PART IV

BASIS FOR SEWERAGE AND DRAINAGE PLANNING

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. Furthermore, 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.*

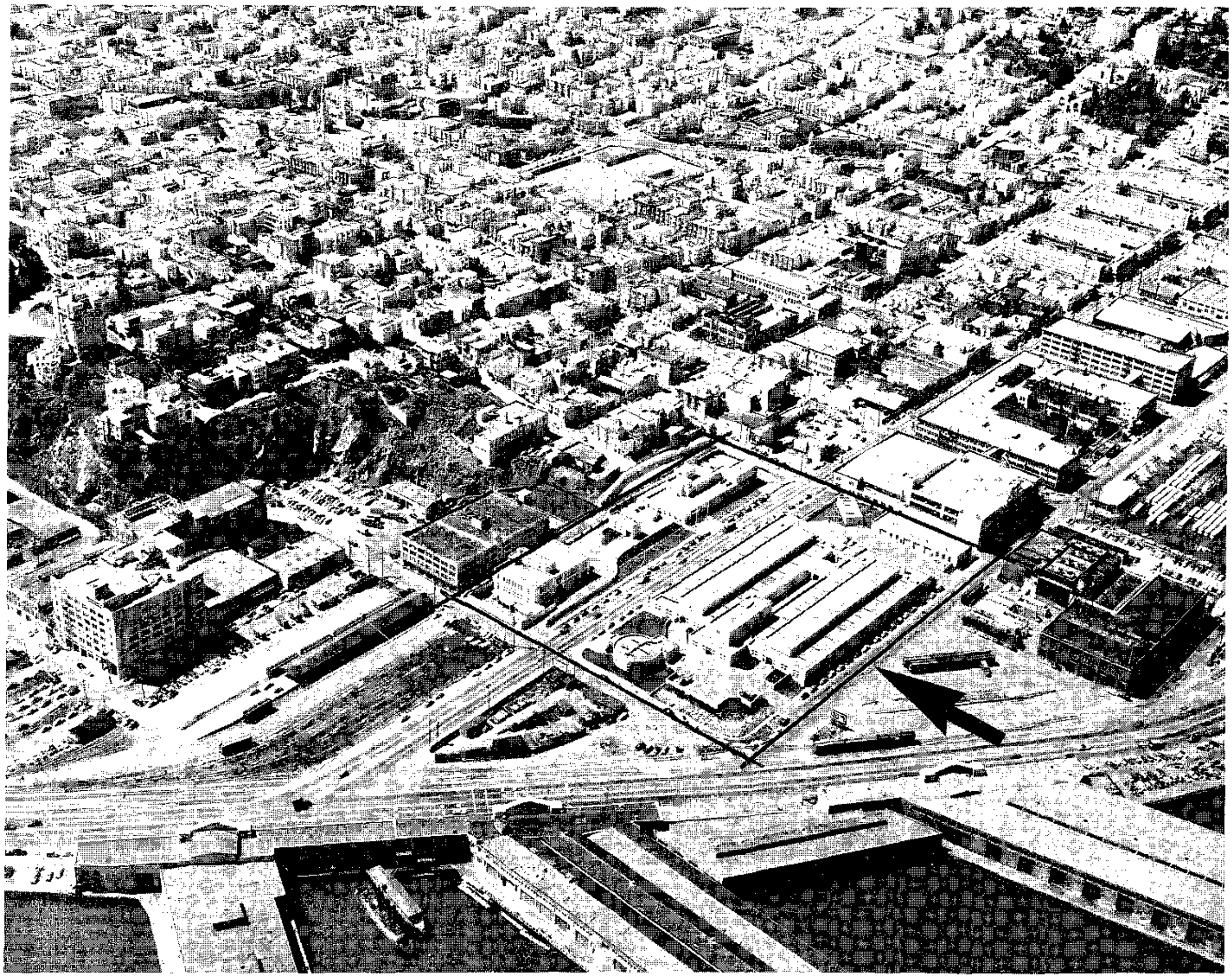
age from the deep sewer to one nearer the surface of the ground.

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

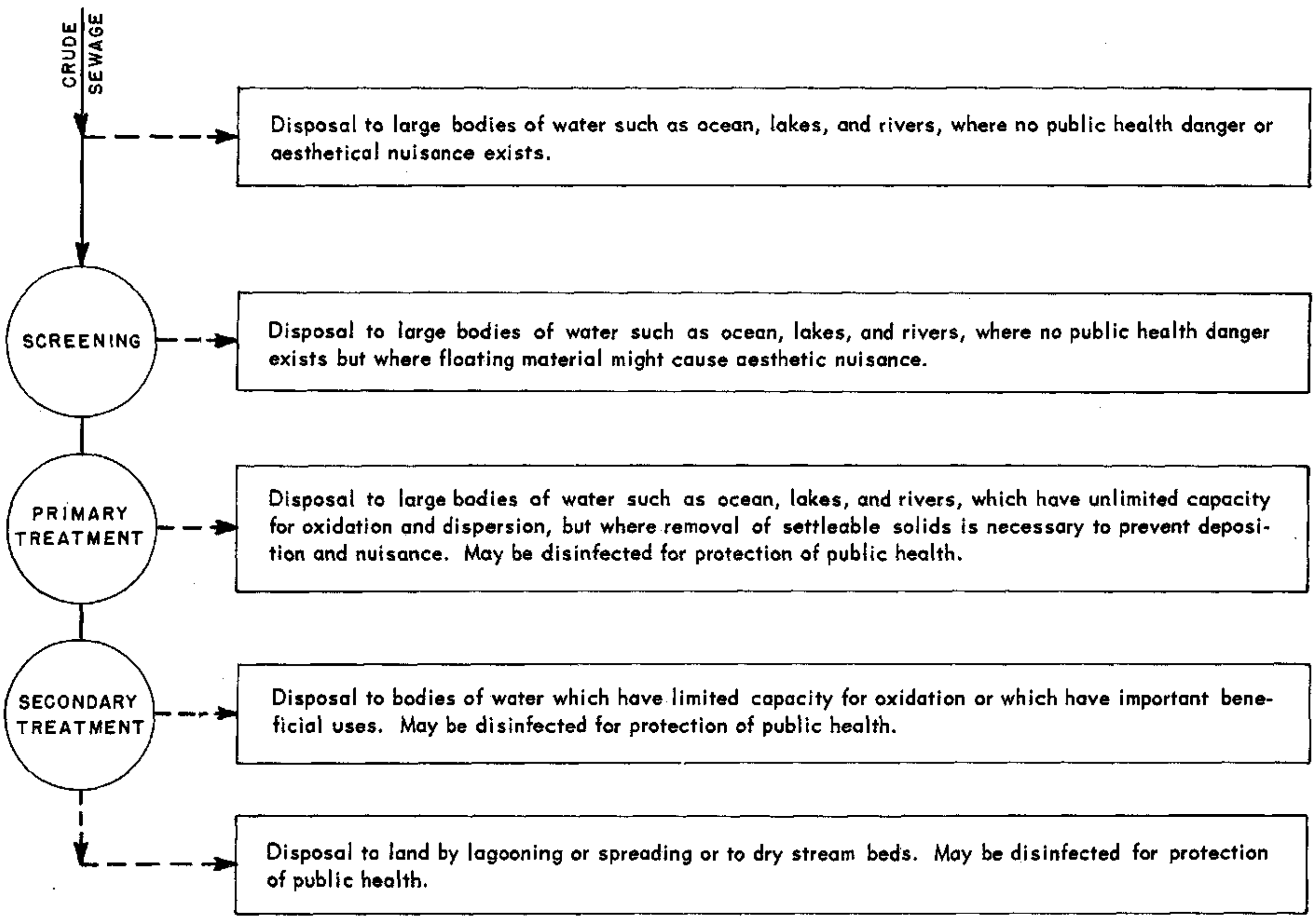


Fig. 9-1. Sewage Treatment Processes

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.

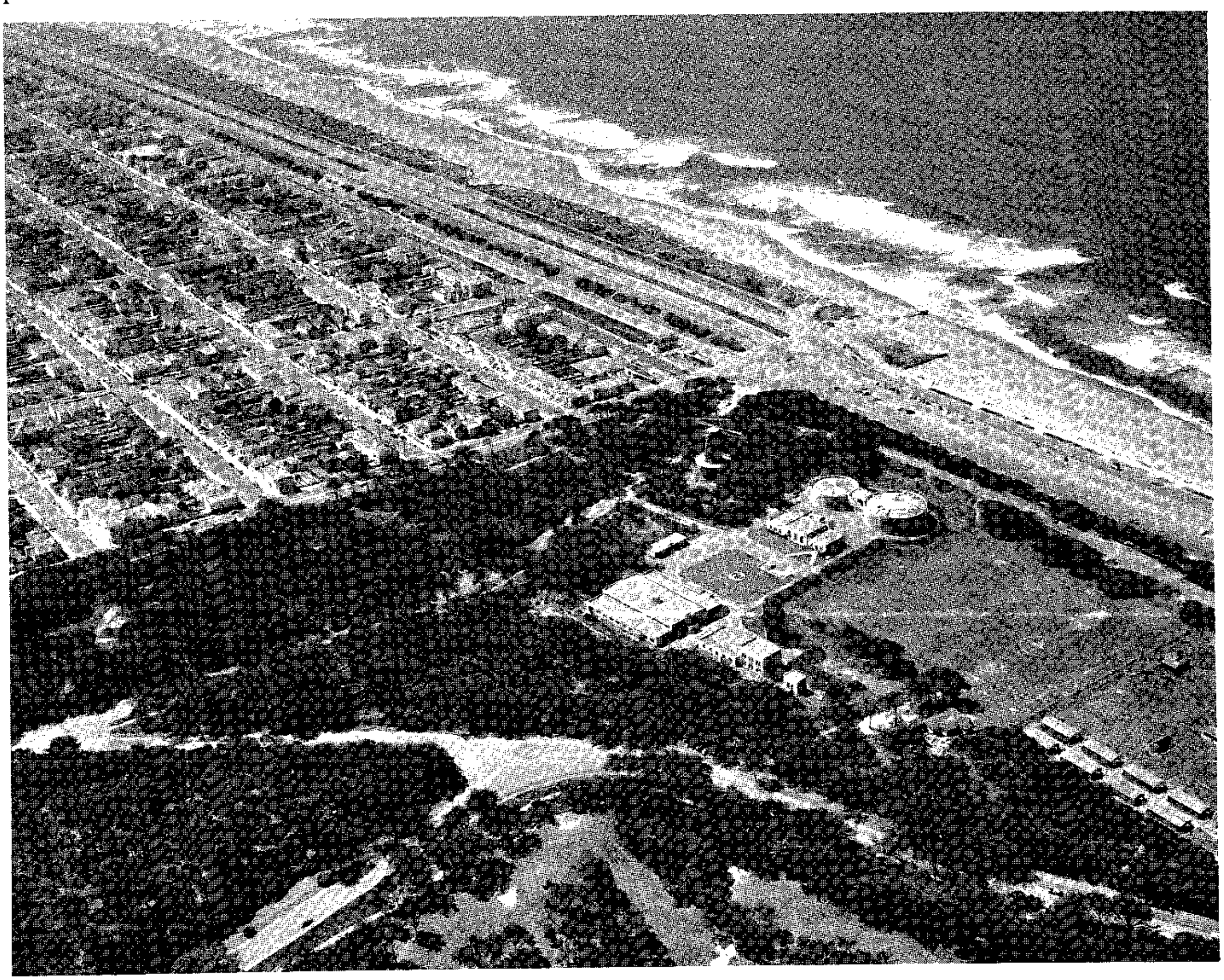
racks or bar screens and by comminuters or cutting screens. Bar racks may be hand cleaned or provided with automatic raking mechanisms. If hand cleaned, the open space between bars may be as small as 1-1/2 inches or as large as 4 inches. Clear spacing for automatically cleaned racks is usually 3/4 or 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.

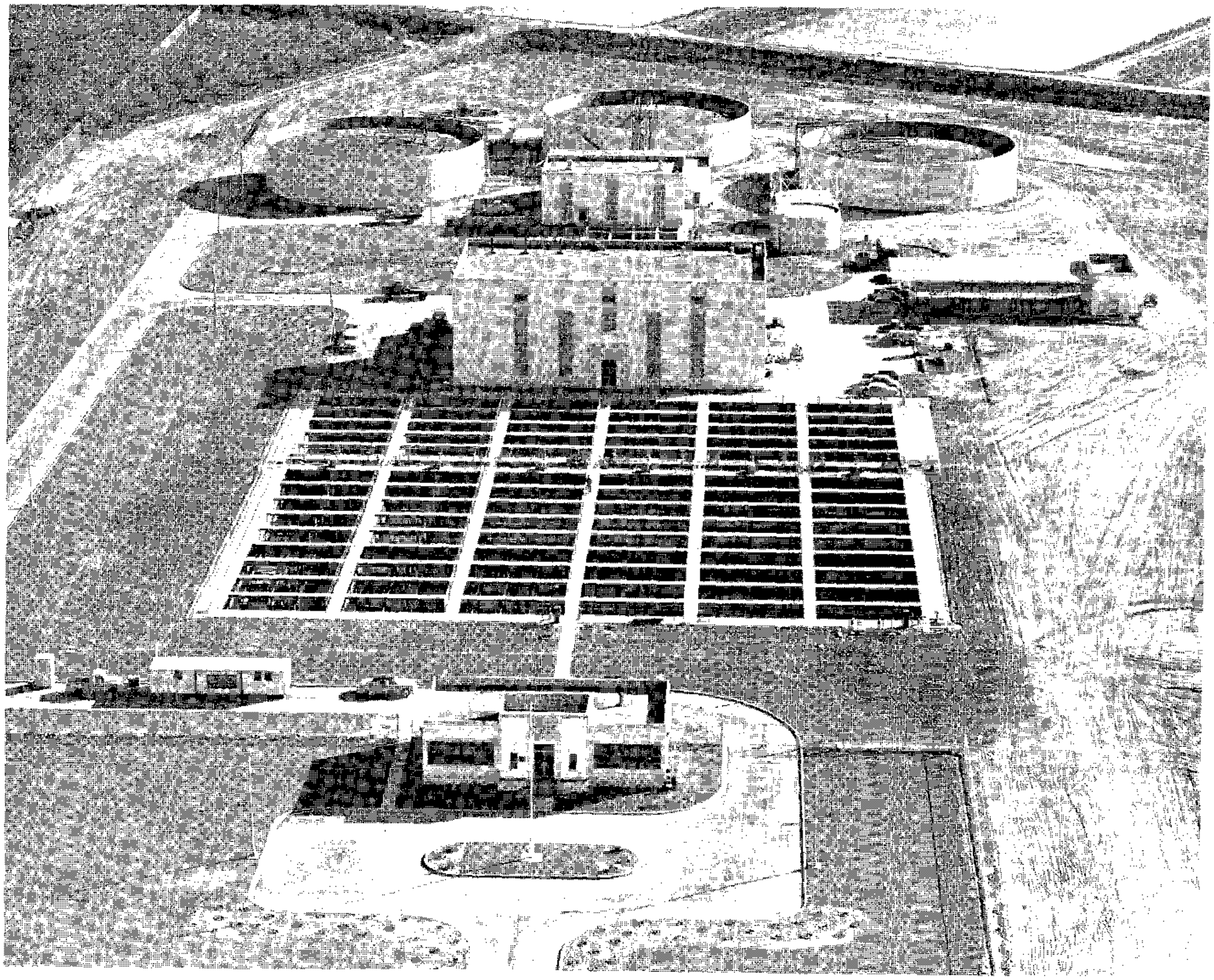
Although a number of biologic oxidation processes have been developed during the last 50 years, only two, high rate trickling filtration and the activated sludge process, are significant in their application to the Seattle area. A third process, oxidation ponds, is worthy of mention since it may be utilized 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 summer 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.

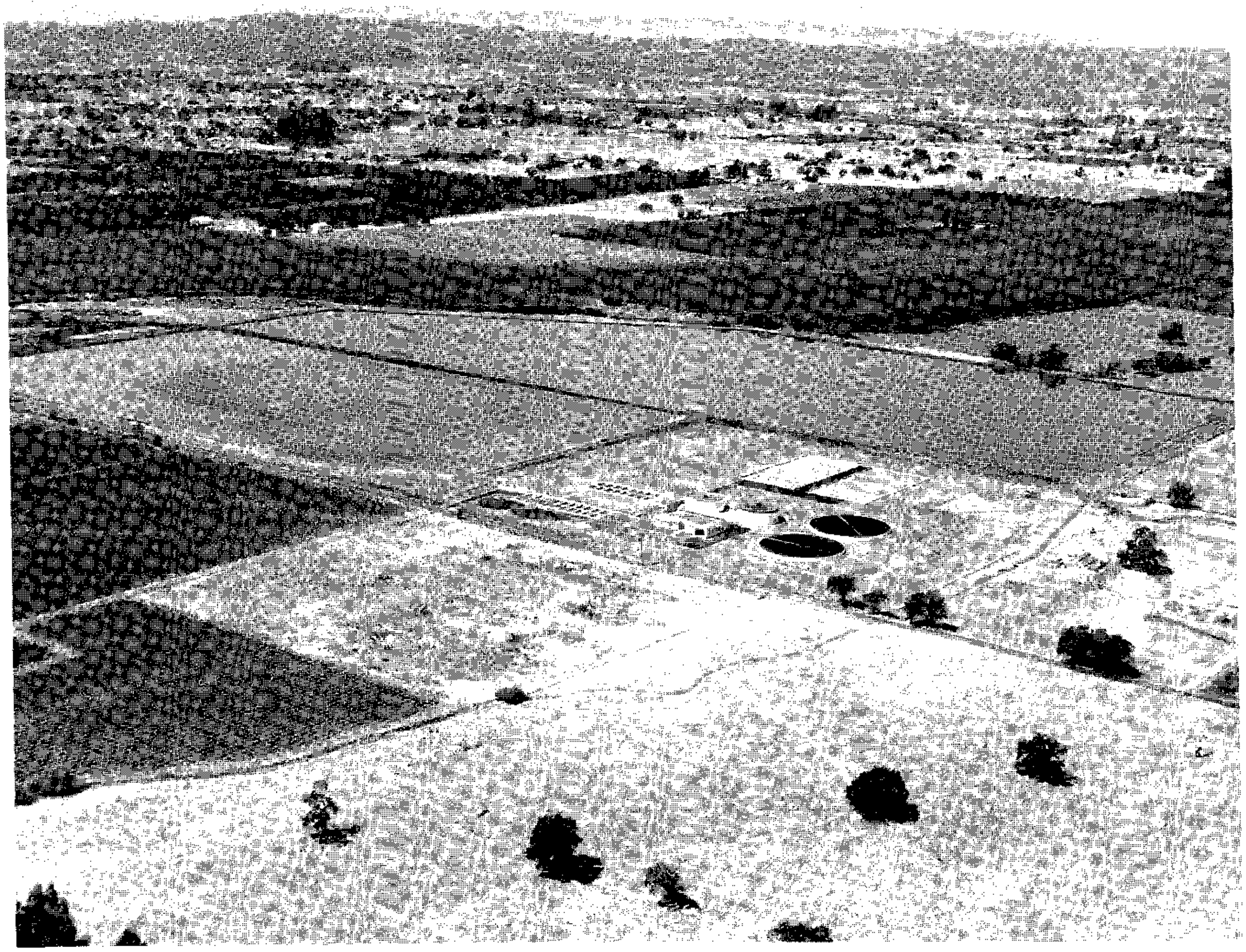
Digested Sludge. Dry sludge contains about 50 per cent of organic matter, of which from 1 to 2 per cent is organic nitrogen. It also contains very small percentages of potassium and phosphate. In all, the characteristics of digested sludge are such that it is an excellent soil builder or soil conditioner. 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 Wa~hington 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.

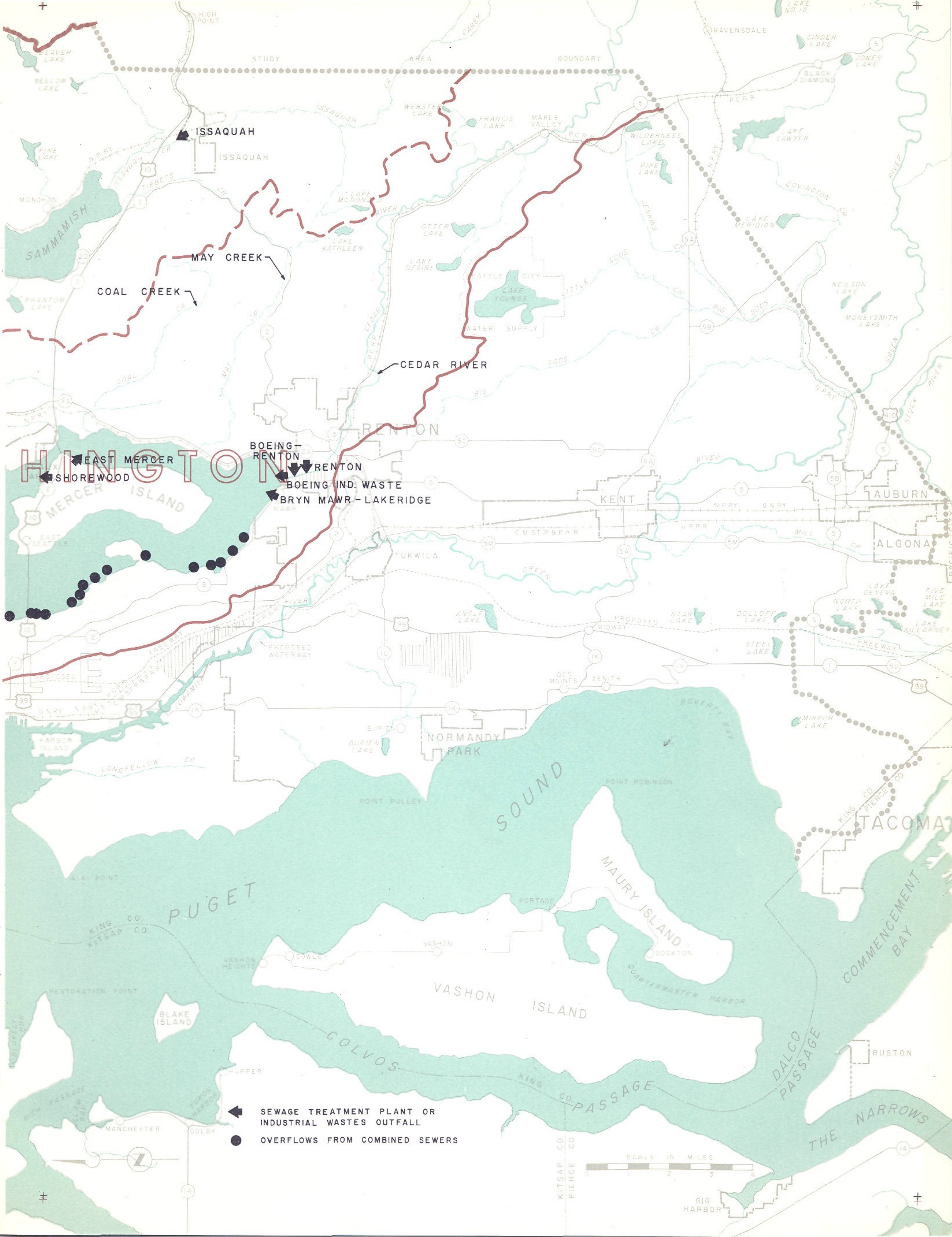
Lake Washington has developed into an outdoor recreational center for the entire metropolitan area. The city of Seattle alone maintains six public bathing beaches and parks along the lake and a number of other communities have developed similar facilities. Thousands of residents and visitors derive recreation 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


ISSAQUAH
MAY CREEK
COAL CREEK
CEDAR RIVER
BOEING-RENTON
RENTON
EAST MERCER
SHOREWOOD
BOEING IND. WASTE
BRYN MAWR - LAKERIDGE
STUDY AREA BOUNDARY
RAVENSDALE
MAPLE VALLEY
LAKE SAMMAMISH
LAKE YOUNGS
SEATTLE CITY WATER SUPPLY
MERCER ISLAND
KENT
AUBURN
ALGONA
TUKWILA
NORMANDY PARK
PUGET SOUND
MAURY ISLAND
VASHON ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
TACOMA
RUSTON
GIG HARBOR
BLAKE ISLAND
KING CO.
KITSAP CO.
PIERCE CO.
SEWAGE TREATMENT PLANT OR INDUSTRIAL WASTES OUTFALL
OVERFLOWS FROM COMBINED SEWERS
SCALE IN MILES
0 1 2 3 4

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.

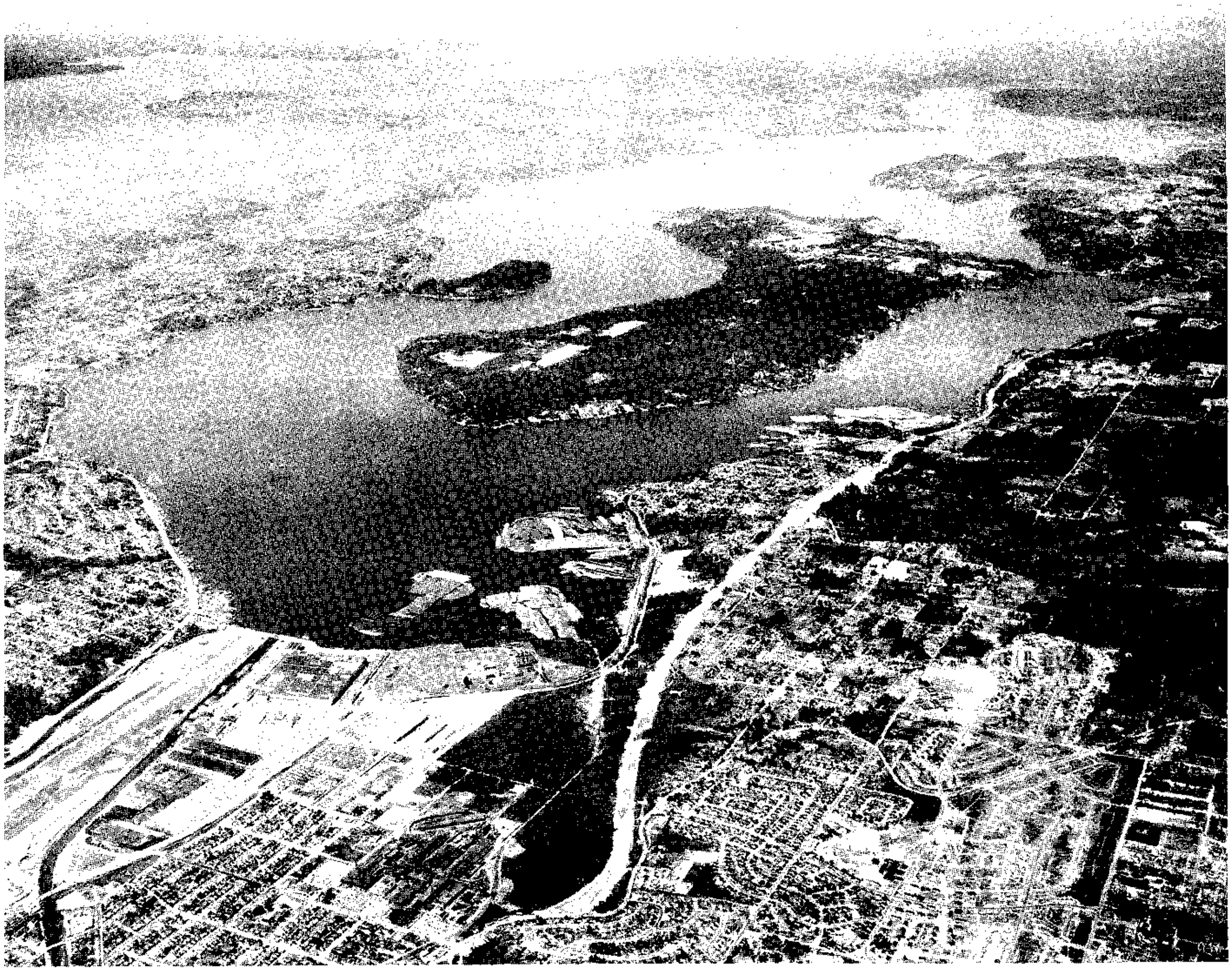
Water Quality Requirements

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:

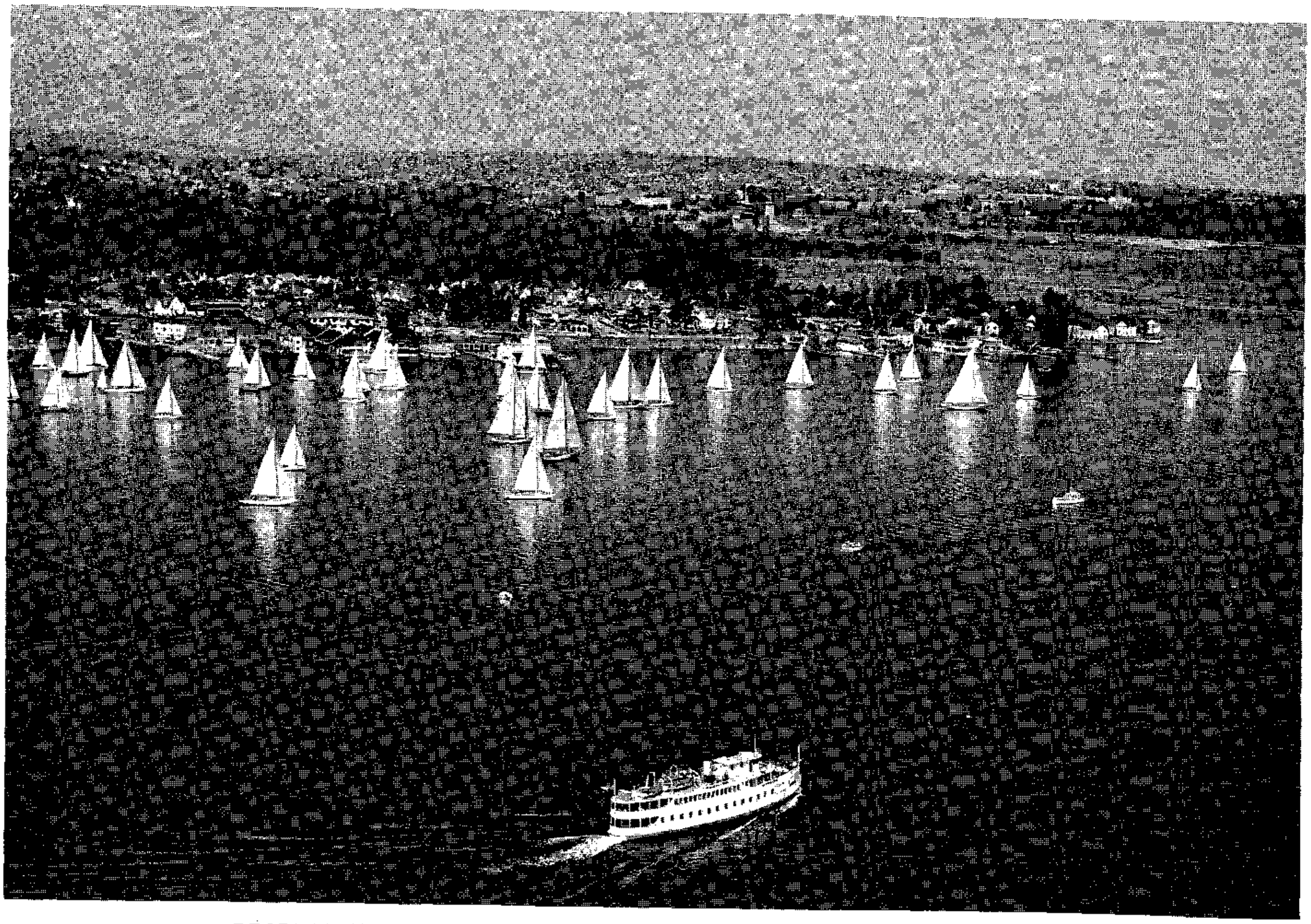
1. Contamination of either recreational or drinking waters with bacteriological organisms of human origin.
2. Physical impairment of the waters by sewage components, including solids, grease and oils.
3. Destruction or inhibition of desirable fish life.
4. Creation of nuisances as evidenced by algal 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 foreseé 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	
		Orthophosphate	Total dissolved phosphate	Orthophosphate	Total dissolved phosphate	Orthophosphate	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	
	Orthophosphate	Total dissolved phosphate	Orthophosphate	Total dissolved phosphate	Orthophosphate	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

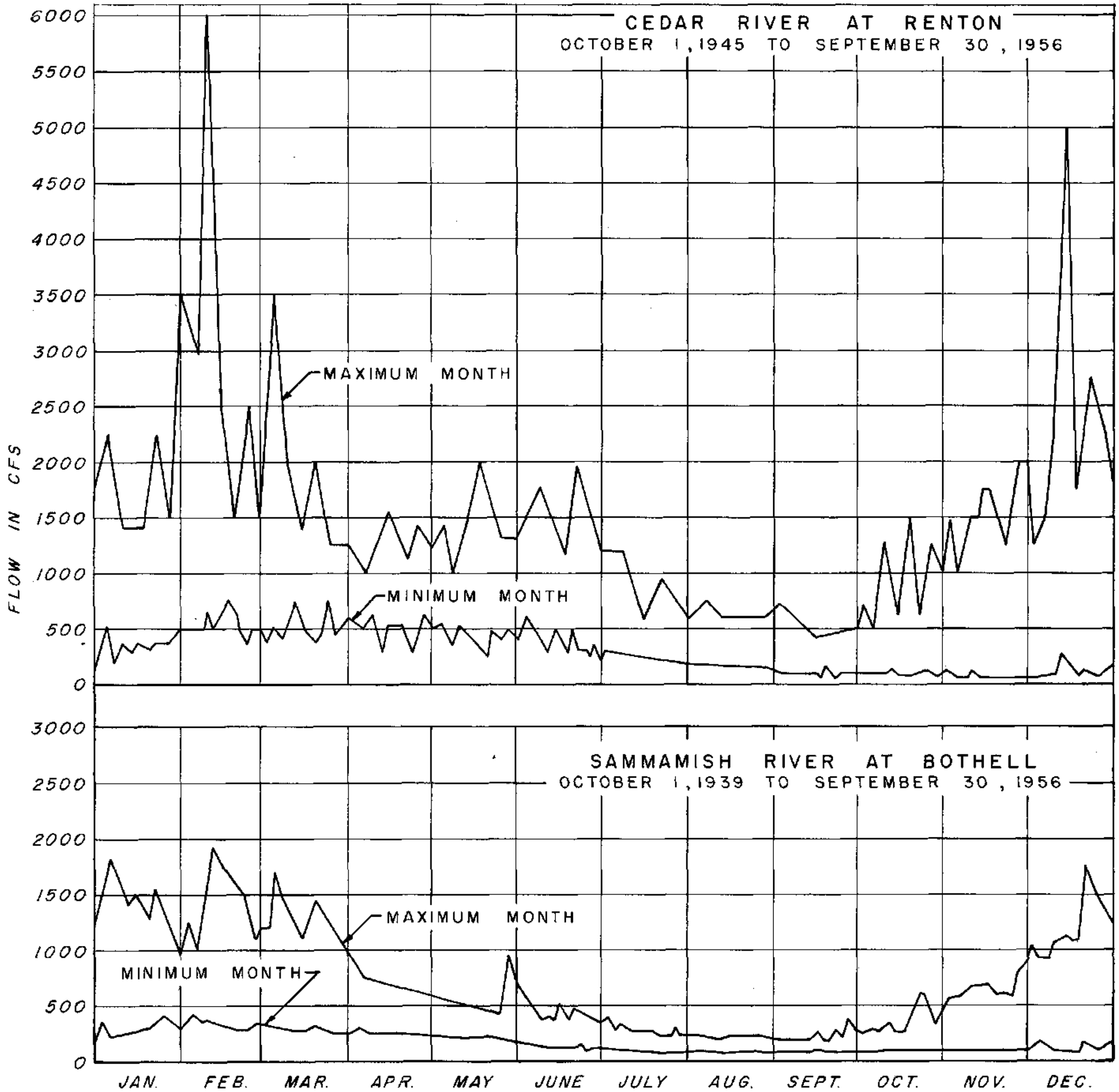


Fig. 10-2. Maximum and Minimum Monthly Flows, Cedar and Sammamish Rivers

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.*

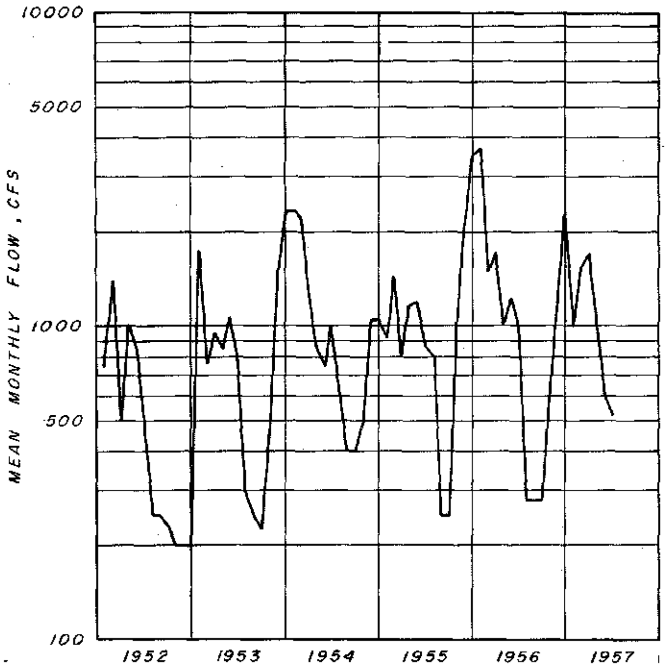


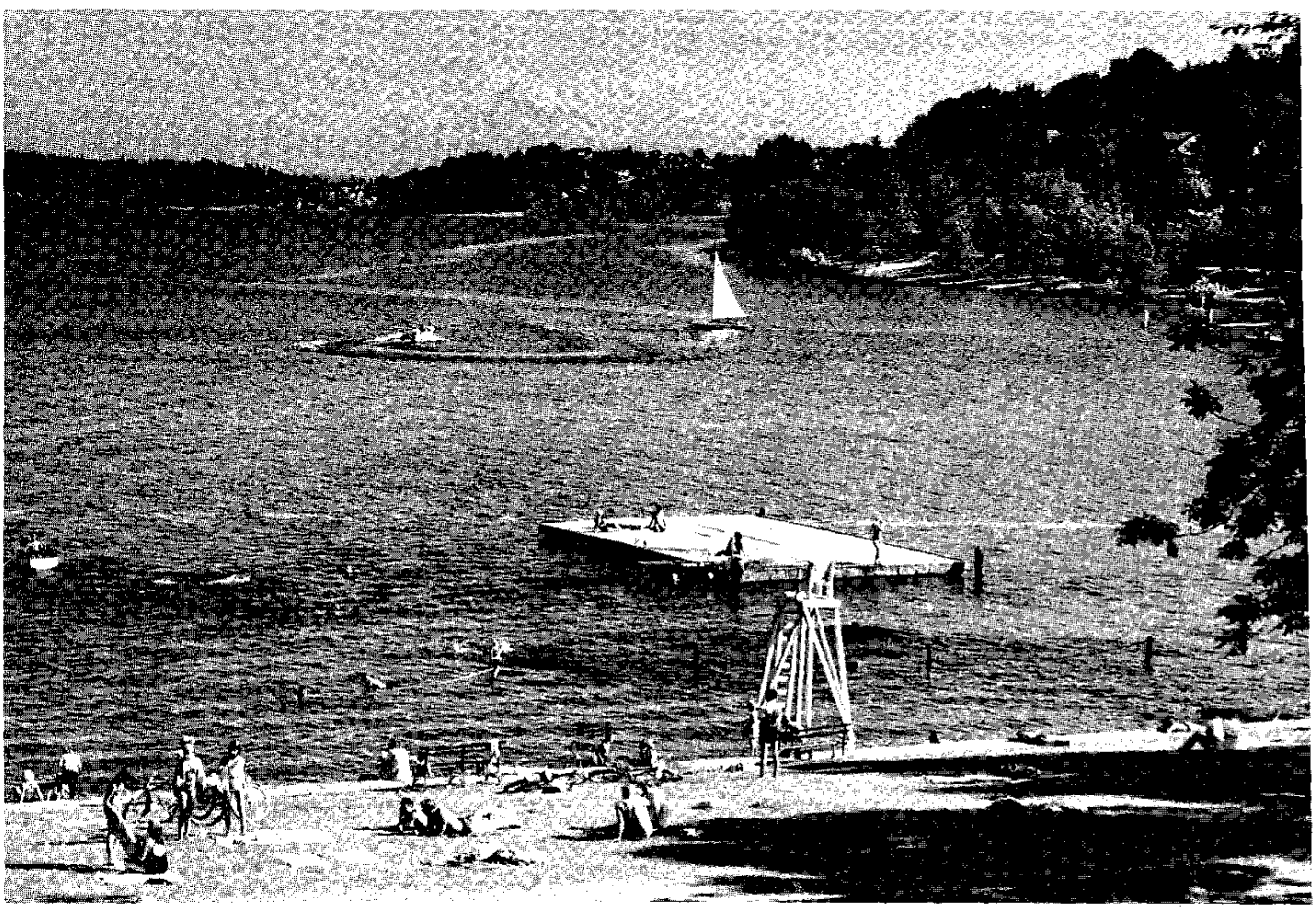
Fig. 10-3. Average Monthly Flows, Lake Washington Ship Canal at Government Locks

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 runoff 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.

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 10-11. 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.

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

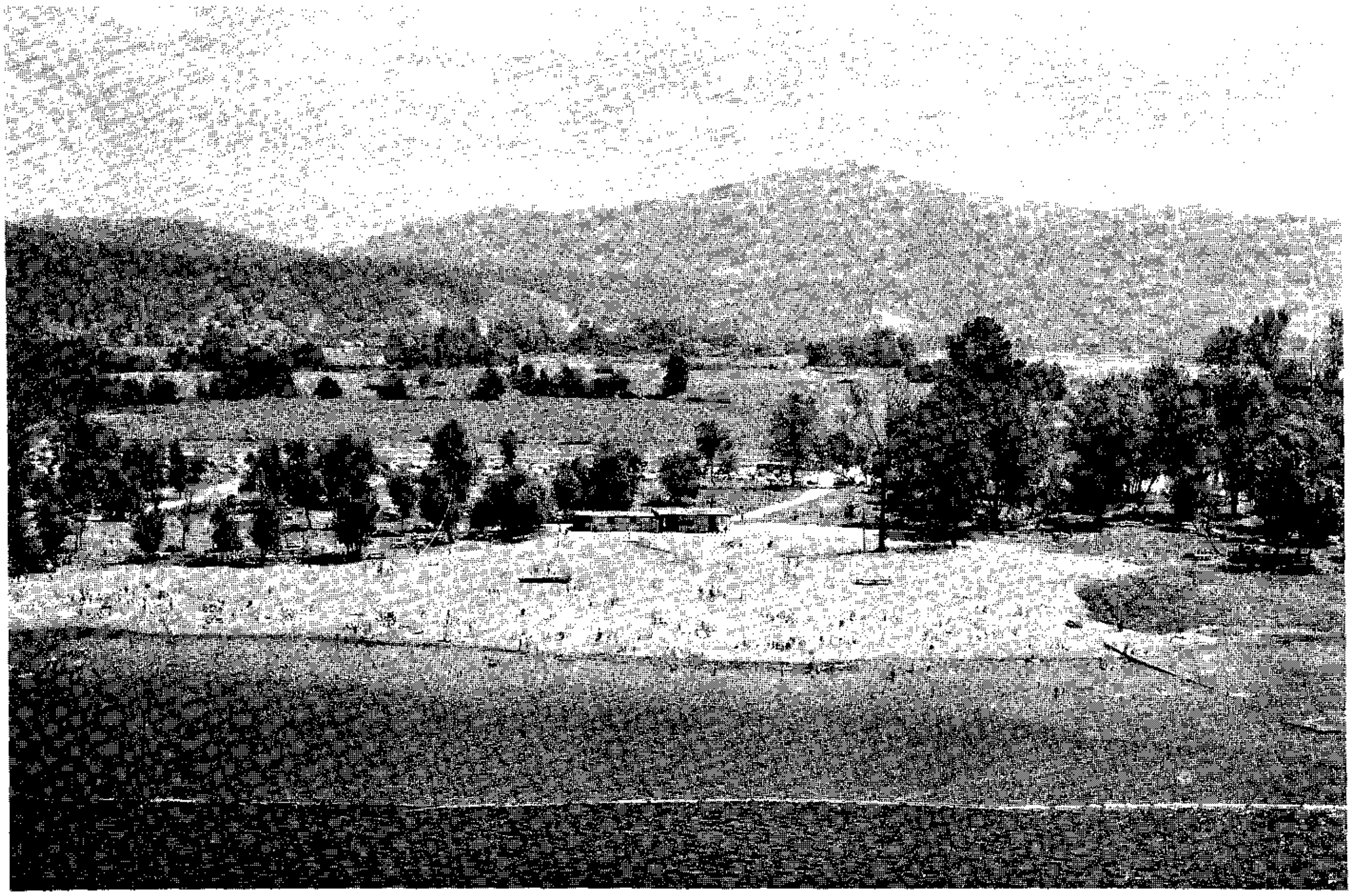
[a]*From Table 10-3.*

[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 $u^3/ml \times 10^{-3}$.

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-

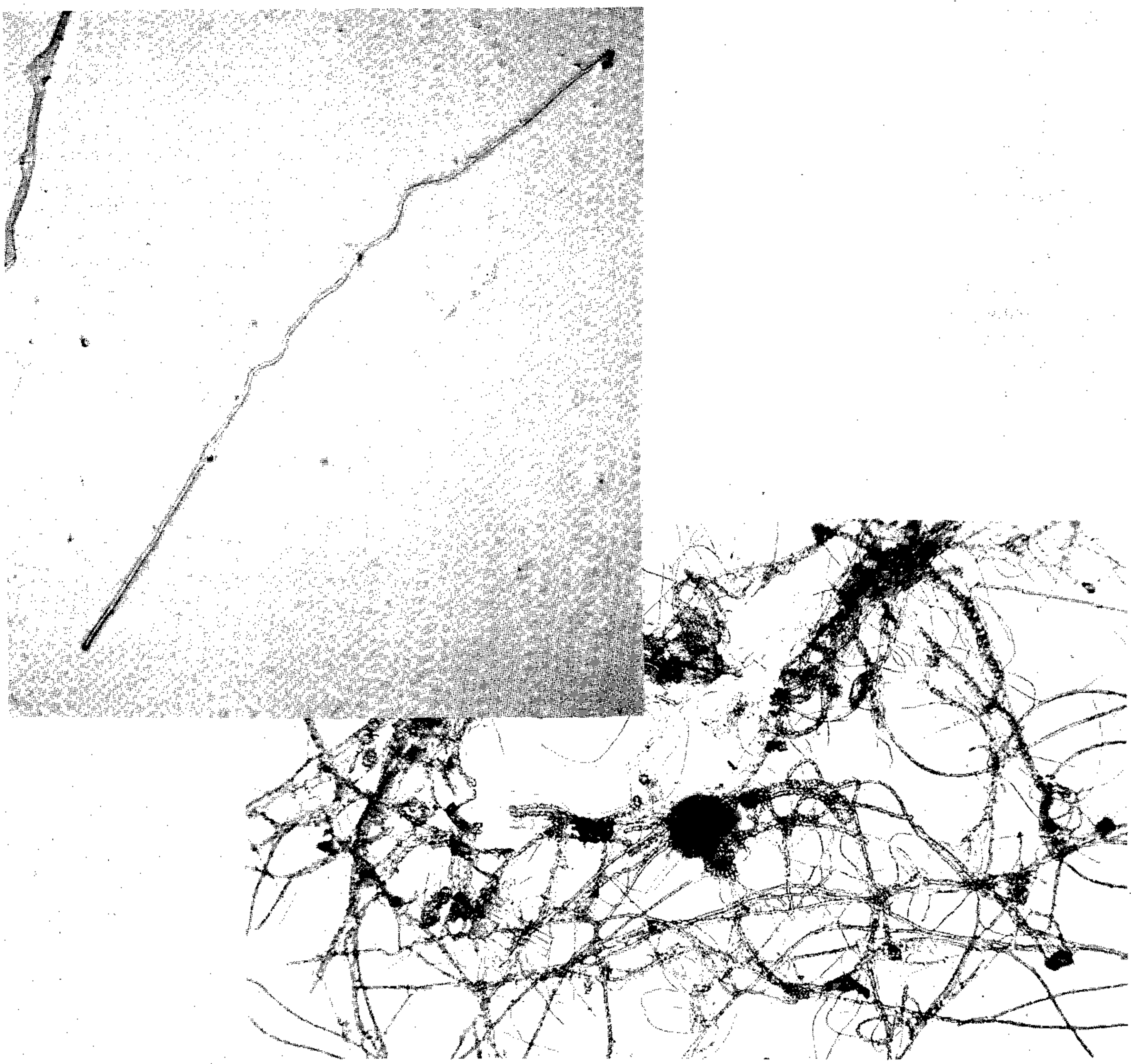


Fig. 10-4. Photomicrographs of Algae Growing in Lake Washington

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.

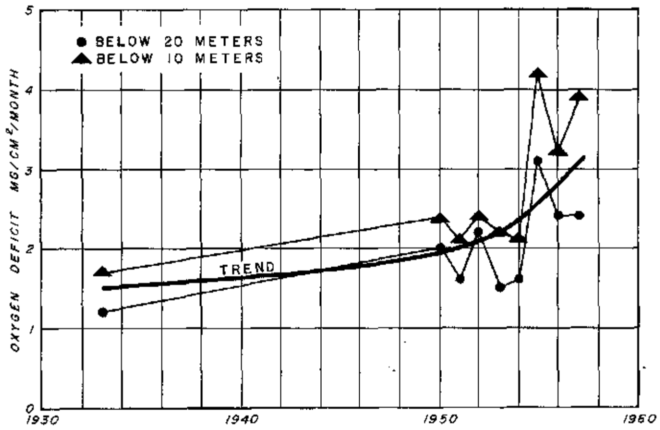


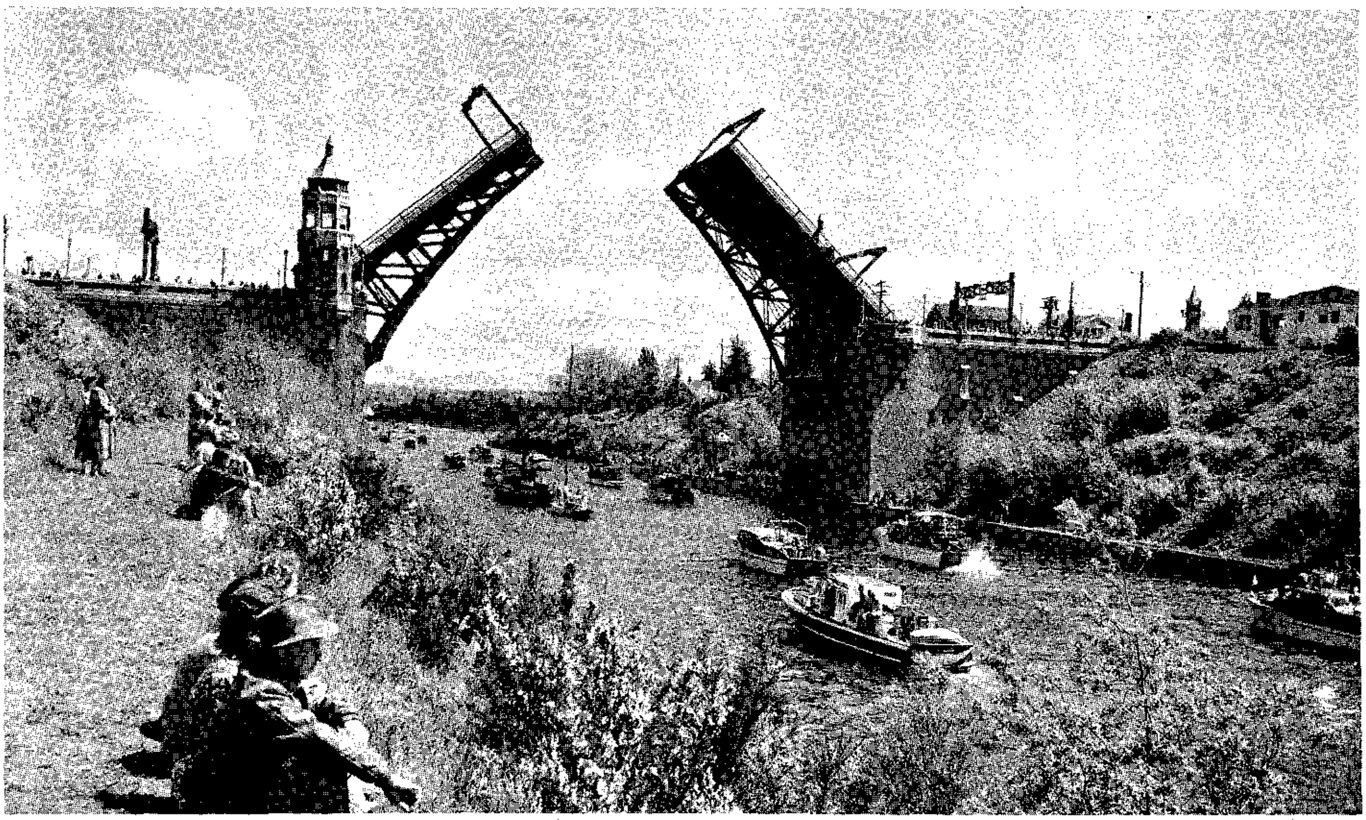
Fig. 10-5. Trend in Hypolimnic Oxygen Deficit in Lake Washington

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.

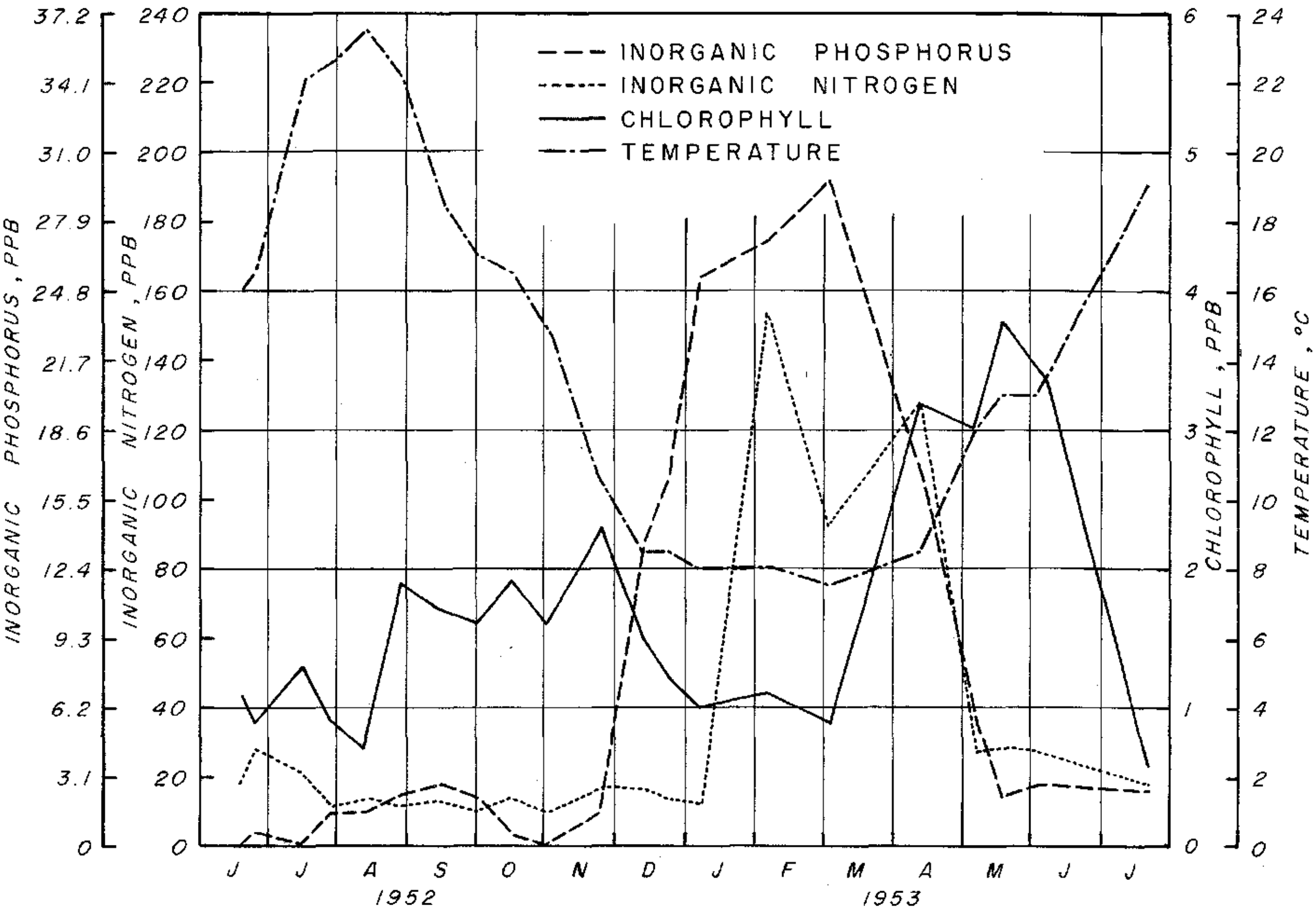
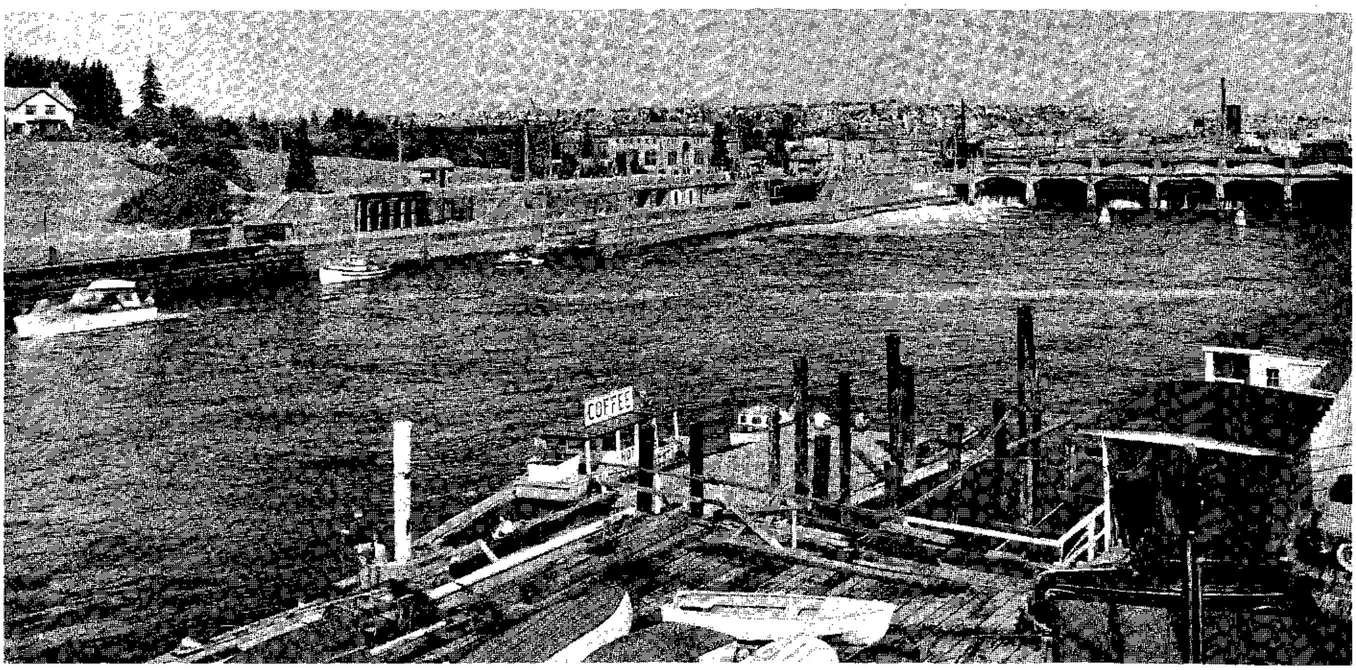


Fig. 10-6. Relationship between Algal Growths, Nutrient Supply and Environmental Conditions in Lake Washington

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

102×10^9 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 102×10^9 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

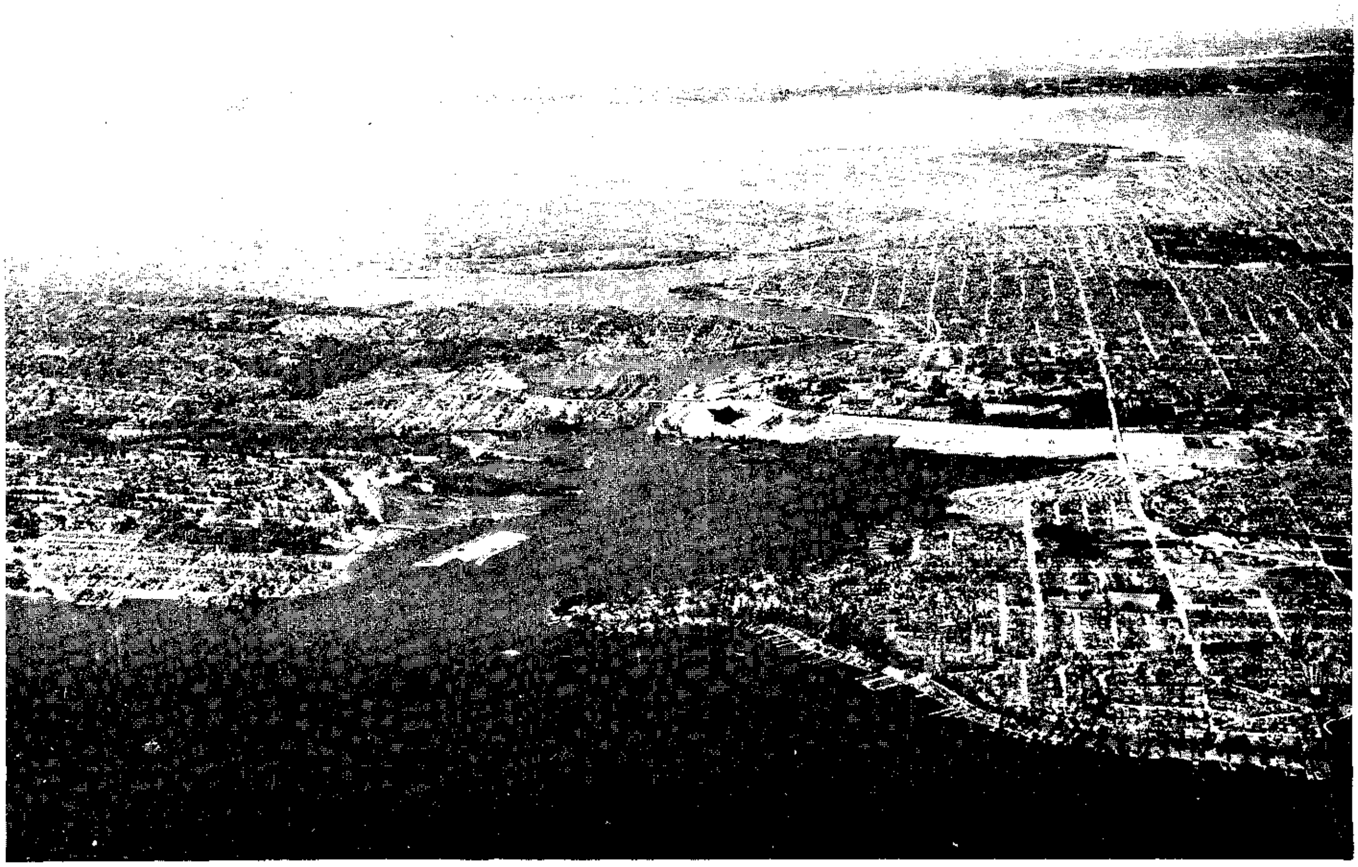
Period	Phosphorus concentration, ppm	
	Total	Inorganic[a]
Pre-1916[b]	0.022	0.011
1916 - 1930	0.039	0.019
1931 - 1940	0.041	0.021
1941 - 1950	0.044	0.022
1957	0.042	0.021

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 102×10^9 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,

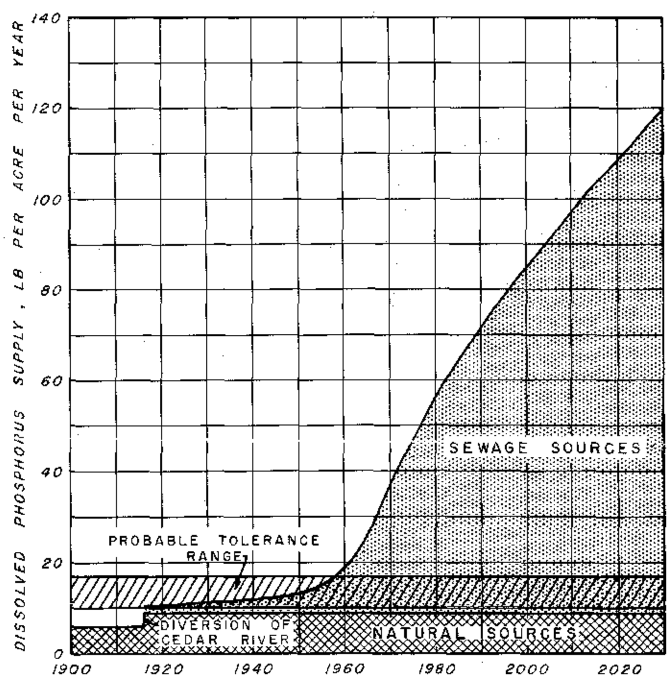
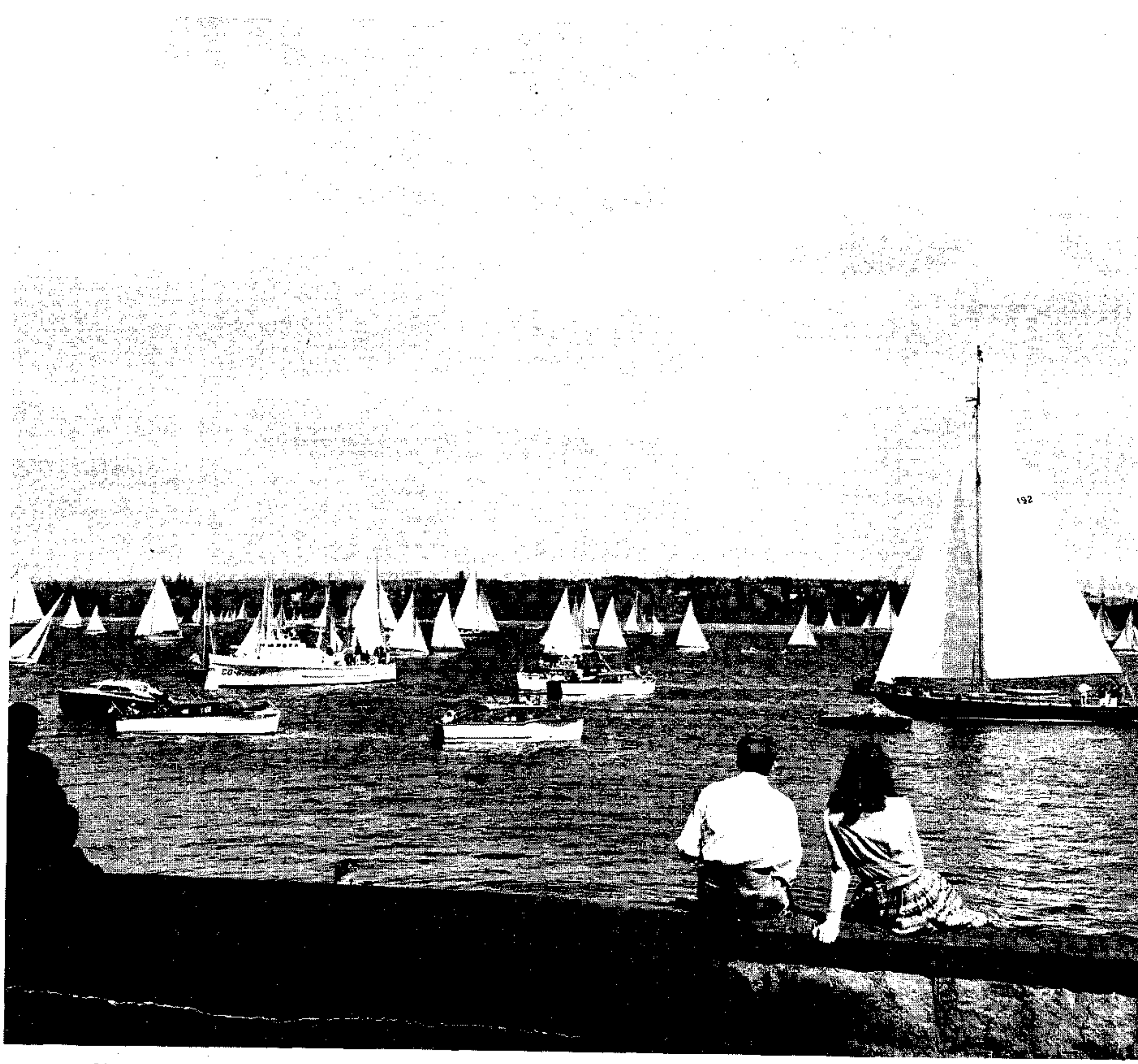


Fig. 10-7. Trend of Phosphorus Enrichment in Lake Washington

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.*

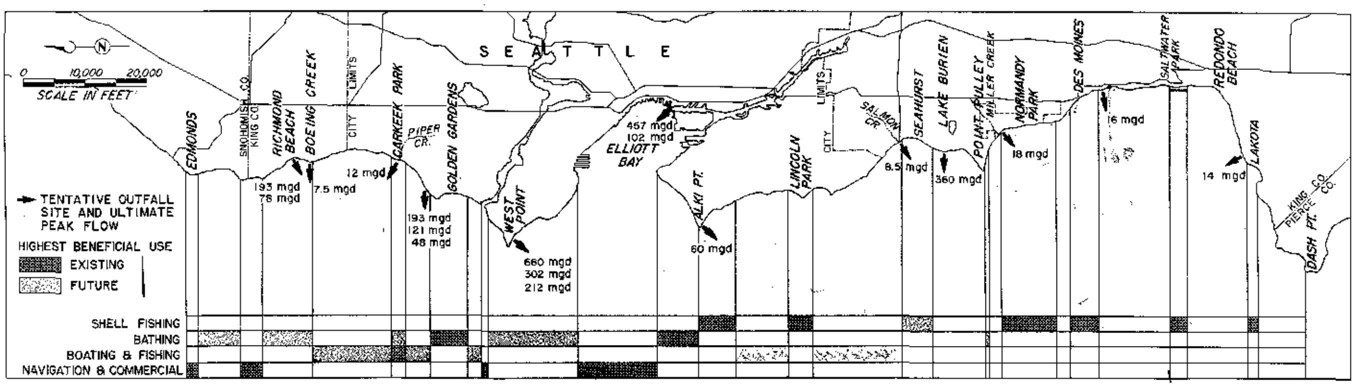


Fig. 11-1. Present and Future Waterfront Uses and Tentative Sewage Disposal Sites

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.

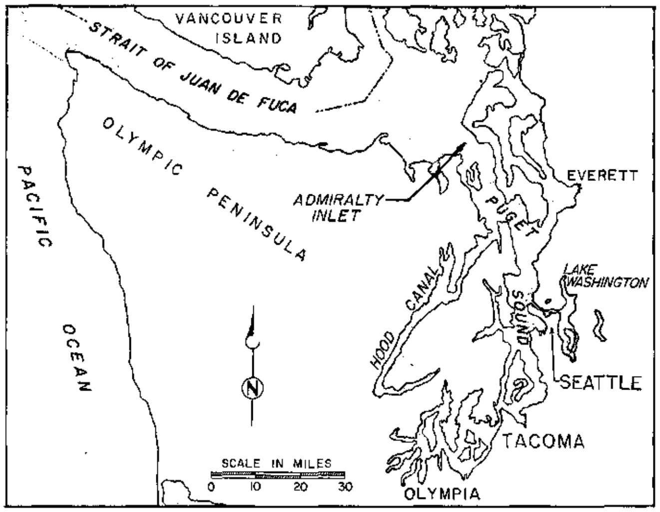


Fig. 11-2. Puget Sound

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

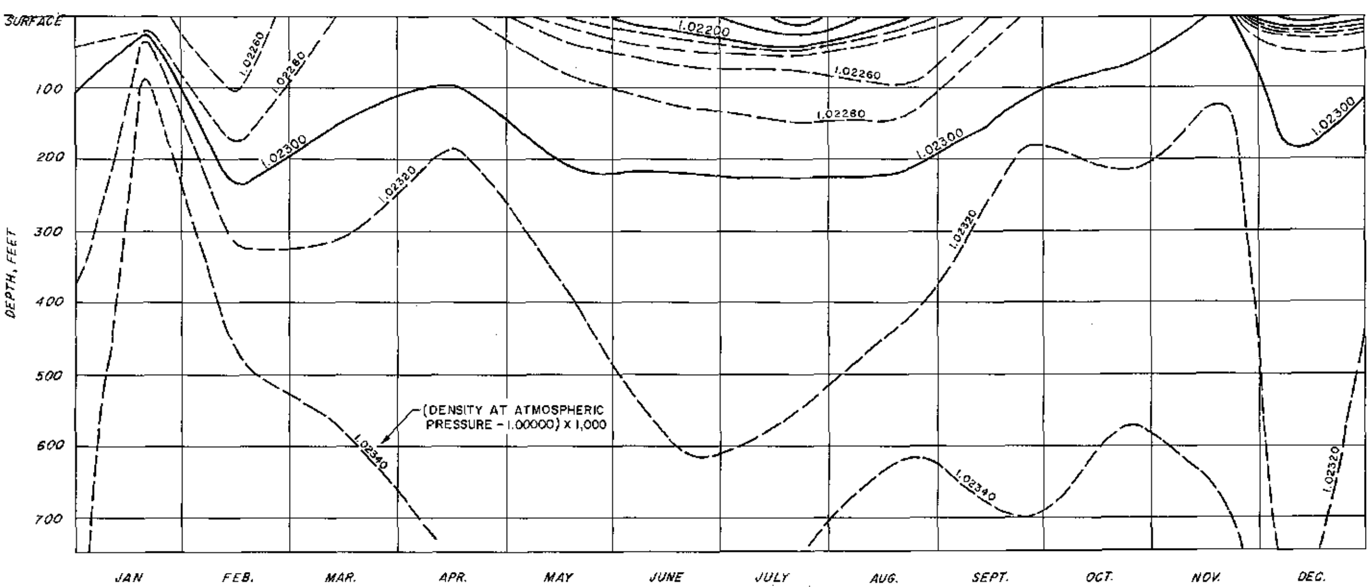


Fig. 11-3. Monthly Variation in Density Structure

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/l 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/l.

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-

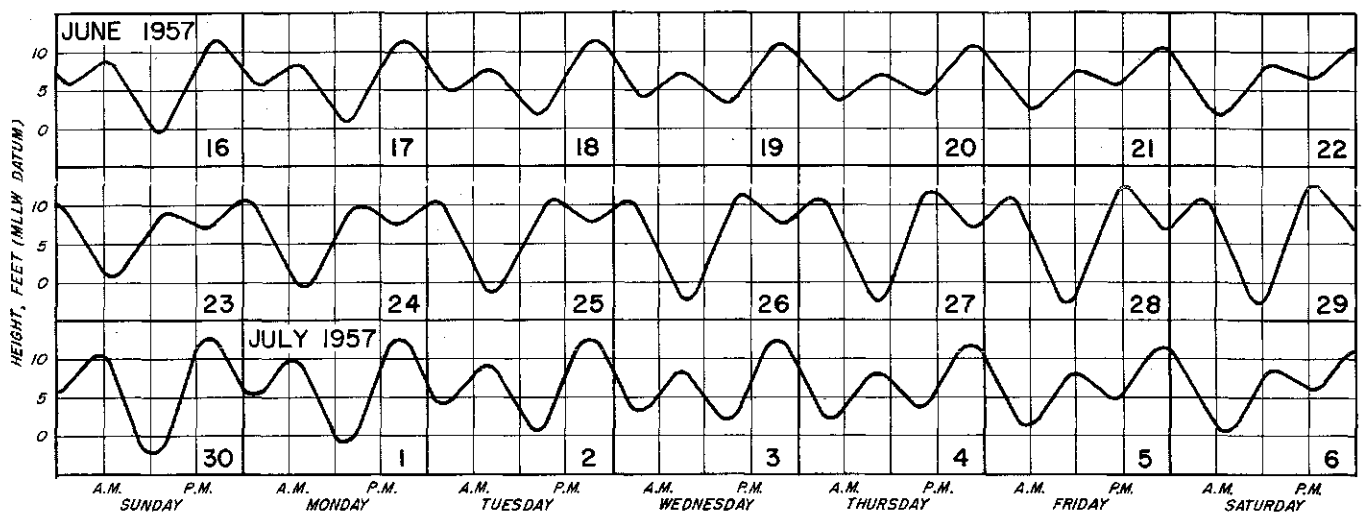
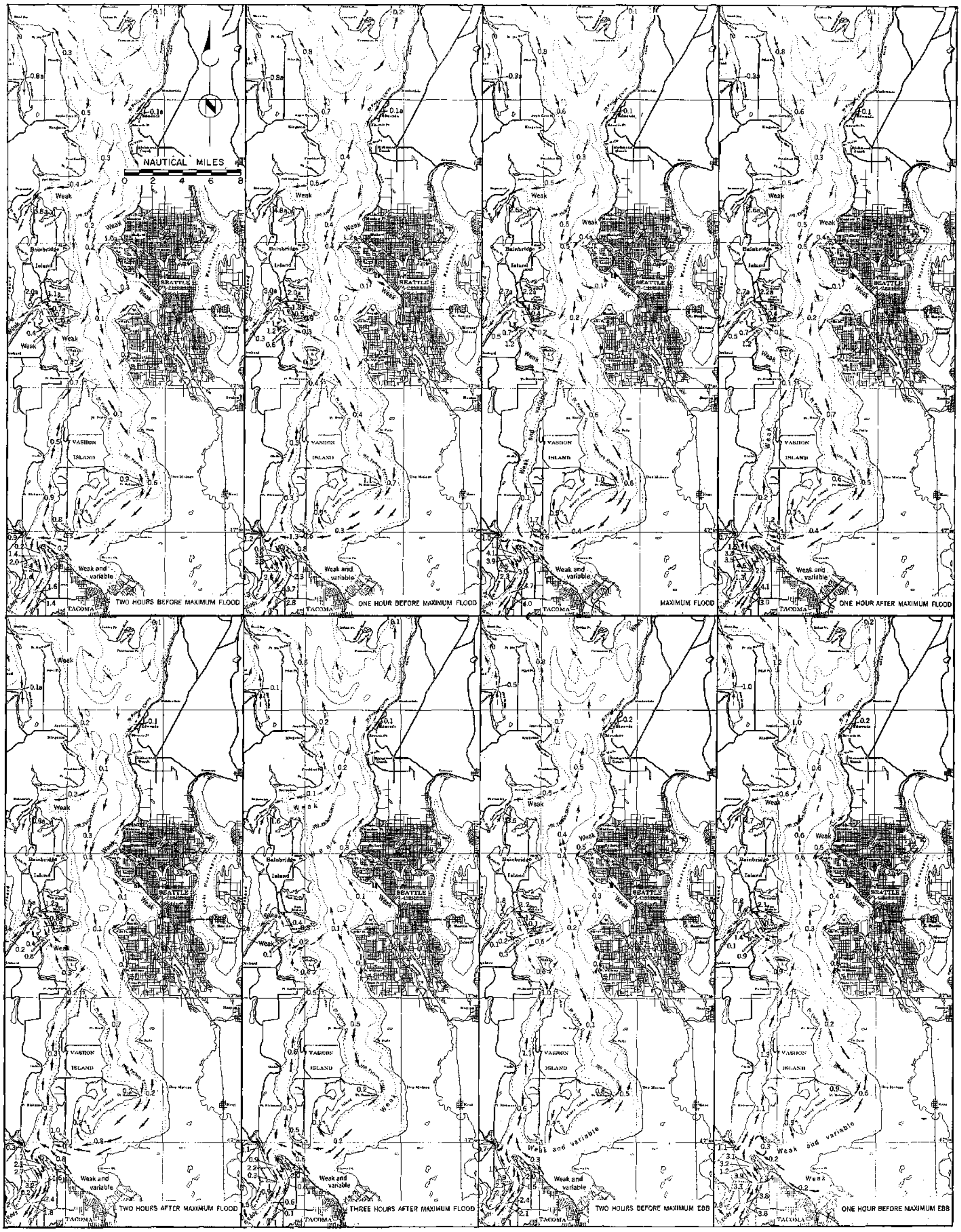


Fig. 11-4. Tidal Height Variation at Seattle

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
N
Bainbridge Island
SEATTLE
VASHON ISLAND
TACOMA
Weak
Weak and variable
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
TWO HOURS BEFORE MAXIMUM EBB
ONE HOUR BEFORE MAXIMUM EBB

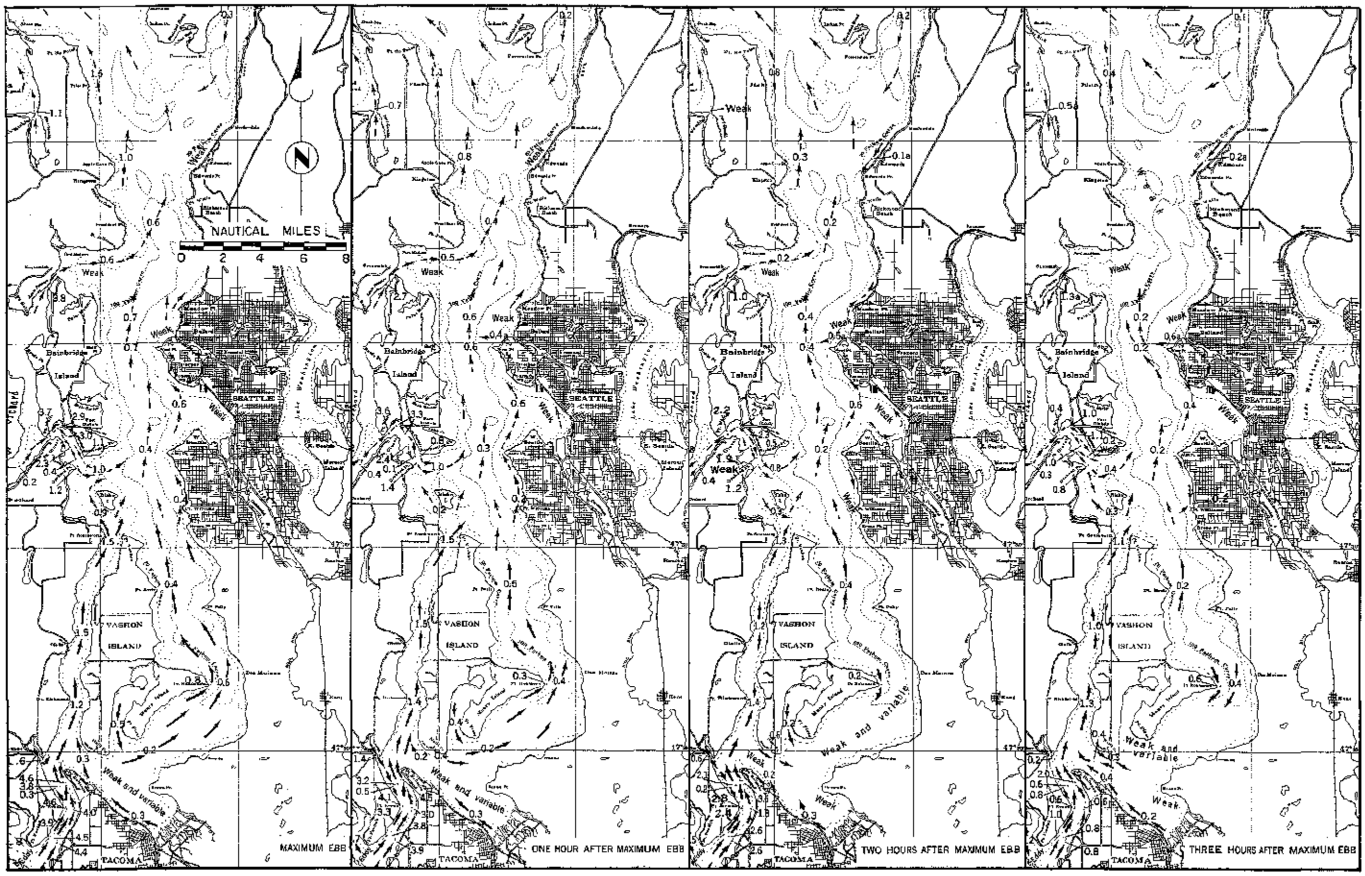


Fig 11-5. Generalized Tidal Currents in Surface Waters

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.*

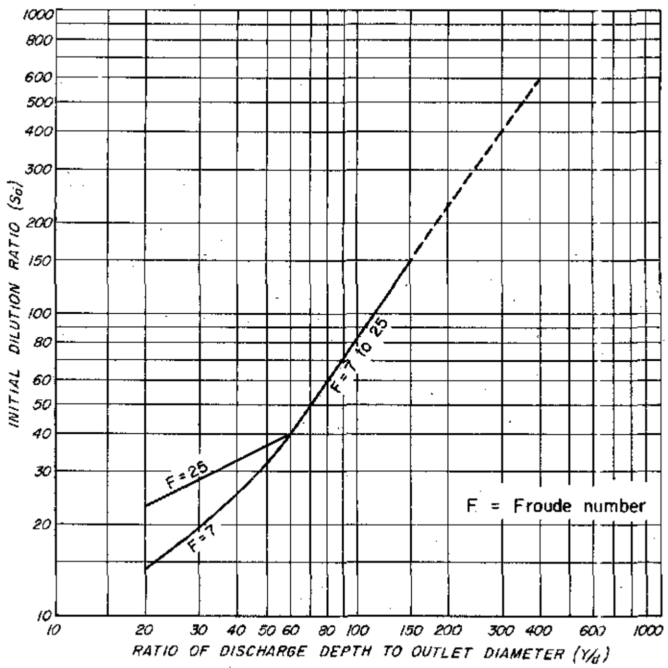


Fig. 11-6. Effect of Outlet Diameter and Depth on Initial Dilution of Sewage

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.

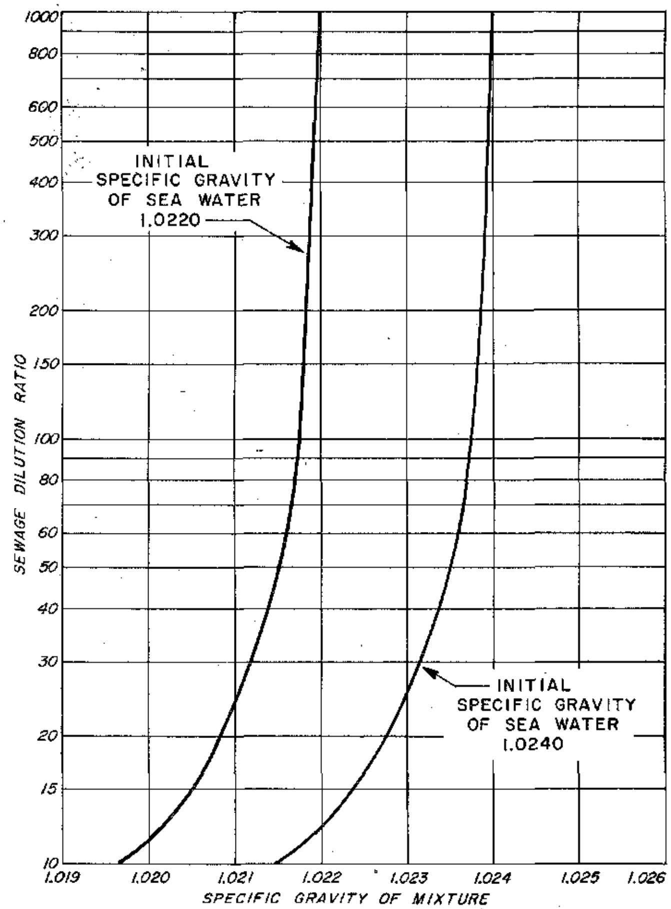


Fig. 11-7. Effect of Dilution on Density of Sewage-Sea Water Mixture

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.*

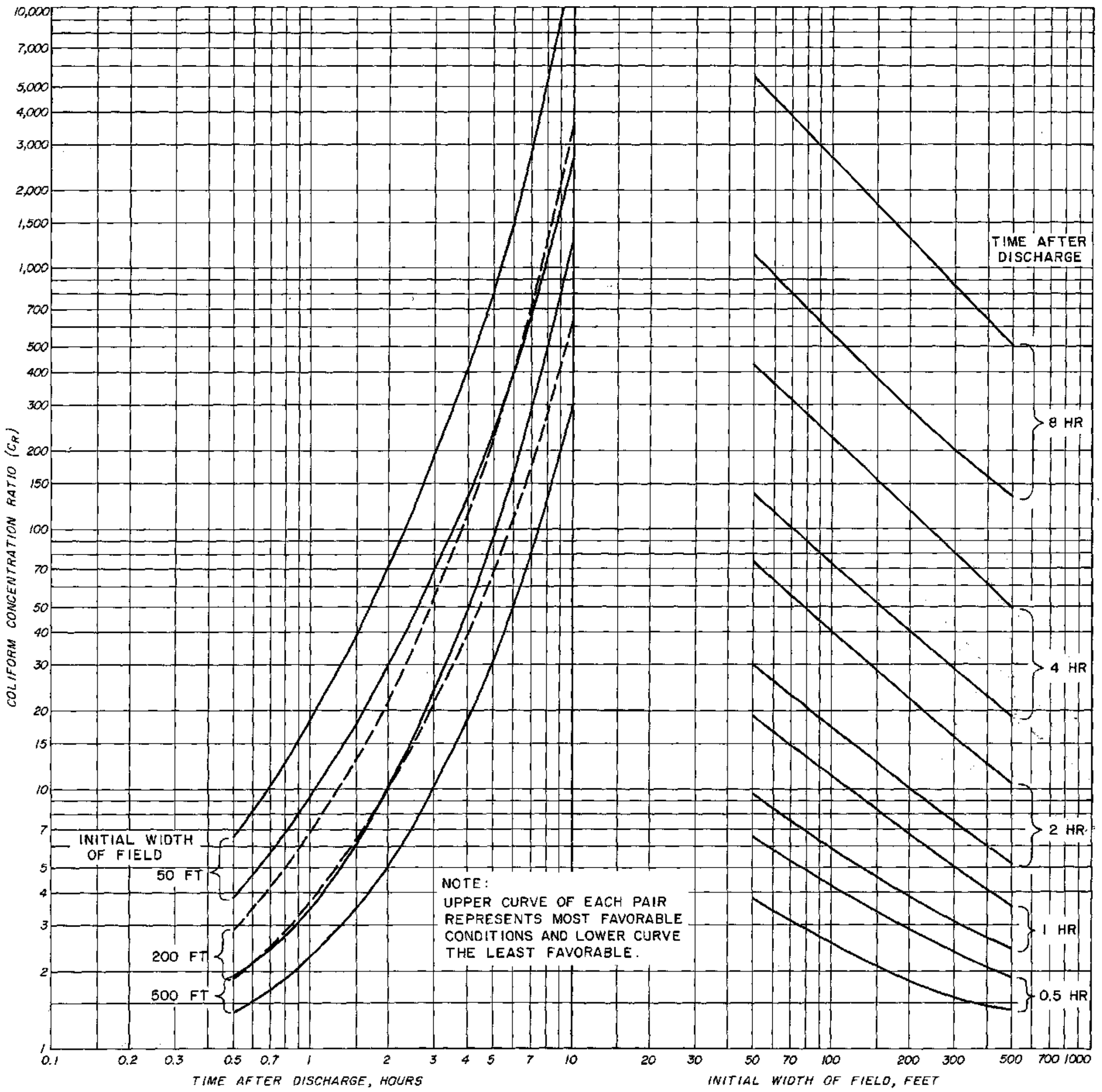


Fig. 11-8. Effect of Time and Size of Field on Coliform Concentration Ratio

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.

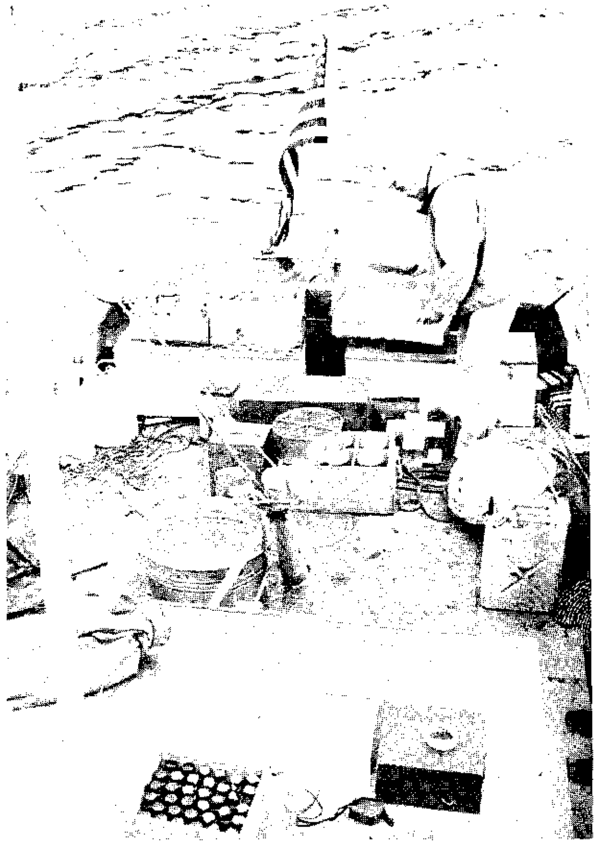
Resuspension. Since current velocity is one of the factors governing deposition, it is evident that increases in velocity would tend to pick up or resuspend particles which have previously settled to the bottom. This tendency is counteracted to some extent by the compaction which takes place after deposition. In the event, however, of either increased turbulence or seasonal upwelling of deep waters, a certain amount of resuspension is likely to occur.

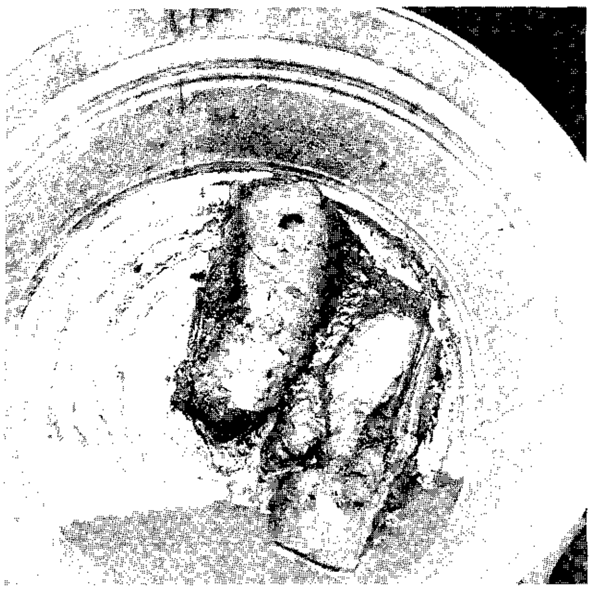
Initially, material will be resuspended from outer fringes of the deposition area where the least dense particles have settled. If conditions become increasingly favorable for resuspension, larger and heavier particles nearer the point of discharge will be picked up. In addition to particles of digested 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.

[12]*Department of Oceanography, University of Washington,* Oceanographic Survey on Submarine Portion of Snohomish-Kitsap, 25 KV Line, *1953.*

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 pcd 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 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

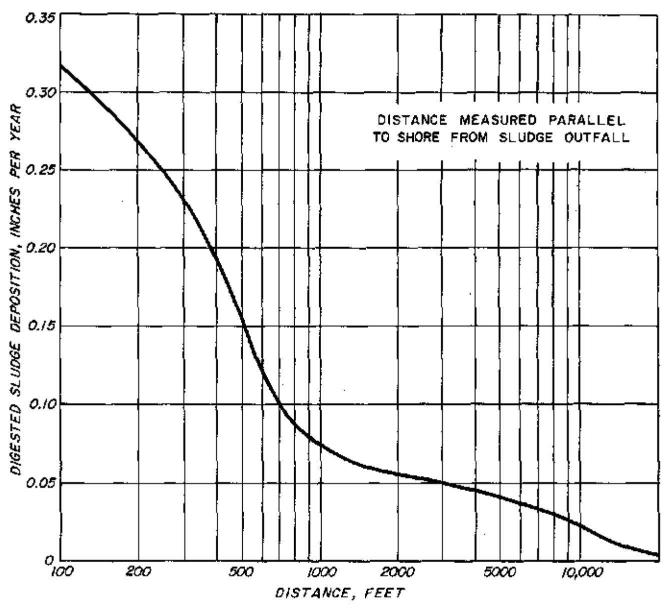


Fig. 11-9. Possible Magnitude of Digested Sludge Deposition from Discharge off West Point

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.*

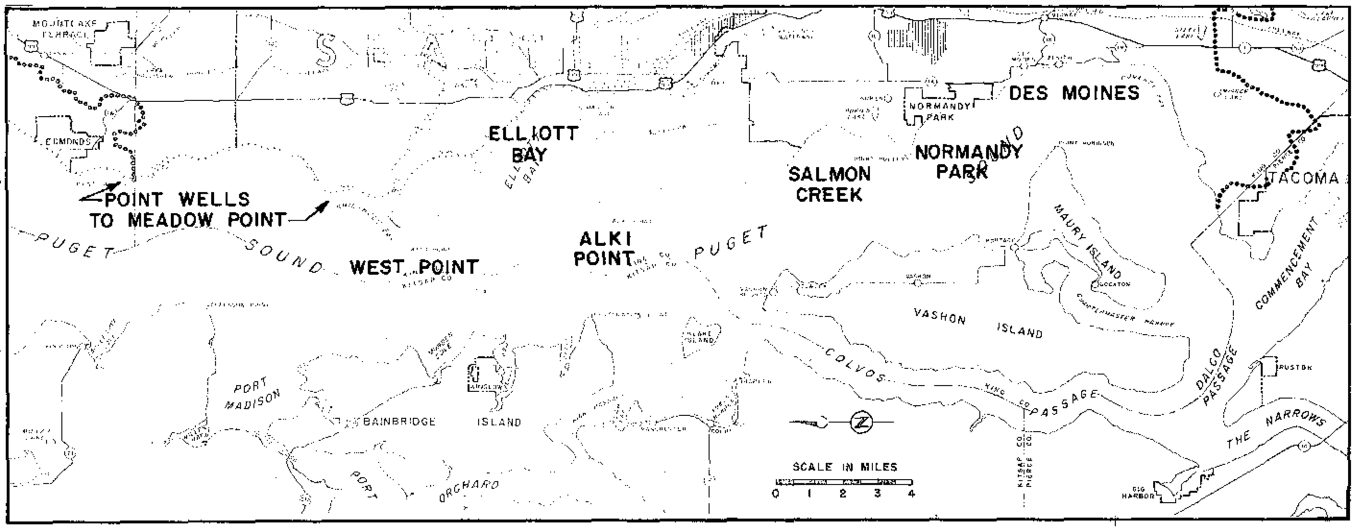


Fig. 11-10. Location of Current Study Areas

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

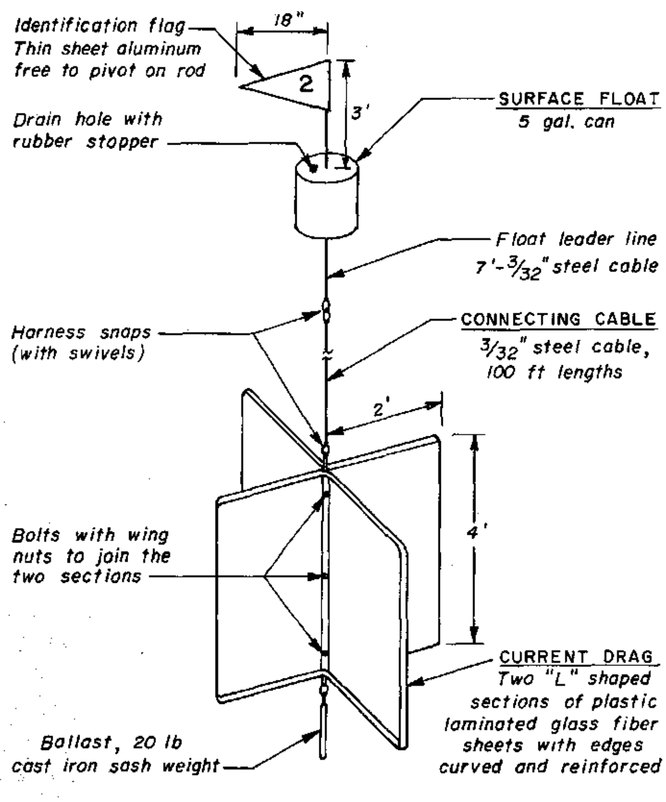
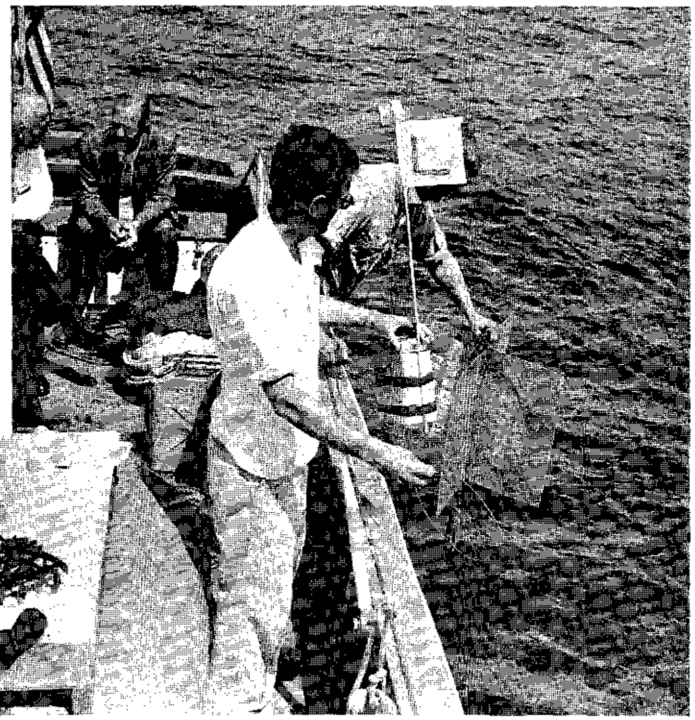
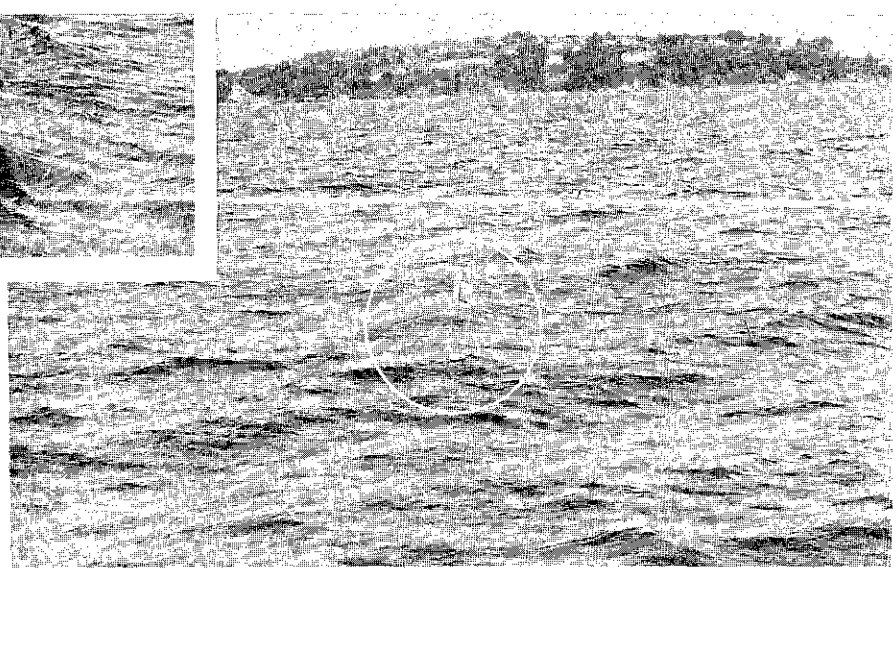


Fig. 11-11. Biplane Drags used to Measure Deep Currents

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.

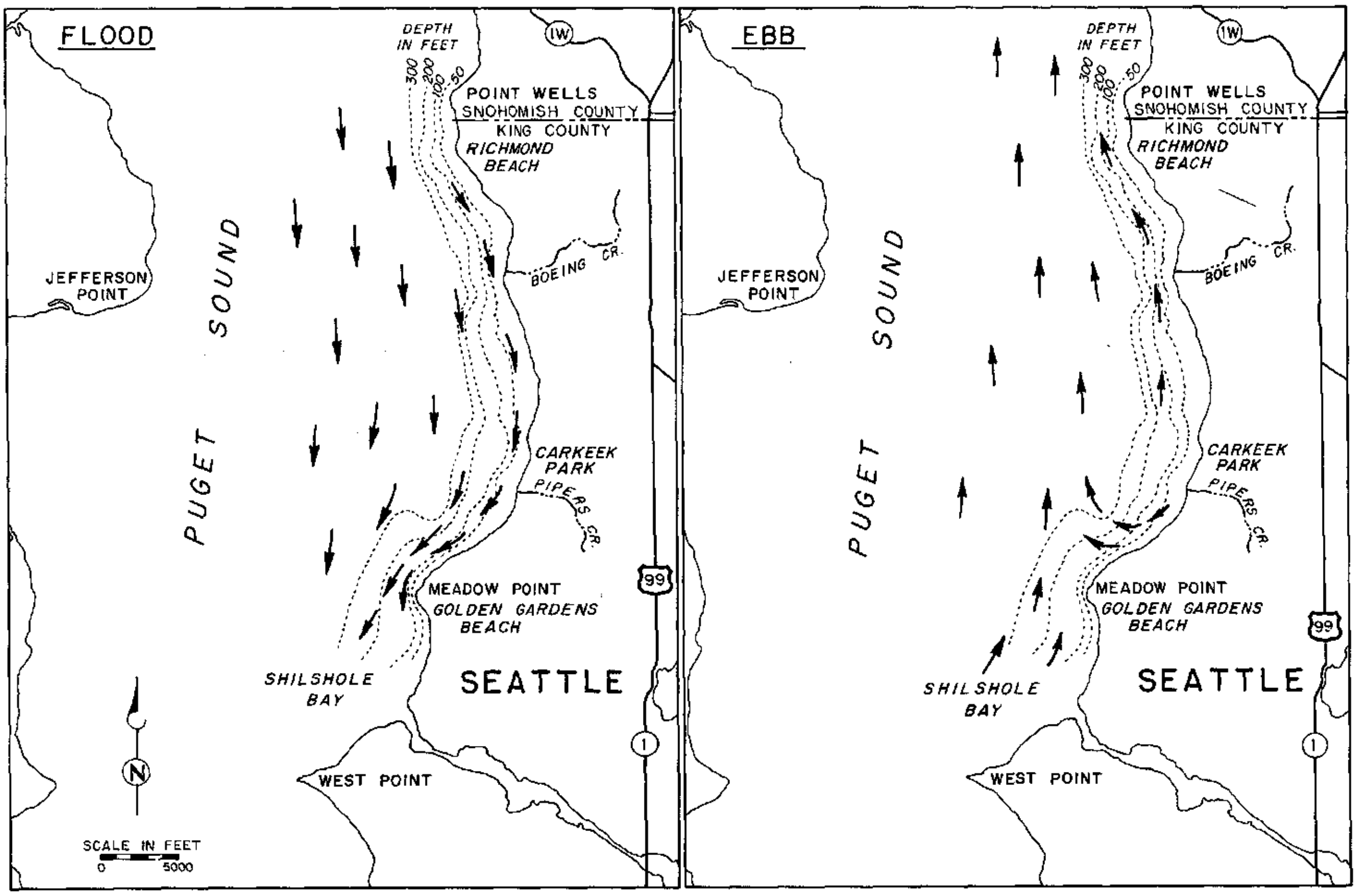


Fig. 11-12. Generalized Current Directions - Point Wells to Meadow Point

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.

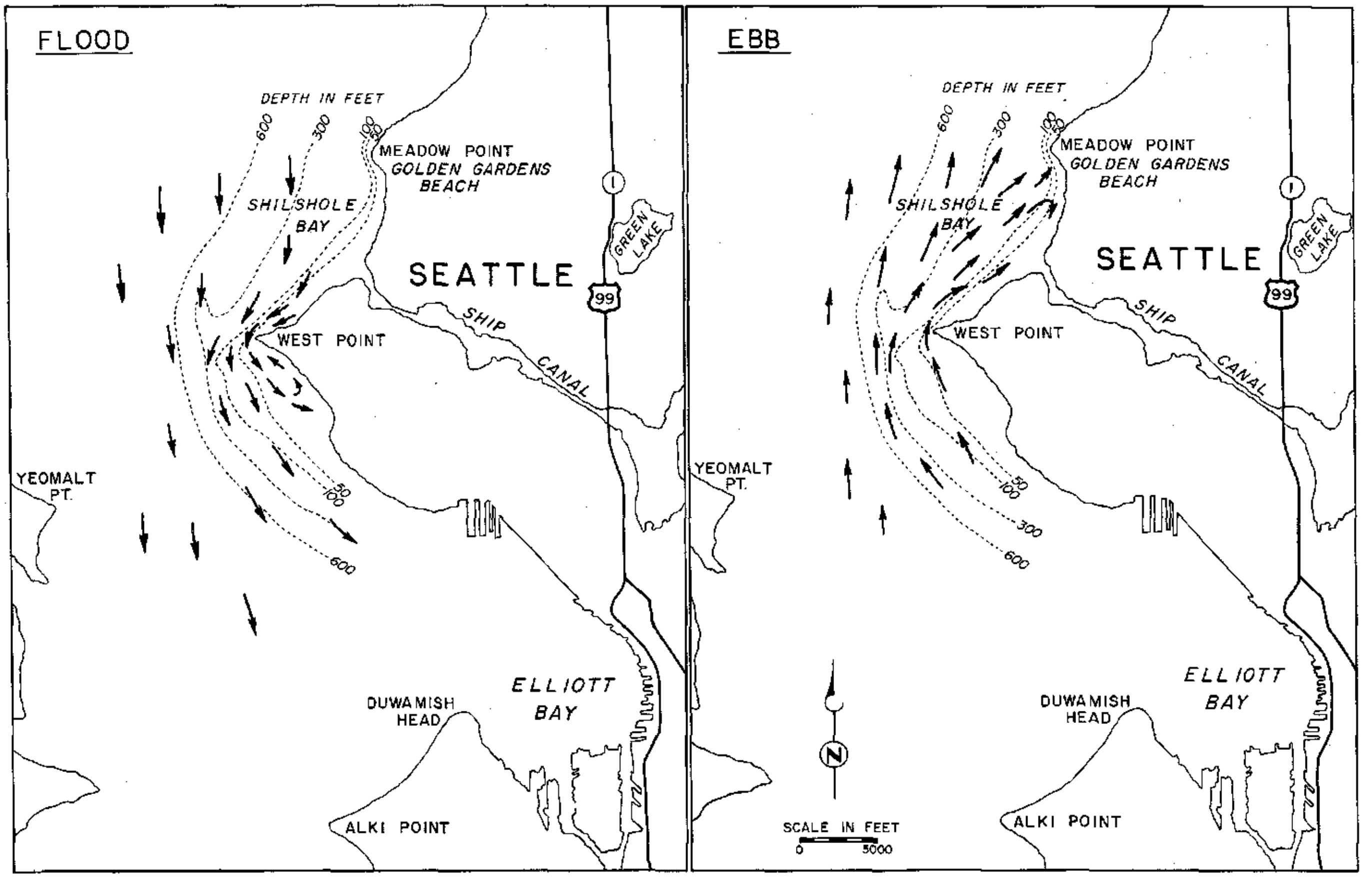


Fig. 11-13. Generalized Current Directions - West Point

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

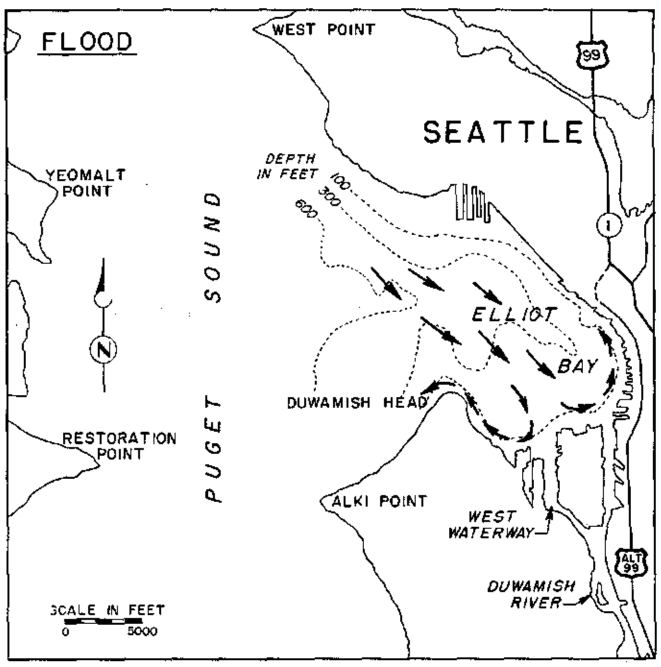


Fig. 11-14. Generalized Current Directions - Elliott Bay

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.

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.

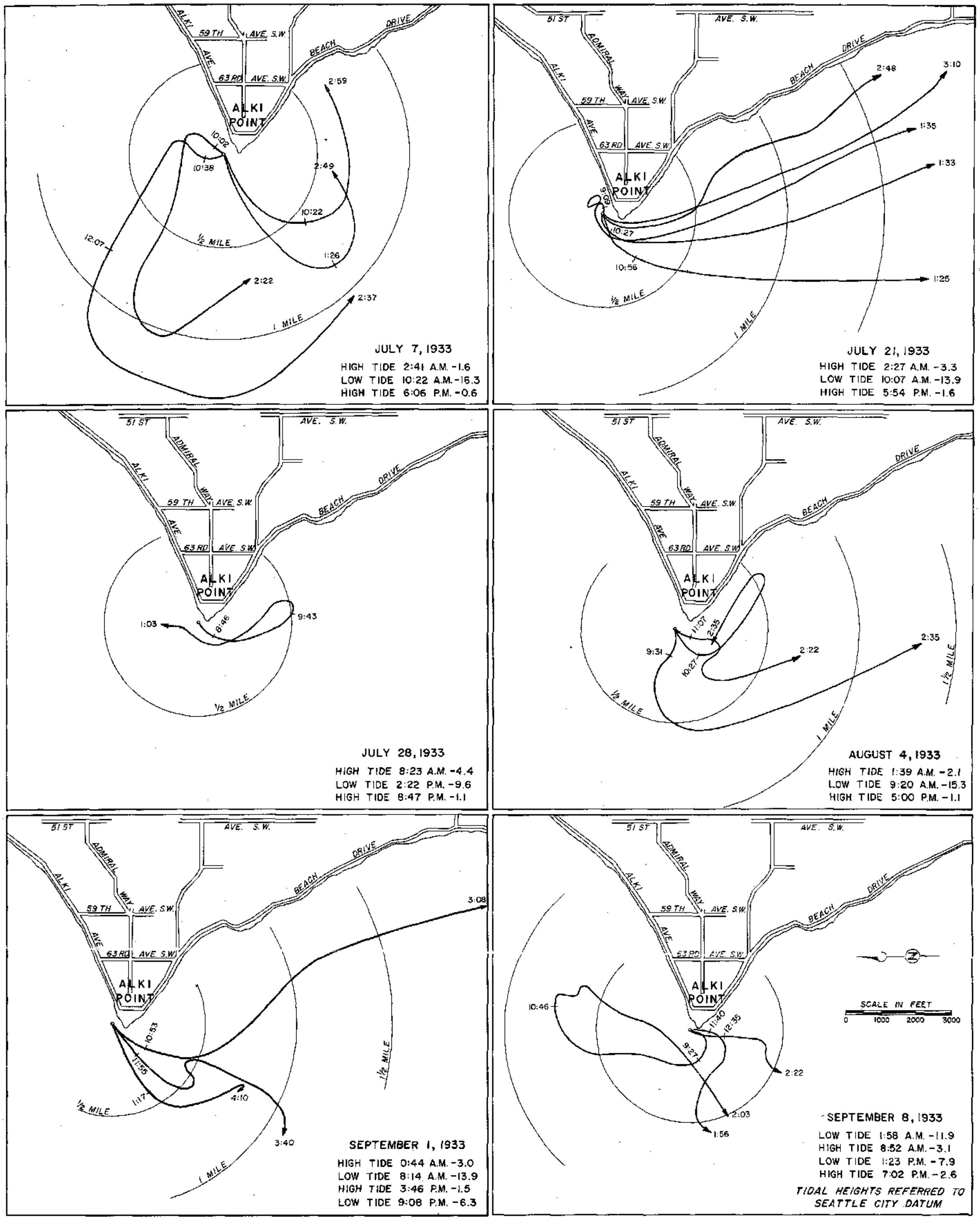


Fig. 11-15. Typical Results of 1933 Current Studies off Alki Point

Float paths shown are based on observations by city of Seattle personnel during the summer of 1933.

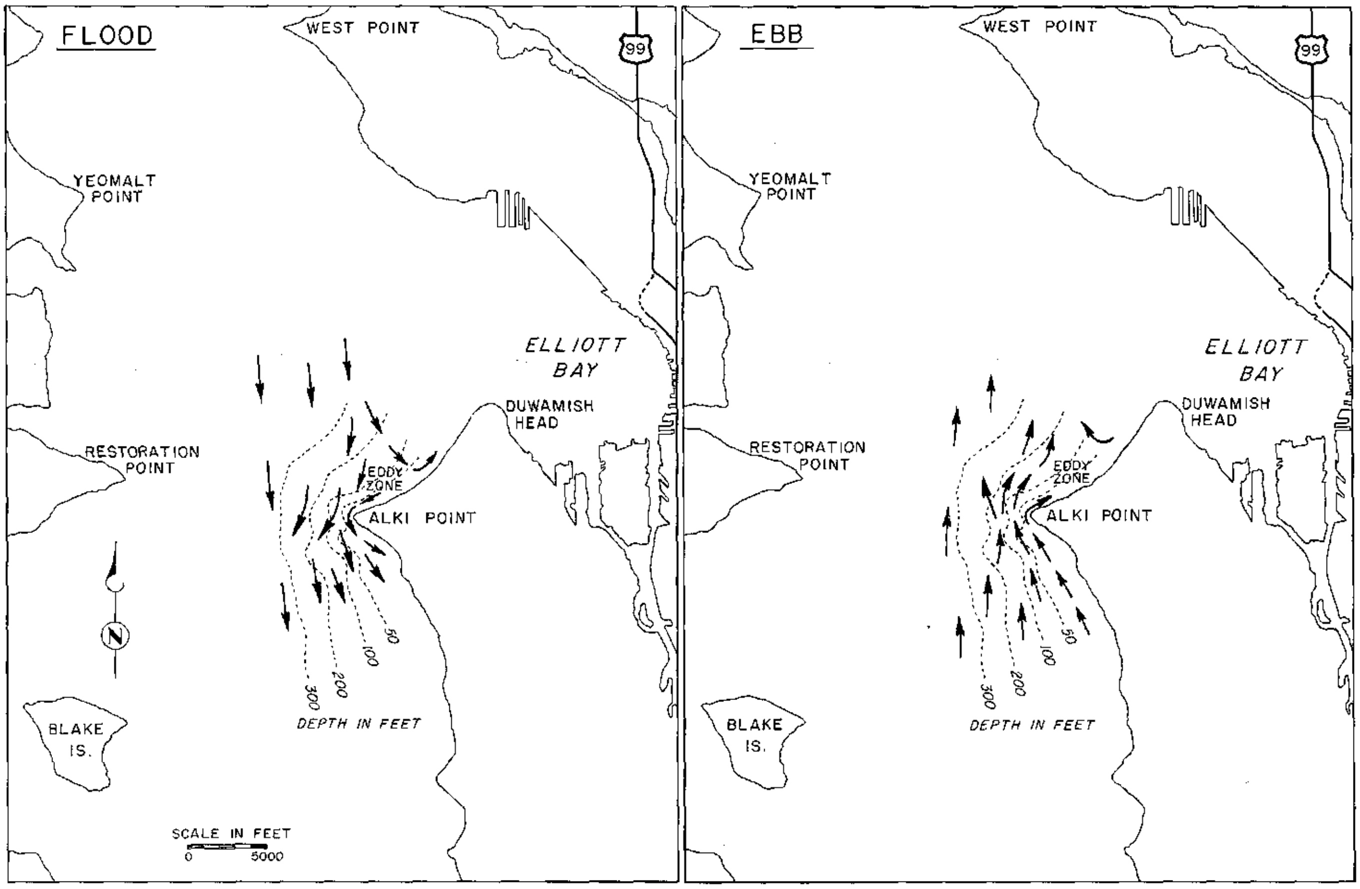


Fig. 11-16. Generalized Current Directions - Alki Point

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.

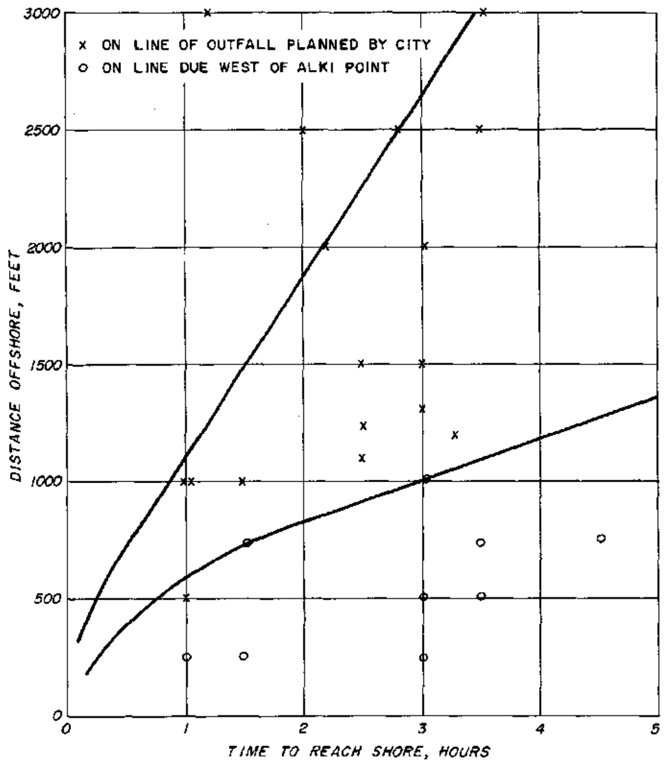


Fig. 11-17. Minimum Travel Times to Shore from Locations off Alki Point

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

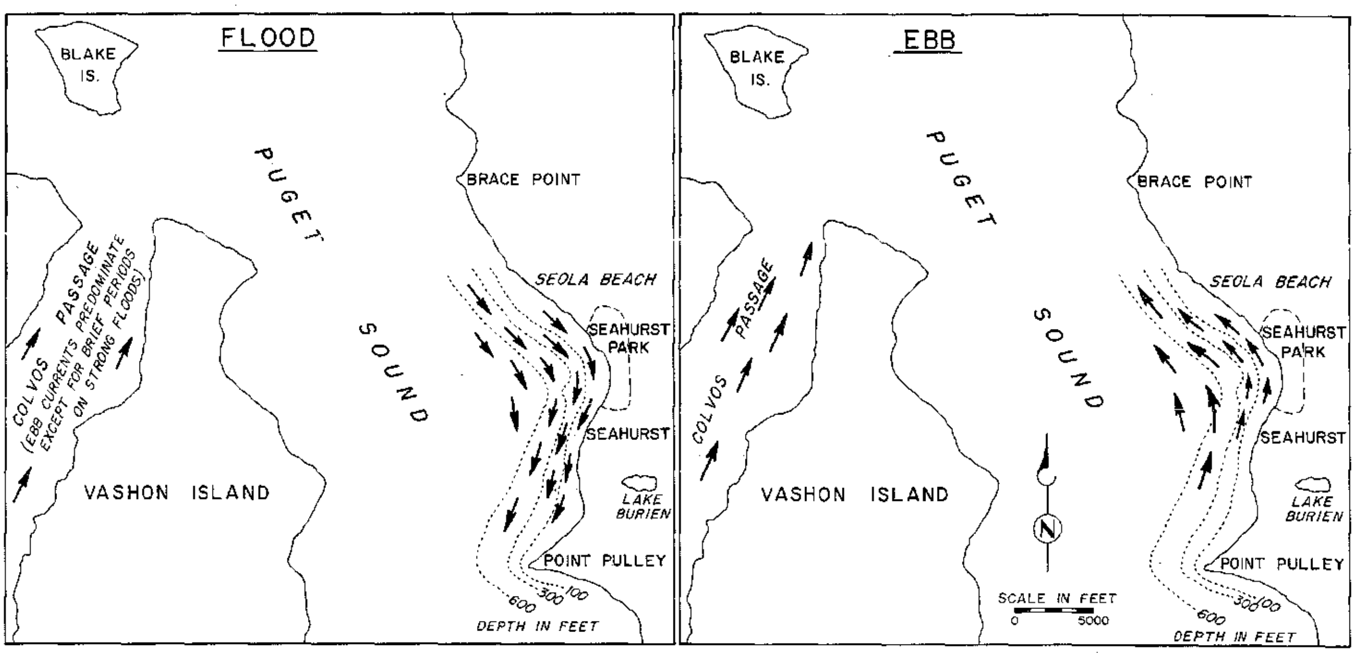


Fig. 11-18. Generalized Current Directions - Salmon Creek

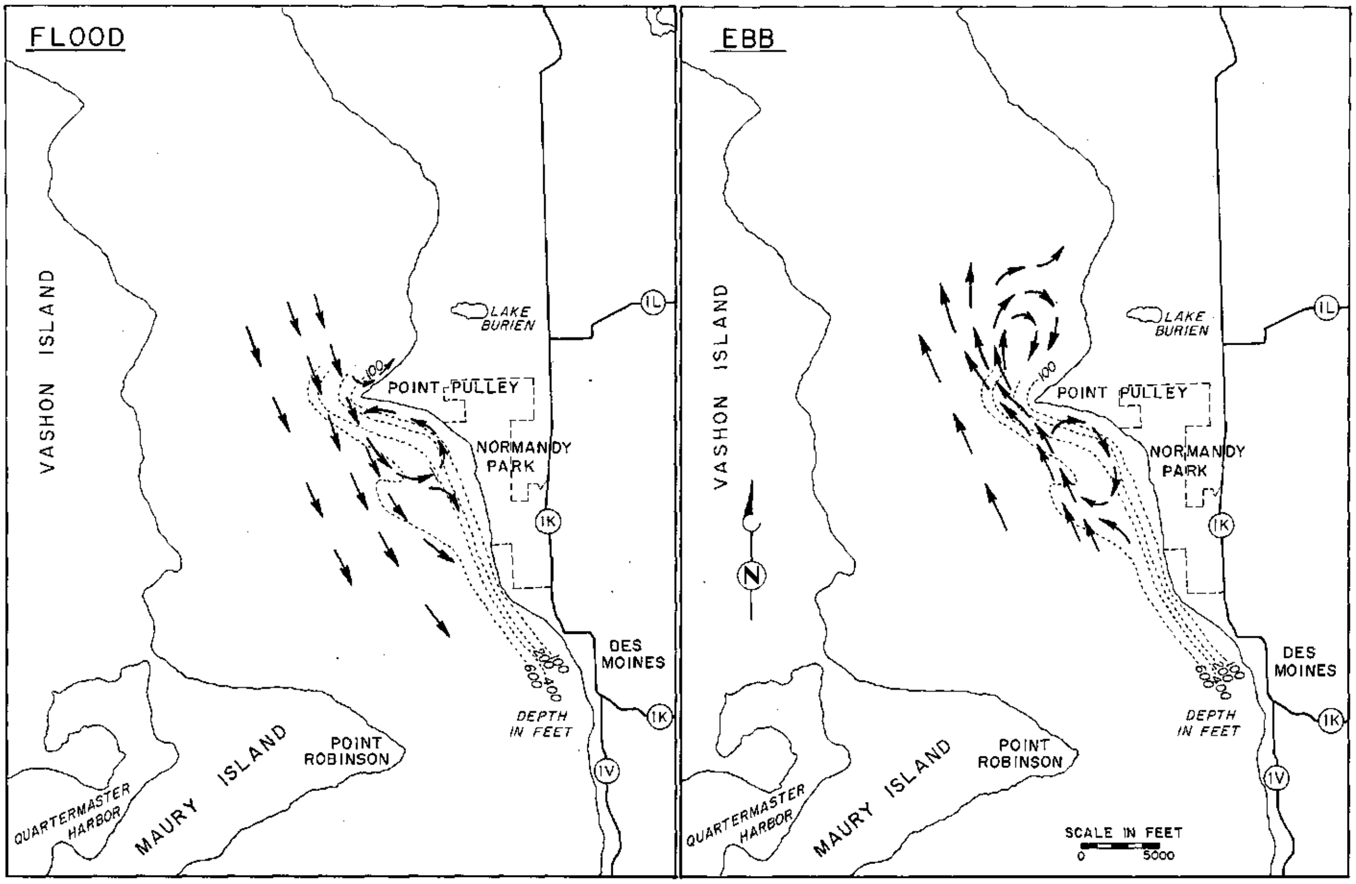


Fig. 11-19. Generalized Current Directions – Point Pulley

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

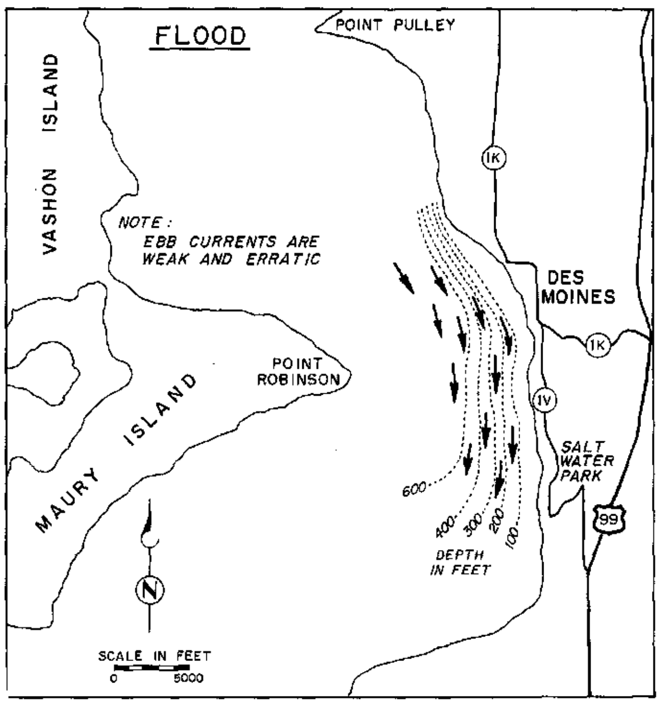


Fig. 11-20. Generalized Current Directions - Des Moines

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 5×10^7 per 100 ml in primary effluent without disinfection and 5×10^6 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

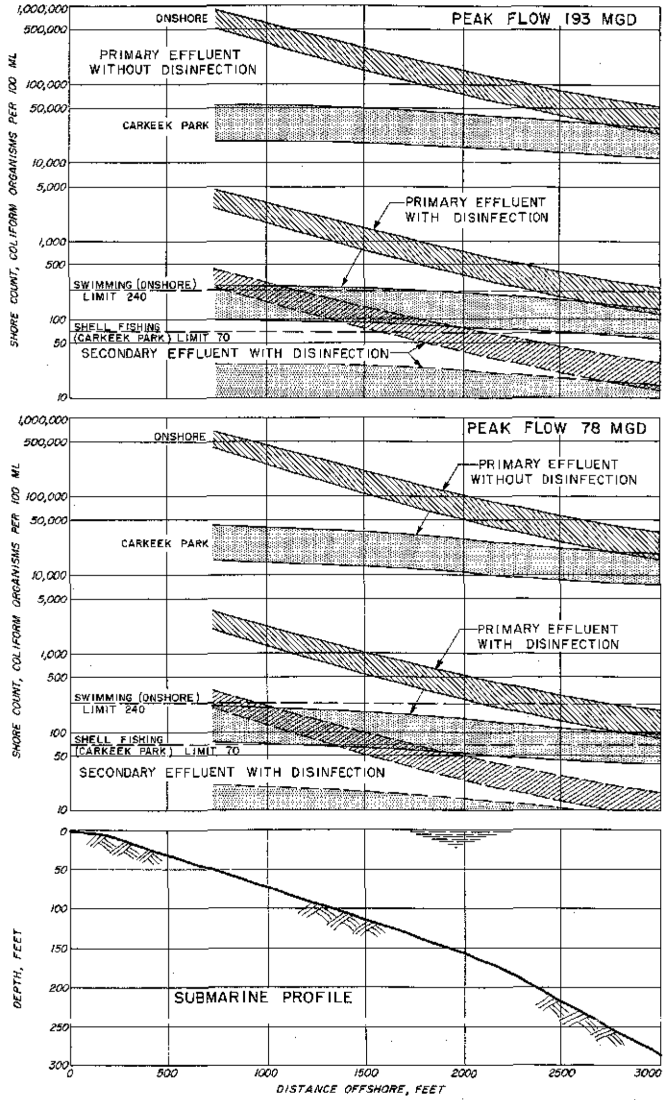


Fig. 11-21. Estimated Maximum Coliform Counts Richmond Beach Site

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.

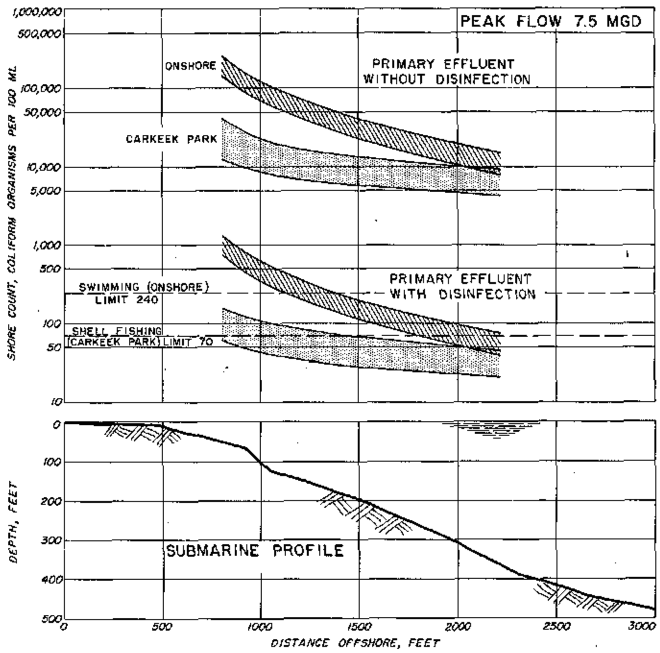


Fig. 11-22. Estimated Maximum Coliform Counts Boeing Creek Site

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

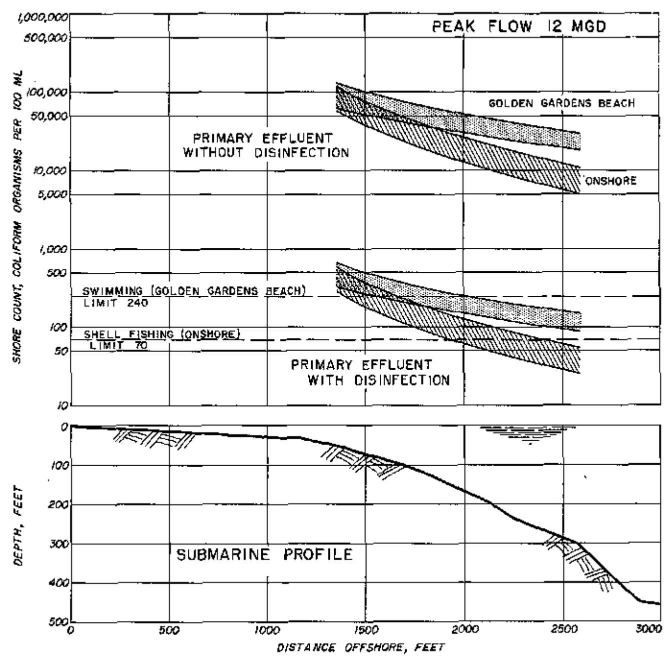


Fig. 11-23. Estimated Maximum Coliform Counts Piper Creek Site

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.

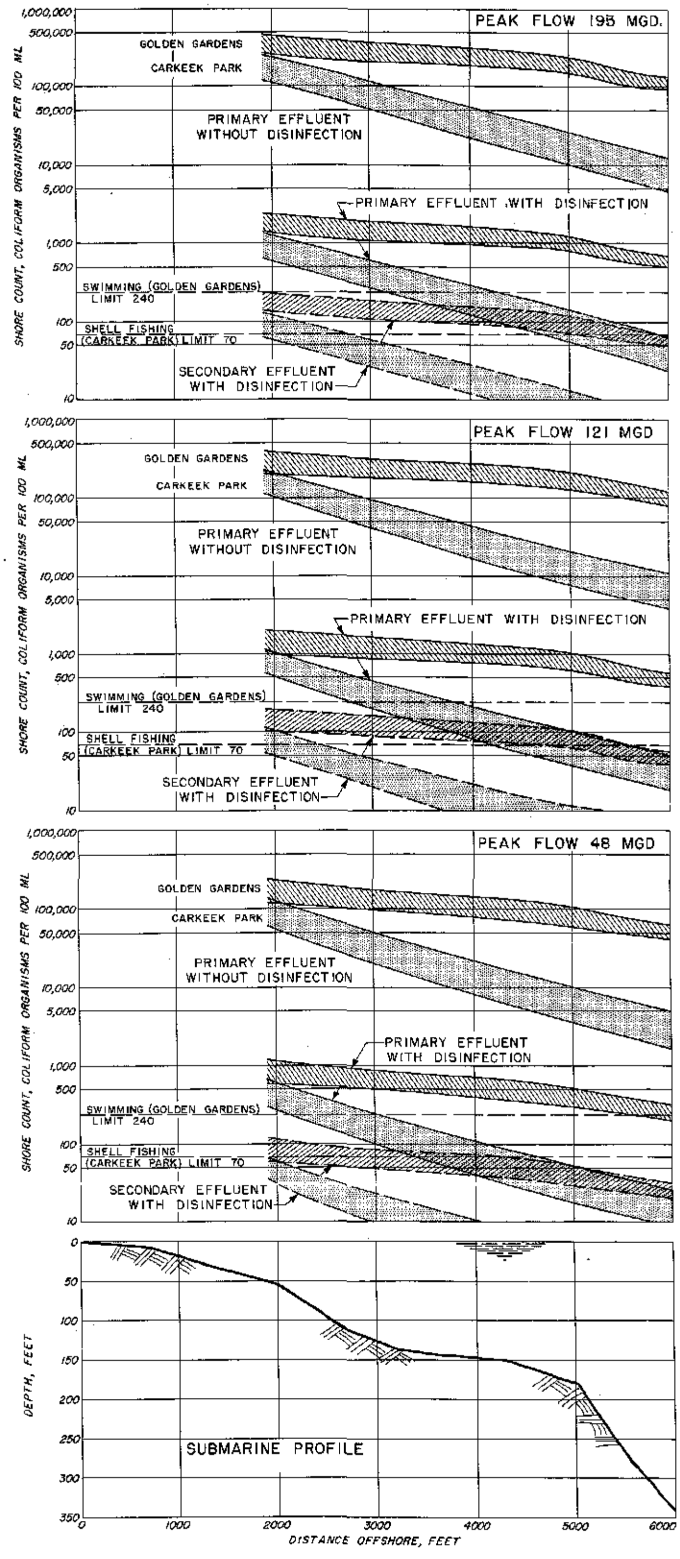


Fig. 11-24. Estimated Maximum Coliform Counts Site South of Piper Creek

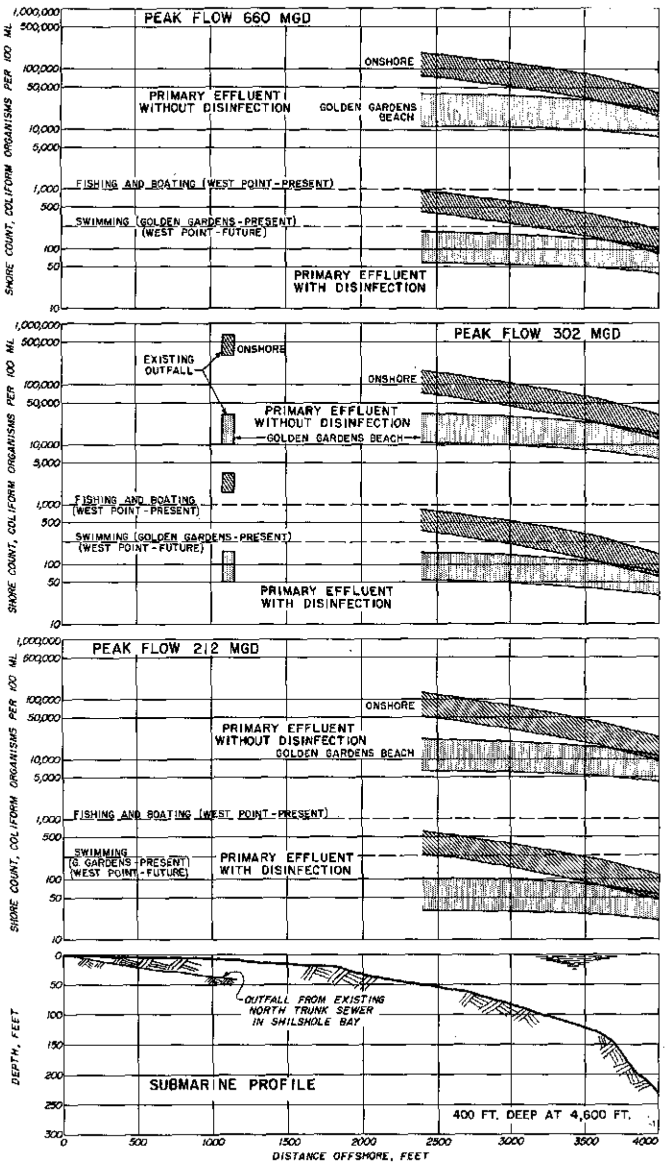


Fig. 11-25. Estimated Maximum Coliform Counts West Point Site

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

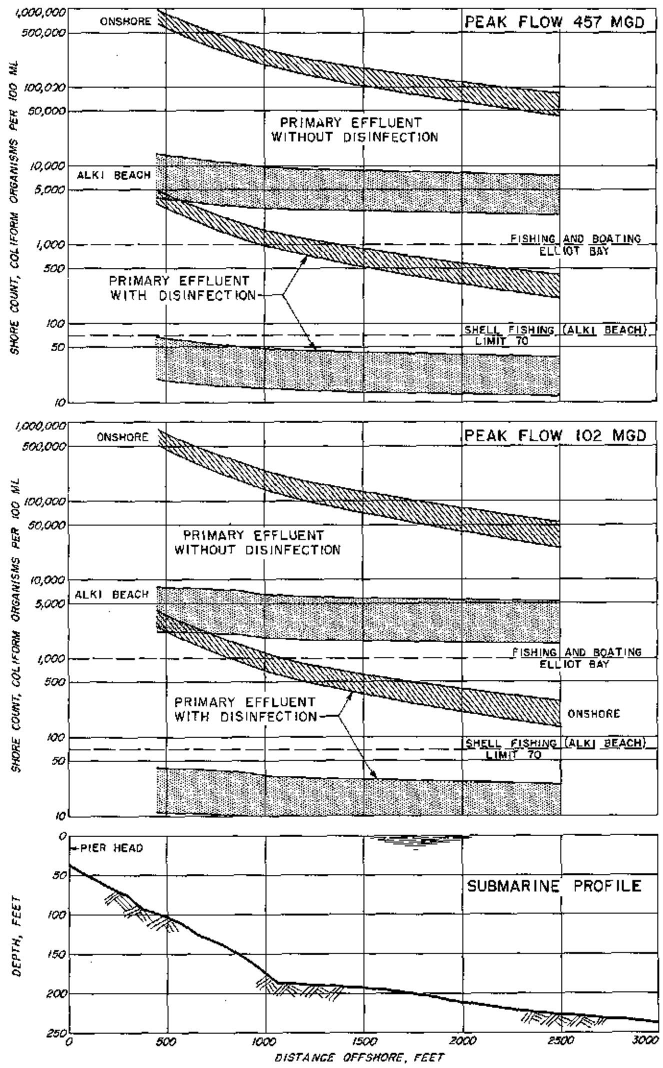


Fig. 11-26. Estimated Maximum Coliform Counts Elliott Bay Site

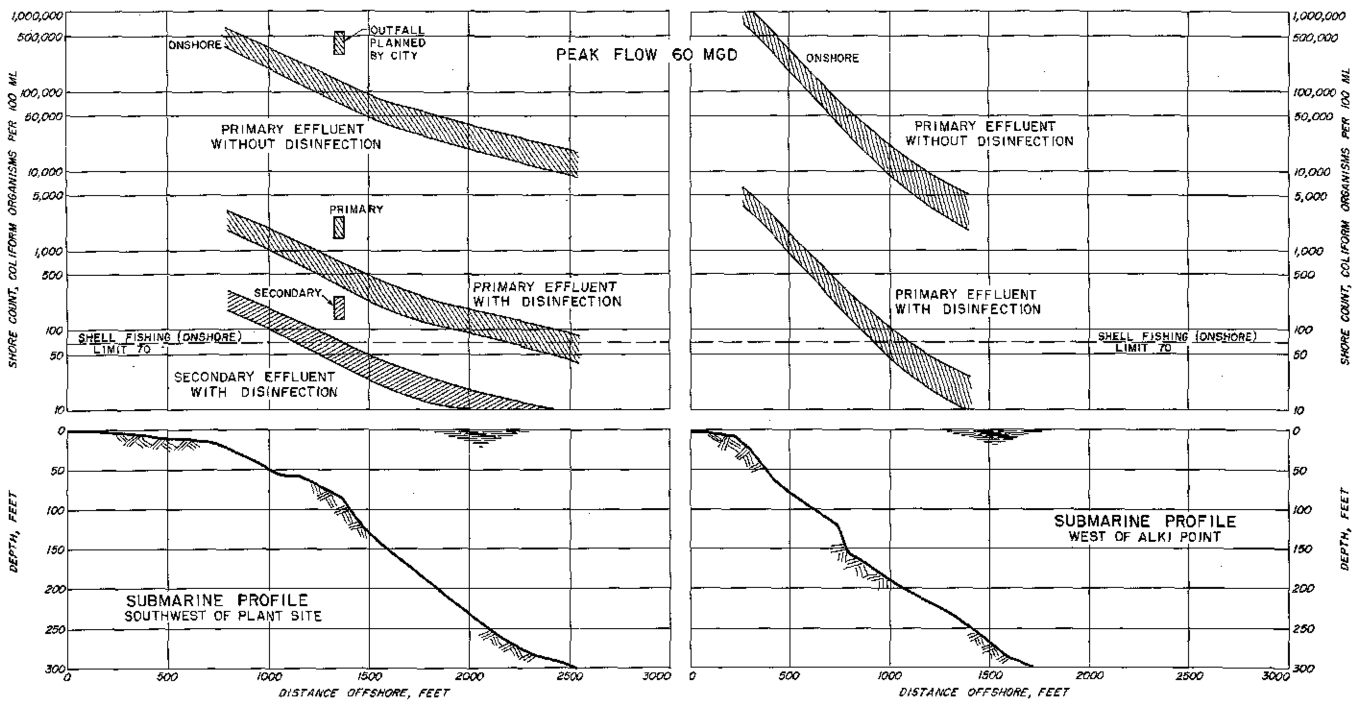


Fig. 11-27. Estimated Maximum Coliform Counts - Alki Point Site

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

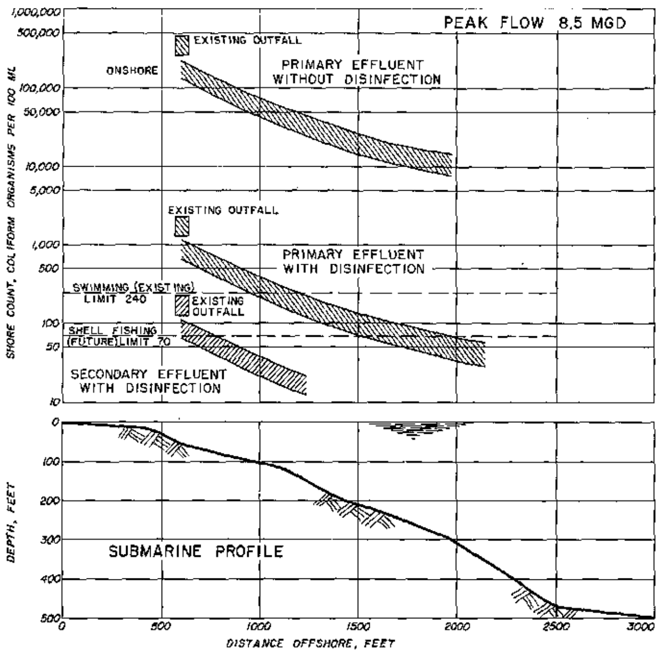


Fig. 11-28. Estimated Maximum Coliform Counts Southwest Suburban Site

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

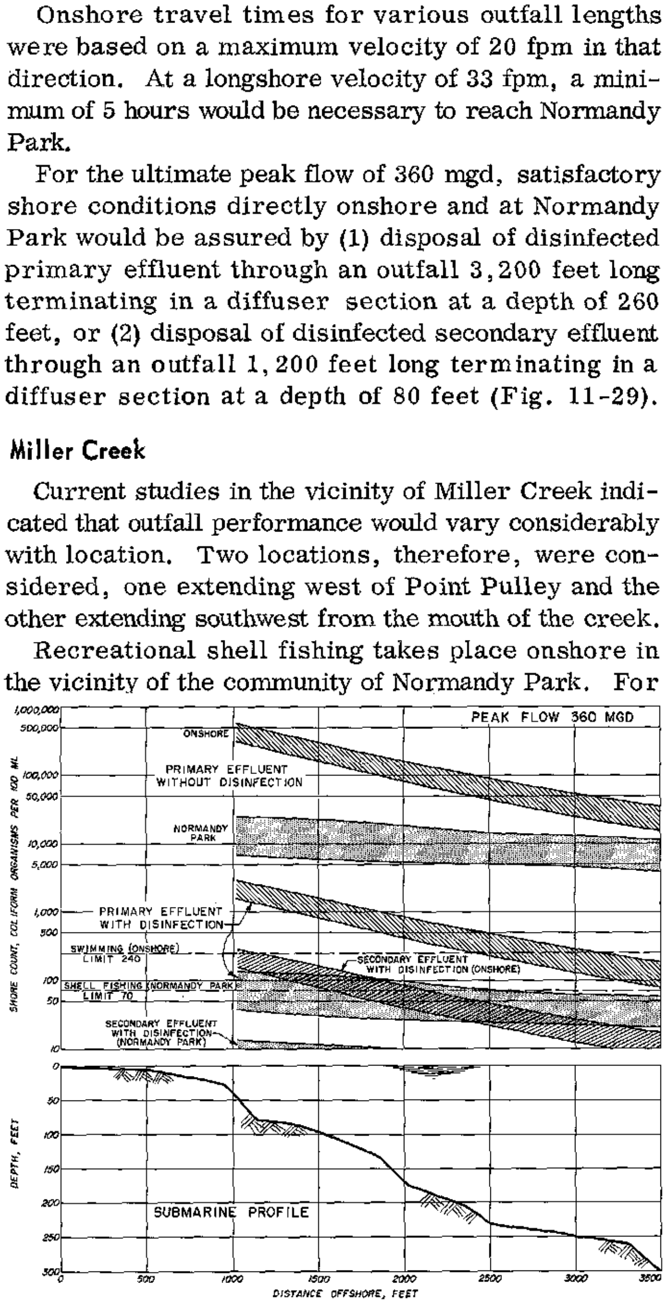


Fig. 11-29. Estimated Maximum Coliform Counts Seahurst Site

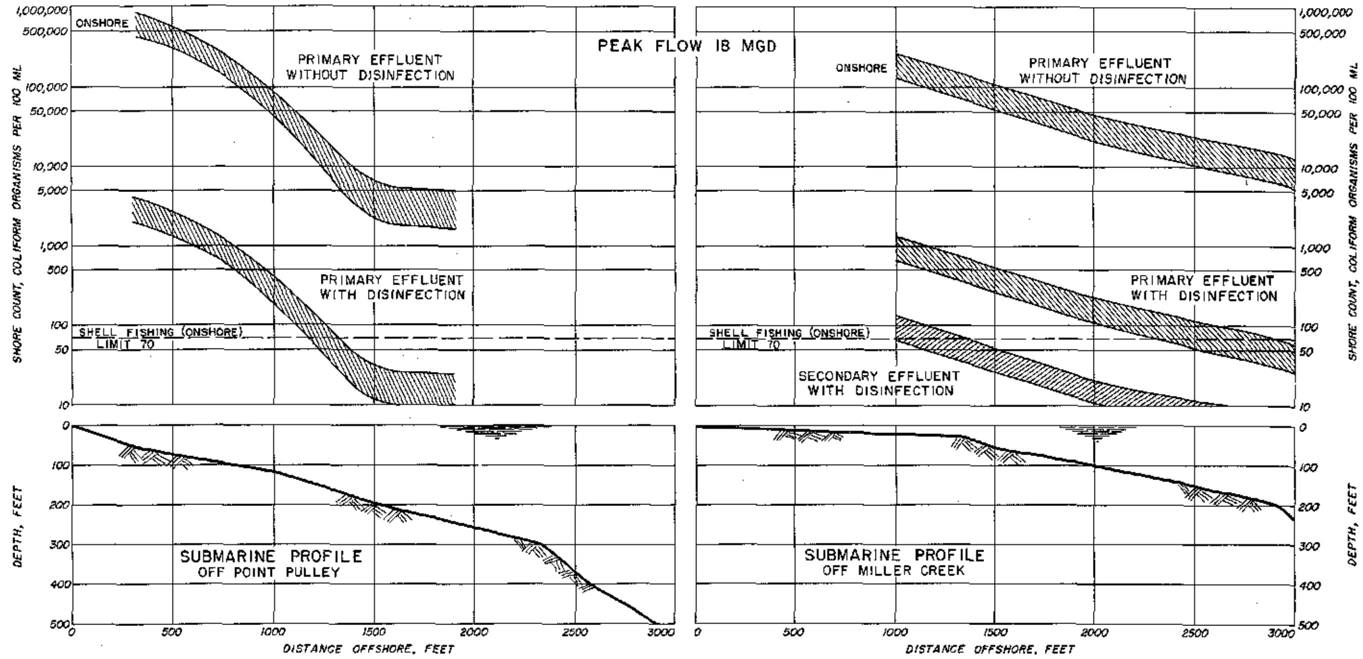


Fig. 11-30. Estimated Maximum Coliform Counts - Miller Creek Site

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

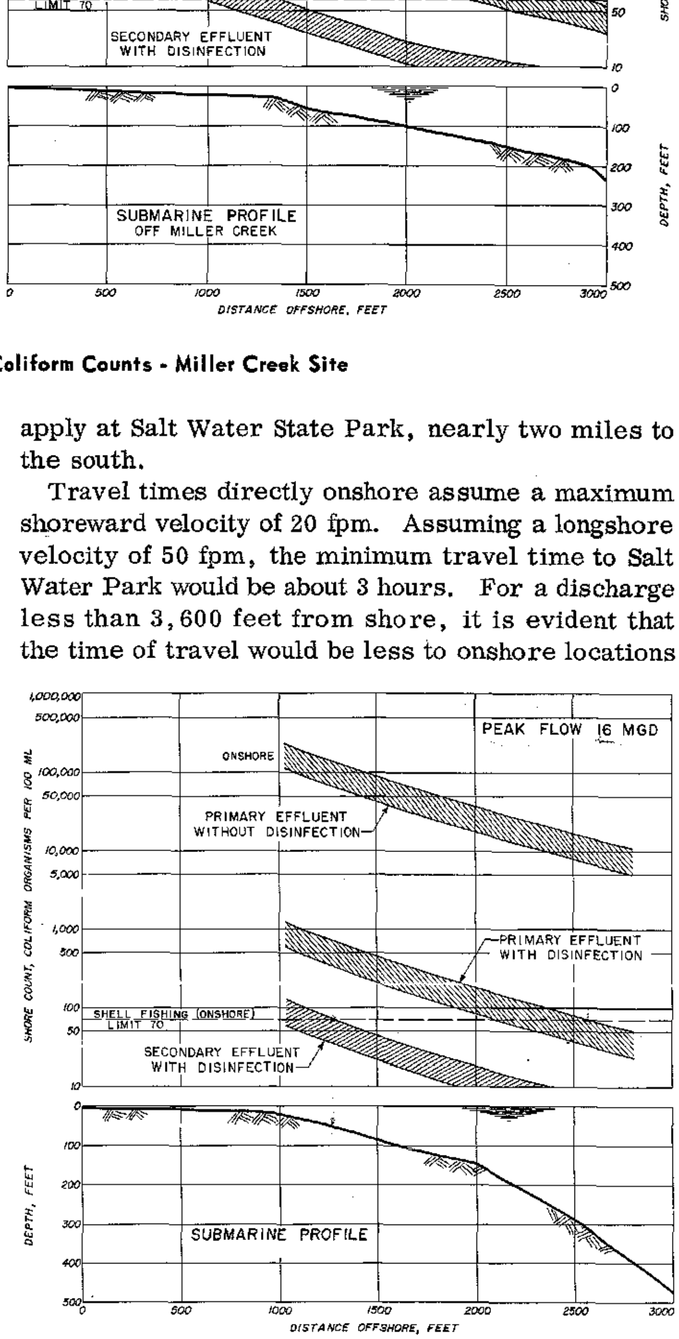


Fig. 11-31. Estimated Maximum Coliform Counts Des Moines Site

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.

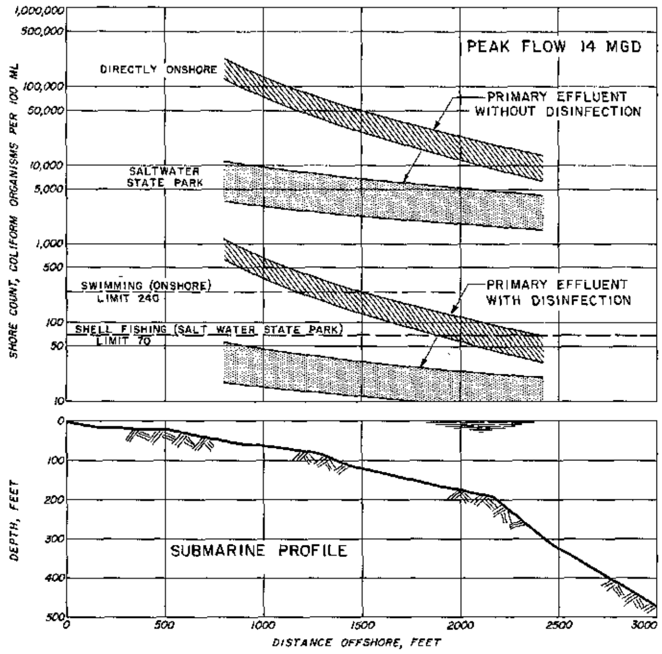


Fig. 11-32. Estimated Maximum Coliform Counts Redondo Beach Site

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.*

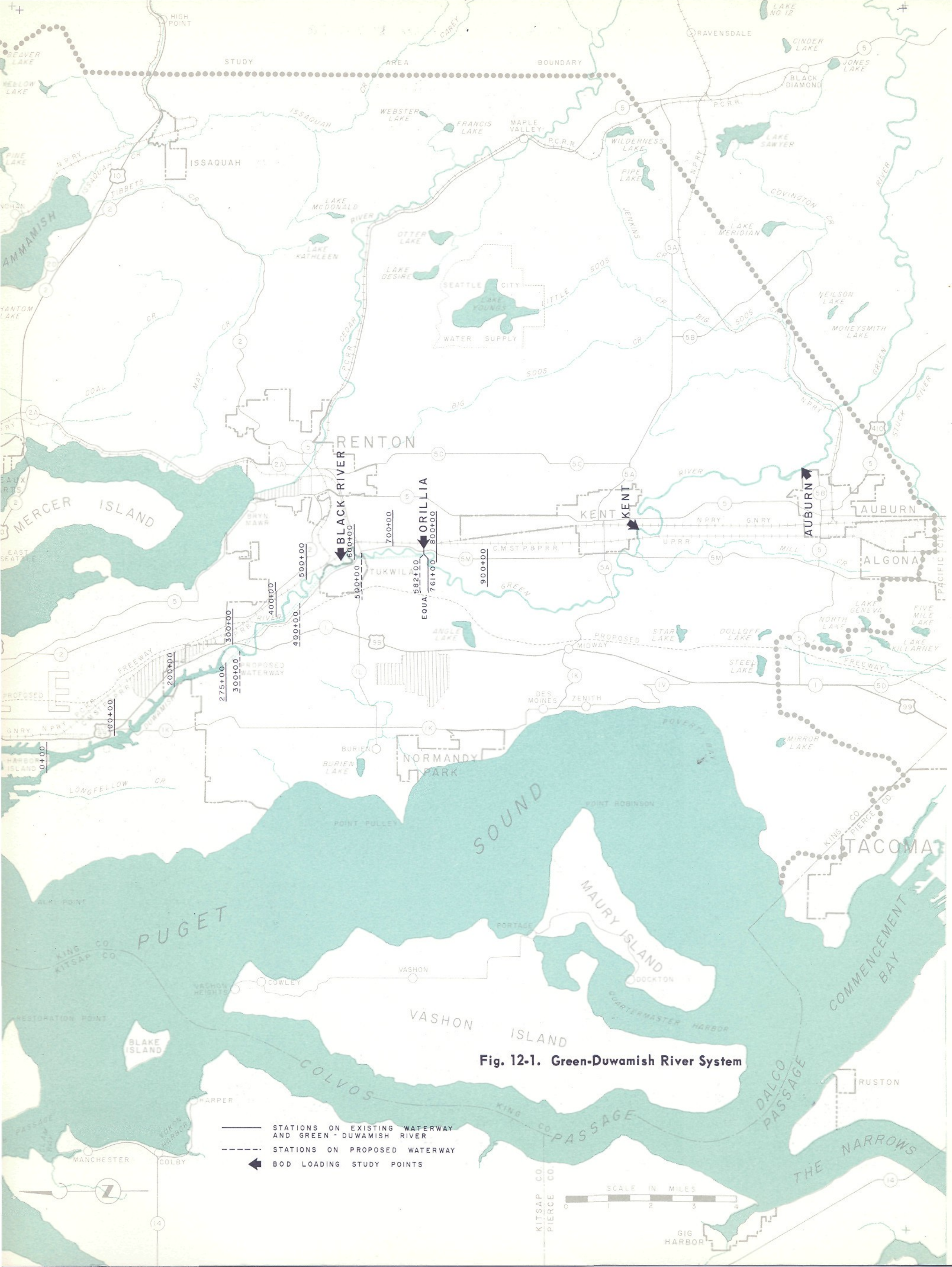


Fig. 12-1. Green-Duwamish River System

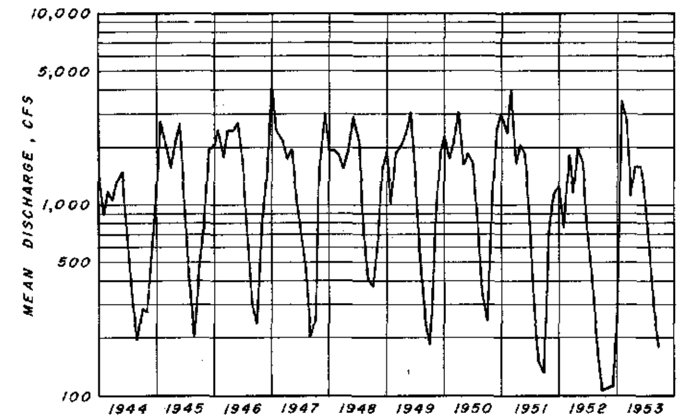


Fig. 12-2. Green River Discharge

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

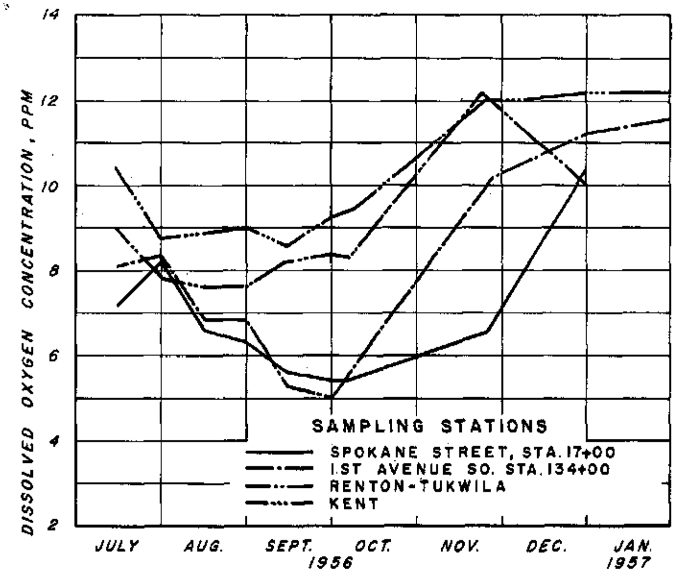


Fig. 12-3. Dissolved Oxygen Concentrations in Green-Duwamish River

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.*

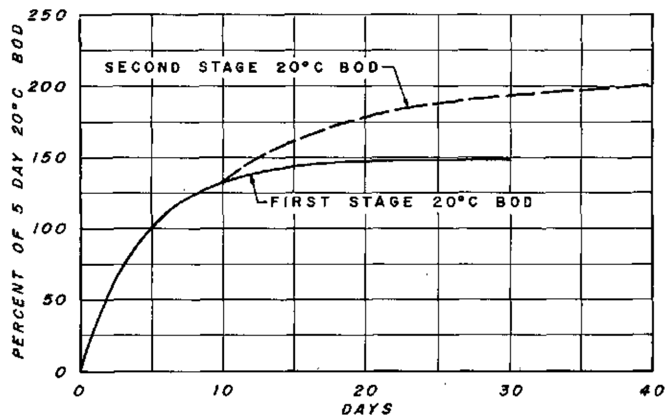


Fig. 12-4. Typical Biochemical Demand Reactions for Domestic Sewage

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.*

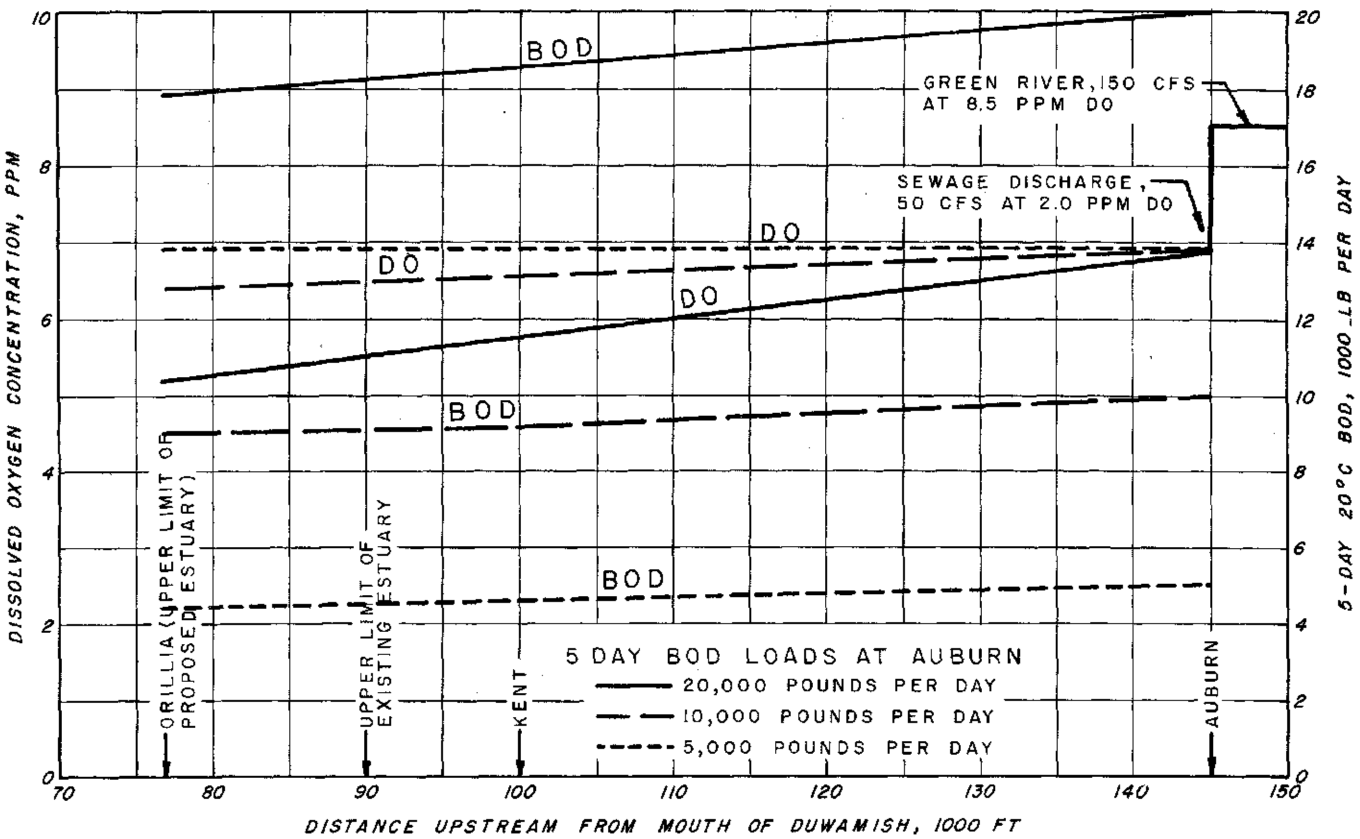


Fig. 12-5. Dissolved Oxygen Concentrations in Green River at Various BOD Loads Applied at Auburn

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

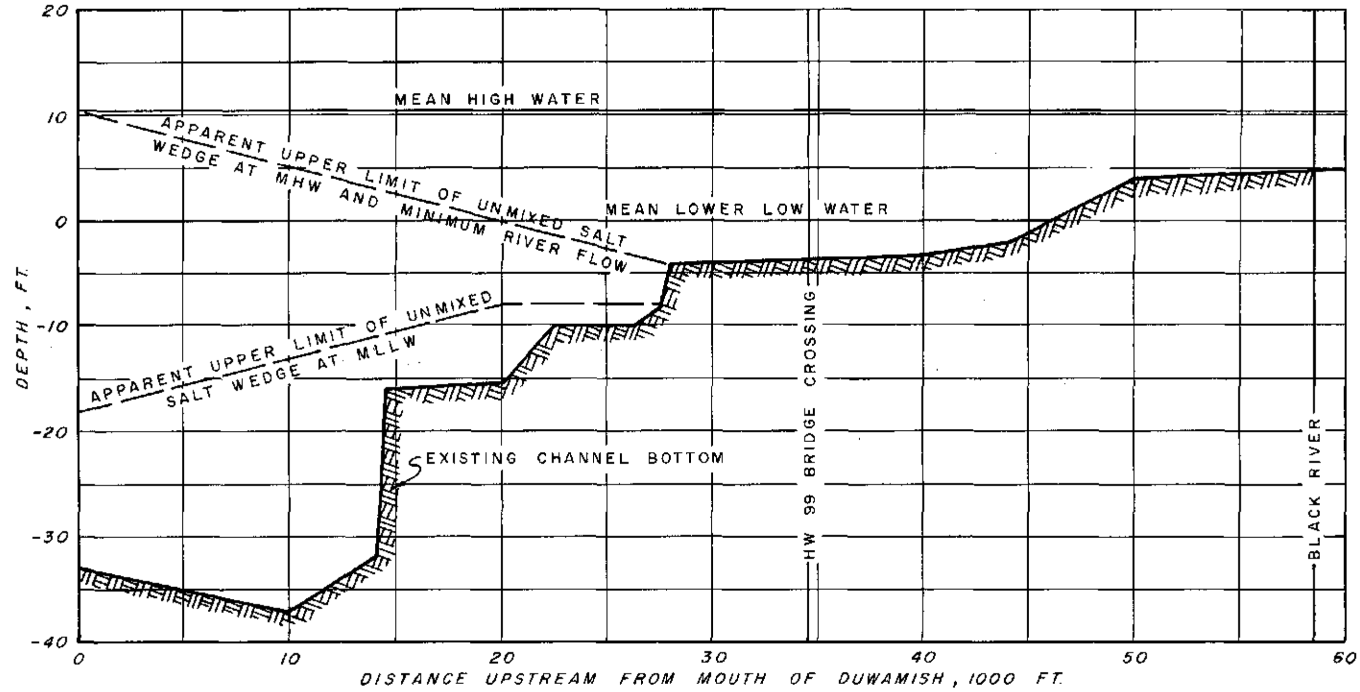


Fig. 12-6. Location of Salt Water Wedge in Existing Duwamish Estuary

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.*

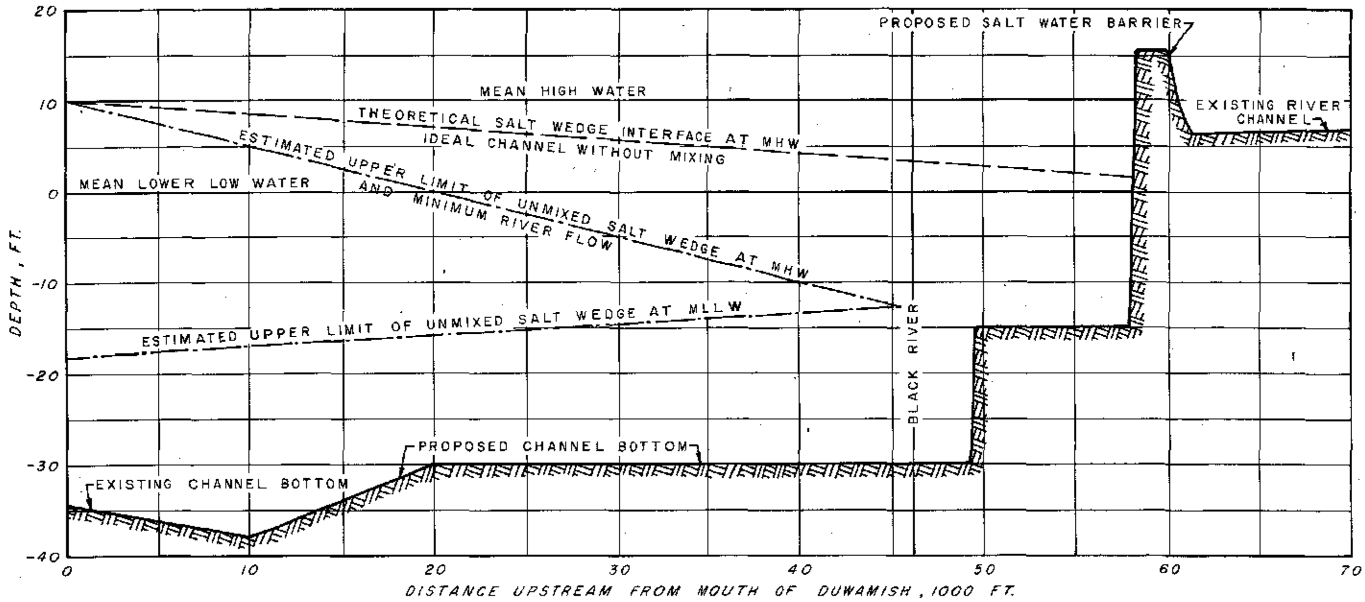


Fig. 12-7. Calculated Location of Salt Water Wedge in Proposed Duwamish Estuary

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).*

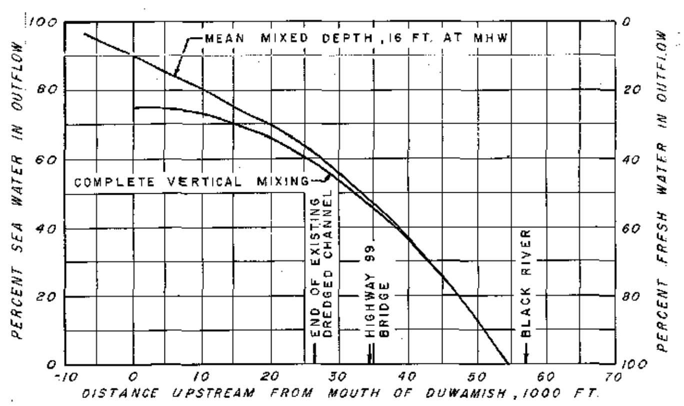


Fig. 12-8. Calculated Salinity in Existing Duwamish Estuary

Based on upland fresh water flow of 200 cfs.

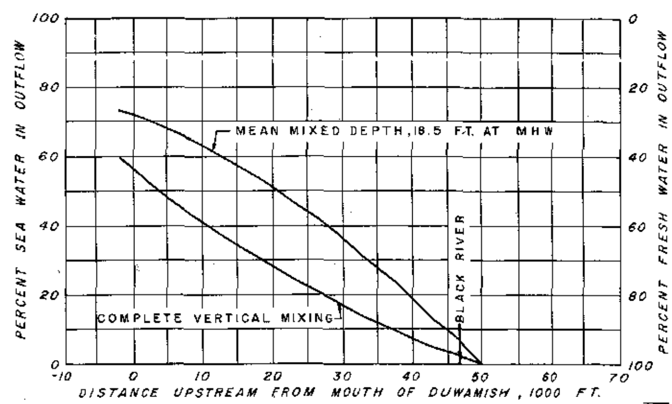


Fig. 12-9. Calculated Salinity in Proposed Duwamish Estuary

Based on upland fresh water flow of 400 cfs.

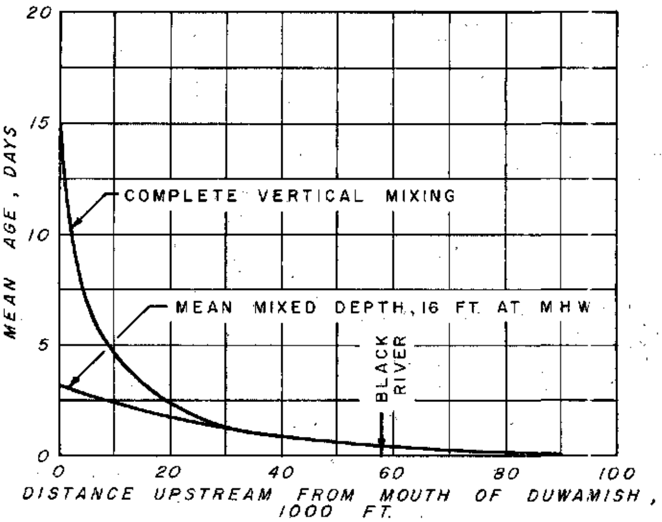


Fig. 12-10. Mean Age of Fresh Water in Existing Duwamish Estuary during Passage Downstream

Based on upland fresh water flow of 200 cfs.

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 dur-

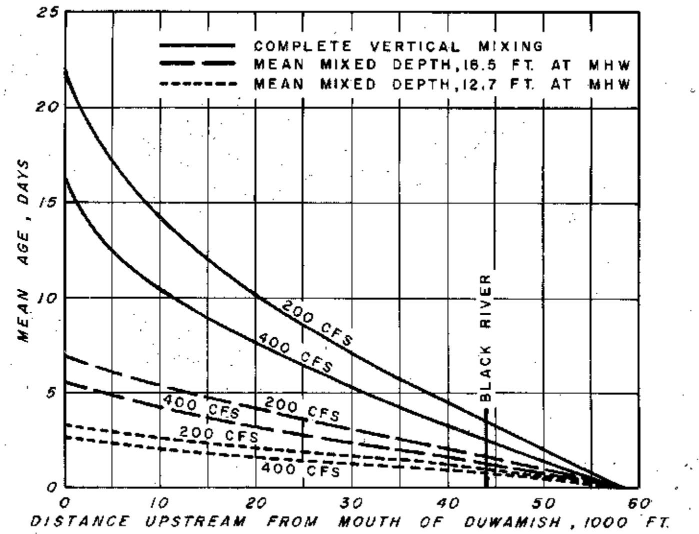


Fig. 12-11. Mean Age of Fresh Water in Proposed Duwamish Estuary during Passage Downstream

The figure shows the influence of mixing depth and fresh water flow on flushing time.

ing 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.

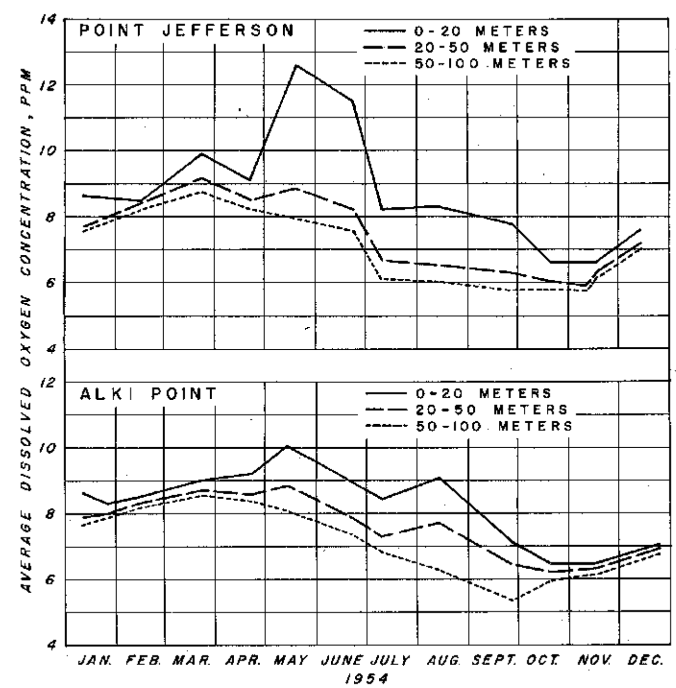


Fig. 12-12. Dissolved Oxygen Concentrations in Puget Sound

Source: Department of Oceanography, University of Washington, "Physical and Chemical Data, Puget Sound and Approaches", Technical Report No. 46, January to December, 1954.

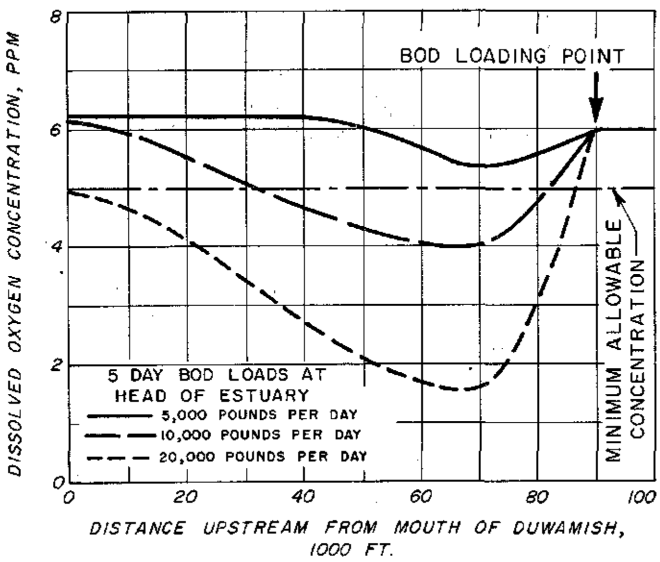


Fig. 12-13. Dissolved Oxygen Concentrations in Existing Duwamish Estuary for Various BOD Loads Applied at Head of Estuary

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

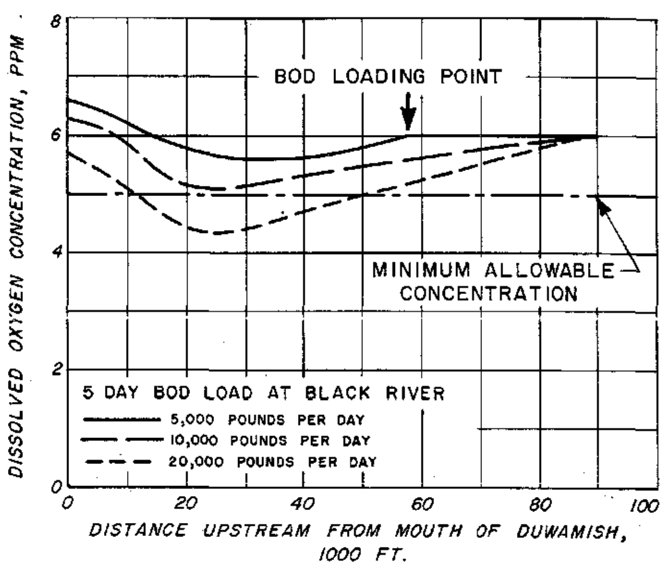


Fig. 12-14. Dissolved Oxygen Concentrations in Existing Duwamish Estuary for Various BOD Loads Applied at Black River

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.*

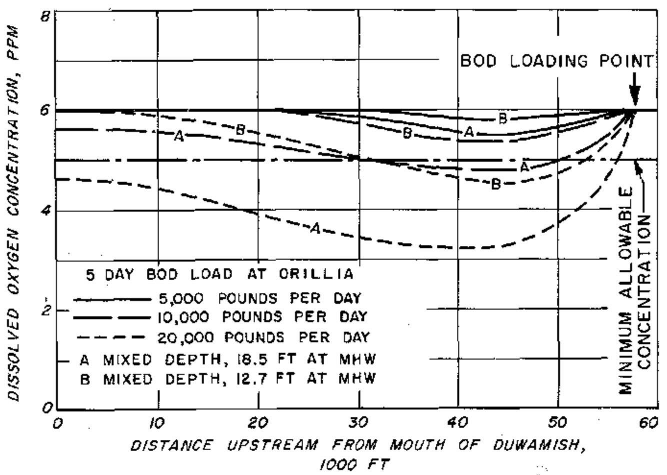


Fig. 12-15. Dissolved Oxygen Concentrations in Existing Duwamish Estuary for Various BOD Loads Applied at Orillia

Based on minimum fresh water flow of 400 cfs and initial dissolved oxygen concentration of 6.0 ppm in both salt and fresh waters.

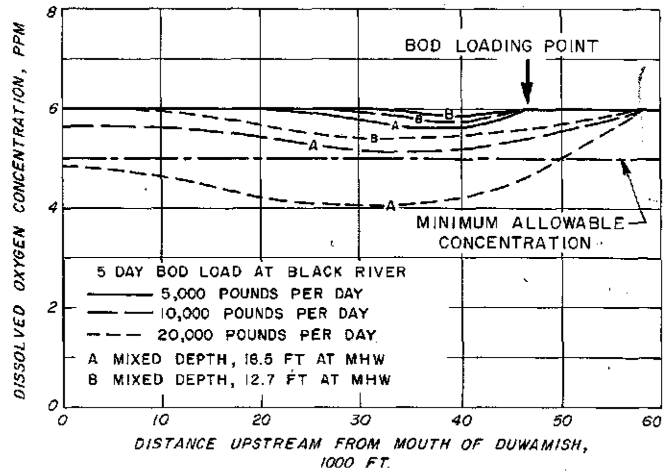


Fig. 12-16. Dissolved Oxygen Concentrations in Proposed Duwamish Estuary for Various BOD Loads Applied at Black River

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-

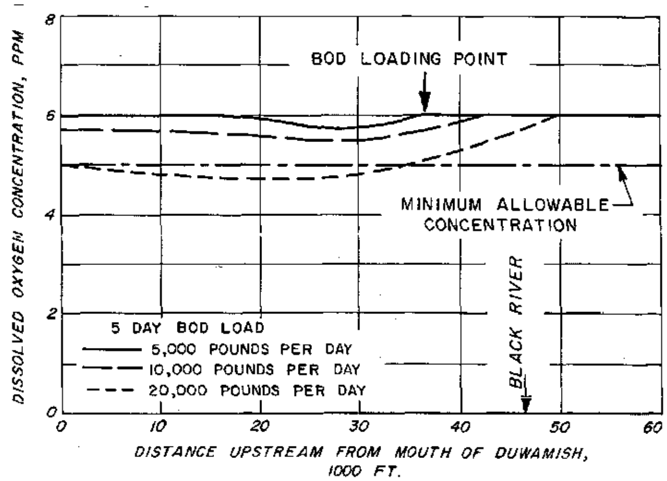


Fig. 12-17. Dissolved Oxygen Concentrations in Proposed Duwamish Estuary for Various BOD Loads Applied Downstream from Black River

Based on minimum fresh water flow of 400cfs, 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.

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.

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.

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

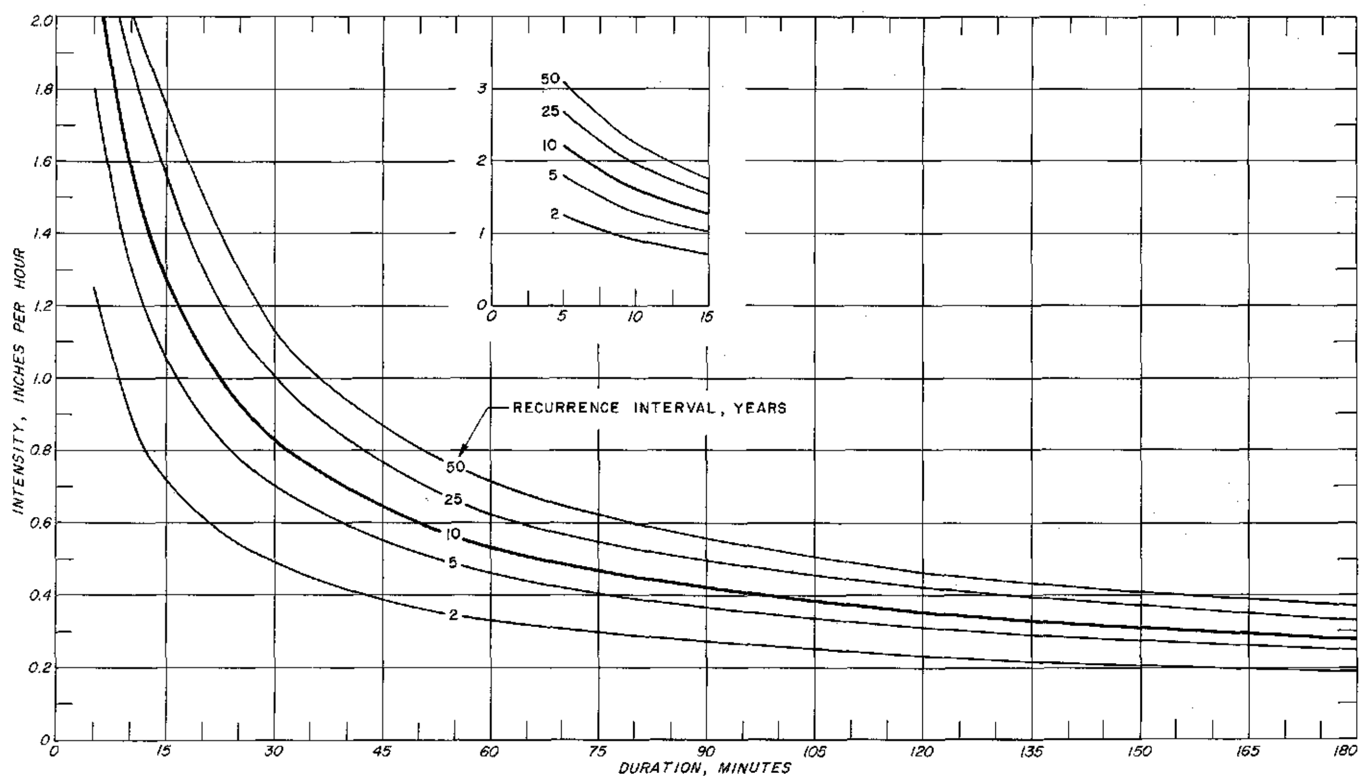


Fig. 13-1. Rainfall Frequency-Intensity-Duration for Seattle, January through December

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

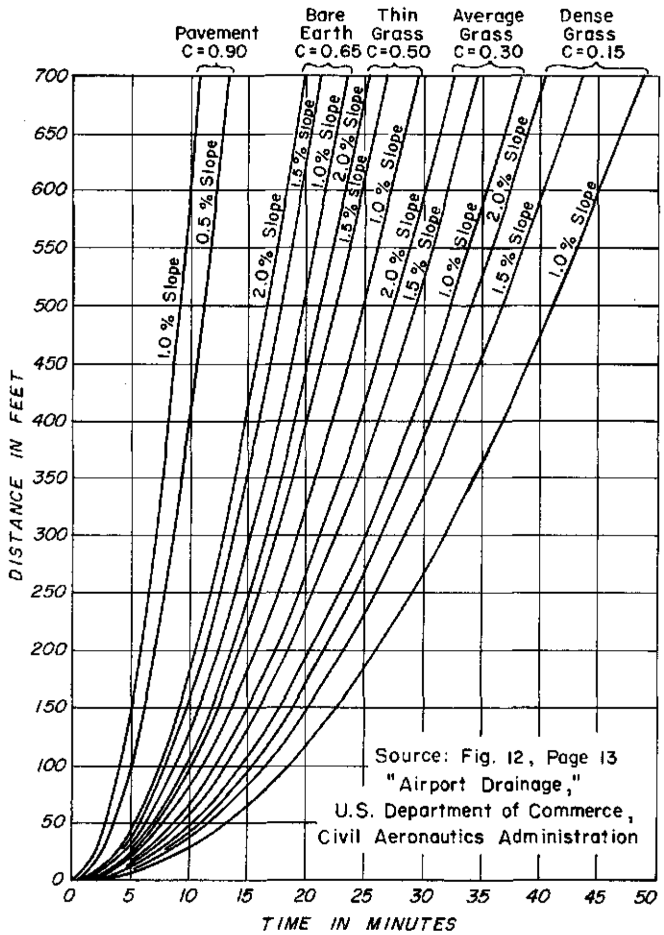


Fig. 13-2. Storm Water Inlet Time

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

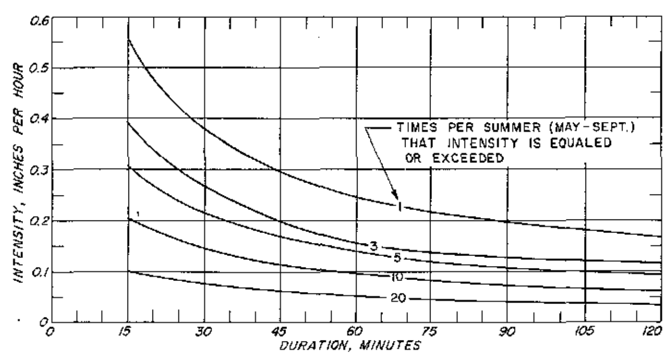


Fig. 13-3. Summer Rainfall Frequency-Intensity-Duration for Seattle, May through September

Based on analysis of summer rainfall intensities during the period 1940-1949, inclusive (Fig. 3-17).

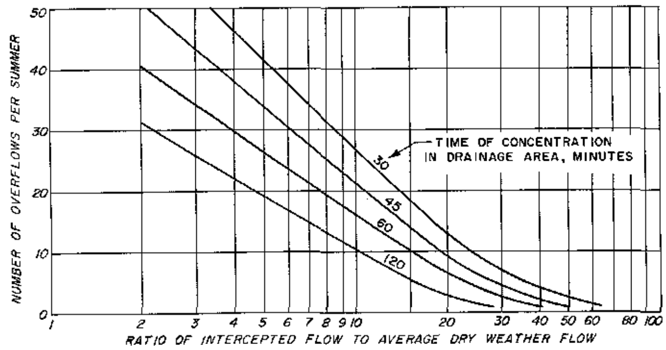


Fig. 13-4. Effect of Interceptor Capacity on Frequency of Overflow from Combined Sewers

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).*

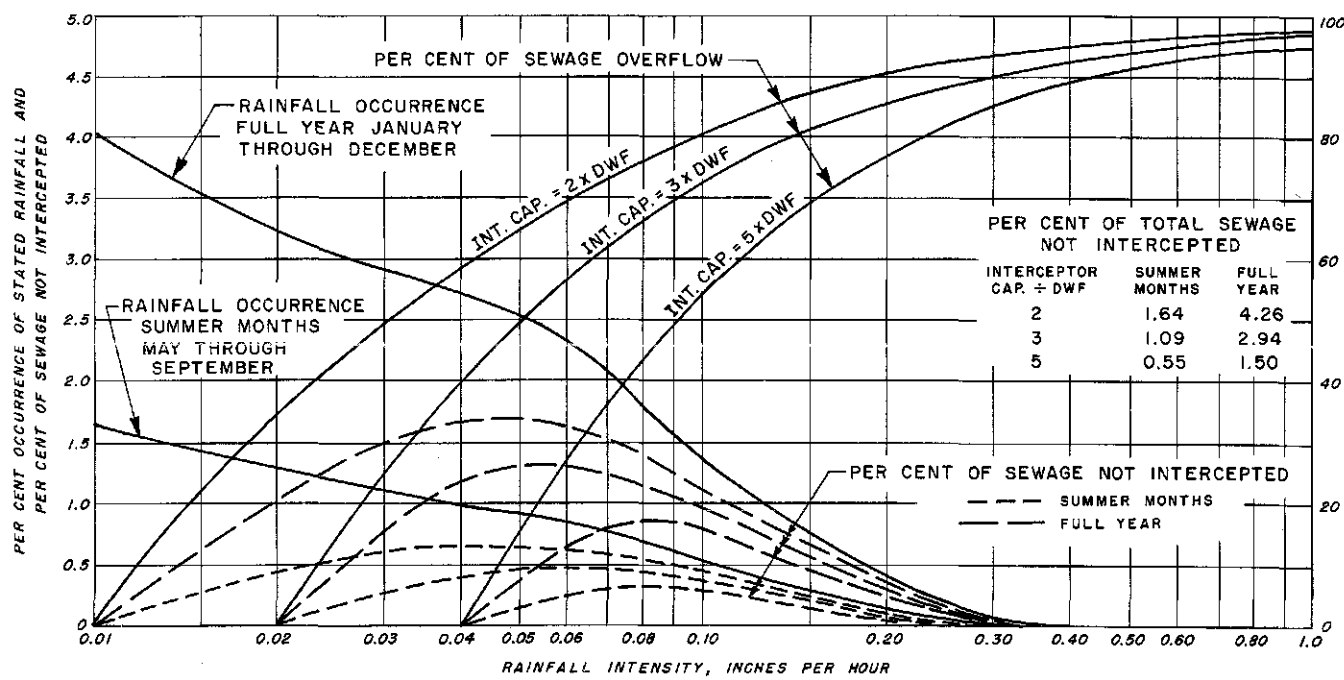


Fig. 13-5. Effect of Interceptor Capacity on Quantity of Sewage Not Intercepted from Combined Systems

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).

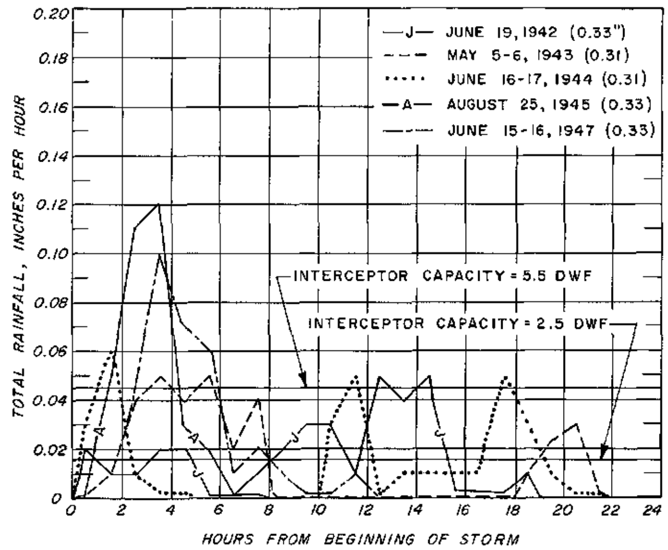


Fig. 13-6. Analyses of Typical Storms in the Six Times per Summer Group

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

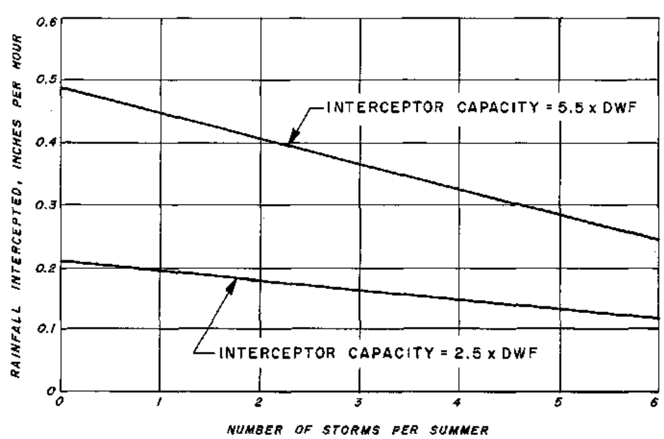


Fig. 13-7. Effect of Interceptor Capacity on Average Quantity of Rainfall Intercepted for Typical Summer Storm

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.

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

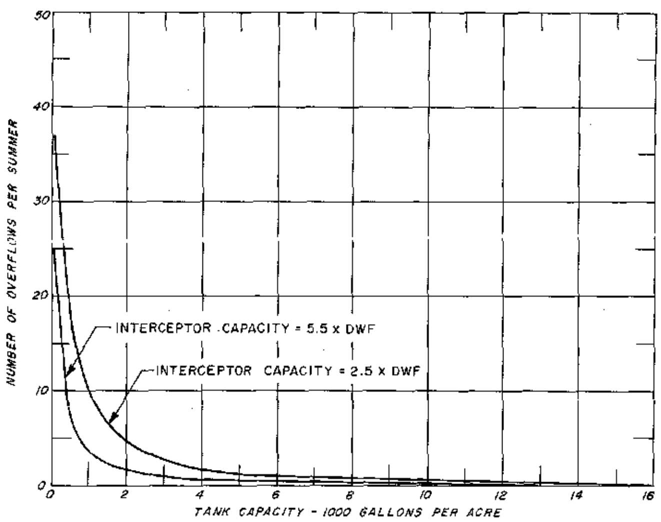


Fig. 13-8. Effect of Interceptor Capacity on Capacity of Storm Water Holding Tanks for Summer Storms of Various Frequencies

Based on Fig. 13-7.

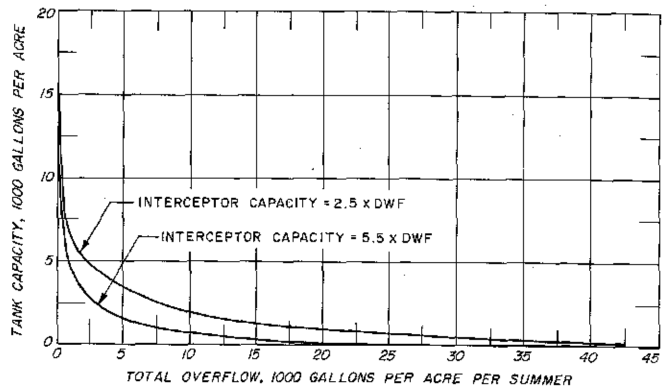


Fig. 13-9. Effect of Interceptor Capacity on Quantity of Overflow from Storm Water Holding Tanks

Based on Fig. 13-7

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.

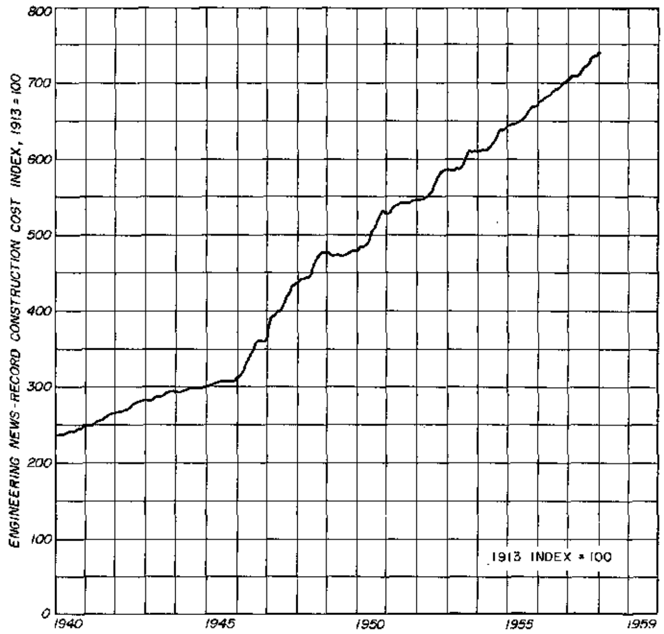


Fig. 13-10. Engineering News-Record Construction Cost Index

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

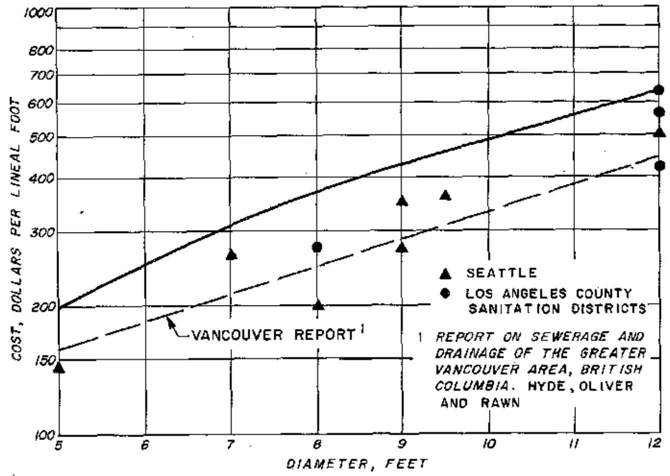


Fig. 13-11. Estimated Unit Costs of Tunnels

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

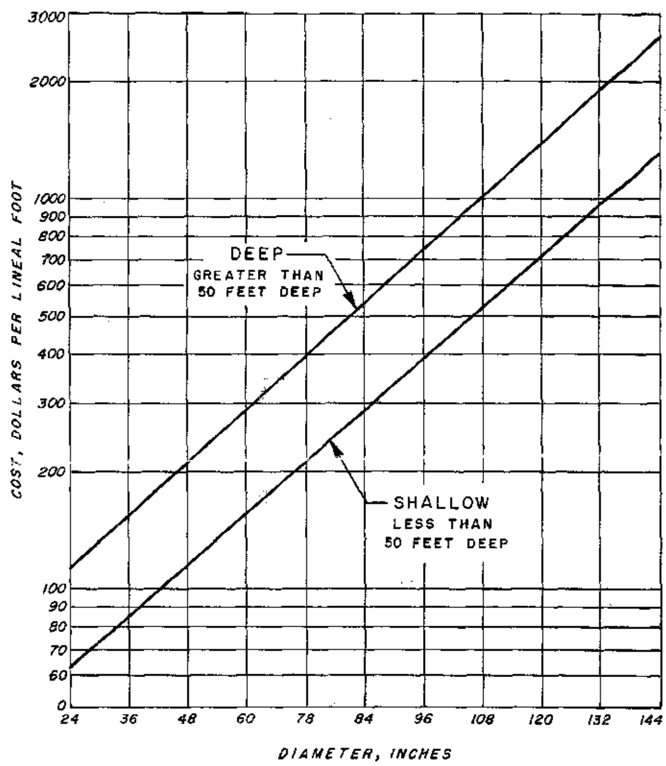


Fig. 13-12. Unit Costs of Submarine Outfalls

Based on analysis of costs of existing outfalls adjusted to ENR construction cost index of 800. Estimates do not include engineering and contingencies.

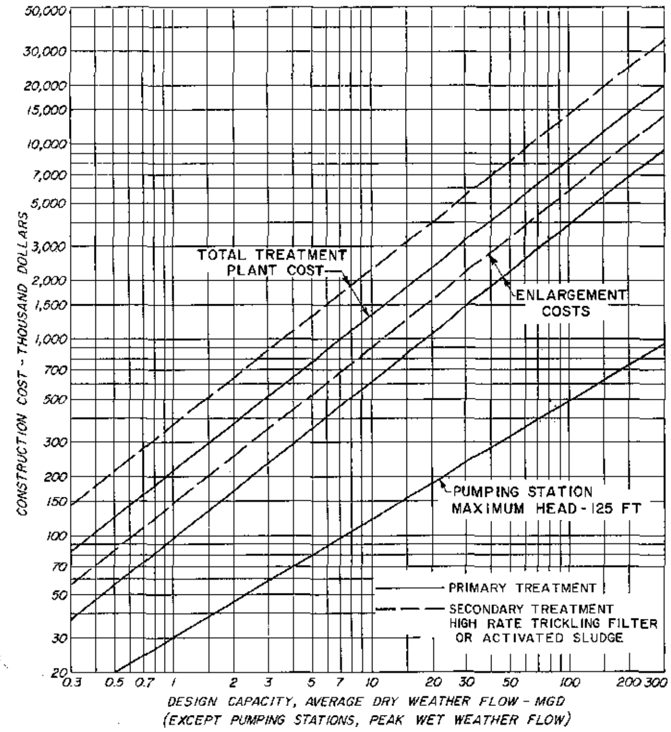


Fig. 13-13. Construction Costs of Sewage Treatment Plants and Pumping Stations

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 sewers 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.

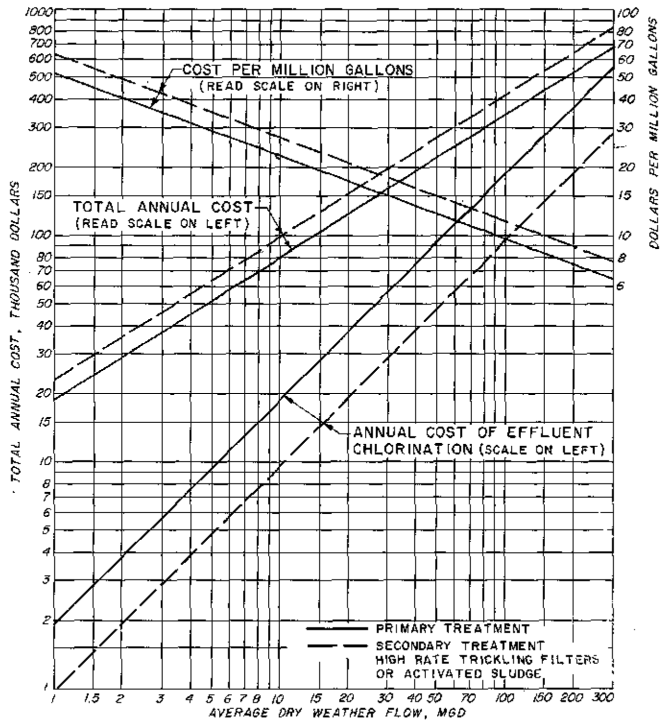


Fig. 13-14. Operating Costs of Sewage Treatment Plants

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.

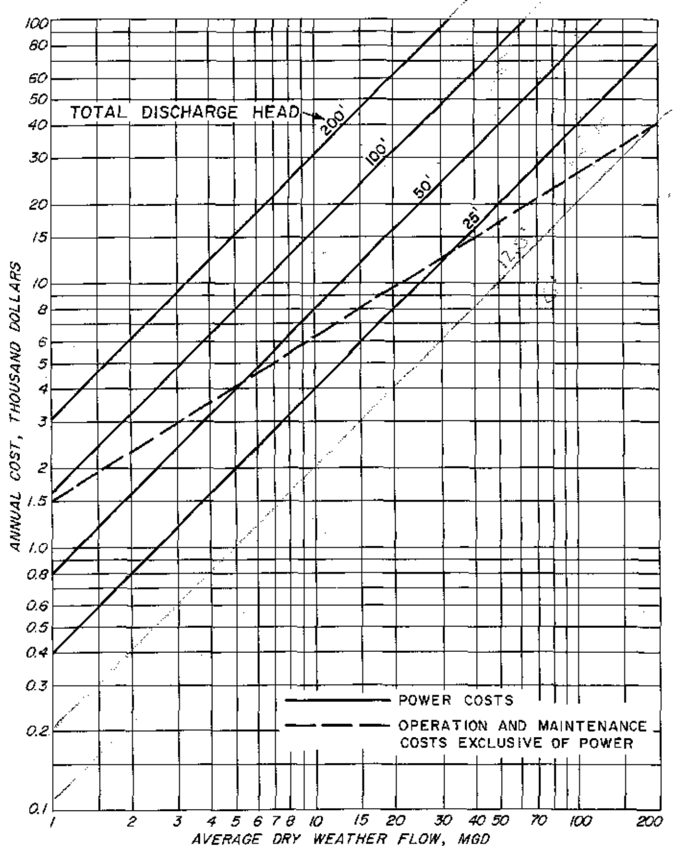


Fig. 13-15. Operating Costs of Sewage Pumping Stations

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.

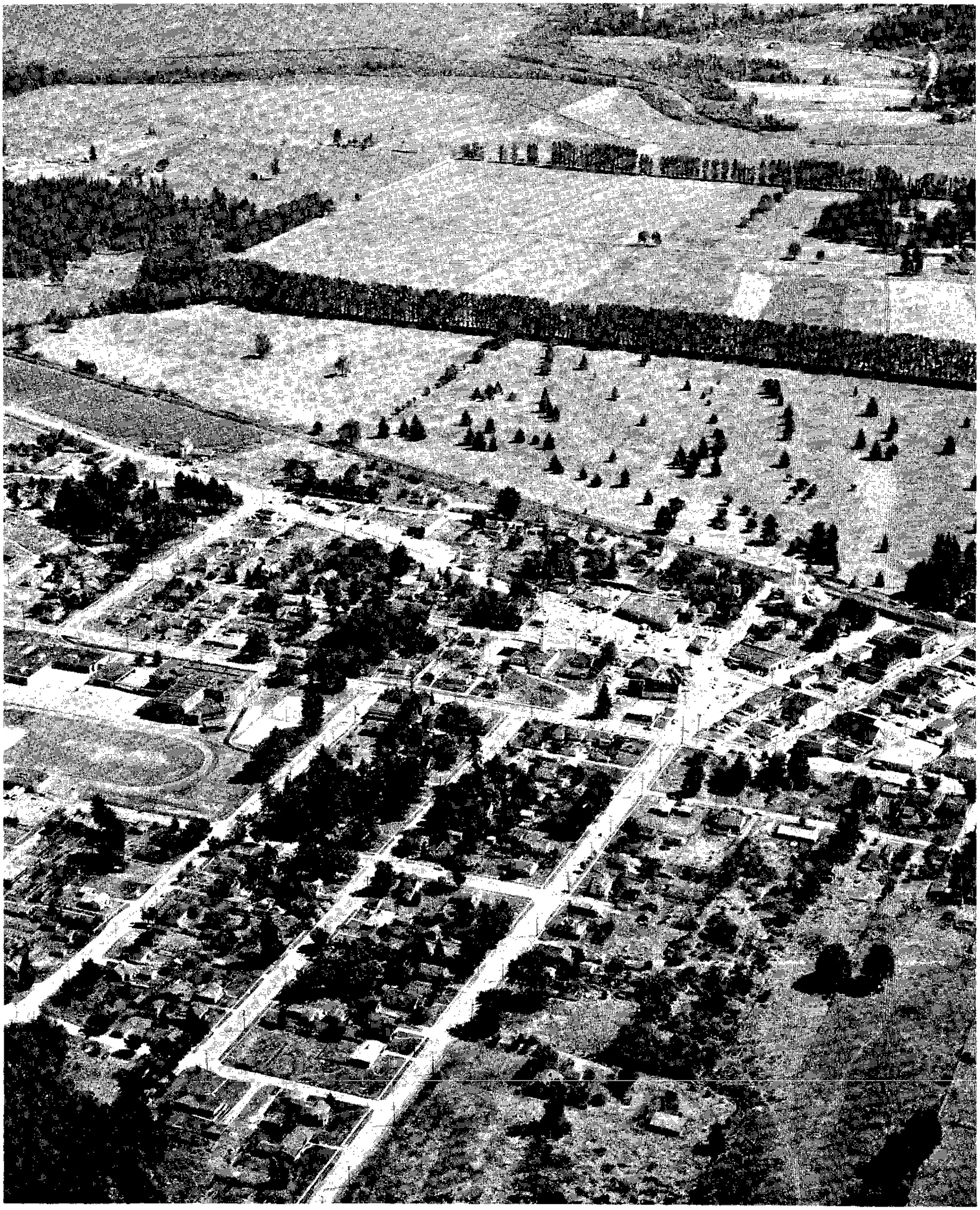
As set forth in Chapter 3, the study area consists of four primary watersheds, namely, Lake Washington, Puget Sound, Green-Duwamish and Lake Union - Ship Canal. For the purposes of sewerage planning, these watersheds are divided into twelve major areas, the divisions conforming generally to natural topographic 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.

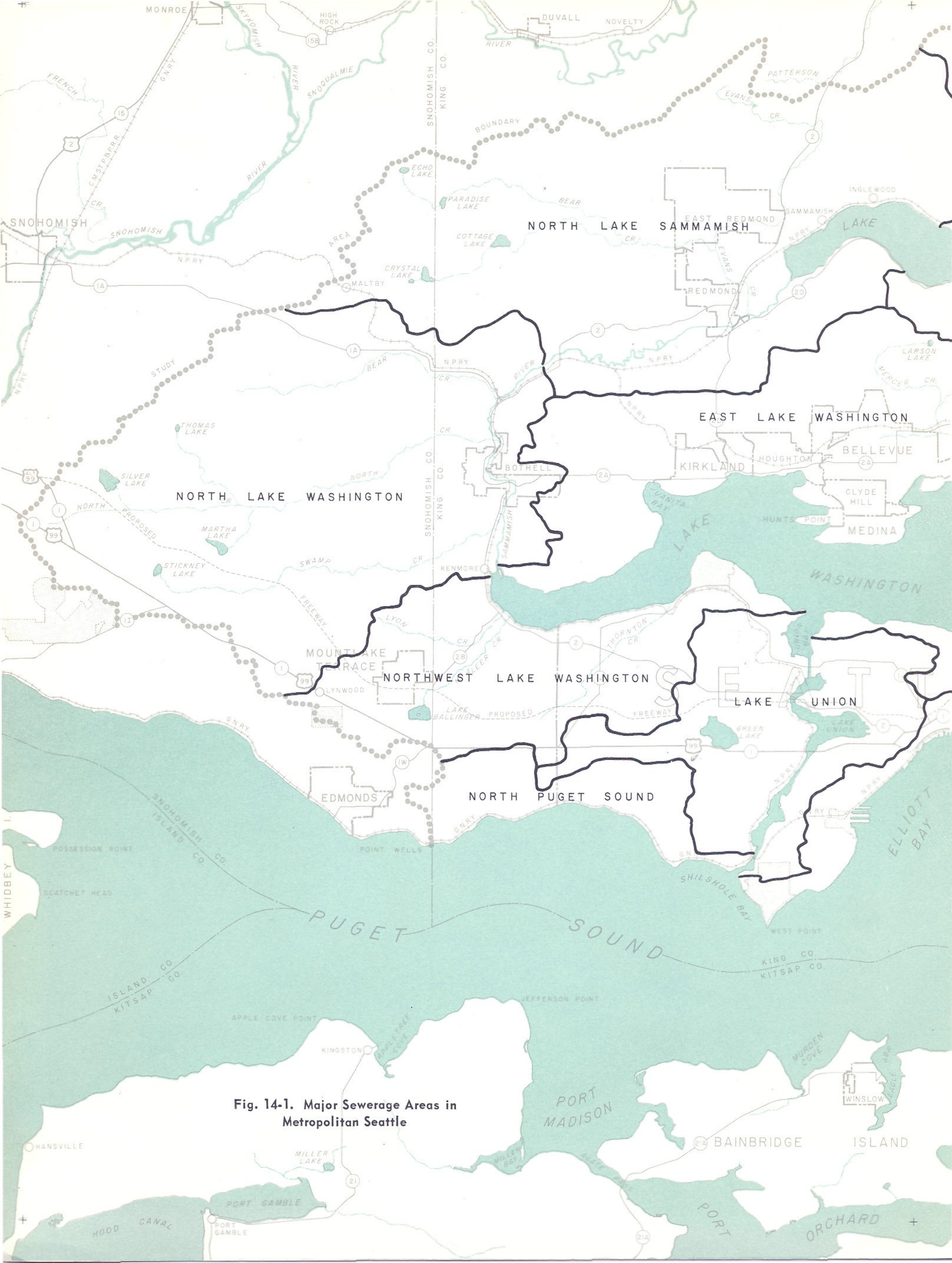
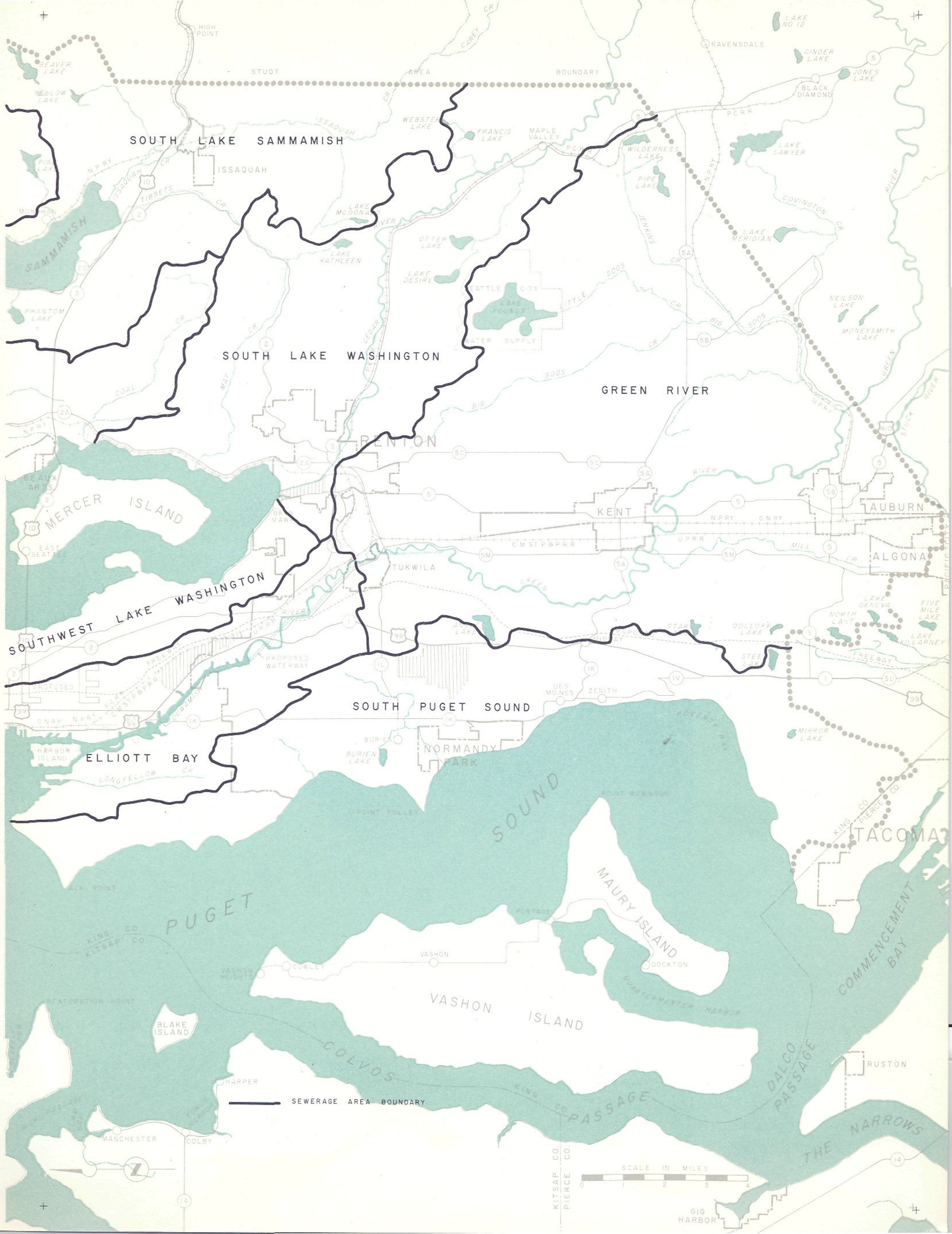


Fig. 14-1. Major Sewerage Areas in Metropolitan Seattle



SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
ELLIOTT BAY
SOUTH PUGET SOUND
STUDY AREA BOUNDARY
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
MERCER ISLAND
NORMANDY PARK
TACOMA
RUSTON
VASHON ISLAND
MAURY ISLAND
PUGET SOUND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
SEWERAGE AREA BOUNDARY
SCALE IN MILES
0 1 2 3 4

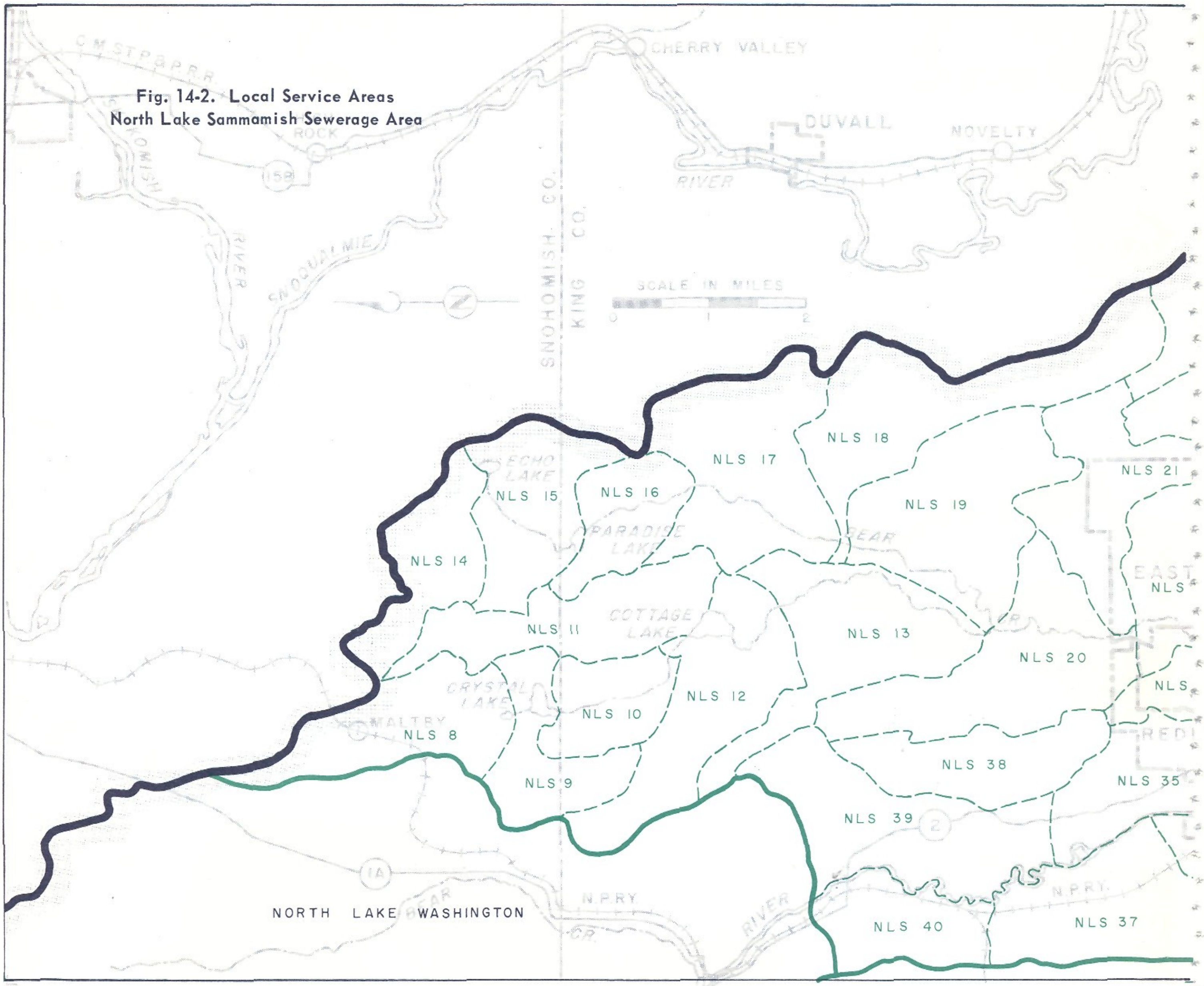


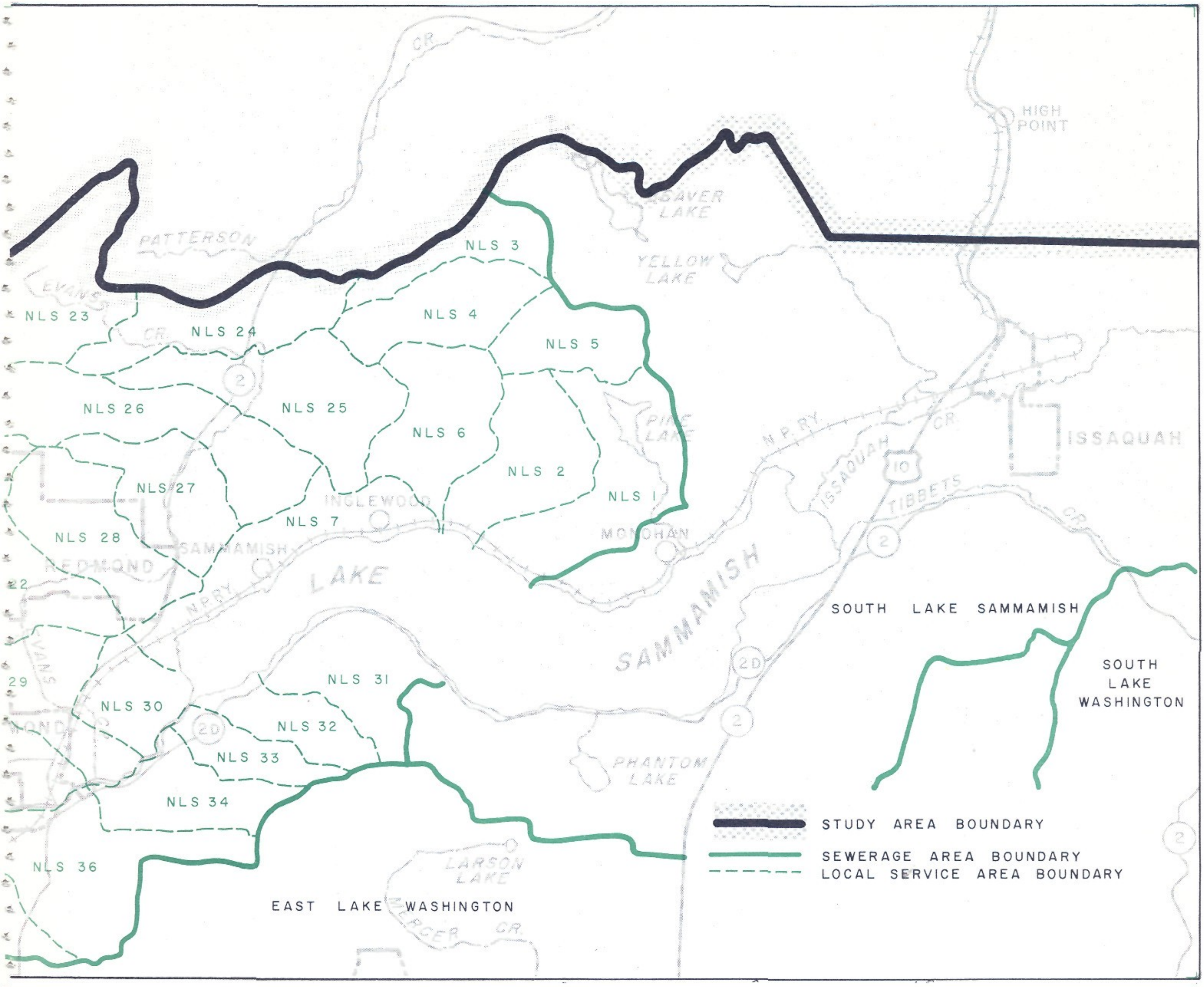
Fig. 14-2. Local Service Areas North Lake Sammamish Sewerage Area

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

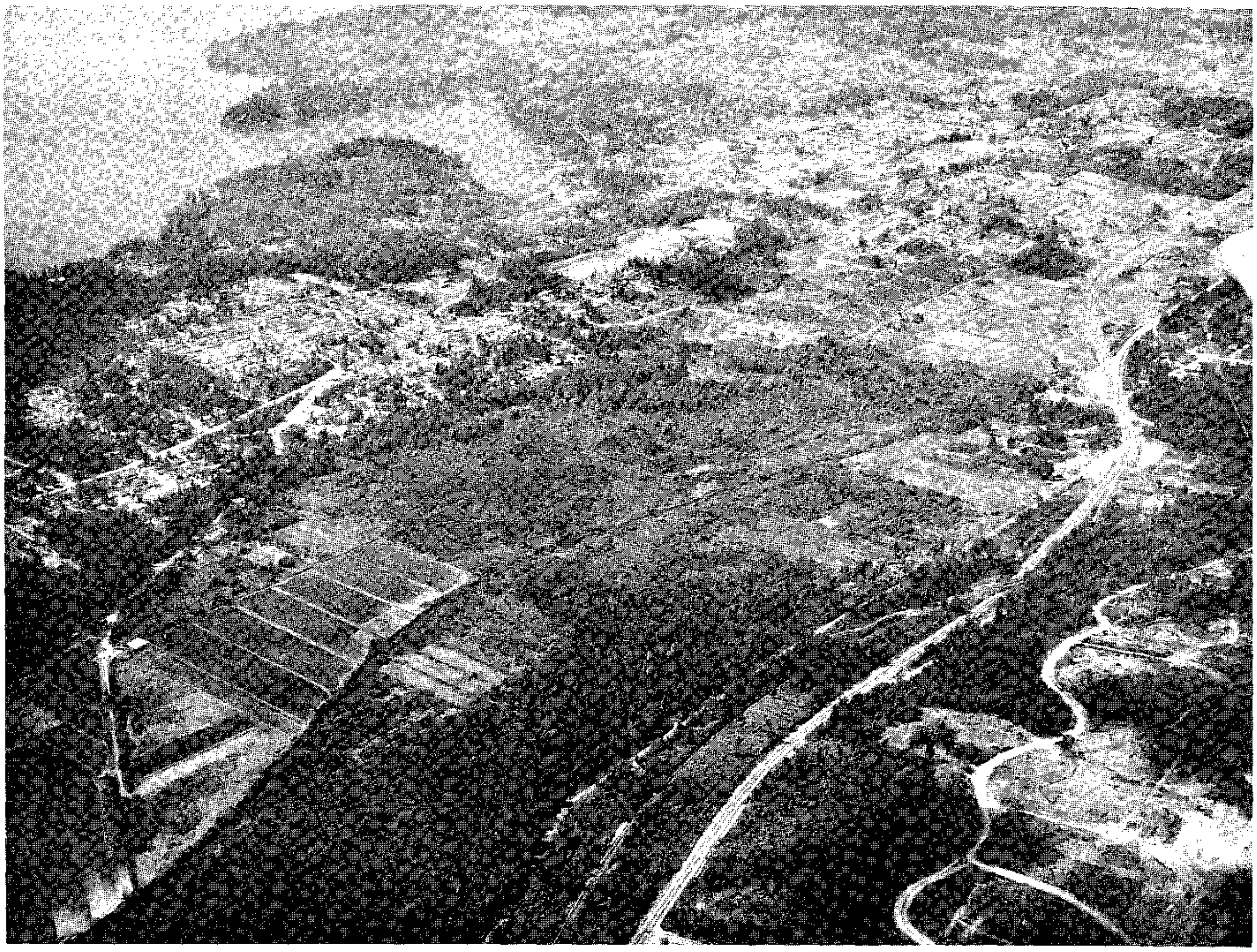
See Fig. 14-2 for location.

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.

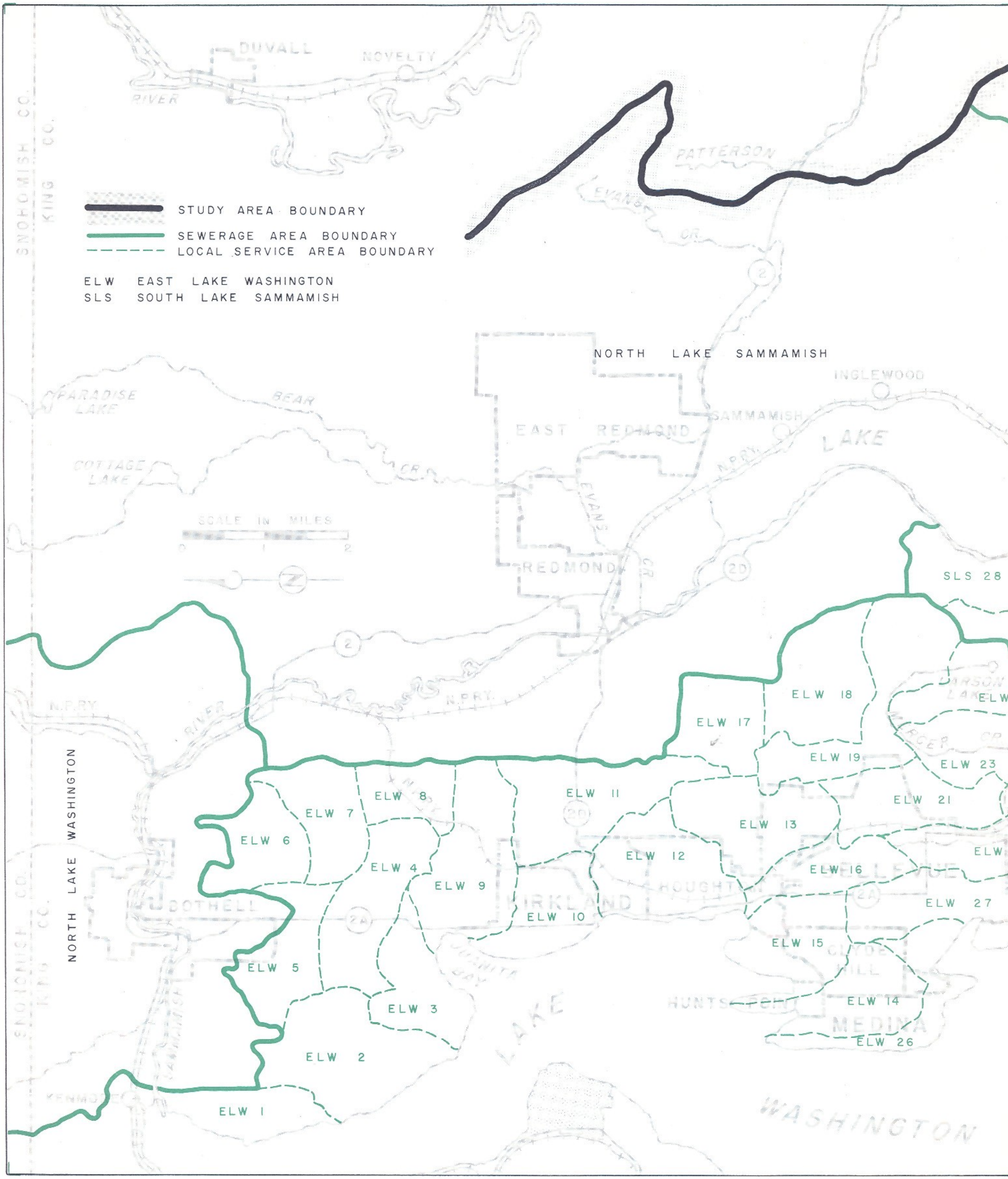
quired. In any event, flows can be transported from the area either southward along the railroad right-of-way to the South Lake Washington sewerage area, or westward across Lake Washington to the Southwest Lake Washington sewerage area.

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
SCALE IN MILES
0 1 2
DUVALL
NOVELTY
RIVER
PATTERSON
EVANS CR.
PARADISE LAKE
BEAR CR.
COTTAGE LAKE
EAST REDMOND
REDMOND
INGLEWOOD
SAMMAMISH
LAKE
N.P.RY.
SNOHOMISH CO.
KING CO.
BOTHELL
KIRKLAND
HOUGHTON
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
KENMORE
LAKE WASHINGTON
ELW 1
ELW 2
ELW 3
ELW 4
ELW 5
ELW 6
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ELW 10
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ELW 15
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ELW 27
SLS 28
LARSON LAKE
MERCER CR.

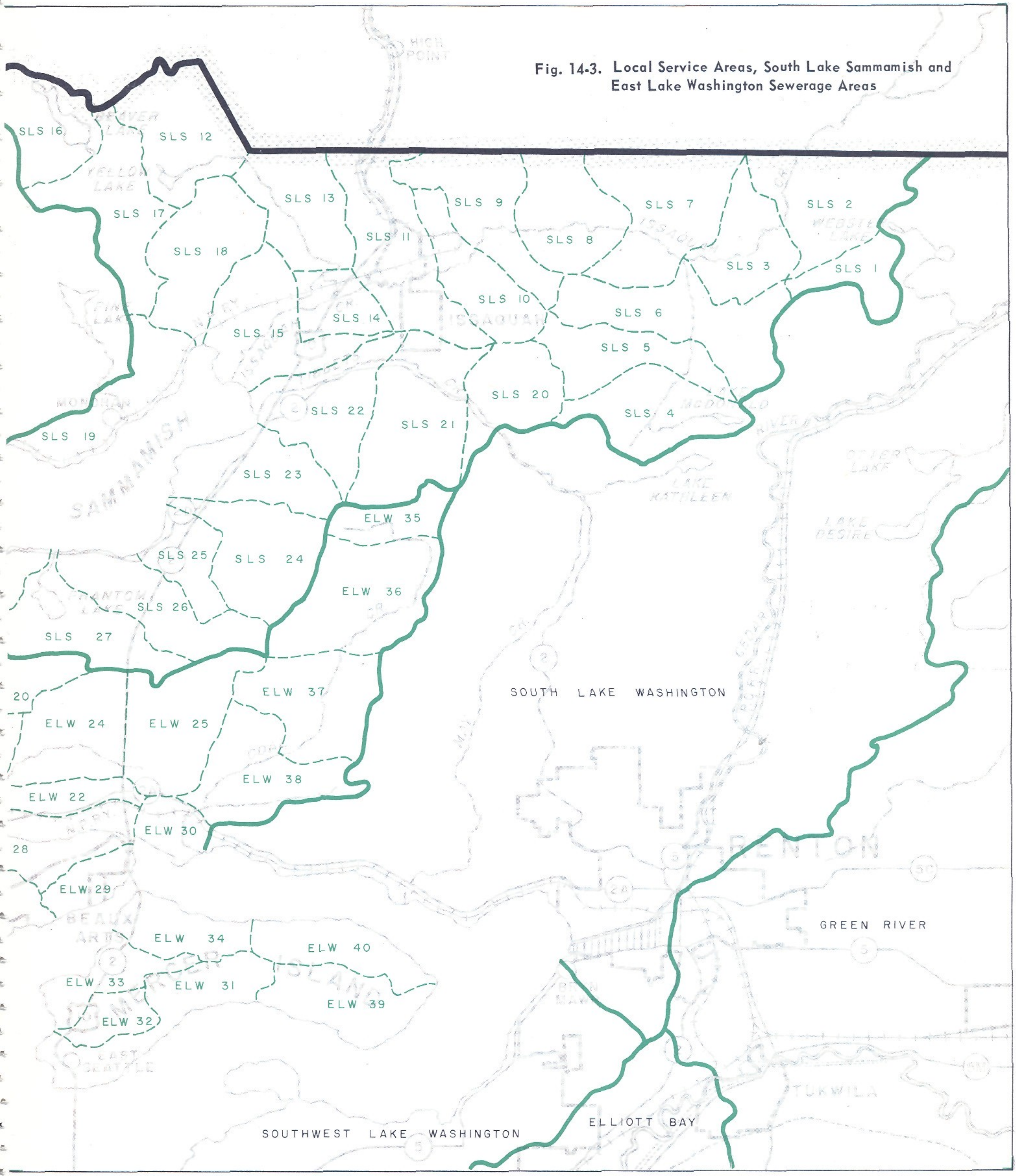


Fig. 14-3. Local Service Areas, South Lake Sammamish and East Lake Washington Sewerage Areas

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).

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.

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, 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

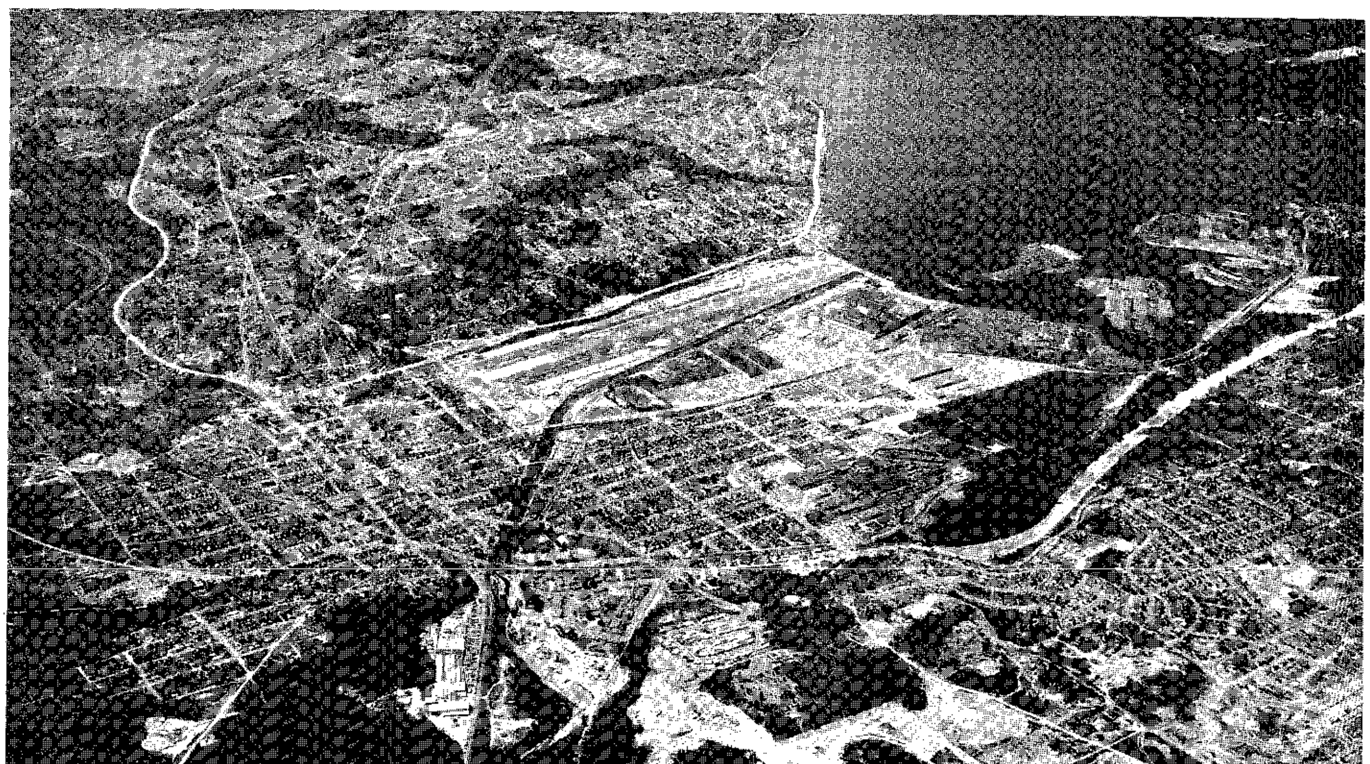
This area contains 34,230 acres and includes all of the Cedar River watershed within the metropolitan

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.

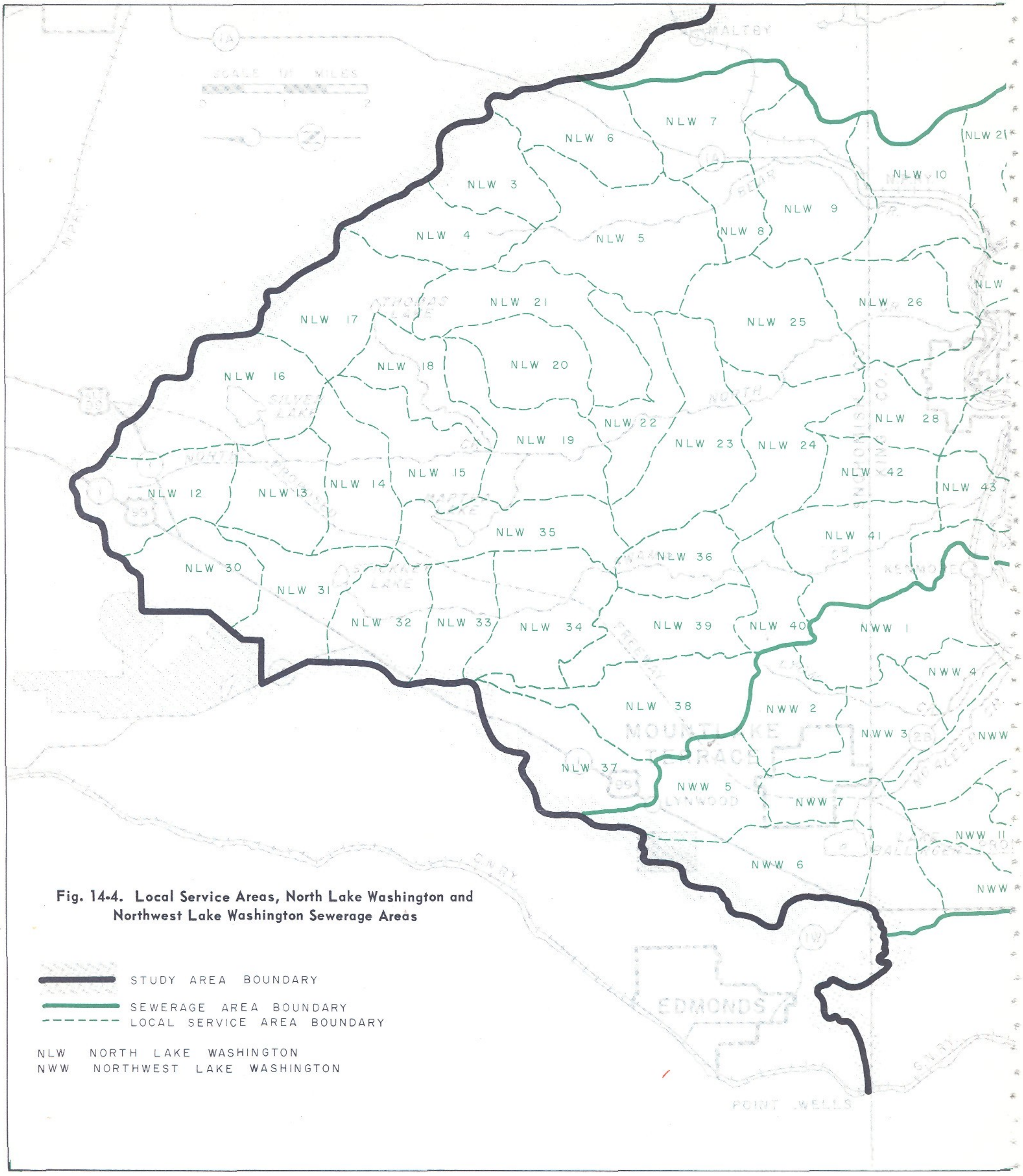
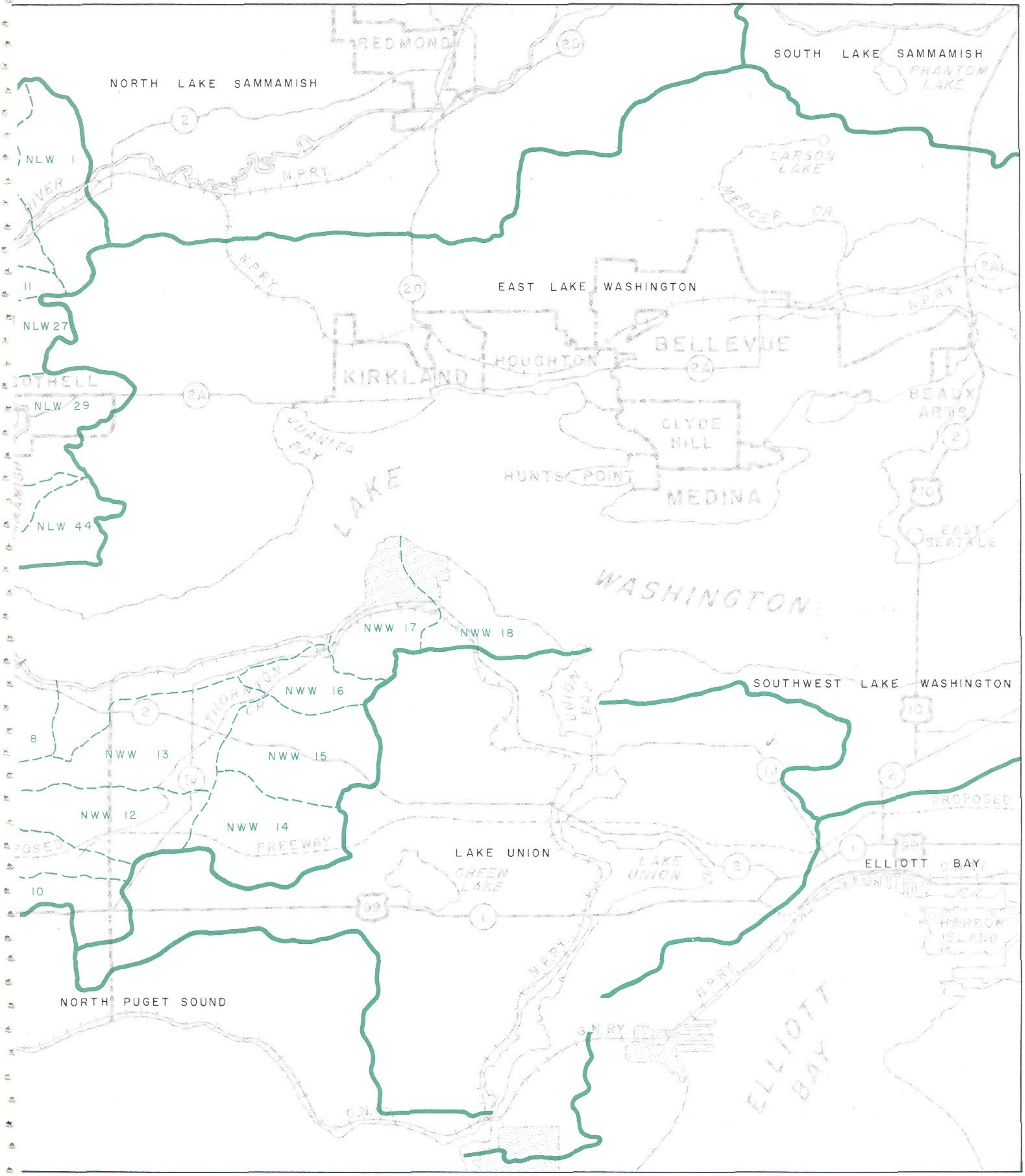


Fig. 14-4. Local Service Areas, North Lake Washington and Northwest Lake Washington Sewerage Areas



REDMOND
NORTH LAKE SAMMAMISH
SOUTH LAKE SAMMAMISH
PHANTOM LAKE
NLW 1
N.P.RY.
LARSON LAKE
MERCER CR.
EAST LAKE WASHINGTON
NLW 27
KIRKLAND
HOUGHTON
BELLEVUE
NLW 29
JUANITA BAY
CLYDE HILL
BEAUX ARTS
HUNTS POINT
MEDINA
LAKE
NLW 44
EAST SEATTLE
WASHINGTON
NWW 17
NWW 18
NWW 16
SOUTHWEST LAKE WASHINGTON
THORNTON CR.
UNION BAY
NWW 13
NWW 15
NWW 12
NWW 14
FREEWAY
PROPOSED
LAKE UNION
GREEN LAKE
LAKE UNION
ELLIOTT BAY
10
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

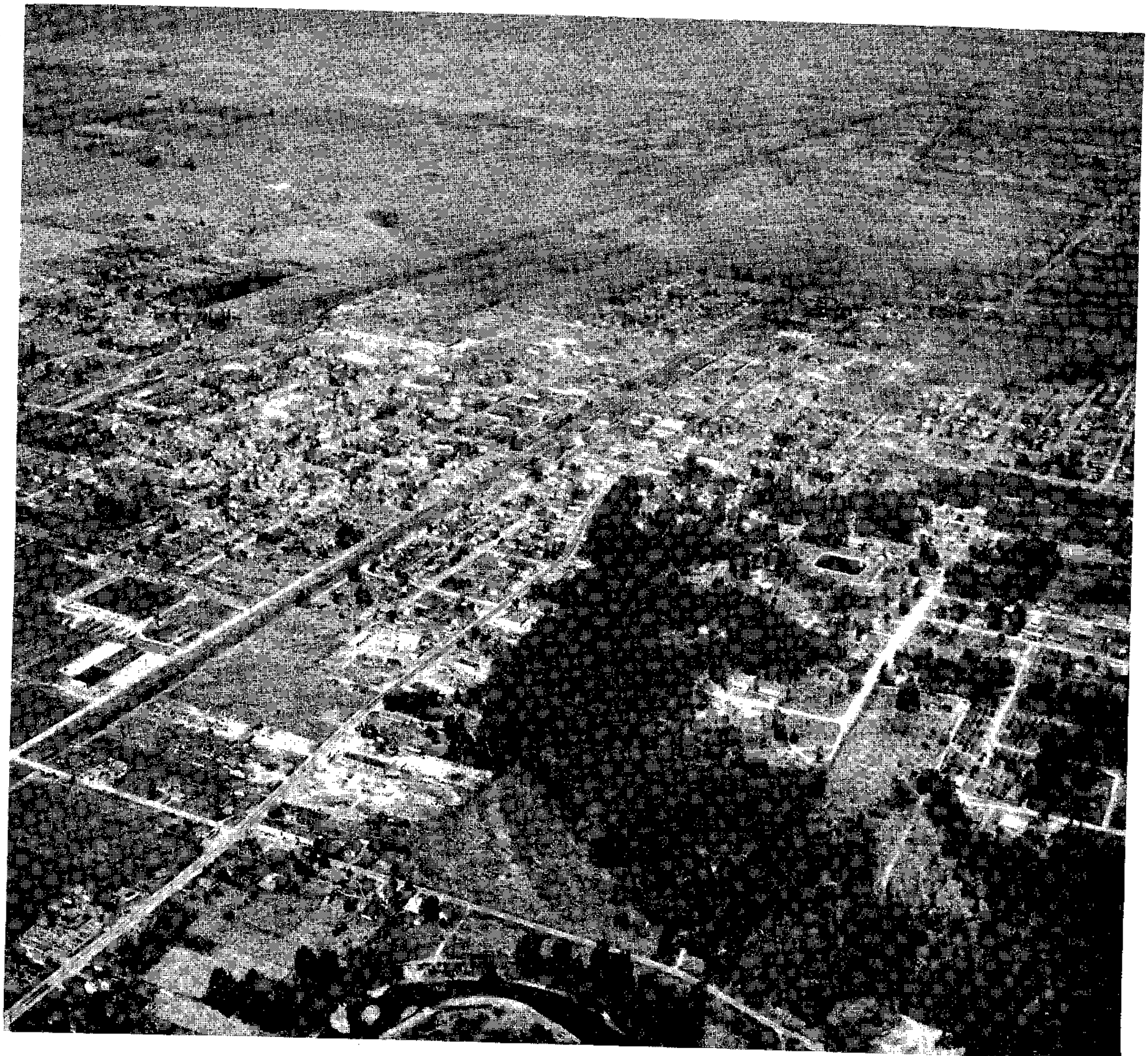
Area designation	Area in acres	Area designation	Area in acres
GR-1	1,020	GR-37	920
GR-2	1,900	GR-38	1,520
GR-3	1,460	GR-39	1,100
GR-4	2,760	GR-40	760
GR-5	1,310	GR-41	910
GR-6	1,500	GR-42	1,000
GR-7	390	GR-43	640
GR-8	980	GR-44	560
GR-9	1,060	GR-45	1,130
GR-10	980	GR-46	720
GR-11	870	GR-47	1,080
GR-12	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).

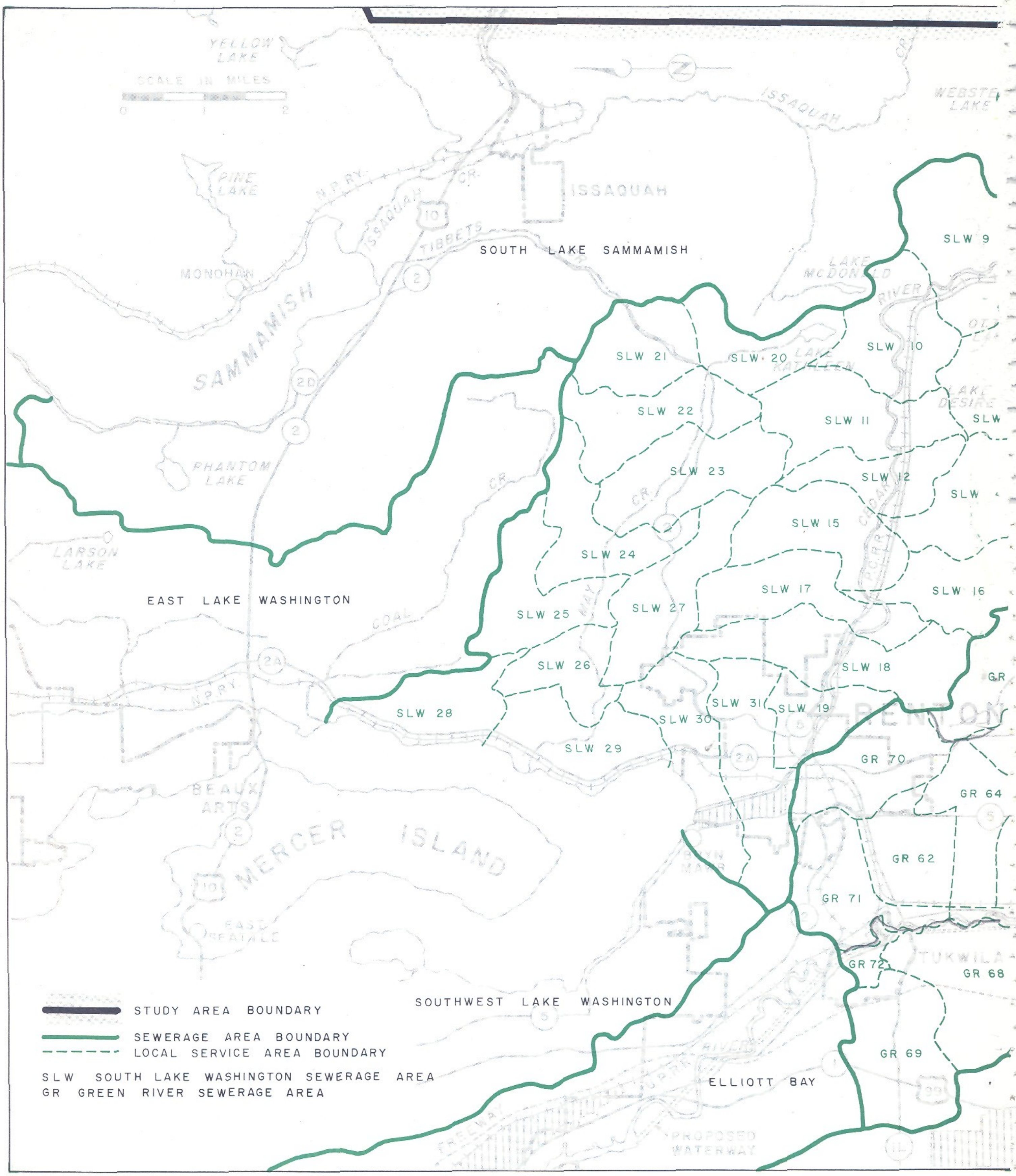
Southwest Lake Washington Sewerage Area

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
ISSAQUAH
WEBSTER LAKE
SOUTH LAKE SAMMAMISH
MONOHAN
SAMMAMISH
PHANTOM LAKE
LARSON LAKE
EAST LAKE WASHINGTON
LAKE McDONALD
LAKE KATHLEEN
LAKE DESIRE
SLW 9
SLW 10
SLW 11
SLW 12
SLW 15
SLW 16
SLW 17
SLW 18
SLW 19
SLW 20
SLW 21
SLW 22
SLW 23
SLW 24
SLW 25
SLW 26
SLW 27
SLW 28
SLW 29
SLW 30
SLW 31
GR 62
GR 64
GR 68
GR 69
GR 70
GR 71
GR 72
RENTON
TUKWILA
BEAUX ARTS
MERCER ISLAND
EAST SEATTLE
BRYN MAWR
SOUTHWEST LAKE WASHINGTON
ELLIOTT BAY
PROPOSED WATERWAY
STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
SLW SOUTH LAKE WASHINGTON SEWERAGE AREA
GR GREEN RIVER SEWERAGE AREA

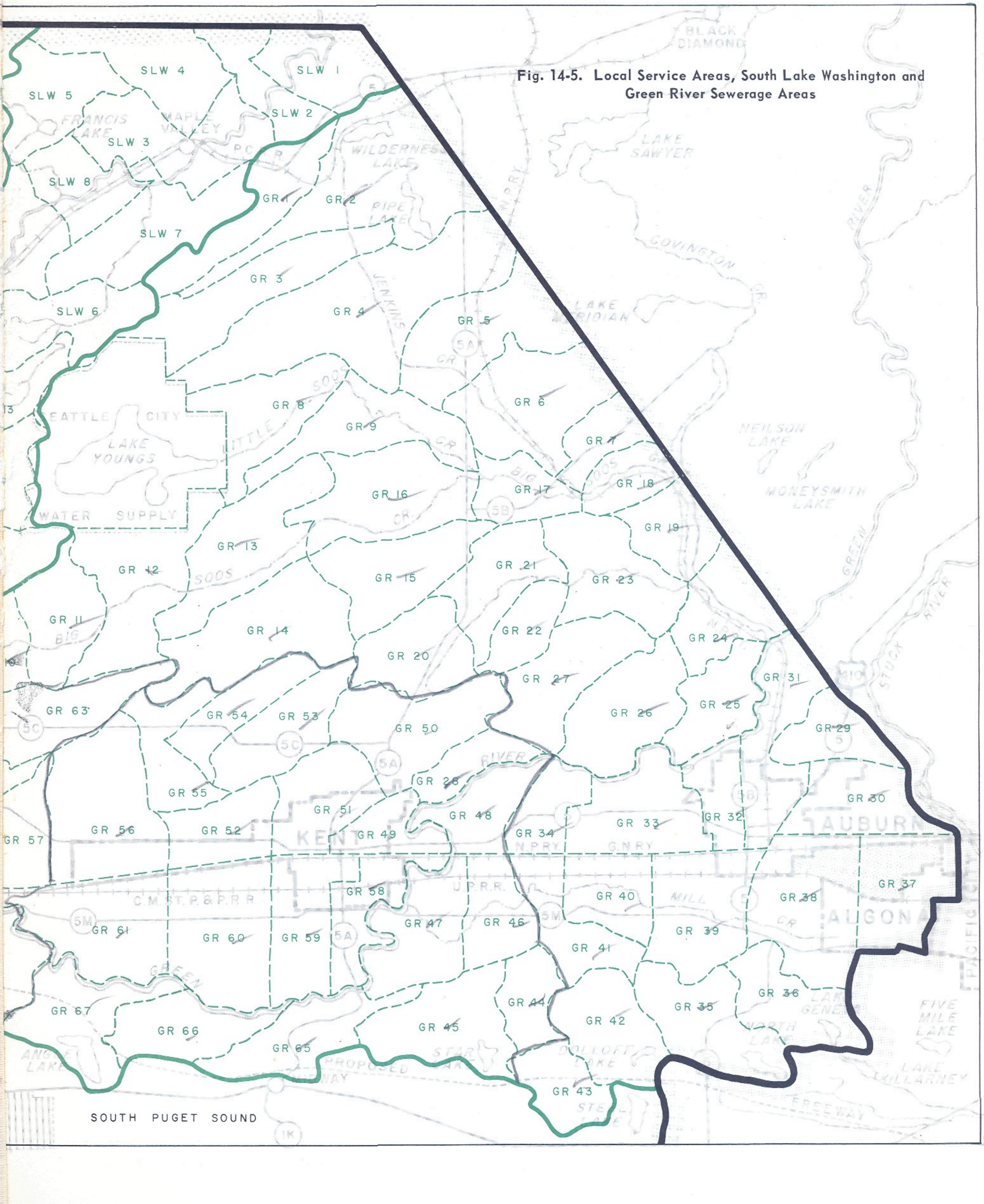


Fig. 14-5. Local Service Areas, South Lake Washington and Green River Sewerage Areas

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.

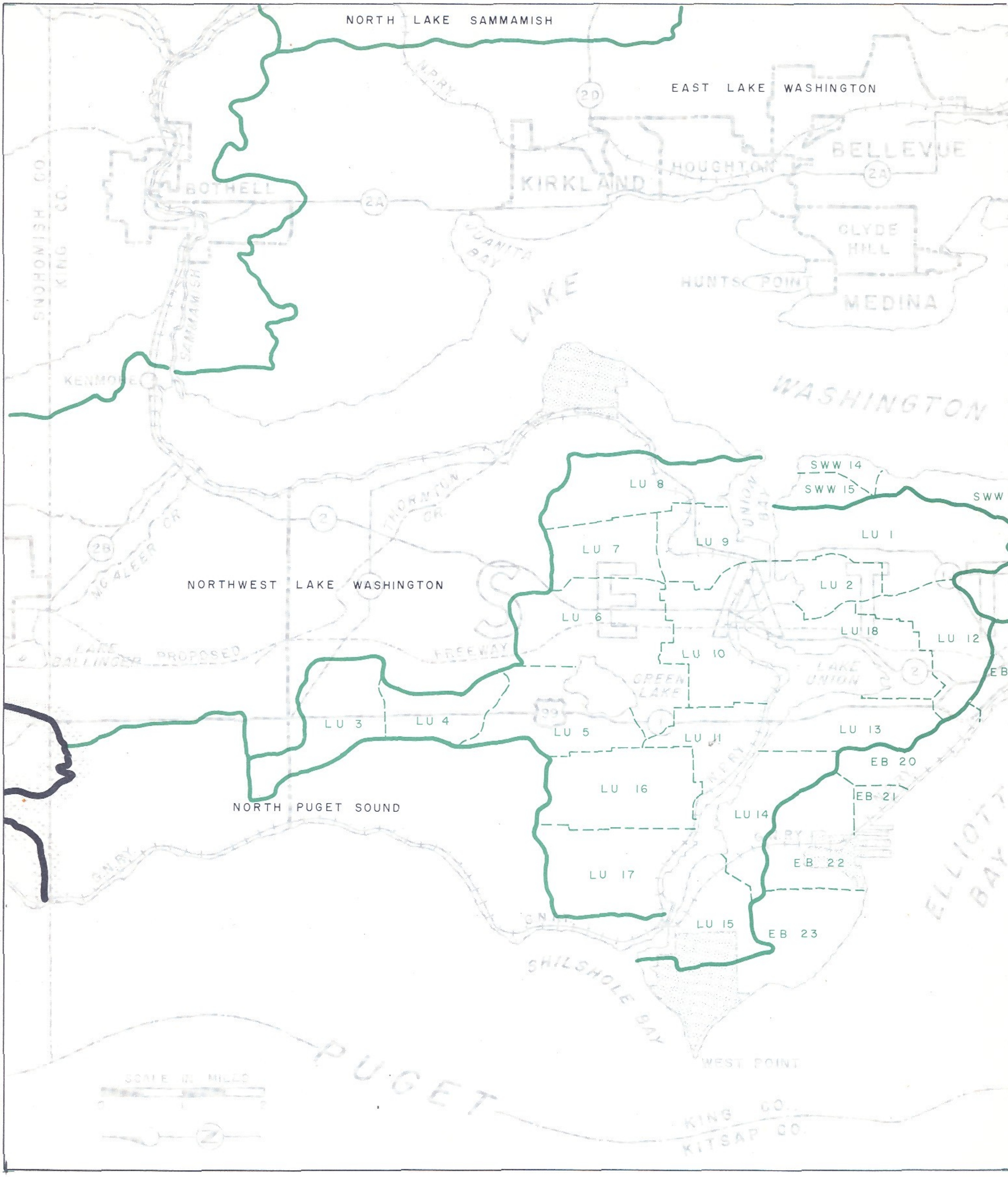
Drainage from the area is generally northward and southward to Union Bay, Lake Washington Ship Canal, Portage Bay, Lake Union and Salmon Bay Waterway. Lakefront interceptors will be necessary at Green Lake, a fresh water body of some 225 acres, lying in the northern portion of the area.

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
HUNTS POINT
MEDINA
SNOHOMISH CO.
KING CO.
JUANITA BAY
LAKE
WASHINGTON
SAMMAMISH
KENMORE
N.P.RY.
THORNTON CR
MC ALEER CR
NORTHWEST LAKE WASHINGTON
LAKE BALLINGER
PROPOSED FREEWAY
LU 8
LU 9
LU 7
LU 1
LU 2
LU 6
LU 18
LU 12
LU 10
GREEN LAKE
LAKE UNION
LU 3
LU 4
LU 5
LU 11
LU 13
LU 16
LU 17
LU 14
LU 15
SWW 14
SWW 15
SWW
UNION BAY
EB
EB 20
EB 21
EB 22
EB 23
NORTH PUGET SOUND
G.N.RY.
SHILSHOLE BAY
WEST POINT
ELLIOTT BAY
PUGET
KING CO.
KITSAP CO.

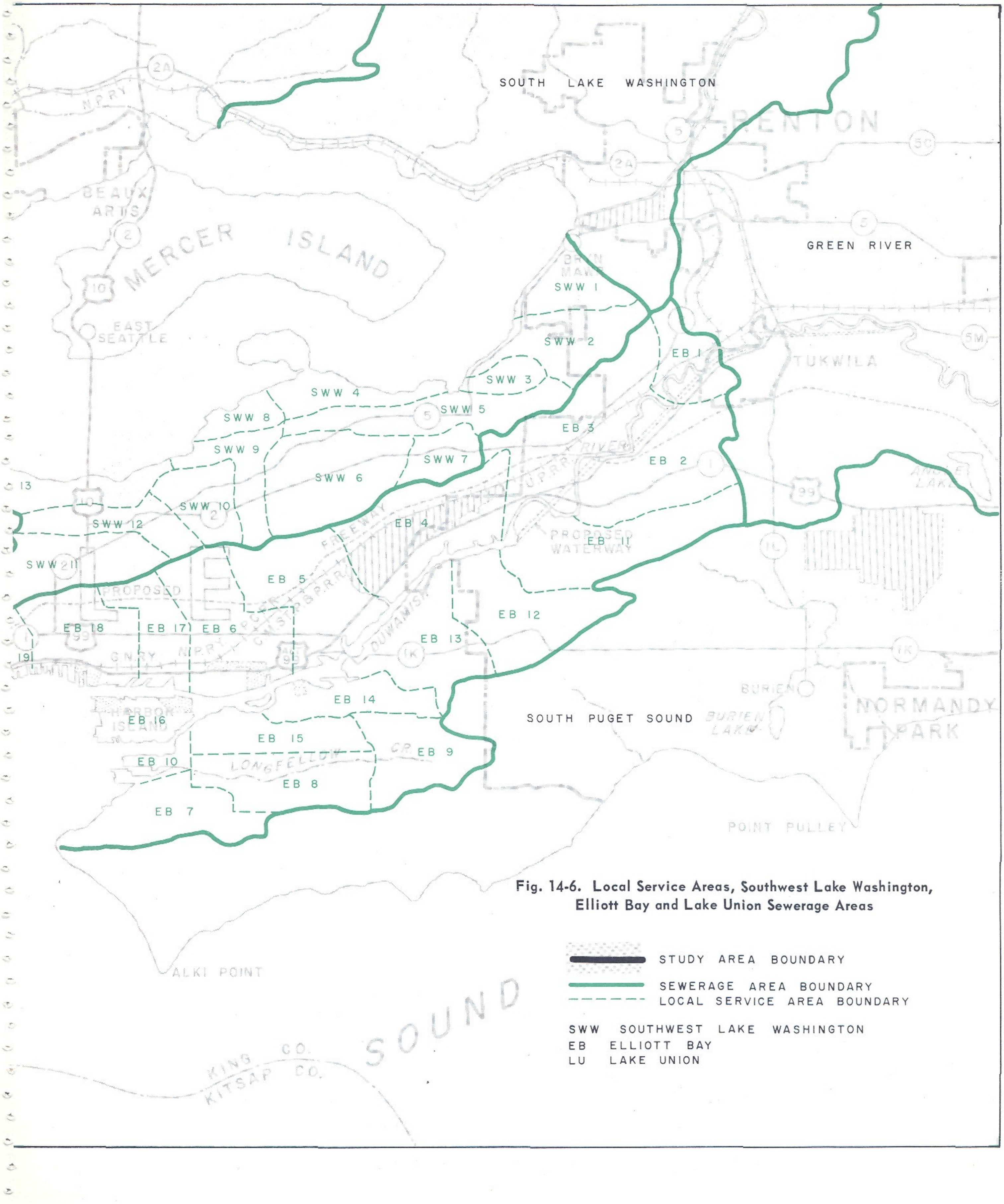


Fig. 14-6. Local Service Areas, Southwest Lake Washington, Elliott Bay and Lake Union Sewerage Areas

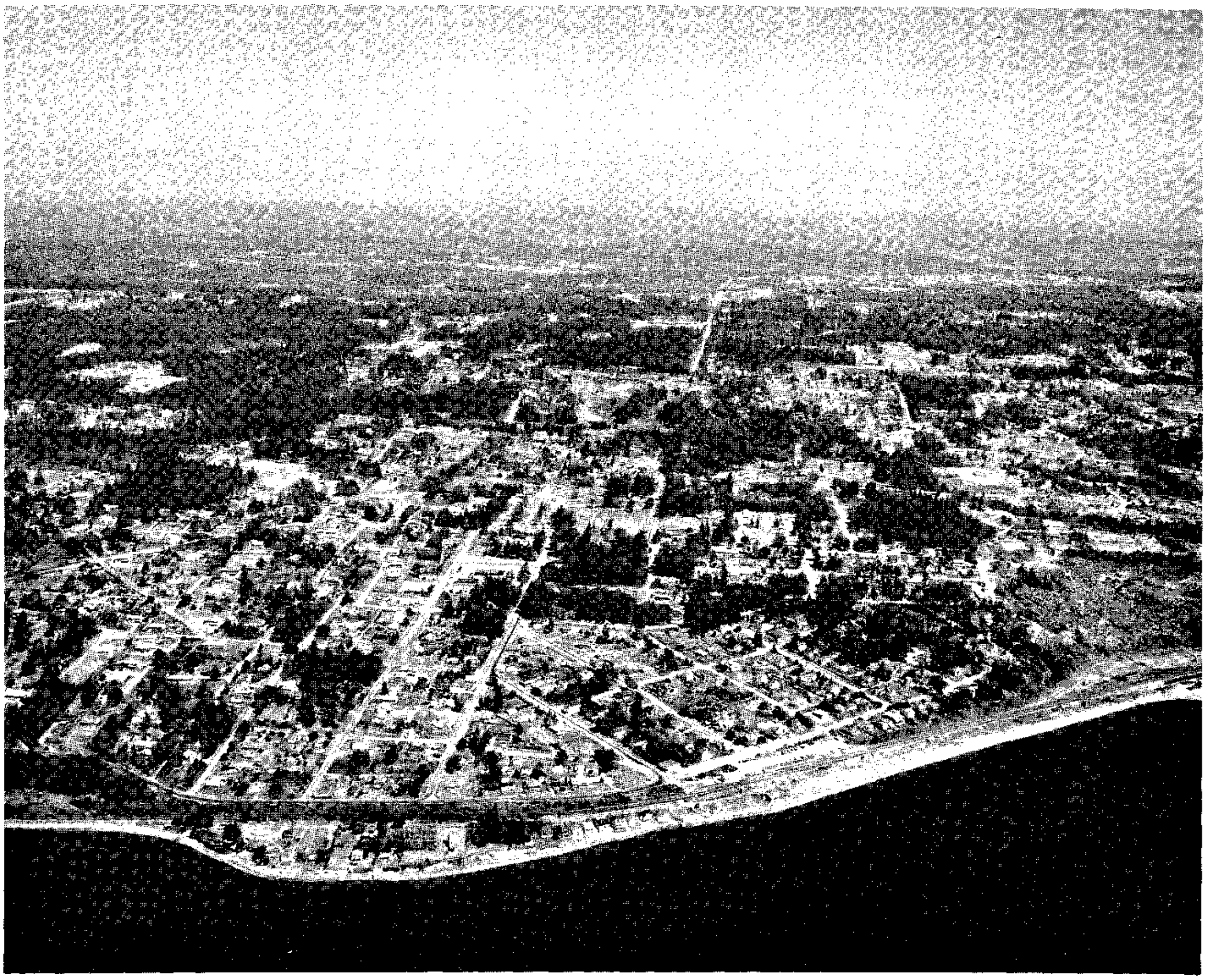
CONTINUED RESIDENTIAL GROWTH is expected in the South Puget Sound drainage area. The photo shows the city of Normandy Park and the development to the north and east of that city.

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.

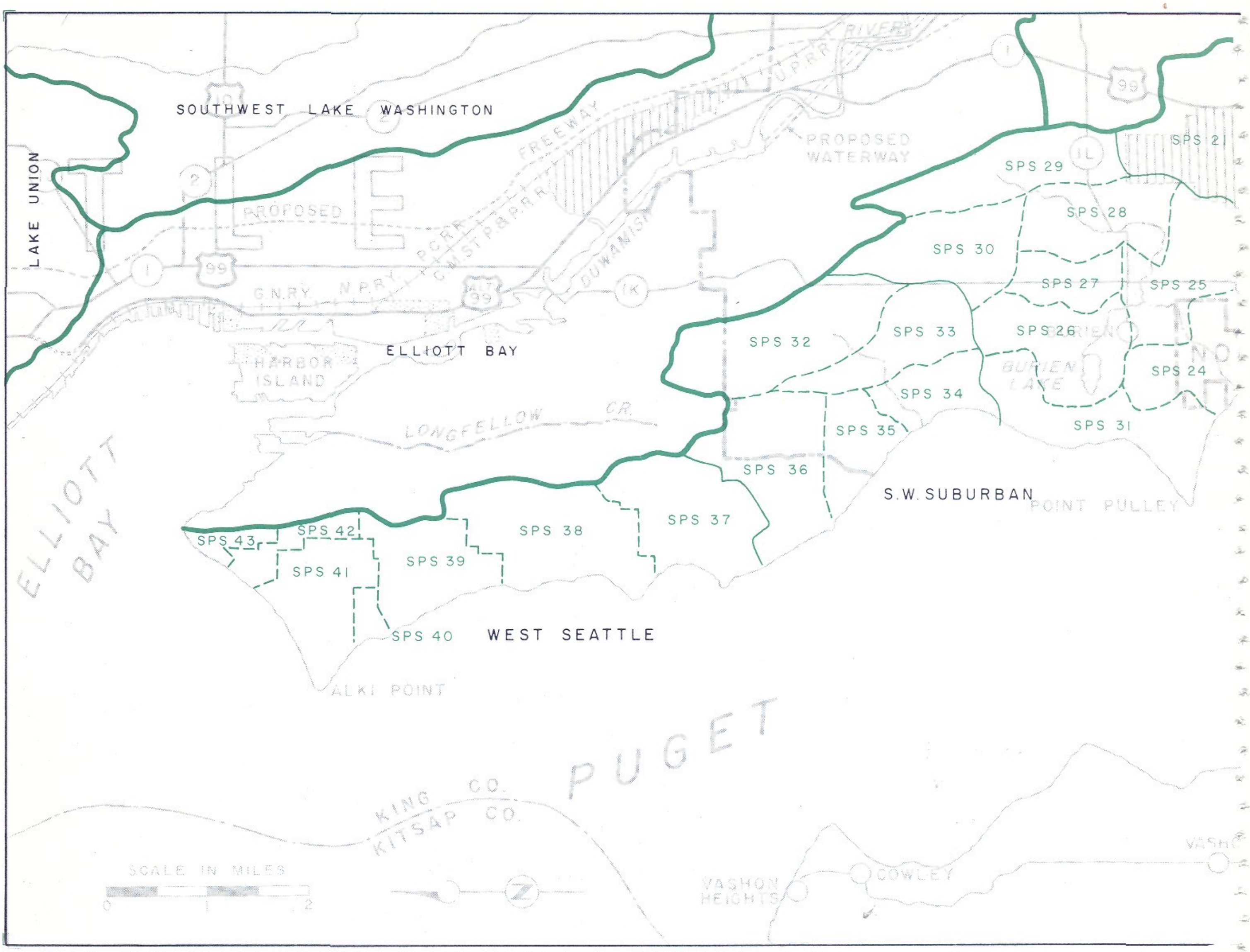
Topographically, the subarea is traversed by ravines and by many small creeks which discharge to Puget Sound. Much of the shoreline along the sound consists of steep bluffs and sheer cliffs, making it difficult to provide for waterfront interception of sewage. Collection of the sewage at one or more 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.

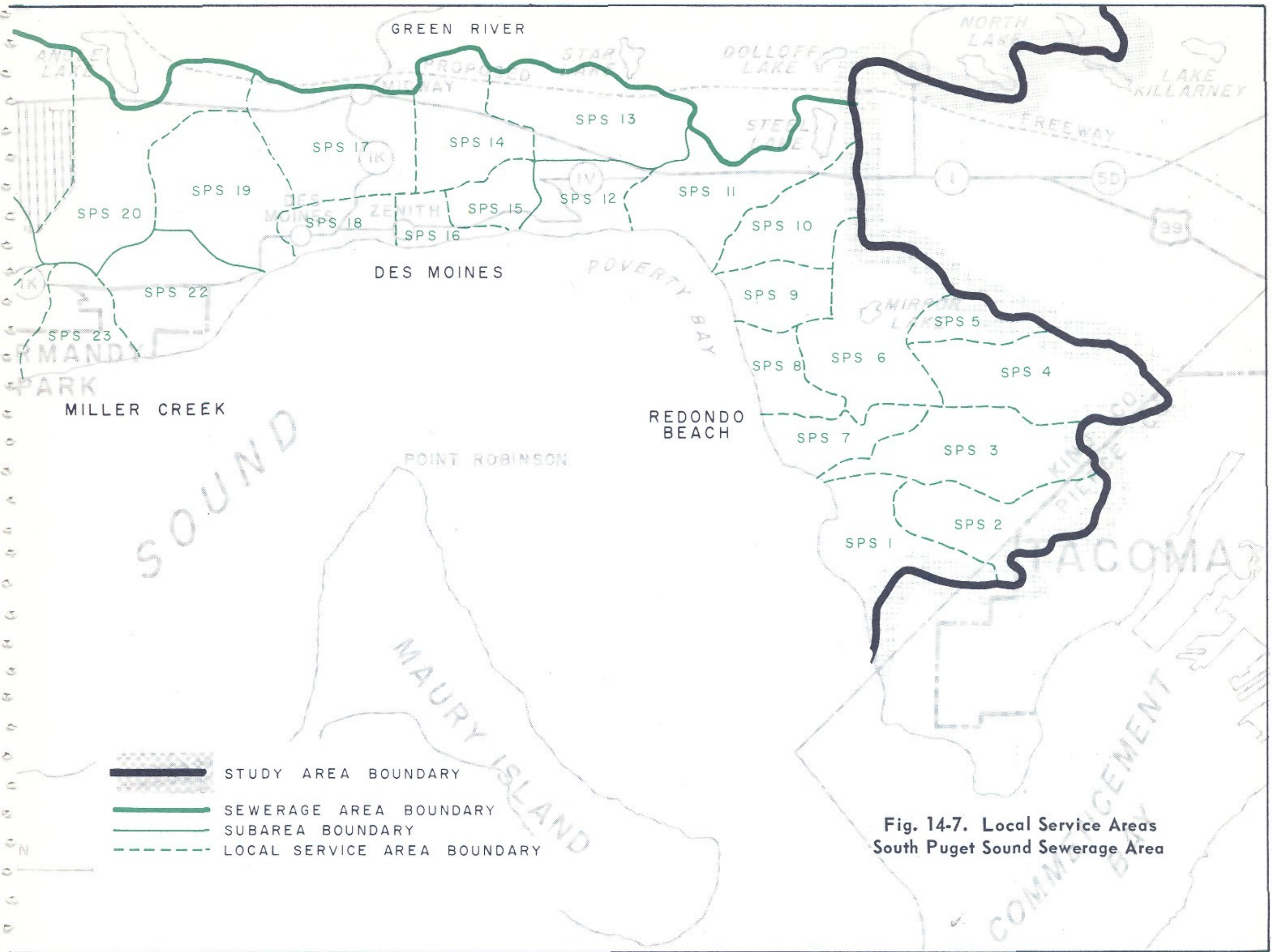


Fig. 14-7. Local Service Areas South Puget Sound Sewerage Area

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.

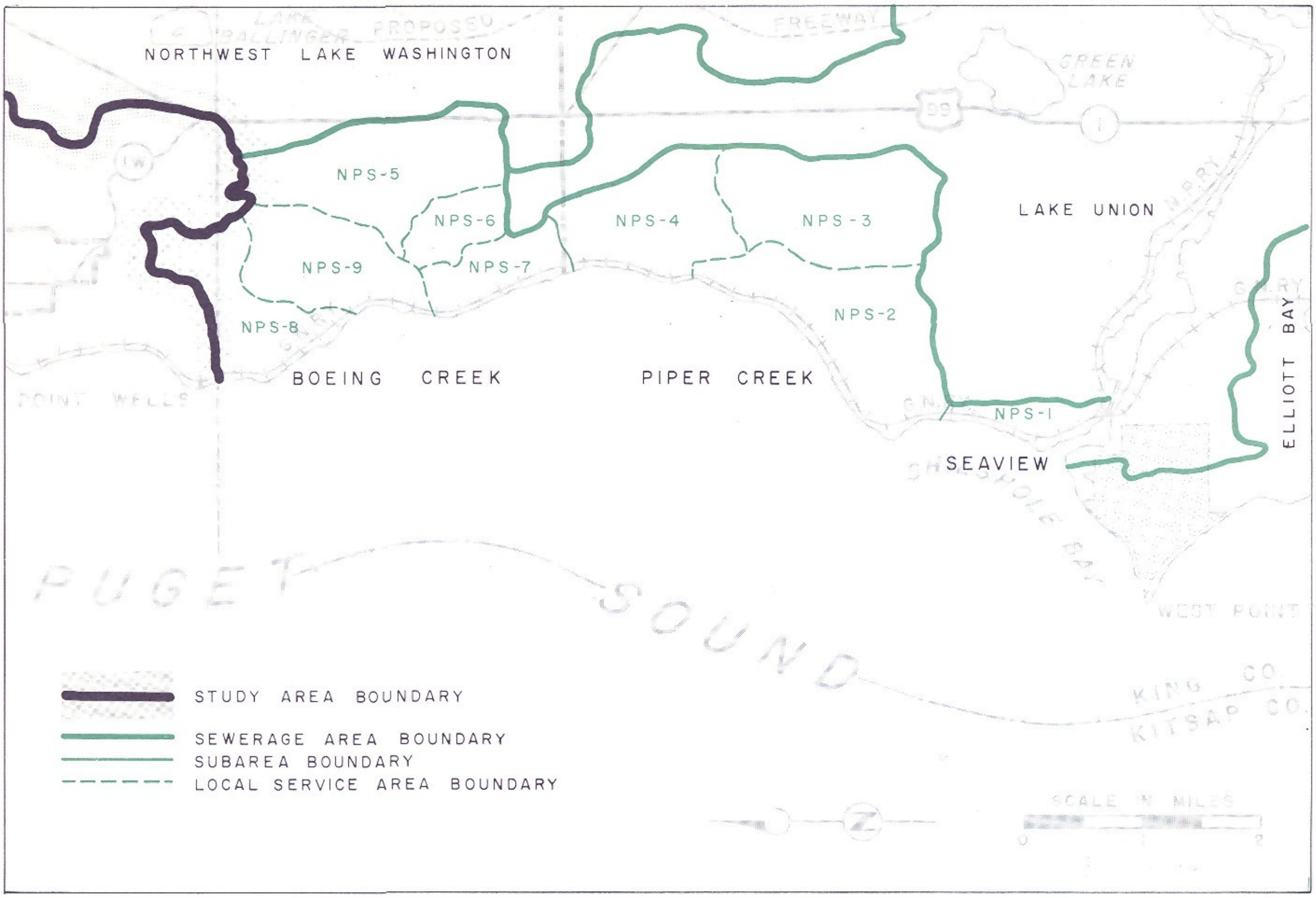


Fig. 14-8. Local Service Areas, North Puget Sound Sewerage Area

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.