depth 150 feet. About 1,800 feet straight out from outfall. Sand, slight sludge deposit. No sewage odor detectable.	Light productivity and fair variety of organisms. 50/5.
D-31	Depth 180 feet. Blue clay overlaid with black sludge-like material.	Light productivity but variety of organisms. 128/8.
D-33	Depth 140 feet. Gray-black mud, clay sand mixture. D-31 and 33 are west and north of outlet, respectively at distance of nearly 2,000 feet.	Light productivity but variety of organisms. 122/9.
D-36	Depth 70 feet. About 1,200 feet northeast of outlet. Gray-black mud and sand - no odor.	Heavy productivity but less variety of organisms than preceding two samples. 2375/5.
D-39	Depth 40 feet. Sand and black mud. Wood chips and bark. About 1,600 feet southwest of sewage "boil" and toward the tip of West Point.	Extremely sparse population of bottom organisms. One of the least productive of all stations. Few kinds of organisms found. 46/6.

TABLE 4. SUMMARIZED PHYSICAL AND BIOLOGICAL DATA - ZONE III

Within a three-thousand foot radius of sewage "boil", Seattle, North Trunk Outfall

Station	Location and Description	Summarized Results of Bottom Faunal Analysis
S-1	Depth 50 feet. About 200 feet due north of tip of West Point. Sand bottom.	150/6. No evidence of sludge.
S-8 S-9	Close to shore on north side of West Point S-8, 50 feet; S-9, 6 feet deep sharp drop off	79/6 but shifting 36/3 sand bottom and currents make this area undesirable habitat for many bottom animals.
D-40	Depth 155 feet. About 2,000 feet southwest of sewage "boil". Bottom sand overlaid with black sludgy mud.	260/7. Moderately productive - fairly wide variety.
D-10	Depth 200 feet. West and a little south of the outfall. Sand-mud bottom.	258/9. Moderate productivity, good variety.
D-30	Depth 220 feet. West of outfall. Bottom blue-gray mud and sand	133/6.
D-29	Depth 260 feet. Nearly 3,000 feet northwest from outfall. Bottom gray-blue sandy muck.	168/7.
D-20	Depth 15 feet. Near outlet of Union Canal and near edge of Horn Point kelp bed. Weedy sand bottom with no evidence of sewage sludge.	479+/8. Indeterminate numbers of amphipoda crustacea and good variety of other organisms
D-19	Depth 180 feet. Northwest of outfall Sand and blue clay; no evidence of sludge.	1330/9. Teeming with organisms - good variety.

TABLE 5. SUMMARIZED PHYSICAL AND BIOLOGICAL DATA - ZONE IV

Within a four-thousand foot radius of sewage "boil", Seattle, North Trunk outfall

Station	Location and Description	Summarized Results of Bottom Faunal Analysis
S-2	Depth 5 feet from off tip of West Point southeastward	58/5 - 58/5
S-3		37/4 - 37/5
S-4		459/7 - 459/7
S-5	All closely spaced in water less than 15 feet deep. Patches of eel grass on sandy bottoms. Sewage solids in suspension.	Qualitative
S-7	Depth 23 feet. 800 feet west of tip of West Point.	61/7

The remaining samples in this zone came from a small area straight out northwest from the outfall in waters between 240 and 350 feet deep.

D-18	Depth 350 feet. Pocket of sewage sludge, newspapers and leaves.	435/6
D-27	Depth 350 feet. Hard bottom of blue clay.	No sample
D-28	Depth 275 feet. Hard bottom.	No sample
D-34	Depth 240 feet. Clean sand, hard bottom.	10/2. One of least productive areas sampled.
D-35	Depth 255 feet. Gray clay muck and sand. Wood chips and sawdust.	101/1

TABLE 6. BIOLOGICAL DATA - BOTTOM FAUNA FROM PUGET SOUND NEAR WEST POINT SEATTLE, JULY 2, 1957

Organisms	Numbers per square foot of bottom							
	S-1	S-2	S-3	S-4	S-6	S-7	S-8	S-9
Annelid Worms	22	6			114		3	1
Polychaeta Errantia (free living)	93	20	11	330	126	22	21	6
Polychaeta Sedentaria (tube worms or burrowing)	14	10		10	70	7	50	29
Oligochaeta								
Arthropods								
Amphipoda	17	21	24	90	66	21		
Caprella kennerlyi	1							
Isopoda	3			2	48	8	3	
Mysidacea								
Ostracoda			1			1		
Decapoda				1	1	1		
Cirripedia				1				
Round Worms		1						
Flat Worms				1				
Mollusca								
Gastropods (snails)					1	1	1	
Pelecypods (clams)			1	24	14		1	
Total	150	58	37	459	440	61	79	36

TABLE 7. BIOLOGICAL DATA - BOTTOM FAUNA FROM PUGET SOUND NEAR WEST POINT, SEATTLE

Organisms	Numbers per square foot of bottom									
	D-10 7/2/57	D-12 7/3/57	D-13 7/3/57	D-17 7/3/57	D-18 7/3/57	D-19 7/3/57	D-20 7/22/57	D-21 7/22/57	D-22 7/22/57	D-23 7/22/57
Annelid Worms	1						45	56		
Polychaeta Errantia	79	65	40	62	11	27	40	115	62	
Polychaeta Sedentaria	100	40	41*	31	400	1200	17	140	Abun-dant	
Oligochaeta									2300	72
Gephyrea										
Physcosoma agassizzi						7				
Arthropods										
Amphipoda	12	8	1	3	8	38	1000≠	92	500	
Caprella kennerlyi	14						264			
Isopoda	8		8			14	62		1060	
Decapoda										
Mysidacea							17	45		
Ostracoda	3							185	56	
Round Worms - Nemathelminthes										56
Mollusca										
Gastropods	21	11	11	10	1	24	6			
Pelecypods	20	25	18		12	14	28			
Echinoderms										
Ophiuroidea				4						
Echinoidea				3	3	1				
Holothuroidea						6				
Total	258	149	119	113	435	1331	479	633	3983	128

* Other tube worms
abundant - very small
≠ Not included in total count

TABLE 8. BIOLOGICAL DATA - BOTTOM FAUNA FROM PUGET SOUND NEAR WEST POINT, SEATTLE, JULY 22, 1957.

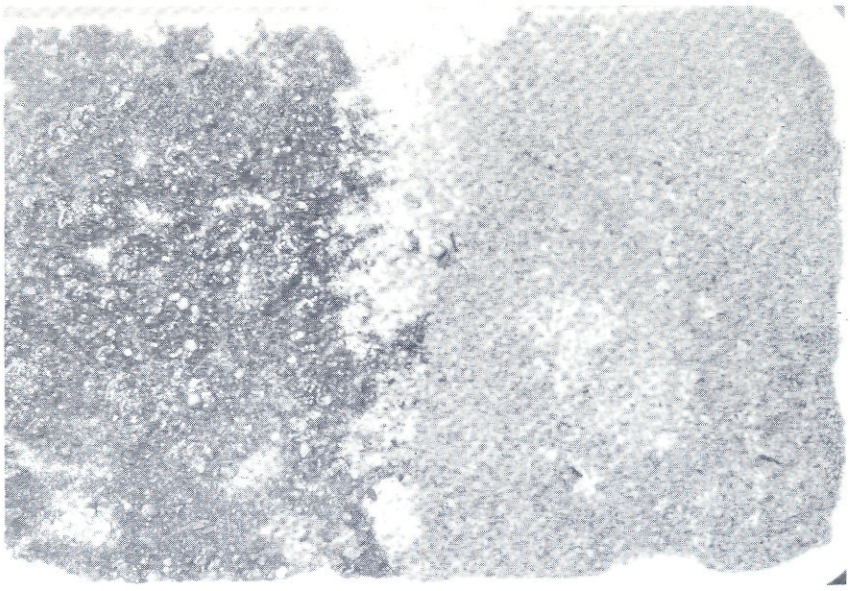
Organisms	Numbers per square foot of bottom									
	D-24	D-25	D-26	D-29	D-30	D-31	D-32	D-33	D-34	D-35
Annelid Worms										
Polychaeta Errantia	11	40*	39*	63	56	25	730	72*	4*	101*
Polychaeta Sedentaria	50			49	51	74	2,400			
Oligochaeta	23	8	4			1				
Hirudinea								1		
Arthropods										
Amphipoda	39	87	1	7	6	11	Abundant	3		
Decapoda								1		
Isopoda			3	21		4	280	1		
Mysidacea								1		
Ostracoda		1						1		
Round Worms - Nemathelminthes	Abundant						390		36	
Mollusca										
Gastropods				7	12	4		12		
Pelecypods		7	3	14	7	8		30		
Echinoderms										
Ophiuroidea		1		7	1					
Coelenterates										
Anthozoa						1				
Total	123	142	50	168	133	128	3,800	122	10	101

*Includes P. Sedentaria also - tube worms.

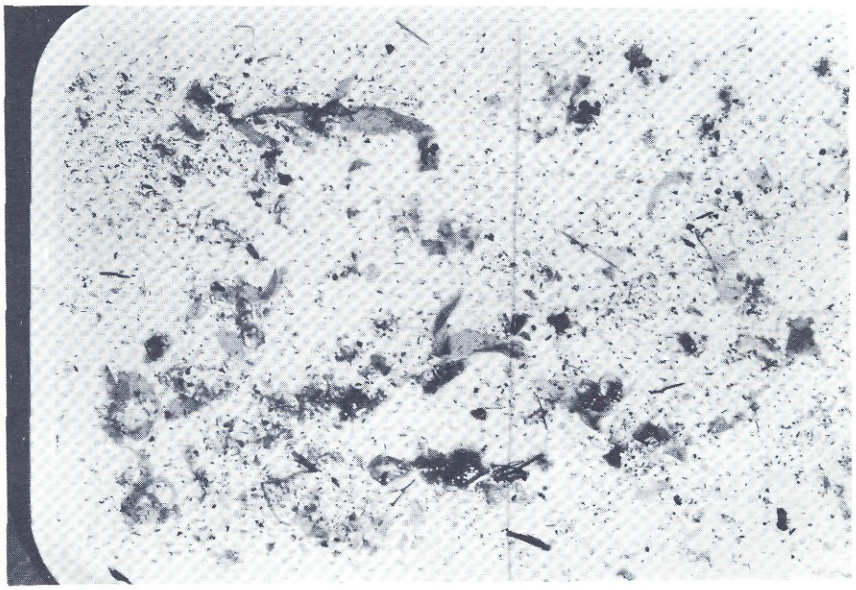
TABLE 9. BIOLOGICAL DATA - BOTTOM FAUNA FROM PUGET SOUND NEAR WEST POINT, SEATTLE AND PIPER CREEK

Organisms	Number per square foot of bottom							
	D-36 7/22/57	D-38 7/22/57	D-39 7/22/57	D-40 7/22/57	North of Piper Cr. off Spring Beach 7/22/57	E-1 Piper Cr. 7/22/57	E-2 7/22/57	E-4 7/22/57
Annelid Worms	44			6				
Polychaeta Errantia	132	115	28*	56	10	25	12	25
Polychaeta Sedentaria	970	480		140	30	6		
Oligochaeta		1,720	4					
Arthropods								
Amphipoda	22	300	4		27	8	8	6
Decapoda			3					
Isopoda	1,185	650	4	17			1	
Amphipoda: Nebalia sp.		92						
Mysidacea								1
Ostracoda					1	140		6
Cirripedia (skeletons only)								3
Mullusca								
Gastropods				17	10	1	1	30
Pelecypods			3	22	7	4		
Amphineura				2				
Nemathelminthes		160					34	
Echinoderms								
Ophiuroidea					1			
Total	2,375	3,517	46	260	86	240	56	71

*Includes tube worms.

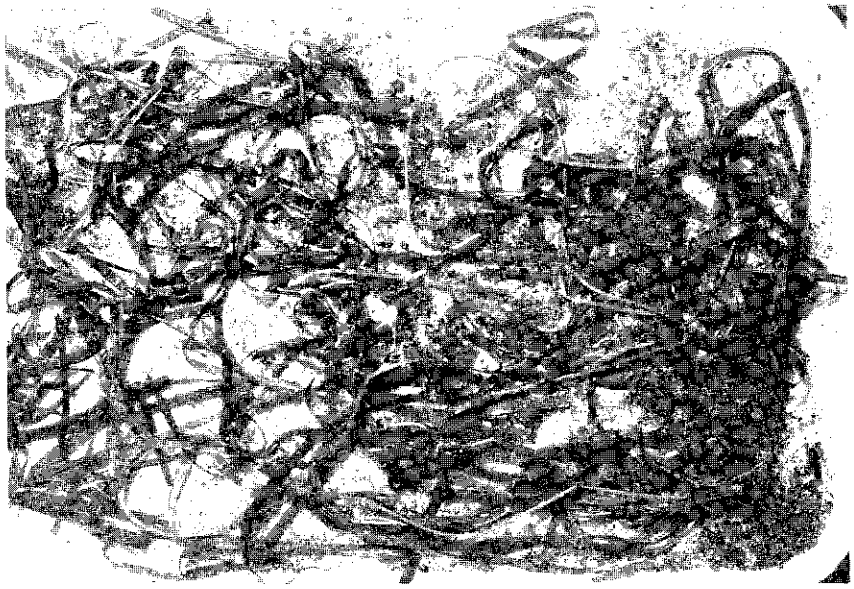


Station D-18. Sludge pocket about 4,000 feet from sewer outfall at 350-foot depth. Sample divided to show light colored sludge particles on right and dark sand and gravel with stick-like tube worms on left.

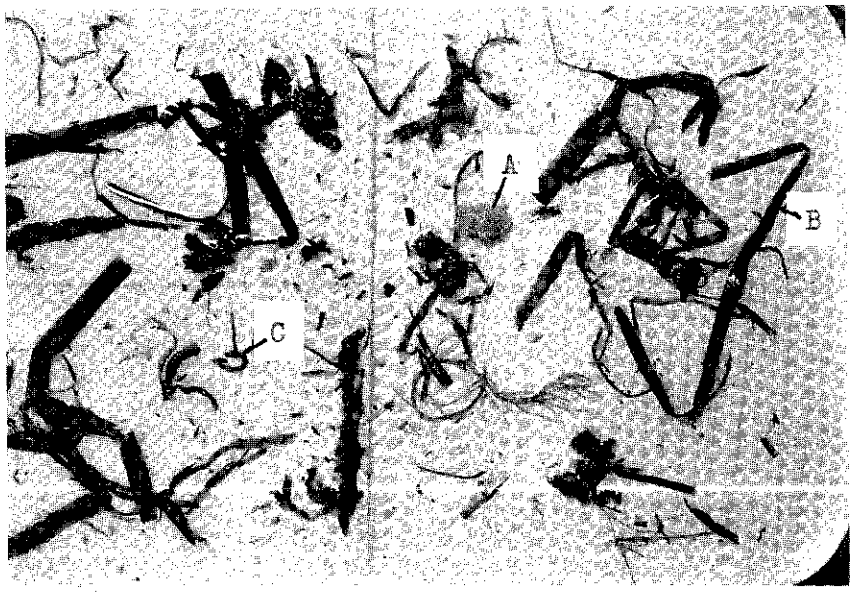


Station D-24. Shoreward side of sewage "boil" near kelp bed. Ribbon-like material is algae.

FIGURE 2

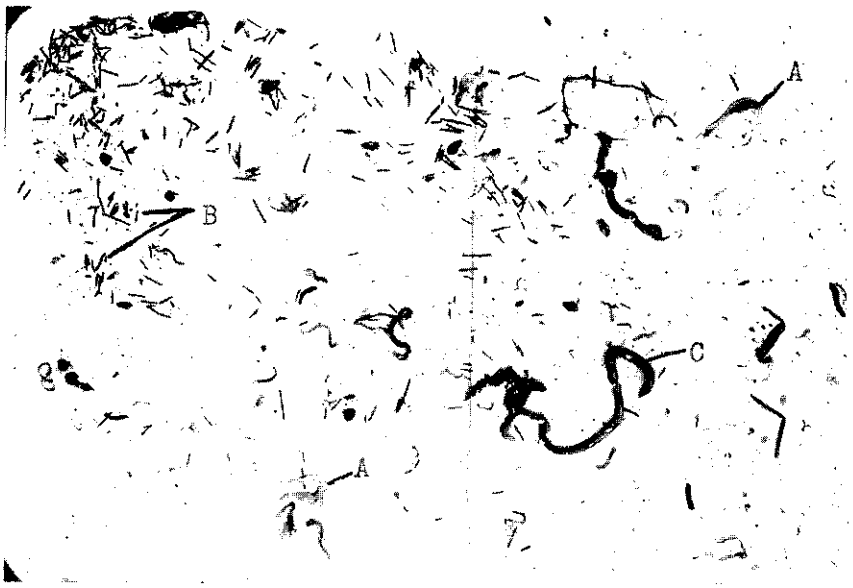


Station S-1. Patch of eelgrass growing on a sand bottom.

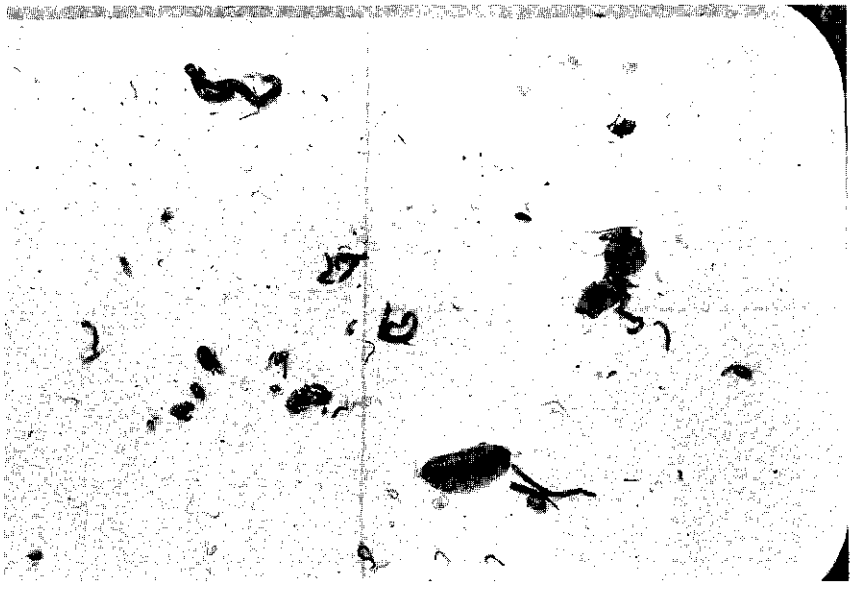


Station S-7. Off West Point at 23-foot depth. Note small crab (A) eelgrass (B) and Polychaete worm (C).

FIGURE 3



Station D-19. Bottom animals from an area free from sludge deposits at 180-foot depth. Peanut worms (A), Tube worms (B), Oligochaete worm (C).



Station D-17. Sample from depth of 550 feet.

FIGURE 4

APPENDIX D

RESULTS OF CURRENT STUDIES IN PUGET SOUND

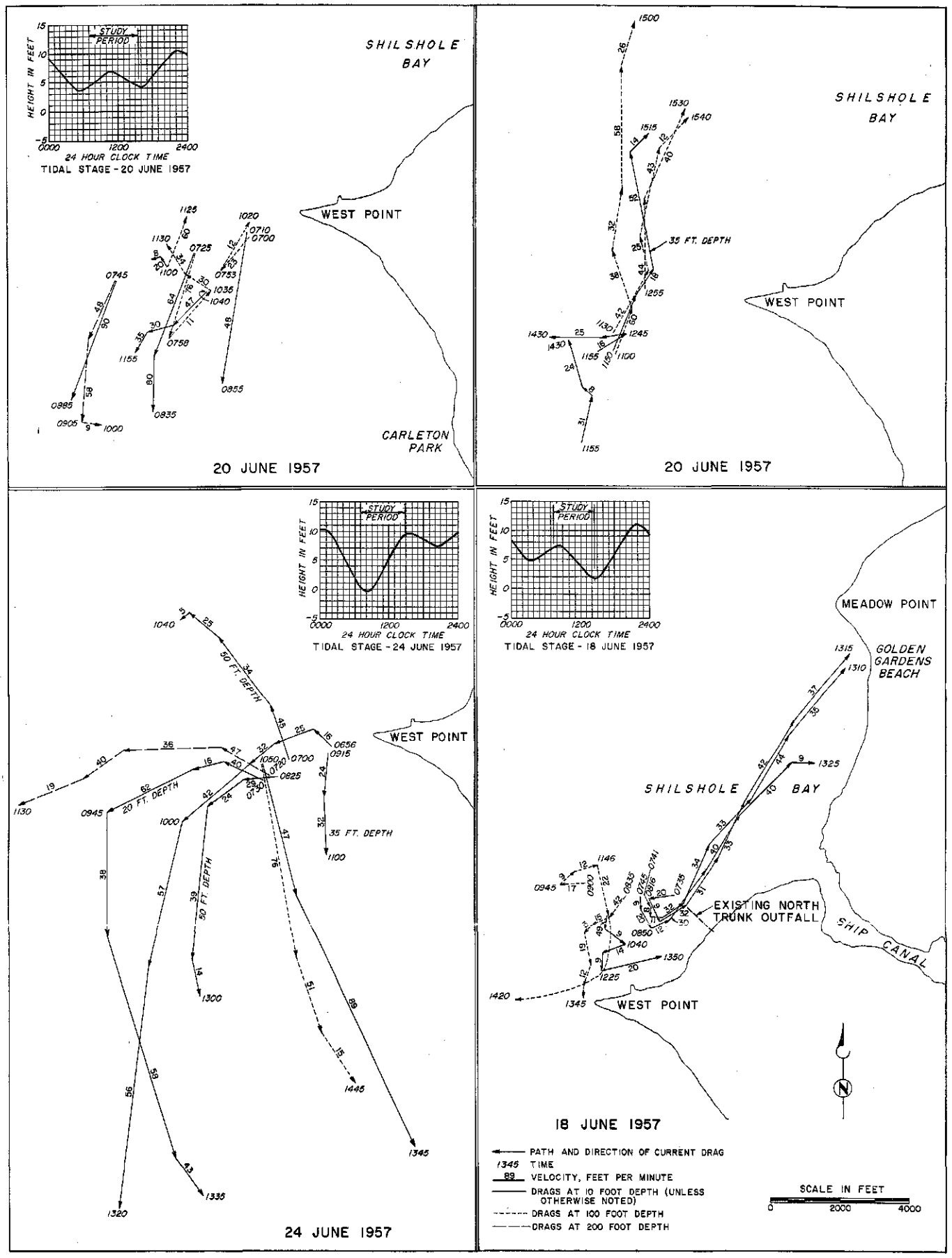


Fig. D-1. West Point Area - June 18, 20 and 24, 1957

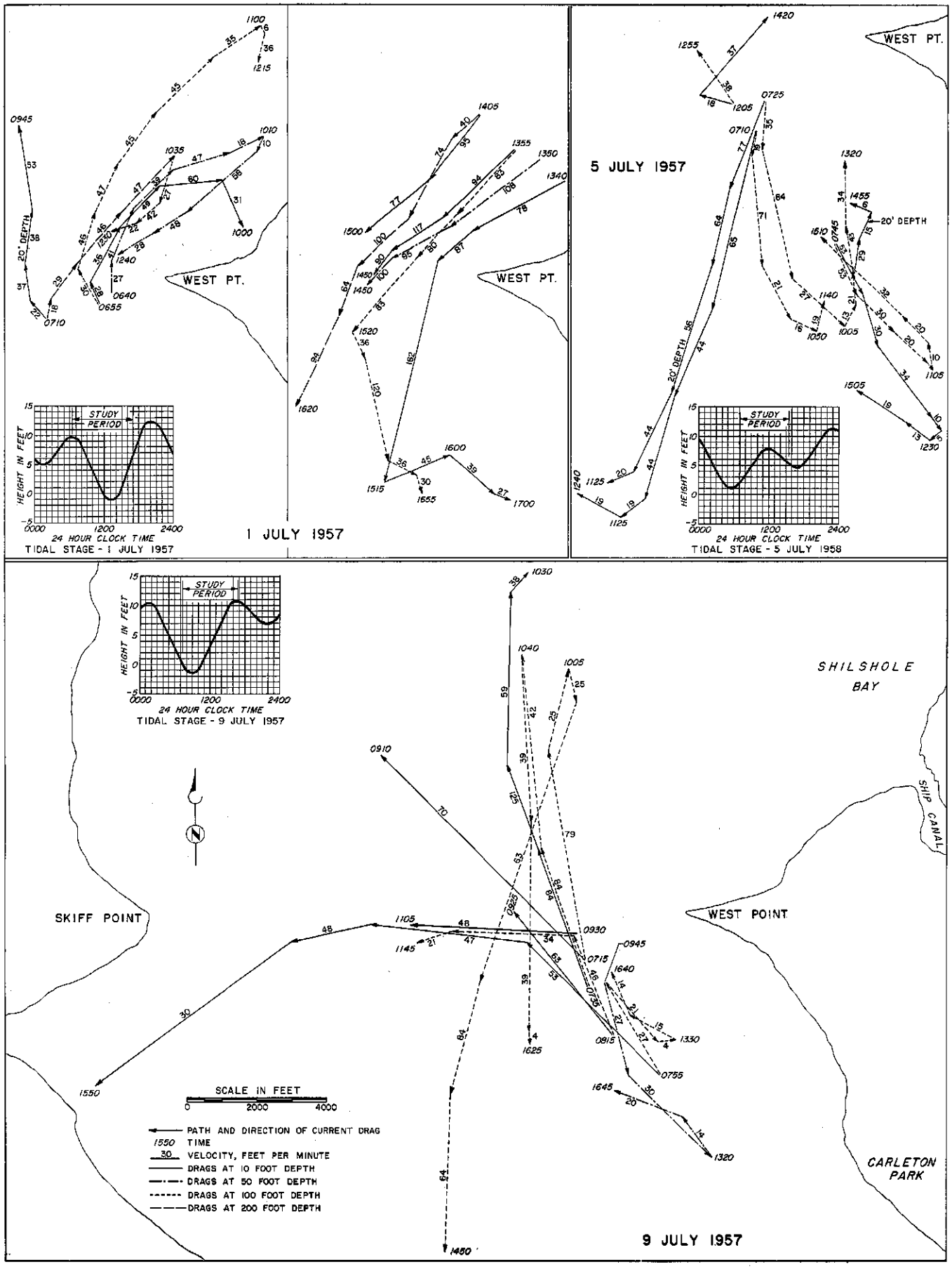


Fig. D-2. West Point Area - July 1, 5 and 9, 1957

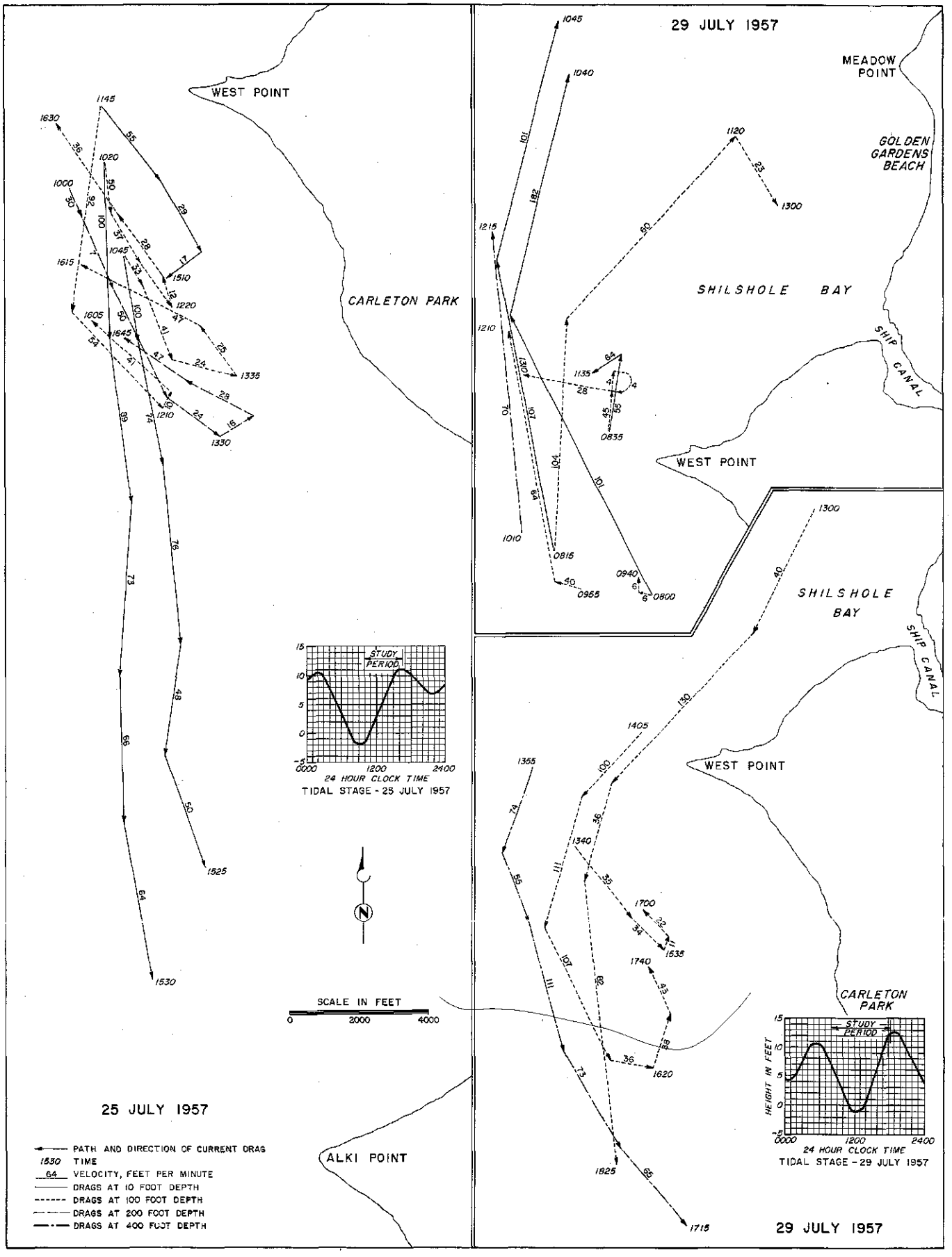


Fig. D-3. West Point Area - July 25 and 29, 1957

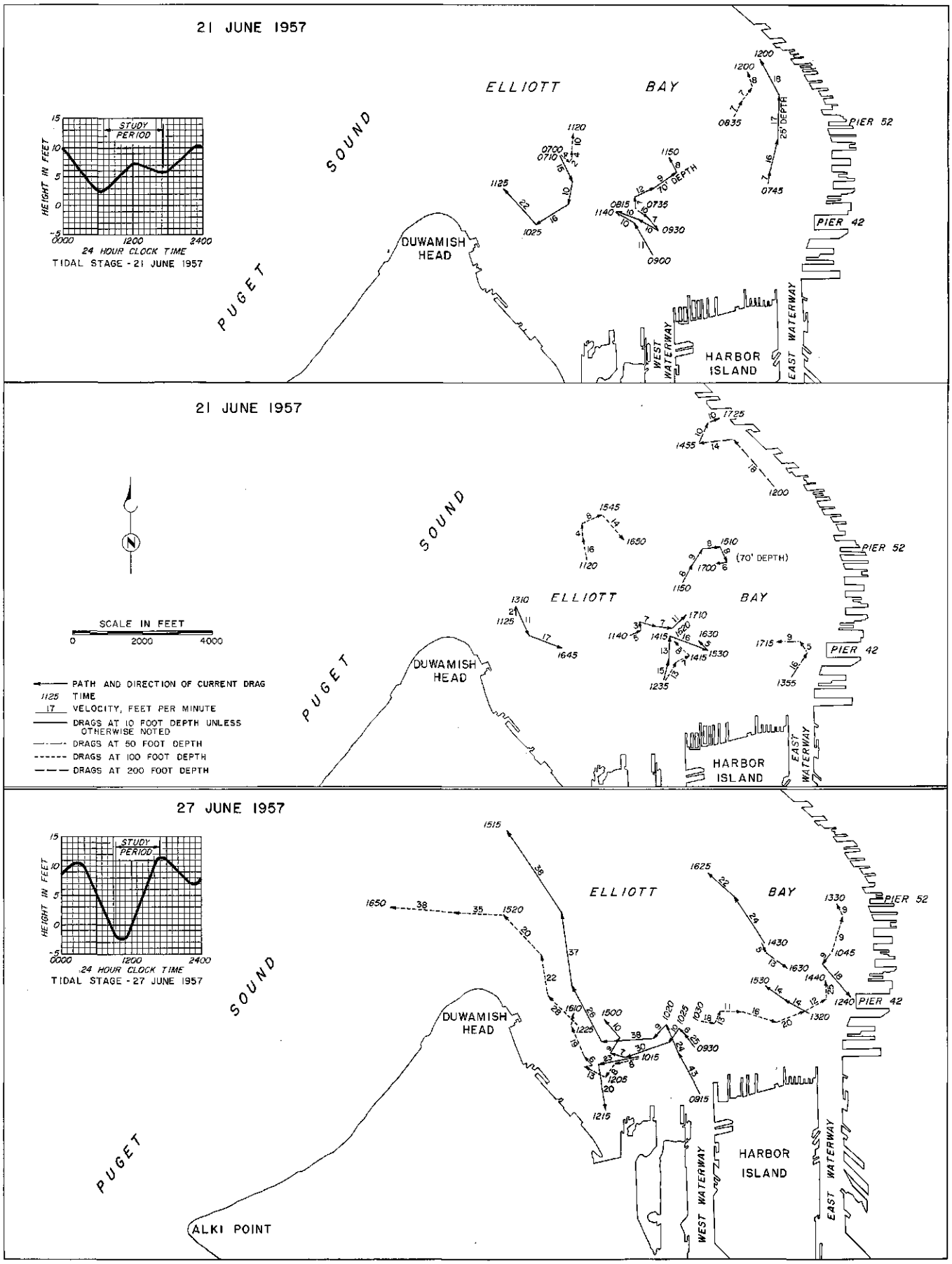


Fig. D-4. Elliott Bay - June 21 and 27, 1957

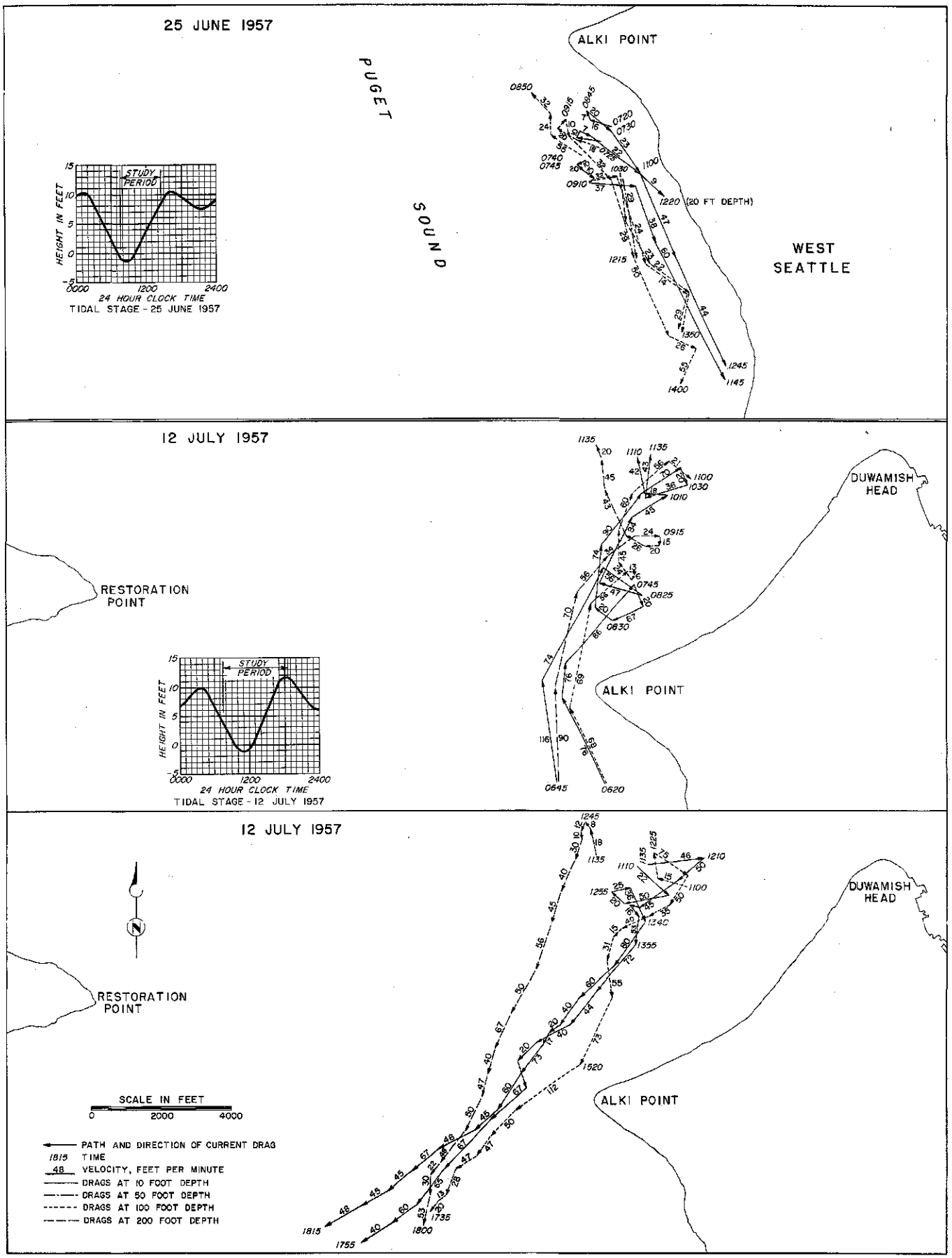


Fig. D-5. Alki Point Area - June 25 and July 12, 1957

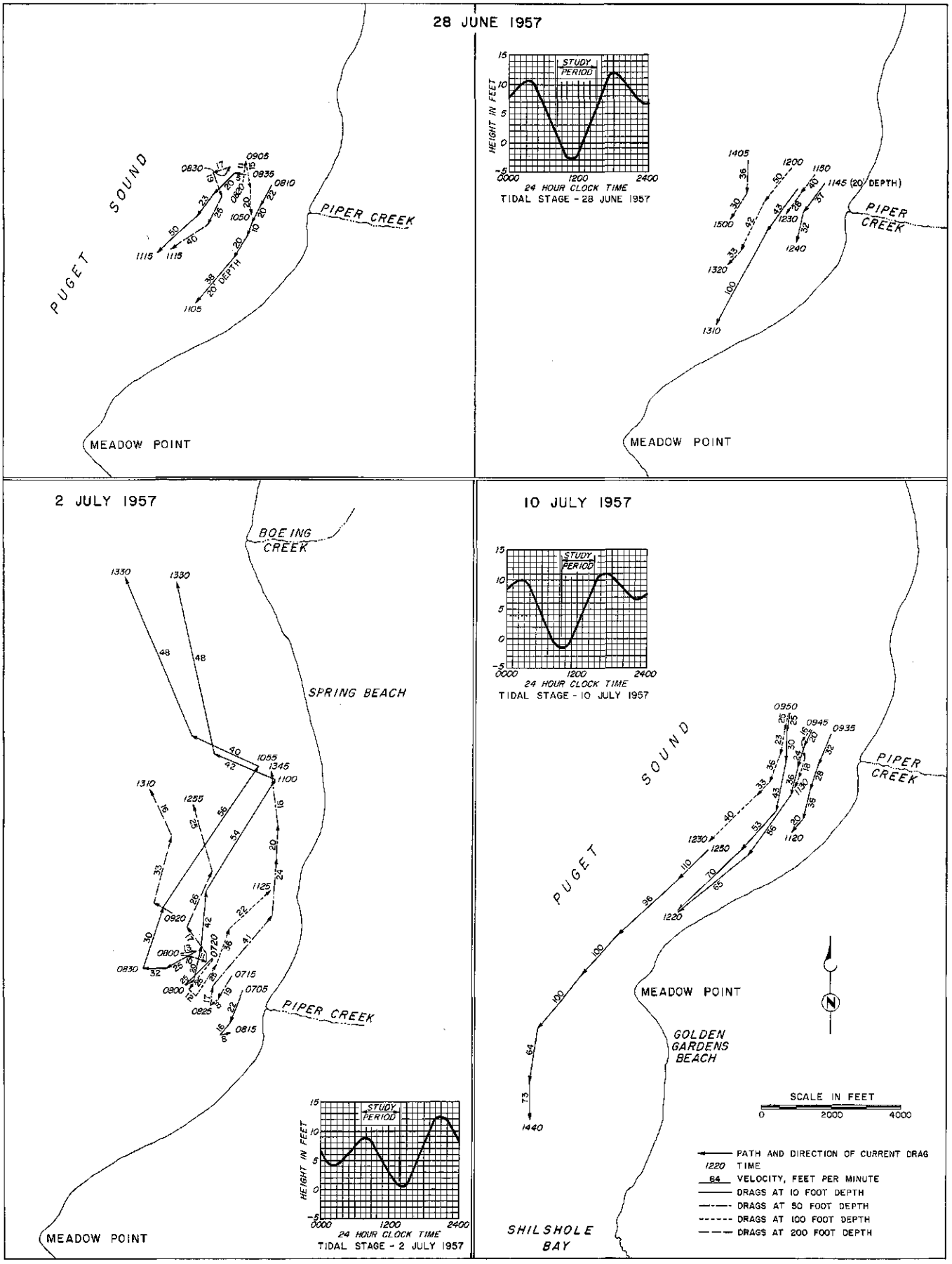


Fig. D-6. Piper Creek Area - June 28, July 2 and 10, 1957

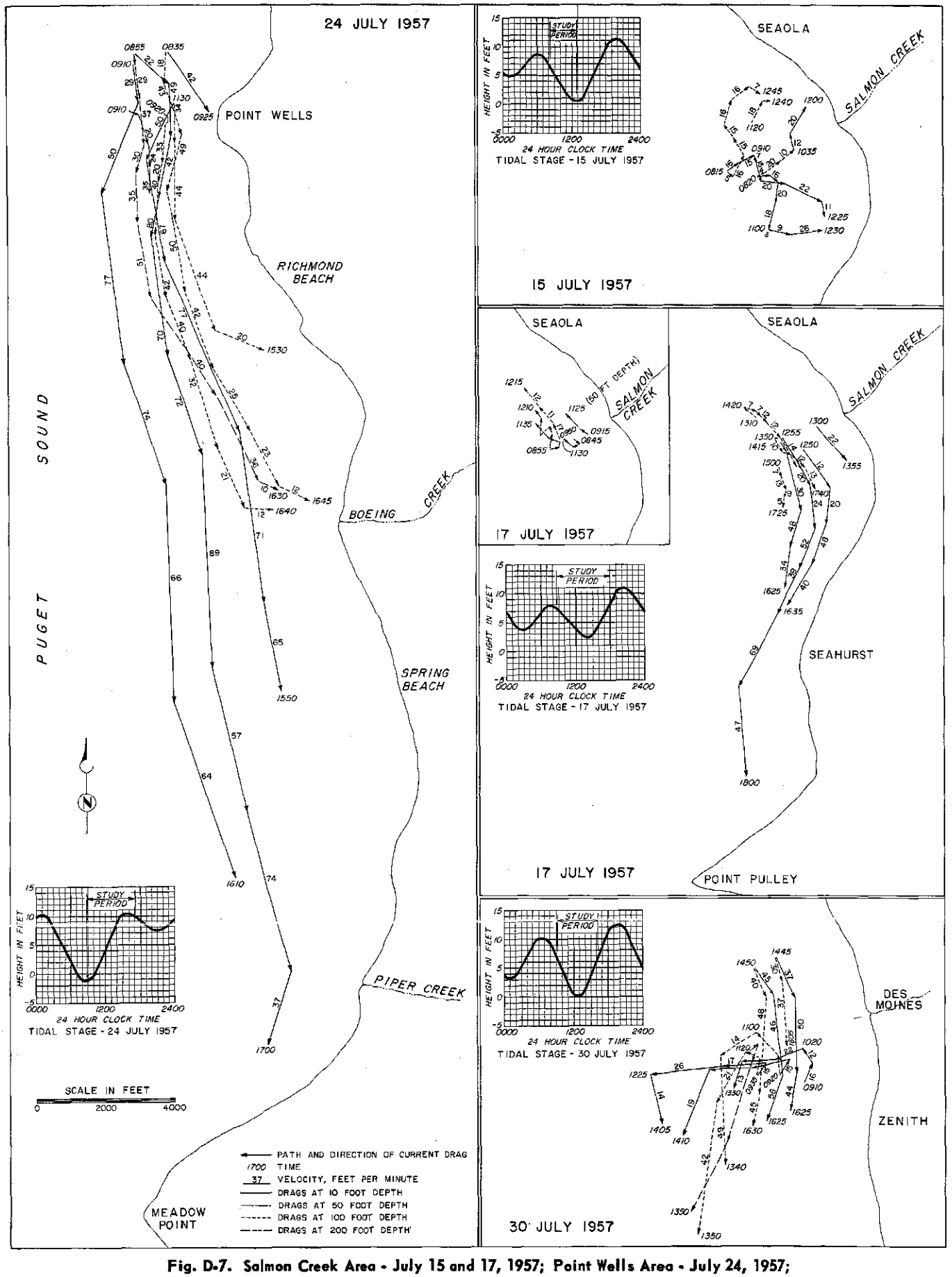


Fig. D-7. Salmon Creek Area - July 15 and 17, 1957; Point Wells Area - July 24, 1957; Des Moines Area - July 30, 1957

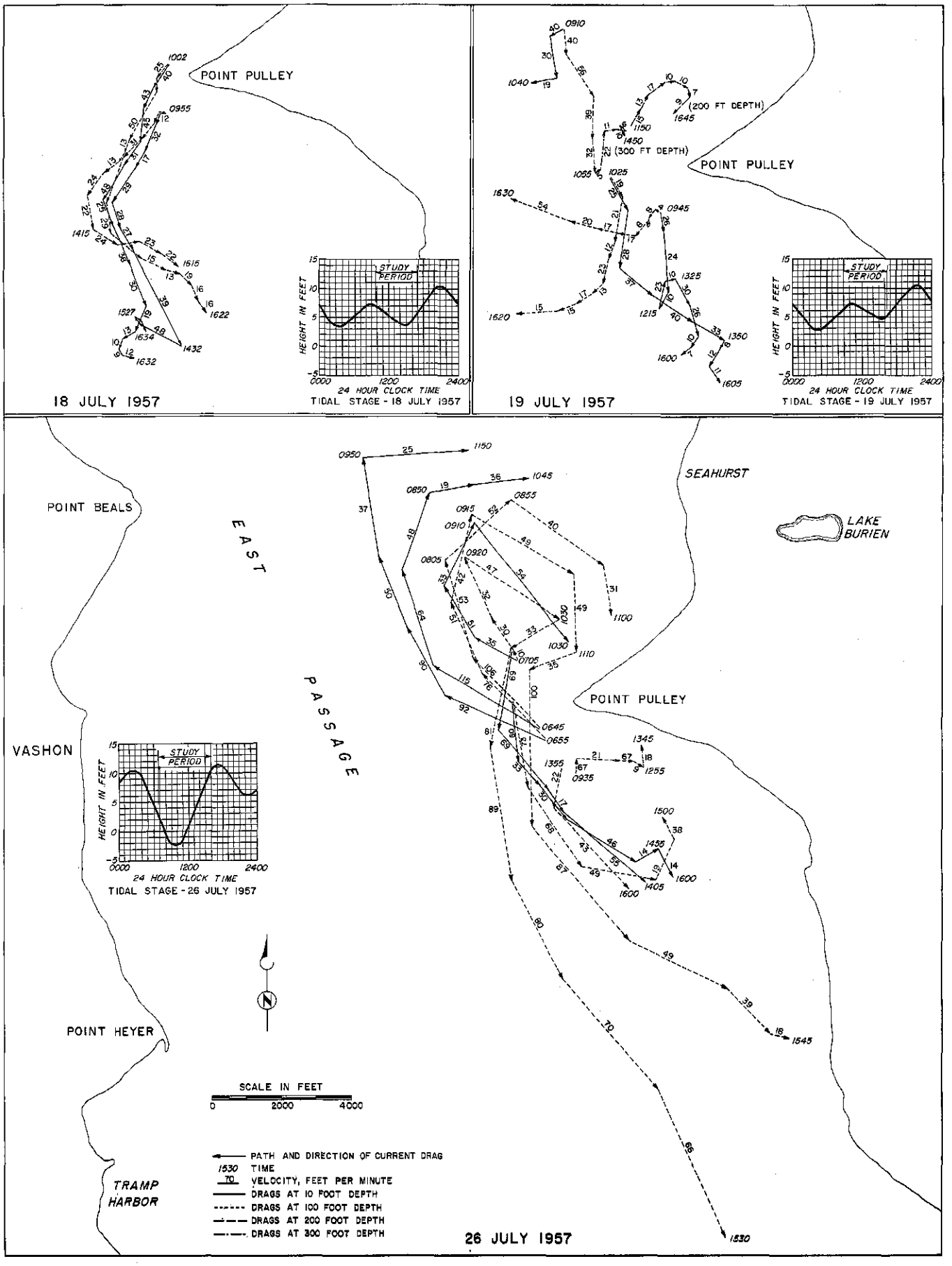
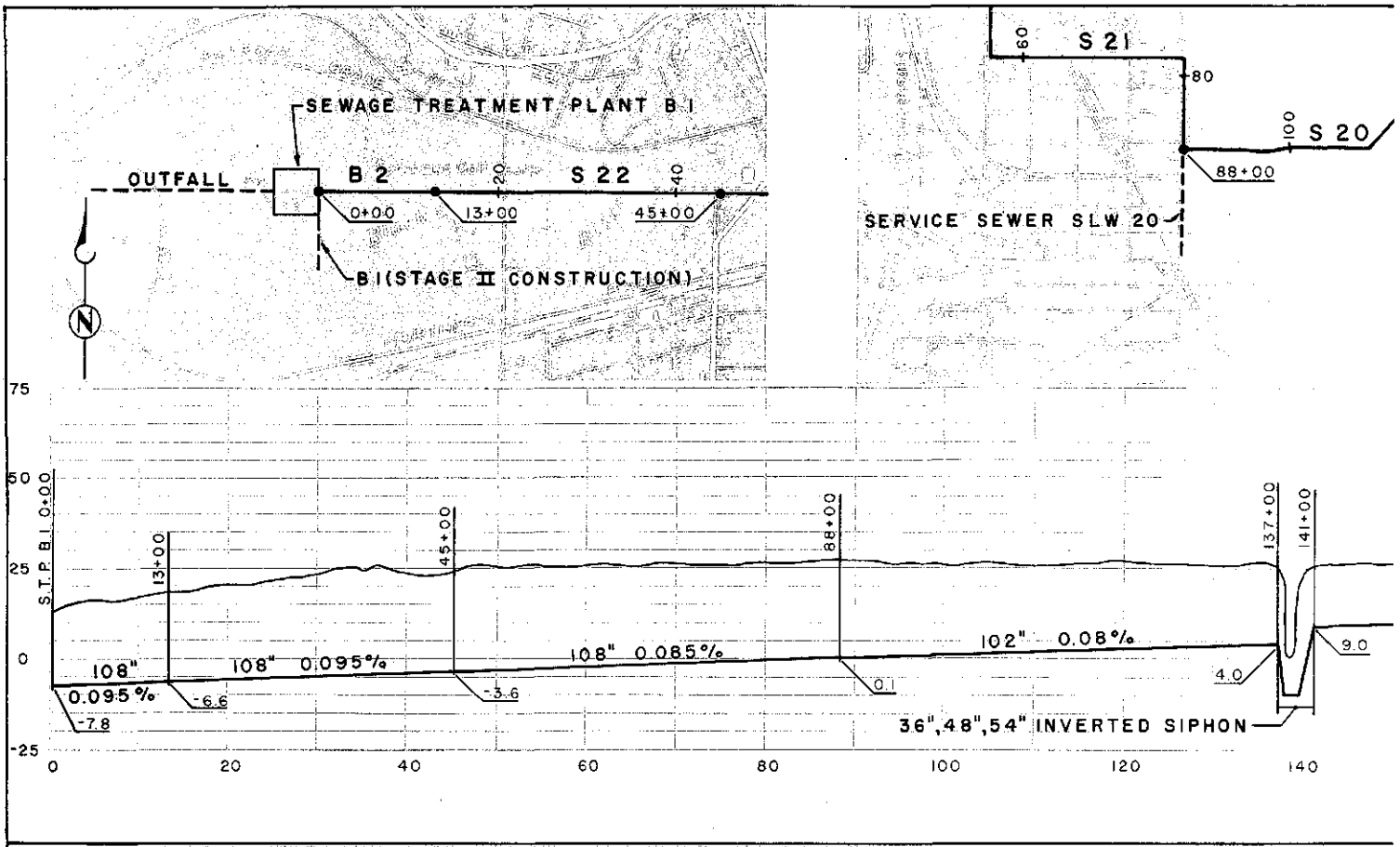
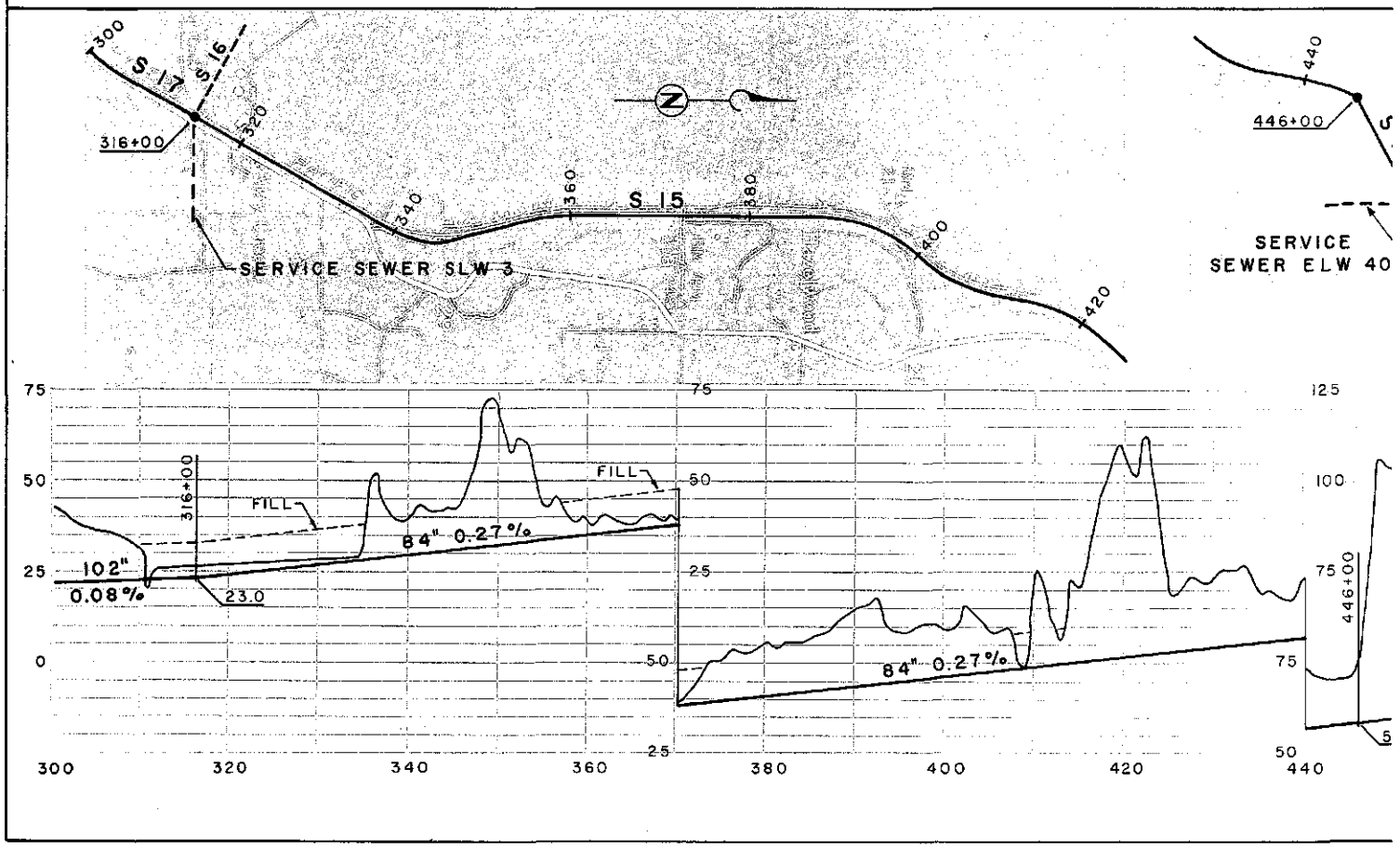


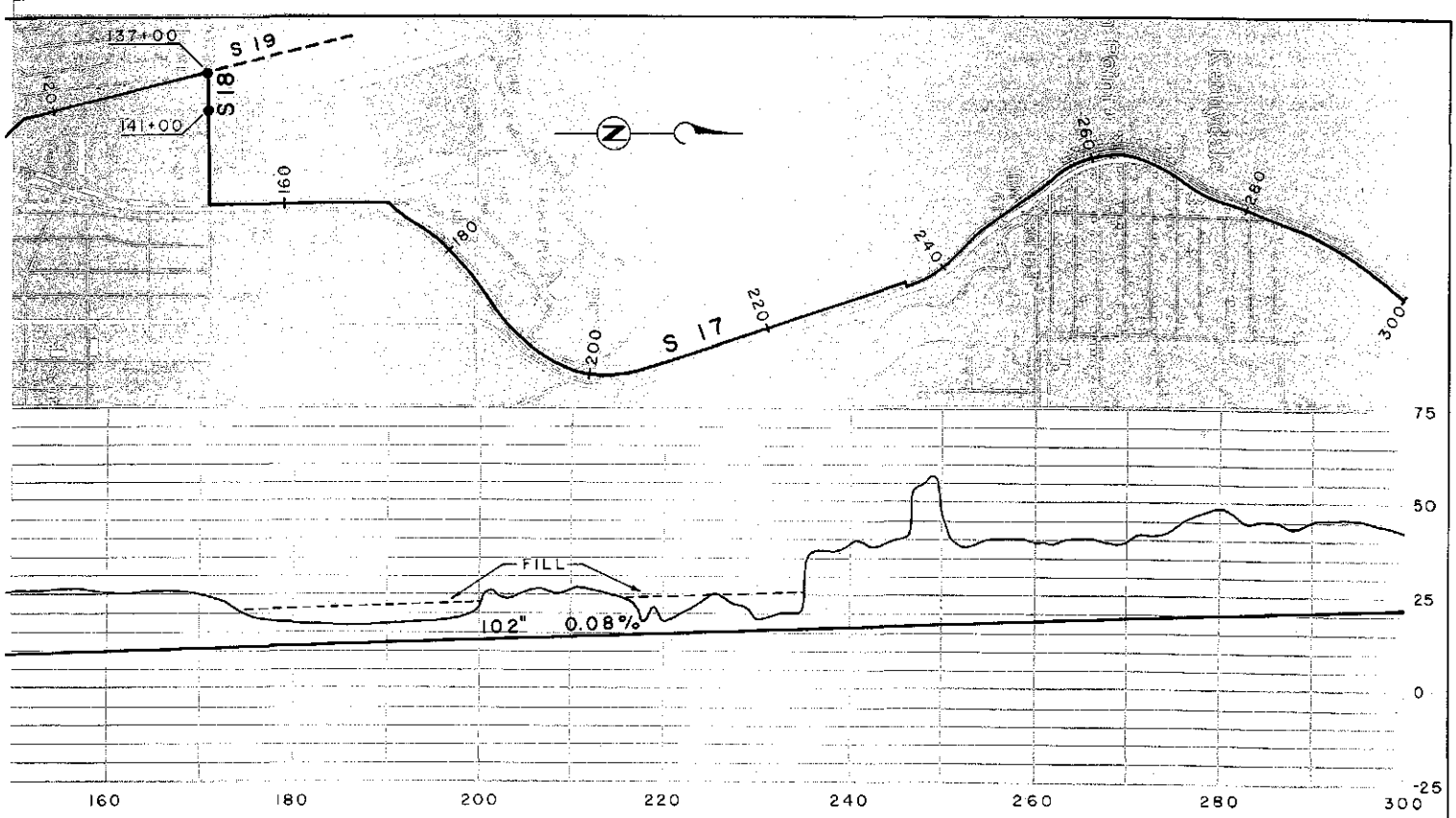
Fig. D-8. Point Pulley Area - July 18 and 19, 1957

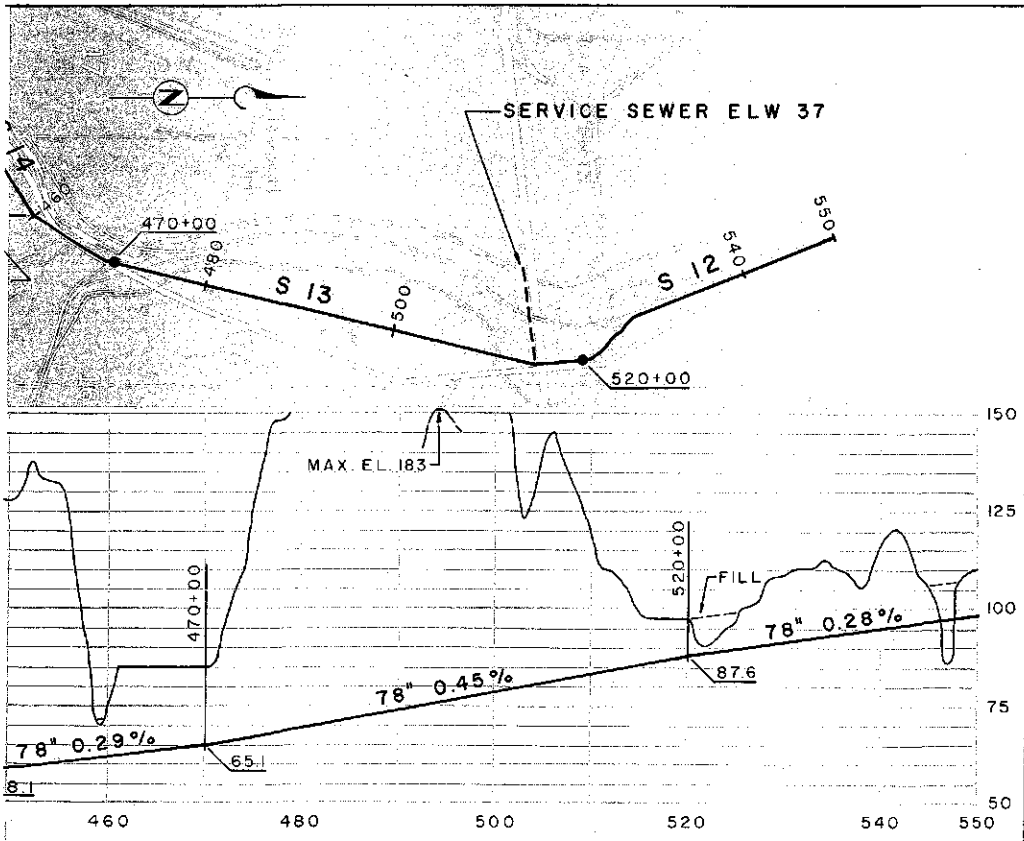
APPENDIX E

PRELIMINARY SEWER PLANS AND PROFILES
STAGE I CONSTRUCTION
CENTRAL SEWERAGE SYSTEM


SEWAGE TREATMENT PLANT B I
OUTFALL
B 2
S 22
0+00
13+00
45+00
B I (STAGE II CONSTRUCTION)
S 21
S 20
88+00
SERVICE SEWER SLW 20
S.T.P. B I 0+00
13+00
45+00
88+00
137+00
141+00
108"
0.095%
108" 0.095%
108" 0.085%
102" 0.08%
-7.8
-6.6
-3.6
0.1
4.0
9.0
36",48",54" INVERTED SIPHON


S 17
S 16
316+00
S 15
SERVICE SEWER SLW 3
446+00
SERVICE SEWER ELW 40
FILL
FILL
102"
0.08%
23.0
84" 0.27%
84" 0.27%
316+00
446+00





B 2 — PROPOSED SEWER AND DESIGNATION

4.0 — SEWER INVERT AND ELEVATION

ELEVATIONS ARE IN FEET AND REFER TO U.S.G.S. MEAN SEA LEVEL DATUM (1929).

GRAVITY SEWERS TO BE CONSTRUCTED OF REINFORCED CONCRETE PIPE WITH RUBBER GASKET TYPE JOINTS.

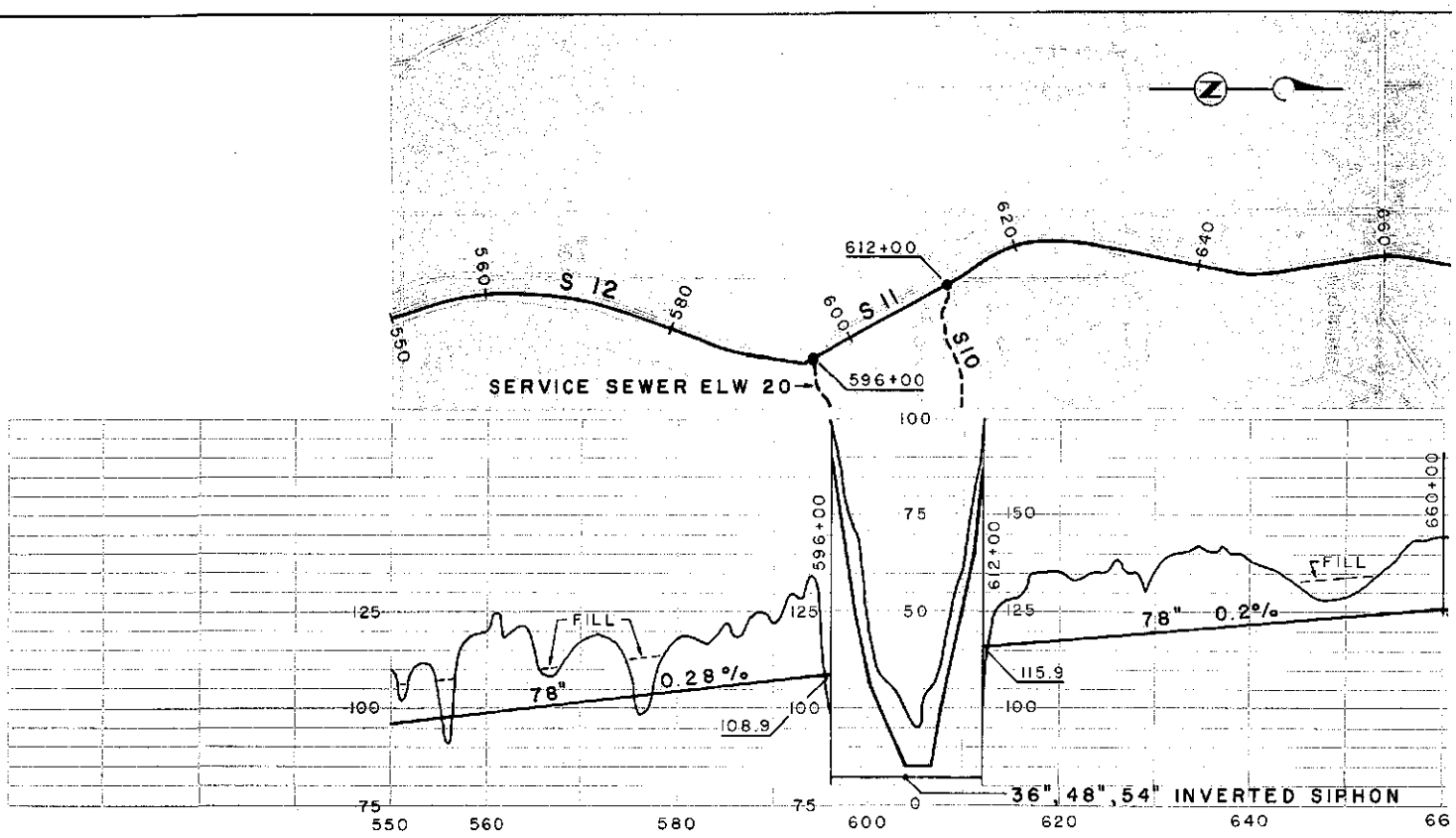
HORIZONTAL SCALE: 1" = 2000'
VERTICAL SCALE AS SHOWN.

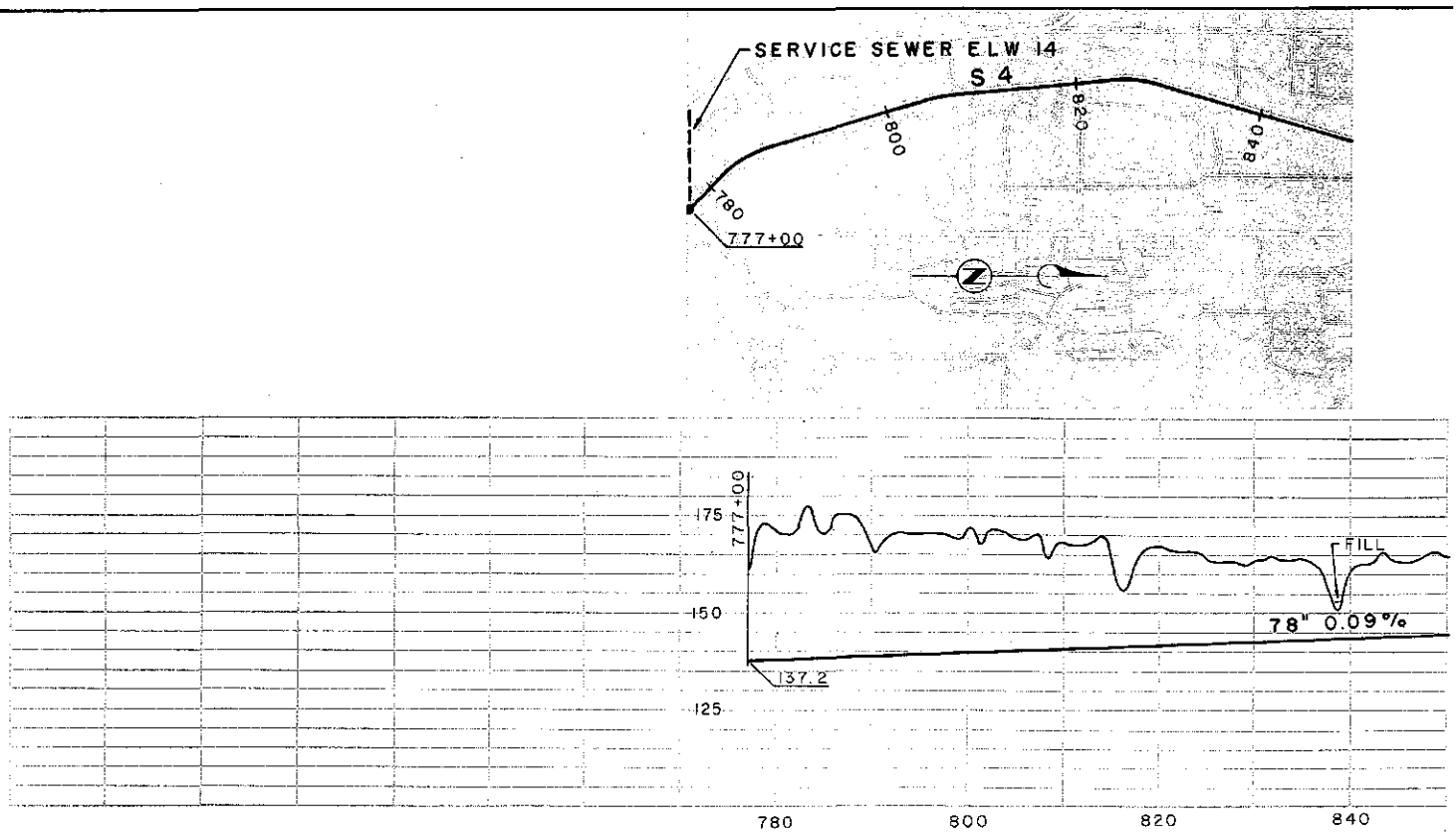
METROPOLITAN SEATTLE SEWERAGE AND DRAINAGE SURVEY

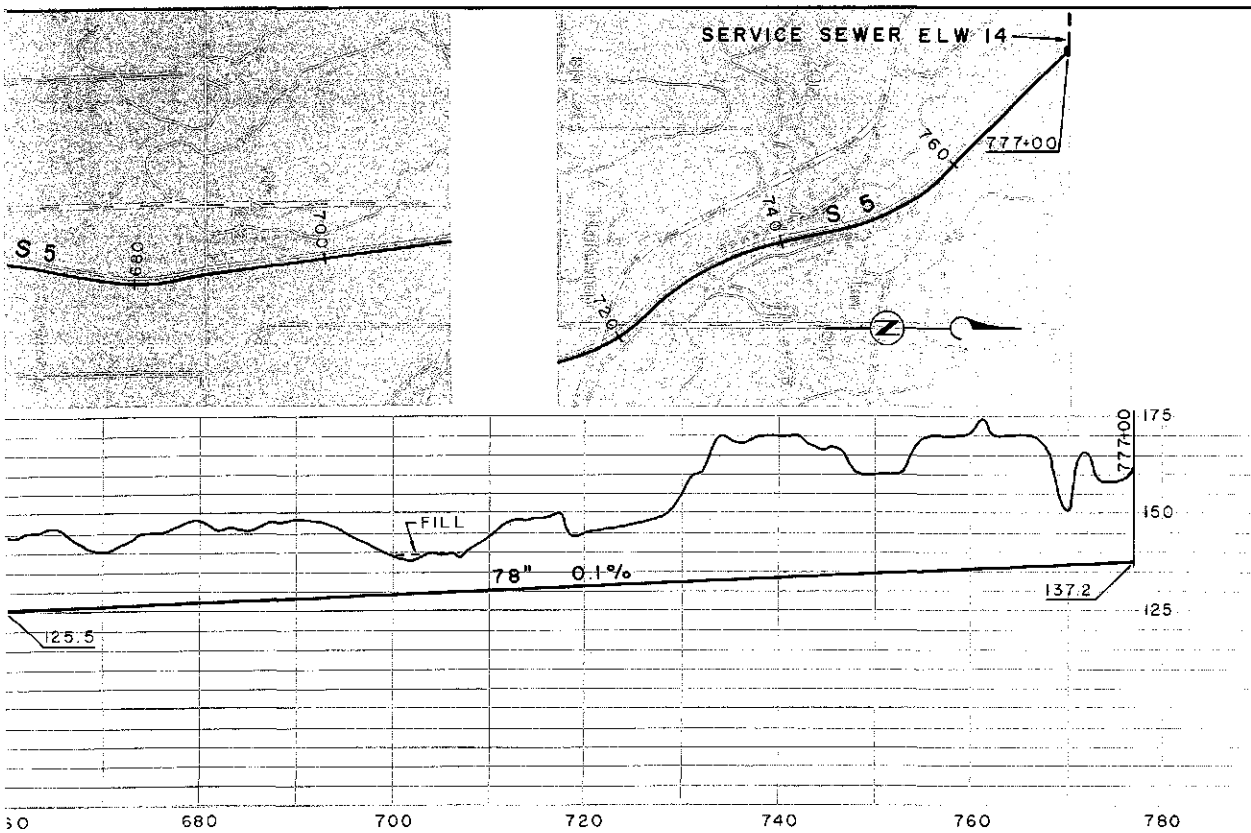
PRELIMINARY PLAN AND PROFILE

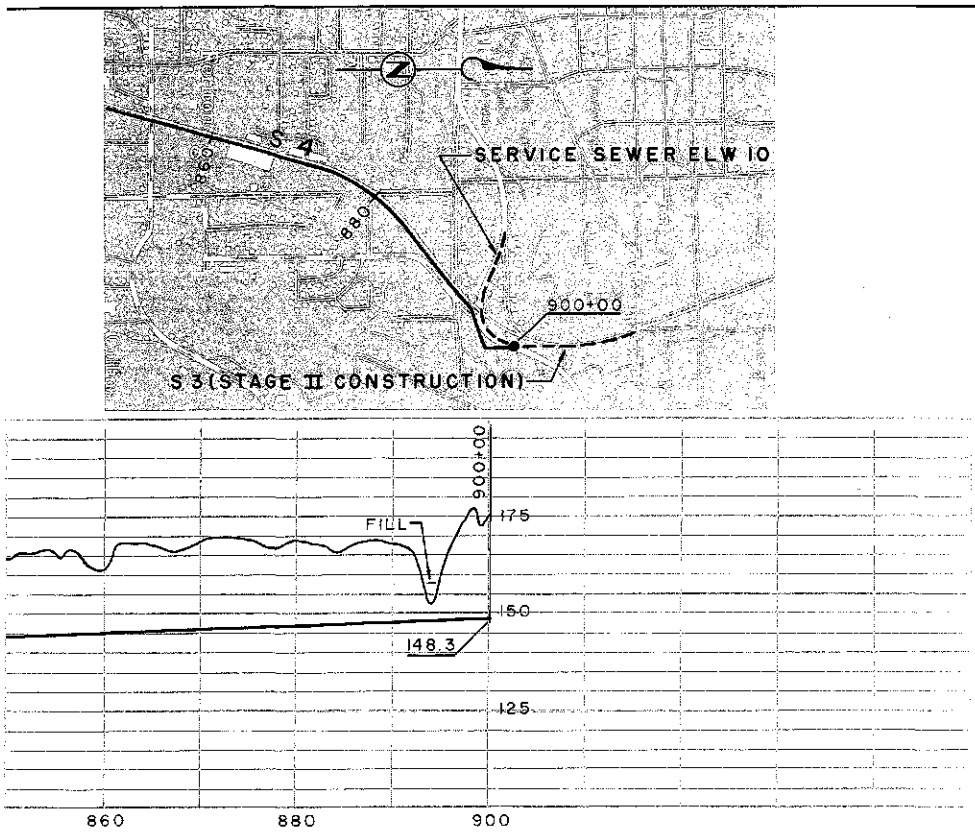
RENTON SYSTEM INTERCEPTING SEWERS

BROWN AND CALDWELL CIVIL AND CHEMICAL ENGINEERS
SAN FRANCISCO, CALIFORNIA. SEATTLE, WASHINGTON


S 12
S 11
S 10
550
560
580
600
620
640
660
612+00
596+00
SERVICE SEWER ELW 20
596+00
612+00
660+00
FILL
78"
0.28 %
108.9
115.9
78" 0.2%
36",48",54" INVERTED SIPHON
100
75
50
0
125
150
175
550
560
580
600
620
640
66


SERVICE SEWER ELW 14
S 4
780
800
820
840
777+00
777+00
FILL
78" 0.09%
137.2
175
150
125
780
800
820
840





S 4 — PROPOSED SEWER AND DESIGNATION

115.9 — SEWER INVERT AND ELEVATION

ELEVATIONS ARE IN FEET AND REFER TO U.S.G.S. MEAN SEA LEVEL DATUM (1929).

GRAVITY SEWERS TO BE CONSTRUCTED OF REINFORCED CONCRETE PIPE WITH RUBBER GASKET TYPE JOINTS.

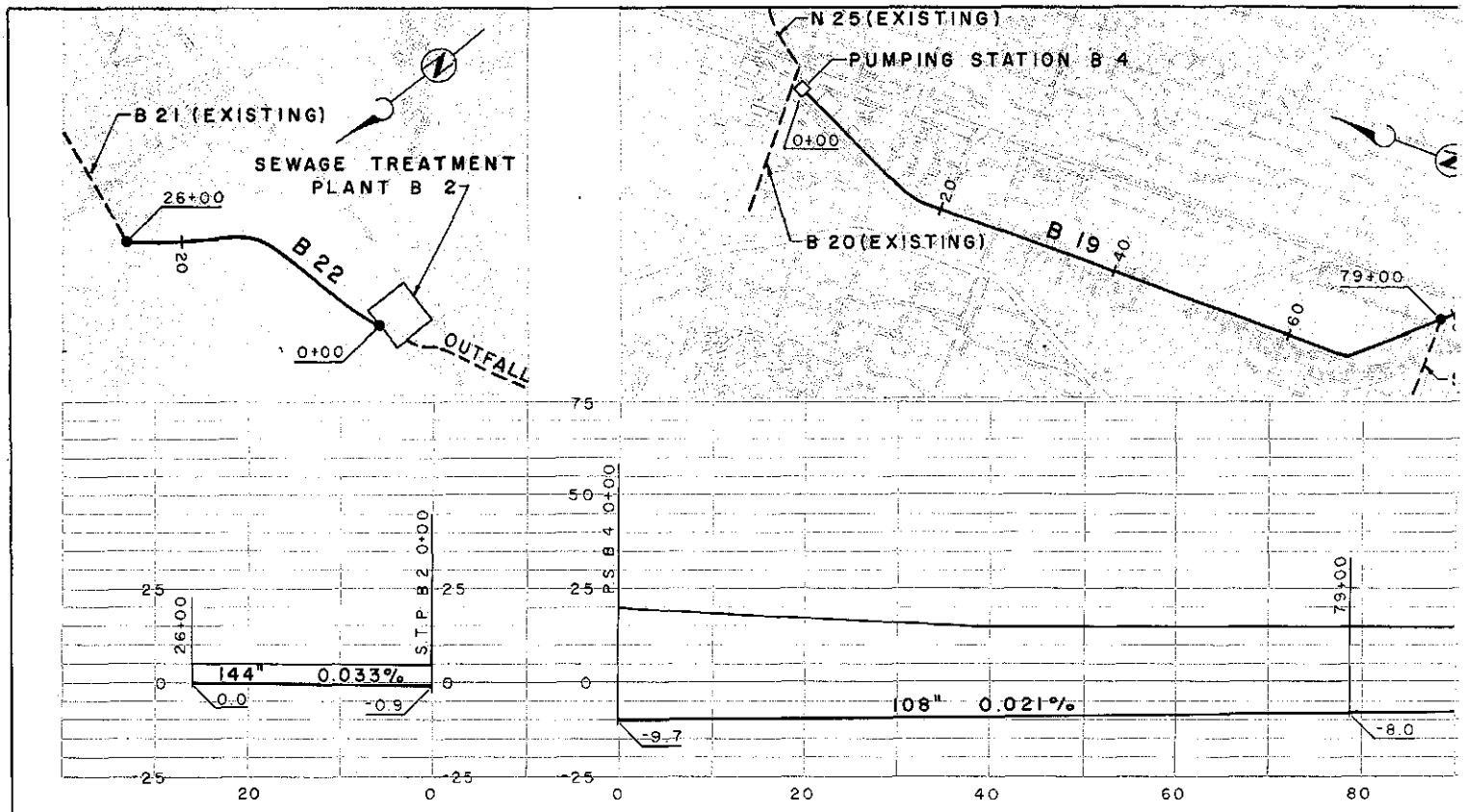
HORIZONTAL SCALE: 1" = 2000'
VERTICAL SCALE AS SHOWN.

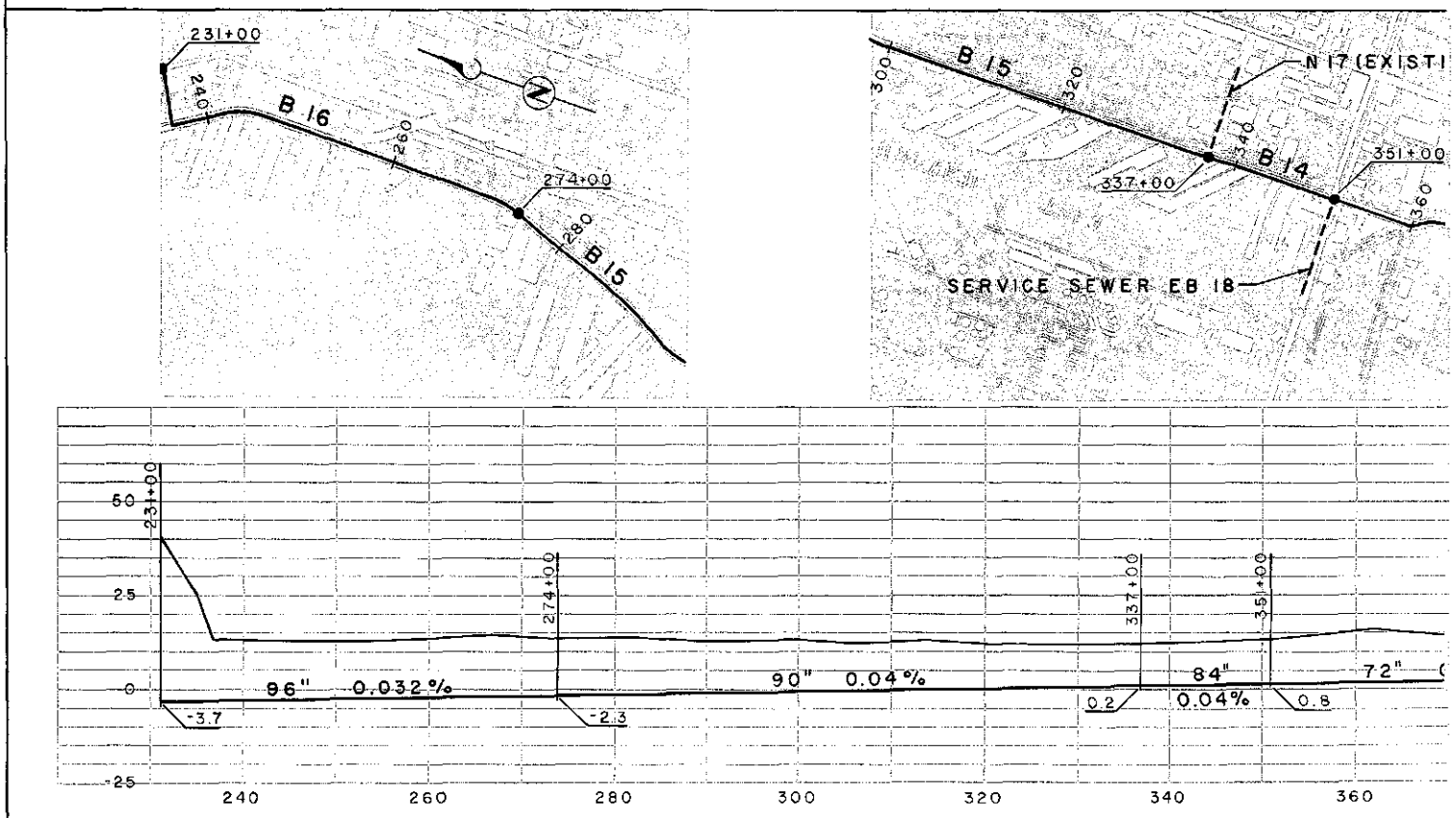
METROPOLITAN SEATTLE SEWERAGE AND DRAINAGE SURVEY

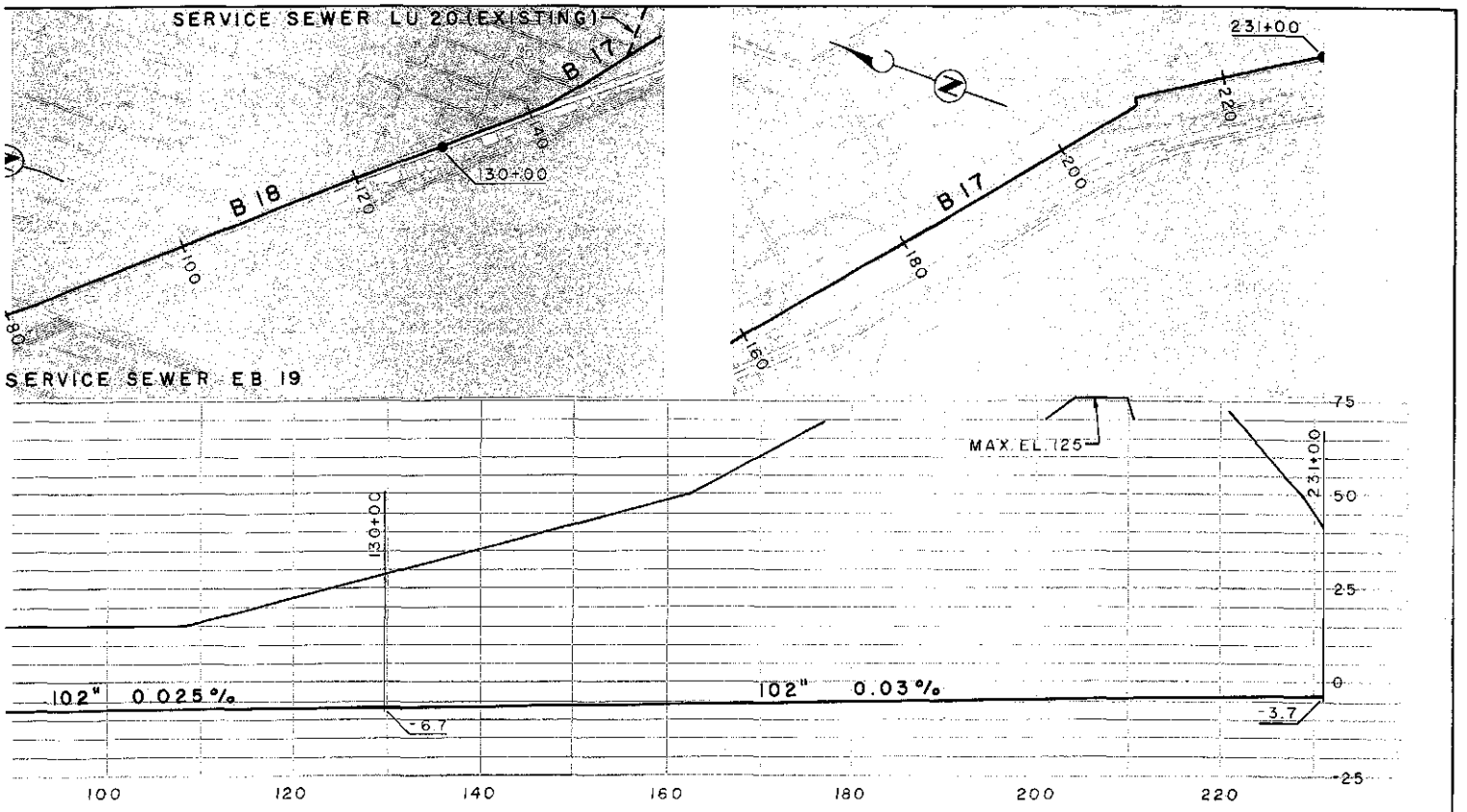
PRELIMINARY PLAN AND PROFILE

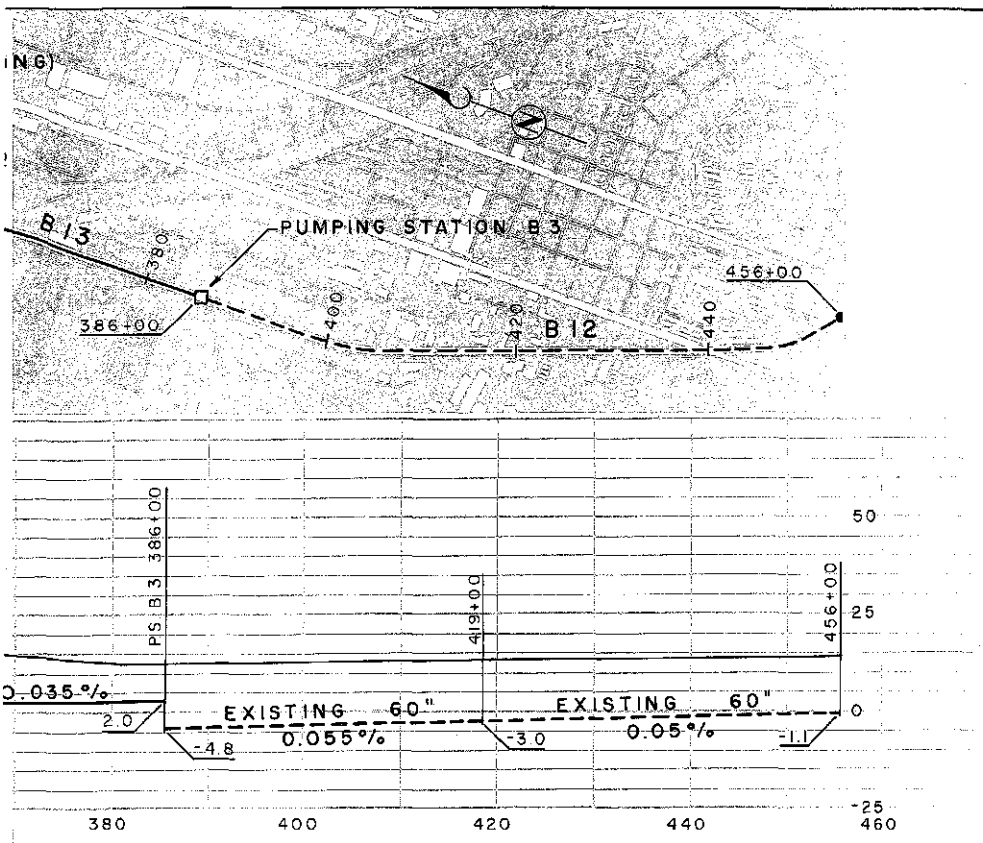
RENTON SYSTEM
INTERCEPTING SEWERS

BROWN AND CALDWELL CIVIL AND CHEMICAL ENGINEERS
SAN FRANCISCO, CALIFORNIA SEATTLE, WASHINGTON


B 21 (EXISTING)
SEWAGE TREATMENT PLANT B 2
26+00
B 22
0+00
OUTFALL
N 25 (EXISTING)
PUMPING STATION B 4
0+00
B 20 (EXISTING)
B 19
79+00
144" 0.033%
0.0
-0.9
S.T.P. B 2 0+00
P.S. B 4 0+00
108" 0.021%
-9.7
-8.0


231+00
B 16
274+00
B 15
B 15
N 17 (EXISTI
B 14
337+00
351+00
SERVICE SEWER EB 18
96" 0.032%
-3.7
-2.3
90" 0.04%
0.2
84" 0.04%
0.8
72"





B 12 — PROPOSED SEWER AND DESIGNATION

0.0 — SEWER INVERT AND ELEVATION

ELEVATIONS ARE IN FEET AND REFER TO U.S.G.S. MEAN SEA LEVEL DATUM (1929).

GRAVITY SEWERS TO BE CONSTRUCTED OF REINFORCED CONCRETE PIPE WITH RUBBER GASKET TYPE JOINTS.

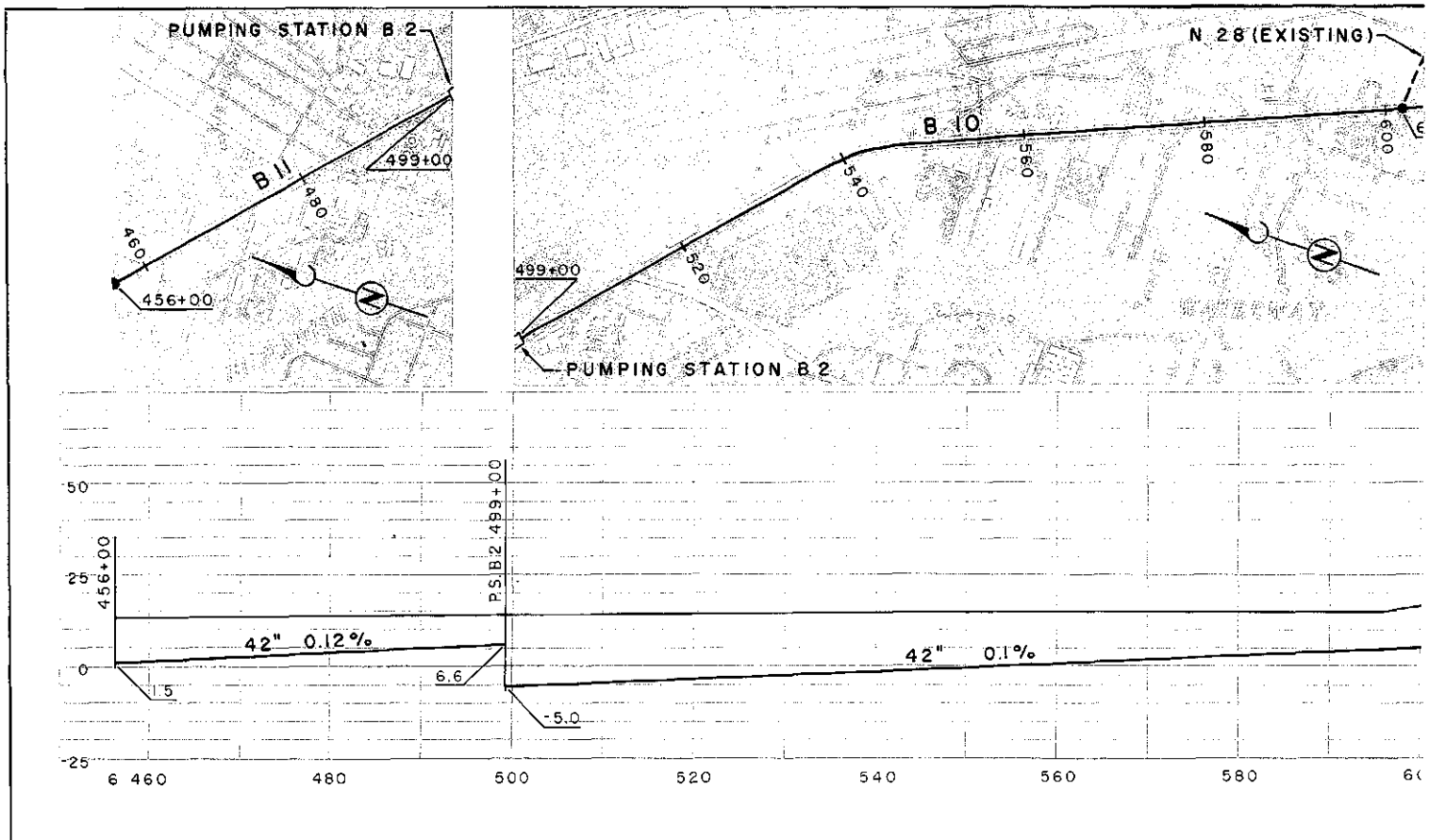
HORIZONTAL SCALE: 1" = 2000'
VERTICAL SCALE AS SHOWN

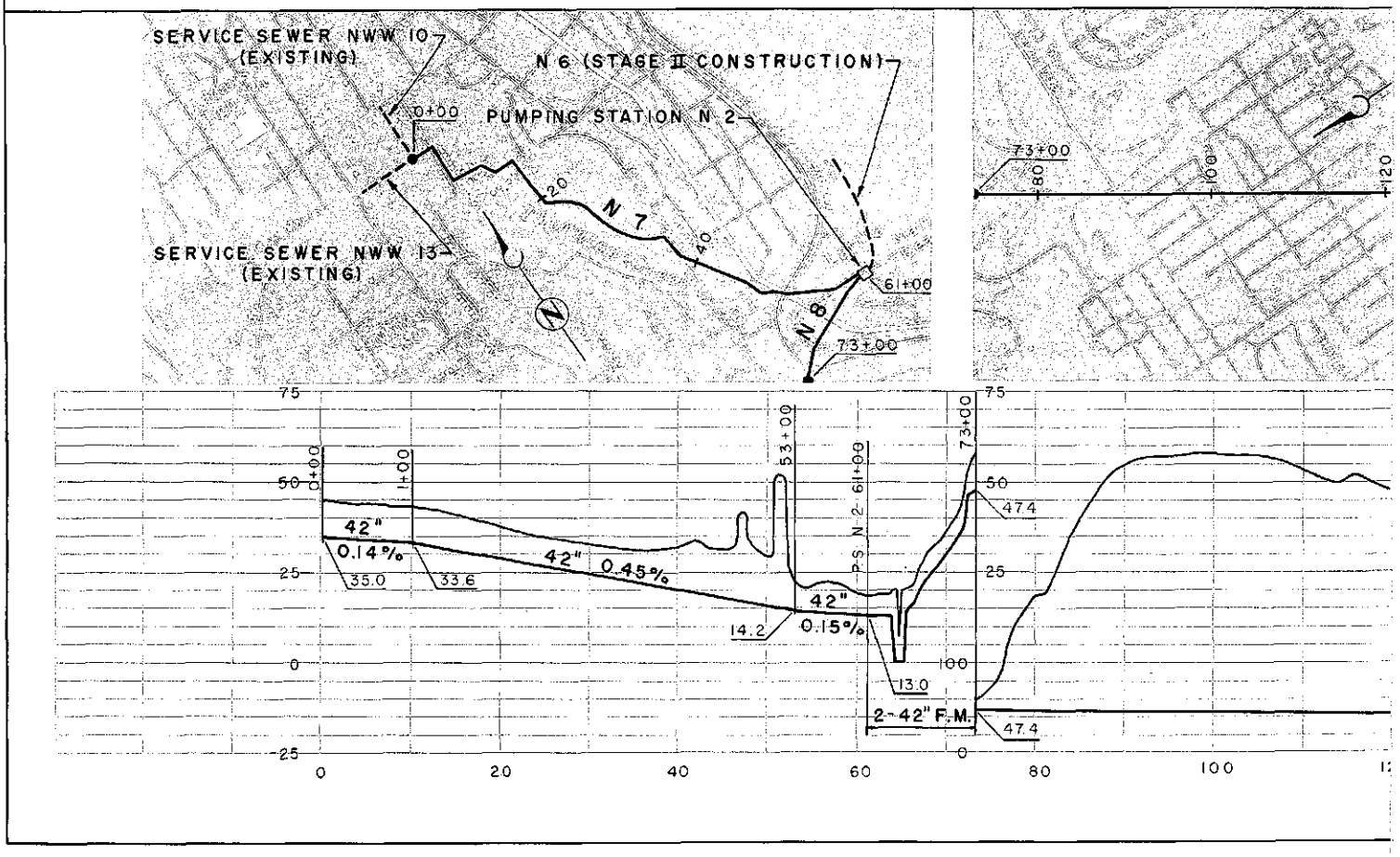
METROPOLITAN SEATTLE SEWERAGE AND DRAINAGE SURVEY

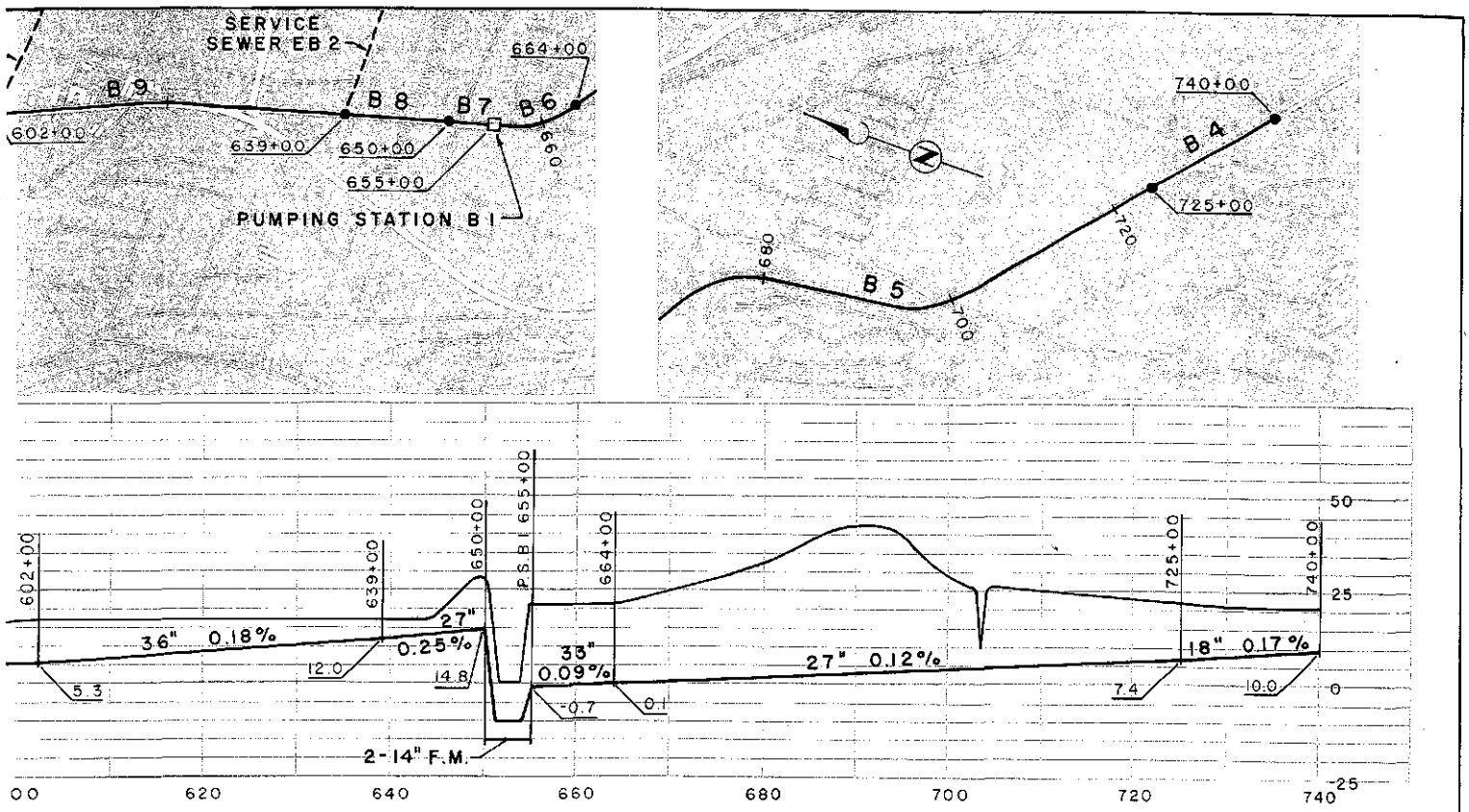
PRELIMINARY PLAN AND PROFILE

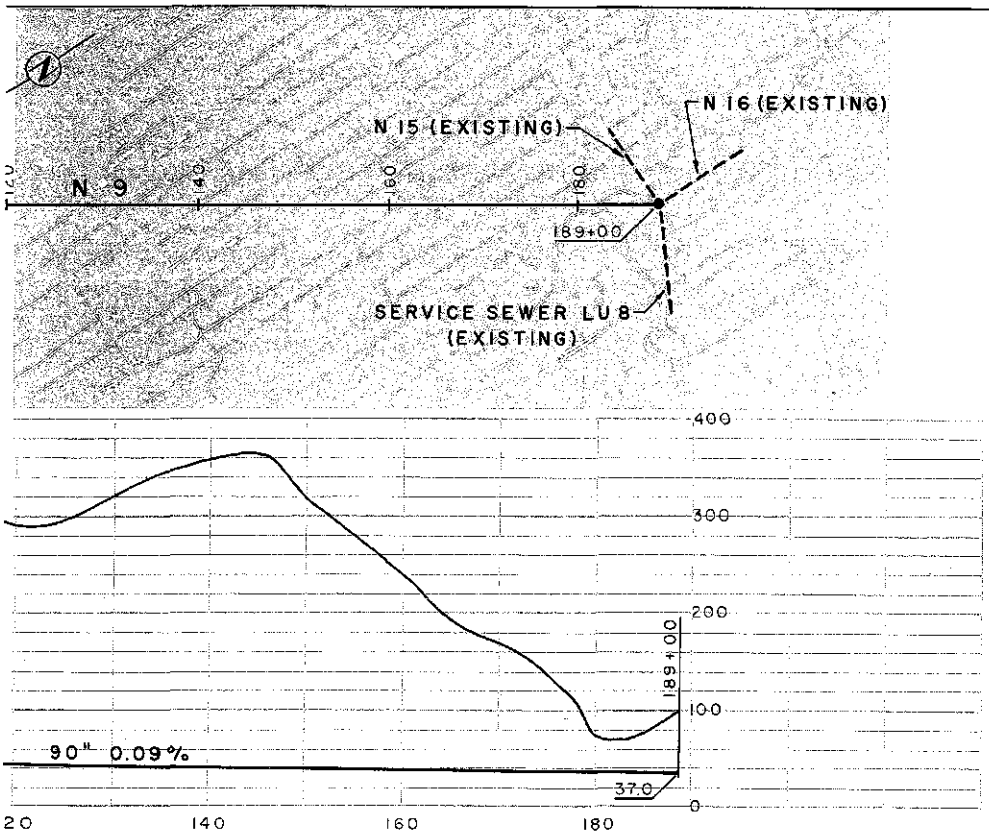
WEST POINT SYSTEM
INTERCEPTING SEWERS

BROWN AND CALDWELL CIVIL AND CHEMICAL ENGINEERS
SAN FRANCISCO, CALIFORNIA. SEATTLE, WASHINGTON


PUMPING STATION B 2
B 11
499+00
480
460
456+00
N 28 (EXISTING)
B 10
540
560
580
600
520
499+00
PUMPING STATION B 2
P.S. B2 499+00
456+00
42" 0.12%
42" 0.1%
1.5
6.6
-5.0
50
25
0
-25
460
480
500
520
540
560
580


SERVICE SEWER NWW 10 (EXISTING)
N 6 (STAGE II CONSTRUCTION)
0+00
PUMPING STATION N 2
20
N 7
40
SERVICE SEWER NWW 13 (EXISTING)
61+00
N 8
73+00
80
100
120
75
50
25
0
-25
0+00
1+00
53+00
P.S. N 2 61+00
73+00
42"
0.14%
42" 0.45%
35.0
33.6
42"
0.15%
14.2
13.0
47.4
2-42" F.M.
47.4
0
20
40
60
80
100





B 6 — PROPOSED SEWER AND DESIGNATION

37.0 — SEWER INVERT AND ELEVATION

ELEVATIONS ARE IN FEET AND REFER TO U.S.G.S. MEAN SEA LEVEL DATUM (1929).

GRAVITY SEWERS TO BE CONSTRUCTED OF REINFORCED CONCRETE PIPE WITH RUBBER GASKET TYPE JOINTS.

HORIZONTAL SCALE: 1" = 2000'
VERTICAL SCALE AS SHOWN.

METROPOLITAN SEATTLE SEWERAGE AND DRAINAGE SURVEY

PRELIMINARY PLAN AND PROFILE

WEST POINT SYSTEM INTERCEPTING SEWERS

BROWN AND CALDWELL CIVIL AND CHEMICAL ENGINEERS
SAN FRANCISCO, CALIFORNIA. SEATTLE, WASHINGTON

APPENDIX F

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