PART V

SEWERAGE AND DRAINAGE PLANS

Chapter 15. Development of Sewerage Plans

Chapter 16. Stage Construction of Sewerage Facilities

Chapter 17. Development of Storm Drainage Plans

Chapter 18. Separation of Combined Sewers in the City of Seattle

Chapter 19. Financing of Recommended Facilities

Chapter 15
DEVELOPMENT OF SEWERAGE PLANS

In general, the most satisfactory and economic solution of the sewerage problem of a metropolitan area is achieved when sewage from the entire area is delivered either to a single point or to a relatively few points for treatment and disposal. To determine the feasibility of providing central sewerage facilities, it is necessary (1) to outline and analyze all reasonably possible projects, (2) to study the characteristics of each contributing area, and (3) to compare the costs of separate disposal facilities with those of central disposal.

Every project suggested for detailed analysis and comparison must satisfy certain fundamental controlling conditions and requirements. As set forth and discussed in preceding chapters of this report, some of the controlling factors are: geography, topography, geology and climate; recreational and other uses of beaches and waters; population numbers and distribution; value of existing sewerage facilities; characteristics of sewage; and disposal requirements for treated effluents.

METHOD OF ANALYSIS

In determining the best plan for sewerage of the metropolitan area, facilities were first laid out for four basic projects involving delivery of sewage to central locations for treatment and disposal. Each of these projects was analyzed in detail and all of them were compared on the basis of construction cost and total annual cost. In addition, where differences in cost were relatively minor, consideration was given to other aspects which then have to be taken into account in determining the over-all suitability of a sewerage project. As a final step, independent sewerage projects were developed for individual tributary areas and the costs thereof were compared with the costs of participation in the selected central project.

Central Sewerage Projects

In developing central sewerage projects, the first step is to determine what facilities are required in each sewerage area to convey sewage to logical points of concentration. These facilities are required in common regardless of whether the individual area is to have its own treatment and disposal system or is to be served by a central sewerage project. For convenience in reference, the facilities common to both individual and central projects are designated herein as service sewers.

As a second step, it is necessary to determine what facilities are required to convey the sewage from the points of concentration in each sewerage area to a point beyond which alternative plans can be developed for conveyance to appropriate locations for treatment and disposal. These facilities, likewise, are common to all alternative central sewerage projects and are referred to as feeder sewers.

The third step is to develop alternative plans for conveyance of the sewage from the terminus of the feeder sewer system to the final point of treatment and disposal. Facilities thus required, which include main intercepting and main trunk sewers, main pumping stations, treatment works, and outfall sewers, comprise what is referred to hereafter as a core plan.

In determining which of the several possible central sewerage projects would be the most suitable, comparisons need be based only on core plan facilities. This is because both the feeder and service sewer systems are common to each core plan.

Separate Projects for Independent Sewerage Areas

For comparison purposes, it is necessary in the case of each sewerage area (Chapter 14) to determine the cost of its share in the core plan and feeder sewer systems. This is accomplished by using the ratio of the flow from each individual area to the total flow for which the core and feeder facilities were designed.

To determine whether it would be economically feasible for each sewerage area to participate in the central sewerage project, studies were made of all independent projects which reasonably could be expected to provide adequate service either to the individual area or, in some cases, to combinations of such areas. In some areas, the choice between alternatives with respect to independent facilities was relatively simple and decisions could be made accordingly. In others, however, partial or complete cost comparisons were required.

Independent sewerage projects which were laid out for individual sewerage areas were analyzed in terms of construction cost and total annual cost, and figures thus obtained were compared with the corresponding costs of the core plan project. In general, the project recommended for adoption by each sewerage area is the one shown to represent the greatest economy to that area.

POSSIBLE TREATMENT PLANT AND DISPOSAL SITES

Selection of a site for sewage treatment and disposal operations is governed largely by two factors. These are (1) the ease with which sewage from a given area can be conveyed to a particular site, and (2) the requirements with respect to receiving water conditions. Less restrictive receiving water requirements at any one location may well justify the conveyance of sewage over a considerable distance for treatment and disposal.

Disposal Sites

Because of the requirement that all sewage and sewage effluents be removed from the Lake Washington watershed, the choice with respect to final disposal of the sewage of the metropolitan area is limited to Puget Sound and Green-Duwamish River. Of these alternatives, disposal to the salt waters of the sound presents fewer complications.

Puget Sound. For the purpose of the survey, investigations were made of 12 possible disposal sites in Puget Sound. The sites were selected primarily on the basis of sewage delivery, taking into account local disposal conditions and water use criteria. At 8 of the sites, it was found that satisfactory conditions could be maintained under ultimate peak flow rates by a combination of primary treatment followed by effluent disposal through a diffuser-equipped outfall. Secondary treatment will be required, however, at the Des Moines, Southwest Suburban, Meadow Point and Richmond Beach sites to obtain the necessary reduction in coliform organisms (Chapter 11).

It should be recognized, of course, that all of the decisions made herein with respect to treatment and disposal requirements are based on presently foreseeable uses of the waters of Puget Sound. In the event of unforeseeable developments, it is possible that a higher degree of treatment may be required at any one or all of the locations presently regarded as suitable for primary treatment. For that reason, and because future disposal requirements may become more stringent, primary sewage treatment plants discharging effluent to Puget Sound should be designed and planned, including purchase of necessary land, in such a manner that secondary units may be added later if and when the need arises.

Green-Duwamish River. Four possible sewage disposal sites were investigated along Green-Duwamish River (Chapter 12). Satisfactory performance at these sites is governed by the requirement that a minimum dissolved oxygen level be maintained in the river for the preservation of fish life. To meet that requirement, only effluents from plants providing complete treatment will be acceptable.

Treatment Plant Sites

To be fully satisfactory, a sewage treatment plant site should:

1. Be as close as possible to a body of water or a watercourse suitable for final disposal of treated effluent.
2. Be well isolated from residential or commercial developments, both present and future.
3. Be economically accessible to trunk and intercepting sewers and service roadways.
4. Have reasonably good soil characteristics to reduce the cost of special foundations.

In most areas, sites which meet all of the foregoing criteria are usually difficult, if not impossible, to find. Obviously, therefore, the problem is one of selecting sites which most nearly fulfill these requirements.

Core Plan Sites. Four possible treatment plant sites were selected as the most suitable for central sewerage projects. These are:

1. At West Point at the western extremity of Fort Lawton, hereinafter designated the West Point site. Effluent would be disposed of in Puget Sound.
2. In the industrial zone above the Government Locks on the Lake Washington Ship Canal in an area bordered by Commodore Way and the Great Northern Railroad tracks and 20th and 27th Avenues West, hereinafter designated the Government Locks site. Effluent would be disposed of in Puget Sound off West Point or in Lake Washington Ship Canal above the Government Locks.
3. In the industrial zone southeast of Elliott Bay, hereinafter designated the Elliott Bay site. In this area there are three possible sites, each of which is discussed in following sections of this chapter. Effluent would be disposed of in Elliott Bay or in Duwamish River.
4. At Black River Junction west of Renton, hereinafter designated the Renton site. Effluent would be disposed of in Duwamish River.

Of the four sites, the one which most nearly fulfills the requirements previously set forth is that at West Point. This site is (1) immediately adjacent to the final point of disposal, (2) well isolated from all residential and commercial areas, (3) near the present discharge point of the North Trunk sewer of the city of Seattle, and (4) in an area where soil borings indicate that no special foundation provisions need be made. There are, however, two major disadvantages. First, to obtain delivery of all sewage generated in the metropolitan area, a second tunnel will be required under Fort Lawton. This is because the existing North Trunk sewer has insufficient capacity, under gravity conditions, to accommodate the predicted ultimate flow. Second, sufficient land for the treatment plant would probably have to be developed by filling low-lying tideland areas to the north of the site.

In the case of the Government Locks site, the primary advantage would be that all sewage of the area could be readily concentrated at this point, utilizing to the fullest possible extent the existing system of the city of Seattle. For disposal to Puget Sound at West Point, treated effluent would be pumped through the existing Fort Lawton tunnel to a suitable outfall. Disposal to the Lake Washington Ship Canal, adjacent to the site, would require complete treatment. The obvious disadvantage to this site is its lack of isolation, although this could be overcome by proper architectural treatment and landscaping. Other disadvantages are high land values and the need for considerable site development, particularly excavation and leveling.

Available sites on Elliott Bay, while relatively good as far as the delivery of sewage is concerned, are not ideal because of remoteness from the point of effluent disposal, poor foundation conditions, and high land values. Although none of these sites is well isolated, the fact that they are all located in a heavy industrial zone makes isolation a matter of less importance.

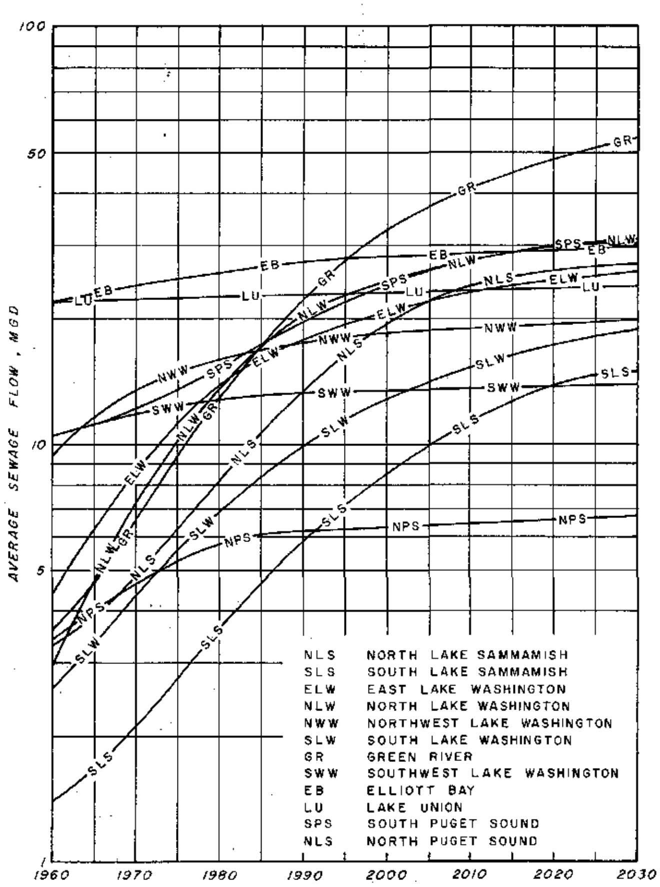


Fig. 15-1. Average Sewage Flows from Sewerage Areas

Disposal requirements in the Duwamish River are such that a plant at the Renton site would have to provide complete treatment. This site, however, offers the advantages of (1) relative ease of sewage delivery from a large part of the metropolitan area, (2) favorable soil and foundation conditions, (3) proximity to the effluent disposal point, and (4) location in an industrial area and consequent lesser need for isolation.

Sites for Plants for Separate Sewerage Areas. In general, the selection of treatment plant sites for individual sewerage areas was made on the basis of logical concentration points for sewage from the tributary area. As such, the locations selected for study purposes were broad rather than specific. Insofar as the present study is concerned, however, the difference costwise between specific sites within these broad locations is not significant in relation to the over-all costs of the individual systems.

PRELIMINARY DESIGN OF SEWERAGE FACILITIES

All plans set forth in this chapter are laid out to serve ultimate development of the tributary area. While it is evident that some of the required facilities, such as treatment plants, can be constructed in stages or increments, the relative economy of the various projects here considered can best be demonstrated by comparing their ultimate costs.

Core Plan Service Area

Because of the pumping required to convey sewage out of the South Puget Sound and North Puget Sound sewerage areas, it became apparent almost at the outset of the study that it probably would be uneconomical for these areas to join in any central sewerage project. Their flows, therefore, were excluded from all core plan facilities as initially conceived. Studies to determine the economic feasibility of their participation in the core plan were made, however, after determining the costs involved in providing separate treatment and disposal facilities.

With the two areas excluded, core plan facilities were laid out for the balance of the metropolitan area. As described in Chapter 14, this service area consists of ten individual sewerage areas.

Sewage Flows

Sewage flows in the facilities herein considered were estimated on the basis of design criteria presented in Chapter 13 and on ultimate population and industrial development of the various sewerage areas (Fig. 15-1 and Table 15-1). These flows are used throughout the report to determine the annual operating costs of treatment plants and pumping stations. They are used also to determine the dates on which facilities

Table 15-1. Average Sewage Flows from Sewerage Areas

Sewerage area	Average flow in mgd during period							Average flow in mgd during design period
	1960-1970	1970-1980	1980-1990	1990-2000	2000-2010	2010-2020	2020-2030	
North Lake Sammamish	4.1	6.5	11.0	16.9	21.0	24.0	26.5	15.7
South Lake Sammamish	1.8	2.9	4.8	7.3	9.7	12.0	14.1	7.5
East Lake Washington	6.7	11.5	15.5	18.8	21.1	23.6	25.8	17.6
North Lake Washington	5.4	11.2	16.9	22.3	26.2	29.0	30.6	20.2
Northwest Lake Washington	11.3	14.8	16.8	18.0	18.6	19.2	19.7	16.9
South Lake Washington	3.8	5.6	8.5	11.5	14.1	16.3	18.2	11.1
Green River	5.5	10.2	17.5	27.8	37.5	45.2	52.0	28.0
Southwest Lake Washington	11.3	12.3	13.0	13.4	13.6	13.8	14.0	13.1
Elliott Bay	23.1	25.0	26.5	27.6	28.7	29.4	29.9	27.2
Lake Union	22.3	22.5	22.8	23.3	23.5	23.7	23.9	23.1
South Puget Sound								
Redondo Beach Subarea	0.5	1.2	2.0	2.7	3.3	4.0	4.7	2.6
Des Moines Subarea	1.6	3.0	4.2	4.9	5.4	5.8	6.3	4.4
Miller Creek Subarea	2.0	3.4	4.8	5.8	6.4	6.8	7.2	5.2
Southwest Suburban Subarea	1.7	2.2	2.7	3.2	3.6	3.8	4.0	3.0
West Seattle Subarea	6.7	6.9	7.0	7.0	7.1	7.1	7.2	7.0
North Puget Sound								
Seaview Subarea	0.2	0.3	0.3	0.4	0.4	0.4	0.4	0.3
Piper Creek Subarea	2.3	3.0	3.3	3.6	3.6	3.7	3.7	3.3
Boeing Creek Subarea	1.6	2.1	2.4	2.5	2.6	2.6	2.6	2.3

should be constructed or enlarged under a stage construction program (Chapter 16).

Use of Existing Facilities

In general, the proposed system of trunk sewers is designed to utilize all local sewerage systems as they now exist. Some of the larger sewers within the more extensive systems, such as those of Southwest Suburban Sewer District and the city of Seattle, including the Lake City Sewer District, are utilized to their full capacity. Other smaller sewers, presently designated as trunk sewers within several of the sewerage agencies, were found to be of such size or in such location that they could not economically be included in any plan of trunk sewerage.

Since many of the existing sewers incorporated in the proposed plans were constructed 50 or more years ago and thus may be structurally weak at some points, their actual utilization will have to be preceded by a thorough inspection. Such an inspection is beyond the scope of this survey, but should be undertaken and completed as soon as possible. Sewers, or sections of sewers, found to be structurally unsound or otherwise damaged should be repaired or replaced. This program, which relates only to structural conditions, would not interfere with design and construction of core plan or other sewerage facilities.

None of the sewage treatment plants presently in use was found to be of the type or size or to be so situated that its inclusion in any long-term comprehensive program for central sewage treatment and disposal could be justified. Certain of the plants, however, particularly those of the city of Seattle at Alki Point and Lake City and that of the Southwest Suburban Sewer District, were found to be of ample capacity and suitable for inclusion in projects designed to serve individual sewerage areas.

INTERCEPTION OF COMBINED SEWERS

Most of the city of Seattle is presently sewered on a combined basis. As such, it presents a difficult problem with regard to the amount of storm water to be allowed for in an interceptor system. Obviously, the provision of interceptor capacity sufficient to accommodate flows from storms occurring at a frequency of once in 10 years would be an economic impossibility. That being the case, the only alternatives are either to provide a capacity which allows overflows from the system at certain specified frequencies, or to provide for complete separation of sanitary sewage from storm water. While the latter, of course, is the more attractive of the two, economic considerations may well preclude its general adoption.

Overflow Frequencies

The frequency at which overflows from a combined system should be allowed is governed by the use of the water into which the overflow occurs. Where the receiving waters are used extensively for recreation or shell fishing, the number of overflows should be limited to the minimum number commensurate with economic feasibility. On the other hand, where water use is predominantly commercial, overflows can be tolerated at relatively frequent intervals.

Based on an analysis of water use in the area surrounding the city of Seattle and on economic considerations, the following overflow frequencies appear to be justifiable:

1. In all waters of Lake Washington and contiguous waters east of Montlake Bridge -- an average of once per summer.

2. In the Lake Washington Ship Canal and contiguous waters west of Montlake Bridge -- unlimited overflows under storm conditions.

3. In Duwamish River and Elliott Bay -- unlimited overflows under storm conditions.

4. In recreational waters of Puget Sound -- the present interceptor system at West Seattle is designed on the basis of 12 overflows per summer in accordance with specific requirements of the State Pollution Control Commission and State Health Department. Experience will demonstrate whether this frequency is satisfactory. If not, additional interceptor capacity, or its equivalent, will have to be provided to reduce the number of overflows.

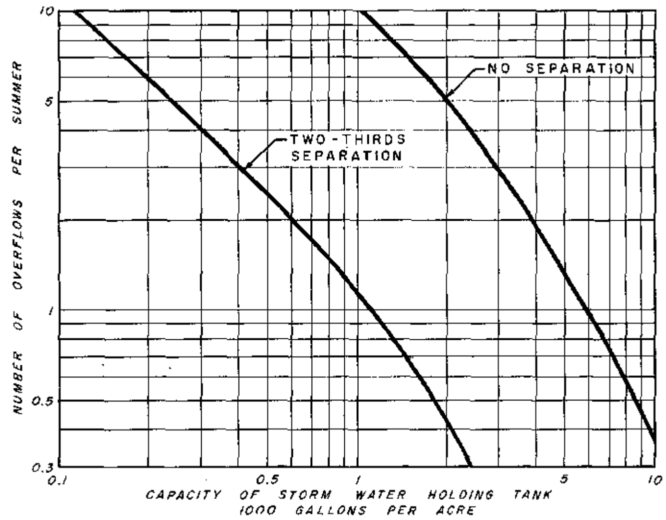


Fig. 15-2. Capacities of Storm Water Holding Tanks for Combined and Partially Separated Systems

The curves are based on an interceptor having a capacity of 2.5 times the average dry weather flow. See Fig. 13-7 for basis of calculations.

Interceptor Capacity

Where the number of overflows is limited because of water use considerations, sufficient capacity must be provided in an interceptor to carry the runoff from a storm having a recurrence interval equal to the overflow frequency. Where storm water overflows are not objectionable, interception of the peak dry weather flow is sufficient.

The capacity, or equivalent capacity, of interceptors from which only a limited number of overflows is permissible can be provided by a number of means including:

1. Construction of an interceptor having a capacity sufficient for the flow.

2. Partial or complete separation of a part or all of the tributary area.

3. Construction of holding tanks at overflow points to store excess flow during periods of rain.

Combined Interceptor. As shown in Fig. 13-3, Chapter 13, a combined interceptor for flows occurring from storms with a recurrence interval of once per summer needs to have a capacity, depending on the time of concentration, of some 30 to 60 times the average dry weather flow. As a matter of comparison, the peak flows for which interceptors for a separate system are designed are usually two to four times average dry weather flow.

Separation. By complete or partial separation of all or part of an area tributary to an interceptor system, the amount of storm water entering the system is appreciably reduced. Complete separation over the entire area would, of course, mean that the system was separate and interceptor design would be as for separate systems. Partial separation, used in conjunction either with large interceptors or with holding tanks, would reduce the size of facilities required in both cases.

Holding Tanks. Provision of holding tanks at points of overflow for the purpose of storing excess water during periods of rain is a feasible alternative in any system where interceptors have inadequate storm flow capacity. In such installations, storm flows in excess of interceptor capacity can be diverted to the holding tanks and then pumped back into the interceptor upon the cessation of rain when capacity is available.

Studies involved in determining the required sizes of holding tanks both for various interceptor capacities and for various overflow frequencies are discussed in Chapter 13. Since partial separation of a tributary area will reduce the size of the holding tank needed for any given interceptor capacity, tank capacities were determined first for areas in which no separation is to be undertaken and second for areas in which two-thirds of the storm water, or roughly that contributed by street drainage only, is to be removed from the combined sewers (Fig. 15-2). These analyses were made on the basis of an interceptor capacity of 2.5 times the average dry weather flow, or about the design capacity of the existing waterfront interceptors in the Lake Washington drainage basin.

Analysis of Interception Methods. To determine which of the three methods outlined above should be adopted for interception of sewage and storm water from combined systems draining to Lake Washington, a study was made of a system in the Southwest Lake Washington sewerage area. This system, which is tributary to a lake front interceptor having a capacity of about 2.5 times average dry weather flow, covers an area of 4,190 acres in lower Rainier Valley from Seward Park on the north to the city limit of Seattle on the south. Consideration was given to the following alternatives:

1. System to remain on a combined basis with intercepting sewers designed for peak wet weather sanitary flow and trunk sewers for a 10-year storm. Storm water holding tanks of a size to allow an average of one overflow per summer would be provided at all overflow points. In addition, since some trunks serving the area do not have enough capacity to handle the flow from a 10-year storm, this alternative would include partial separation of local service areas (Fig. 14-6).

2. System to remain on a combined basis with intercepting sewers designed for storm flows resulting from a rainfall having a recurrence interval of once per summer, and trunk sewers designed for a 10-year storm. As under Alternative 1, this project would include partial separation of local service areas SWW-6 and SWW-7 and a part of SWW-5.

3. System to be partially separated with intercepting sewers designed for peak wet weather sanitary flow and trunk sewers for a 10-year storm. Storm water holding tanks of a size to allow an average of one overflow per summer would be provided at all overflow points. Because the storm water flow would be reduced by partial separation of the entire tributary area, the sizes of holding tanks would be reduced from those proposed under Alternative 1.

4. System to be completely separated with intercepting and trunk sewers designed for peak wet weather sanitary flow.

Estimated construction costs for the four alternatives are given in Table 15-2. As there indicated, the cost of $5,940,000 for Alternative 1, which calls for retention of the existing combined system and provision of storm water holding tanks, is $1,532,000 less than that of the next cheapest alternative. All intercepting sewers for combined systems fronting Lake Washington have, therefore, been designed on the basis (1) of retaining the present systems on a combined basis, (2) of providing holding tanks designed for one overflow per summer, and (3) of separating within the system only to the extent necessary to relieve trunk sewers which now have insufficient capacity.

Table 15-2. Comparison of Construction Costs for Alternative Designs of Trunk and Interceptor Sewers, Southwest Lake Washington Sewerage Area

Alternative designs	Construction cost,[a] dollars
Alternative 1	5,940,000
Alternative 2	7,472,000
Alternative 3	8,873,000
Alternative 4	17,700,000

Partial separation cost, $1,860 per acre.
Complete separation cost, $3,890 per acre.
[a]Includes engineering and contingencies.

DESCRIPTION OF CORE PLANS

As previously defined, core plans include only those facilities which are not common to each of the central sewerage plans herein considered. Four basic plans were investigated as follows:

Core Plan A - delivery of all sewage from the metropolitan area to a single primary type treatment plant at the Government Locks site, with effluent disposal to Puget Sound off West Point.

Core Plan B - delivery of sewage to two treatment plants, the first a primary type plant at the West Point site with effluent disposal to Puget Sound, and the second a complete type plant at the Renton site with effluent disposal to Duwamish River.

Core Plan C - delivery of sewage to two treatment plants, both of the primary type. Of these, the first would be at the West Point site with effluent disposal to Puget Sound, and the second at the Elliott Bay site with effluent disposal to Elliott Bay. Four alternatives, differing with respect to plant location, degree of treatment, and effluent disposal, were considered for the Elliott Bay site.

Core Plan D - delivery of sewage to three treatment plants, one a primary type at the West Point site with effluent disposal to Puget Sound, the second a primary type at the Elliott Bay site with effluent disposal to Elliott Bay, and the third a complete type at the Renton site with effluent disposal to the Duwamish River. Three alternatives, differing only in plant location, were considered for the Elliott Bay site.

In all studies relating to selection of the most appropriate core plan, it was assumed that sewage from the east side of Lake Washington, including that from North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, would be conveyed southward and combined with that from the South Lake Washington and Green River sewerage areas. A study of the possibility of conveying sewage from the east to the west side of Lake Washington across the lake was deferred until the most suitable core plan had been determined. Similarly, other feasible modifications of the selected core plan were studied and evaluated.

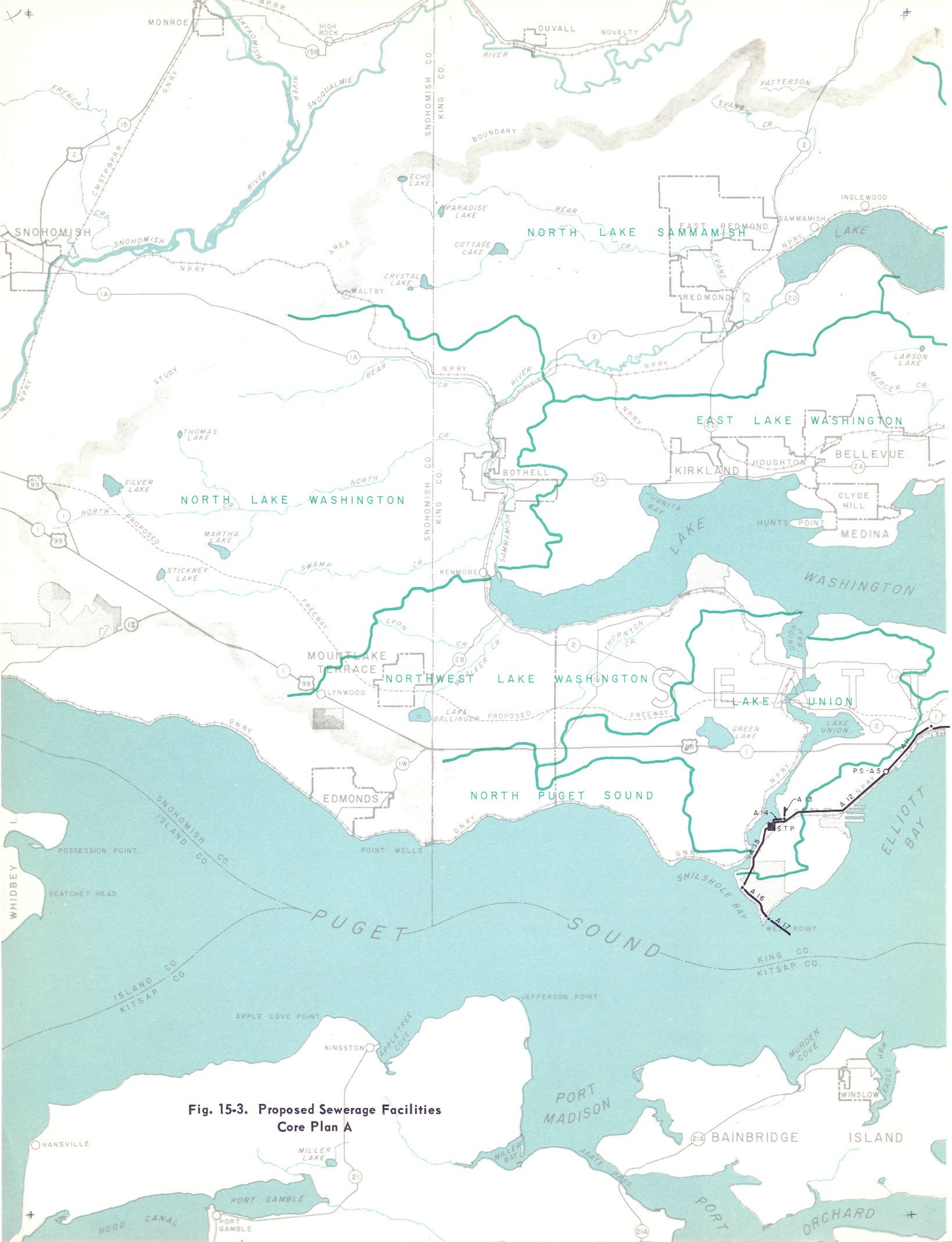


Fig. 15-3. Proposed Sewerage Facilities Core Plan A

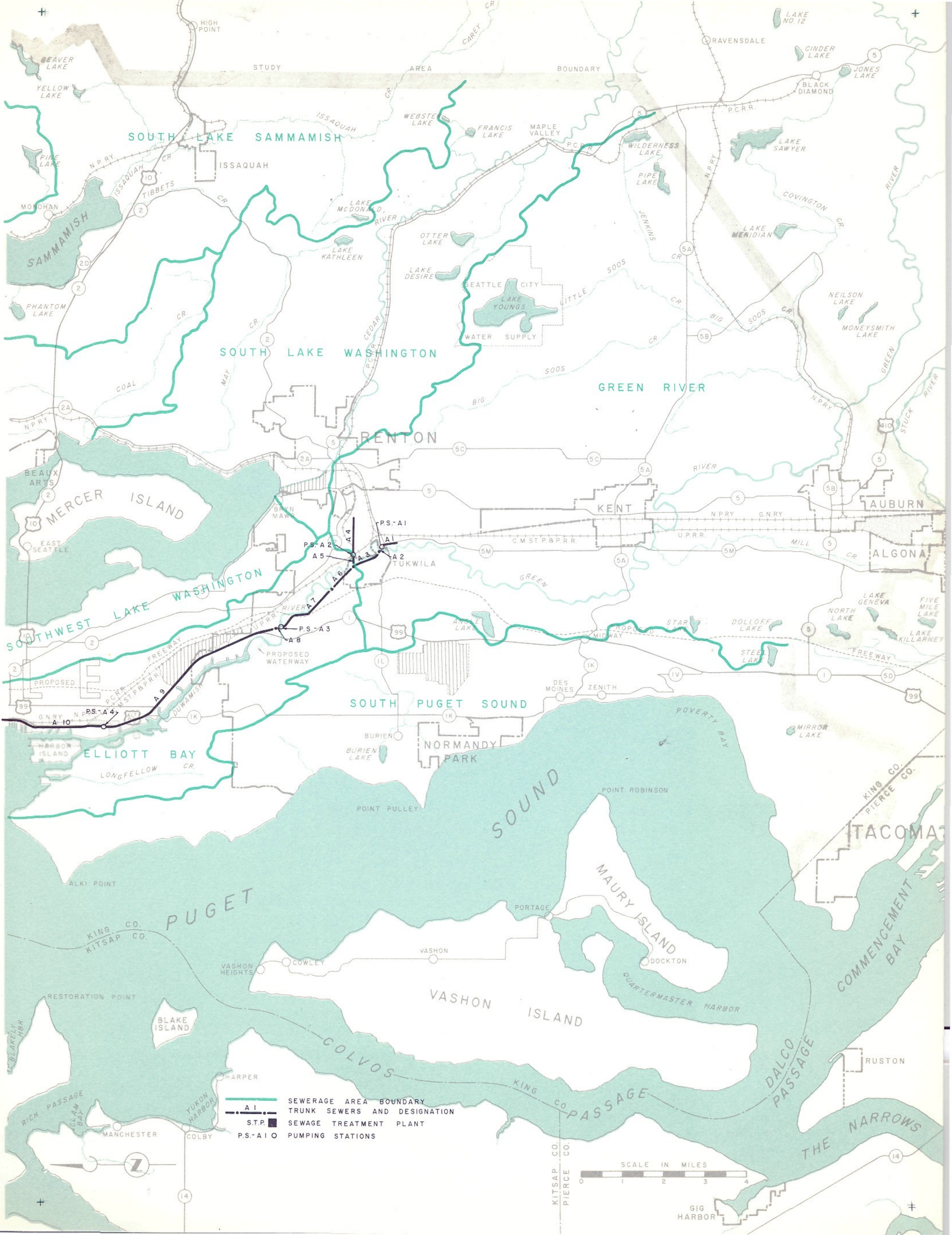

STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
MERCER ISLAND
BEAUX ARTS
EAST SEATTLE
BRYN MAWR
DES MOINES
ZENITH
MIDWAY
BURIEN
NORMANDY PARK
TACOMA
RUSTON
GIG HARBOR
HARPER
MANCHESTER
COLBY
VASHON
VASHON HEIGHTS
COWLEY
PORTAGE
DOCKTON
VASHON ISLAND
MAURY ISLAND
PUGET SOUND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
QUARTERMASTER HARBOR
POINT ROBINSON
POINT PULLEY
ALKI POINT
RESTORATION POINT
BLAKE ISLAND
HARBOR ISLAND
KING CO.
KITSAP CO.
PIERCE CO.
P.S.-A1
P.S.-A2
P.S.-A3
P.S.-A4
A1
A2
A3
A4
A5
A6
A7
A8
A9
A10
PROPOSED WATERWAY
SEWERAGE AREA BOUNDARY
A1 TRUNK SEWERS AND DESIGNATION
S.T.P. SEWAGE TREATMENT PLANT
P.S.-A1 PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 15-3. Description and Estimated Construction Cost, Core Plan A

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
A-1	52	130	2,300 ft of 96-in. RC at 0.05%, average cut 24 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	395,000
A-2	52	130	600 ft of twin 42-in. force mains across Duwamish River	70,000
A-3	52	130	3,300 ft of 96-in. RC at 0.05%, average cut 15 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	401,000
A-4	90	238	5,400 ft of 114-in. RC at 0.07%, average cut 27 - 31 ft, difficult wet, includes sheeting, dewatering and railroad crossing	1,333,000
A-5	90	238	600 ft of parallel 36-in., 42-in. and 48-in. force mains across Duwamish River	110,000
A-6	143	368	2,400 ft of 114-in. RC at 0.16%, average cut 20 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	446,000
A-7	143-146	370-376	10,900 ft of 120-in. RC at 0.13%, average cut 21 - 32 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	2,948,000
A-8	146	376	500 ft of parallel 48-in. and twin 54-in. force mains across Duwamish River	113,000
A-9	146-160	376-411	26,200 ft of 132-in. RC at 0.075 - 0.086%, average cut 16 - 30 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and overflow structure on tributary sewer	7,550,000
A-10	162-181	417-468	16,800 ft of 144-in. RC at 0.058 - 0.068%, average cut 17 - 32 ft, difficult wet, through congested industrial area and through debris fill containing wood and cinders at site of old Yessler Mill, includes imported backfill, repaving, sheeting, dewatering, and underpinning of structures adjacent to 1st Avenue	5,015,000
A-11	181	468	8,400 ft of 144-in. RC tunnel at 0.068%, includes allowance of 20% for uncertainties	6,451,000
A-12	182-189	471-489	15,500 ft of 144-in. RC at 0.068 - 0.075%, average cut 18 - 31 ft, difficult wet, includes 2,000 ft on piles, imported backfill, repaving, sheeting, dewatering, railroad and highway crossings, and overflow structure on tributary sewer	4,841,000
A-13	70	175	1,600 ft of existing 138-in. at 0.035%	Existing
A-14	70	175	2,900 ft of existing 144-in. at 0.032%	Existing
Subtotal, sewers				29,673,000
PS-A-1	52	130	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 28 ft, structure about 30 ft below ground, difficult wet, includes sheeting and dewatering	588,000
PS-A-2	90	238	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 34 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	863,000
PS-A-3	146	376	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 37 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	1,113,000
PS-A-4	160	411	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 31 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	1,173,000
PS-A-5	181	468	Pumping station, single stage, motor and diesel engine driven, static lift 30 ft, total head at peak flow 34 ft, structure about 40 ft below ground, difficult wet, includes sheeting and dewatering	1,273,000
Subtotal, pumping stations				5,010,000

Continued on next page

Table 15-3. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[a] dollars
	Average DWF	Maximum WWF		
STP	265	660	Sewage treatment plant, primary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes purchase of 50 acres of land and site development	19,950,000
A-15	265	660	8,000 ft of existing 144-in. at 0.19%, to be converted to effluent outfall	Existing
A-16	265	660	4,700 ft of 144-in. RC effluent outfall, includes construction of 2,500 ft under tidal conditions, and dewatering	1,259,000
A-17	265	660	3,900 ft of twin 84-in. RC submarine outfalls to a water depth of 210 ft, includes diffuser sections over last 475 ft	3,254,000
Subtotal, outfall				4,513,000
Total contract cost, Core Plan A				59,146,000
Engineering and contingencies, 25 per cent				14,786,000
Total construction cost, Core Plan A				73,932,000

See Fig. 15-3 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

Core Plan A

Sewers, pumping stations, treatment works and outfalls called for under Core Plan A are designated as to location in Fig. 15-3 and are described in Table 15-3. Under this plan, effluent from the treatment plant would be pumped through the existing North Trunk sewer of the city of Seattle for final disposal in Puget Sound off West Point.

Intercepting Sewers. Intercepting sewers under Core Plan A include two branches, a south and a north. The south branch would intercept all sewage from the North Lake Sammamish, South Lake Sammamish, East Lake Washington, South Lake Washington and Green River sewerage areas at a point east of Renton and would convey this flow northward, following generally the route of State Highway 5M and East Marginal Way to the Elliott Bay waterfront. Along the way, additional sewage would be picked up from the Southwest Lake Washington and Elliott Bay sewerage areas and all existing outfalls would be intercepted. On the Elliott Bay waterfront, the sewer would be laid in Alaskan Way and would intercept all industrial and sanitary sewage outfalls.

North of Madison Street, Alaskan Way is constructed on a pile-supported platform, thus precluding the possibility of continuing the south branch in open cut. It would be necessary, therefore, to route the sewer east at Columbia Street to First Avenue, from which point a tunnel would be constructed northwestward along First and Western Avenue alignments to terminate at the intersection of Western and Elliott Avenues. At its deepest point, this tunnel would be about 105 feet below ground surface. From the tunnel exit, the interceptor would be constructed by open cut along Elliott Avenue to north of Pier 91 where it would turn north along the Great Northern Railroad tracks to the treatment plant. To avoid excessive cuts, a total of five pumping stations would be required.

The north branch would consist of a short leg extending eastward from the treatment plant and would convey sewage from the North Lake Washington, Northwest Lake Washington and Lake Union sewerage areas to the plant. This section would utilize a portion of the existing North Trunk sewer of the city of Seattle.

Sewage Treatment Plant. Designed for an average dry weather flow of 265 mgd and a peak storm flow of 660 mgd, the sewage treatment plant would be of the primary type and would require influent pumping from both interceptors. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, as about 30 minutes detention time would be available in the outfall sewer even at the peak flow of 660 mgd. Digested sludge would be hauled away in tank trucks for disposal elsewhere. Sludge gas

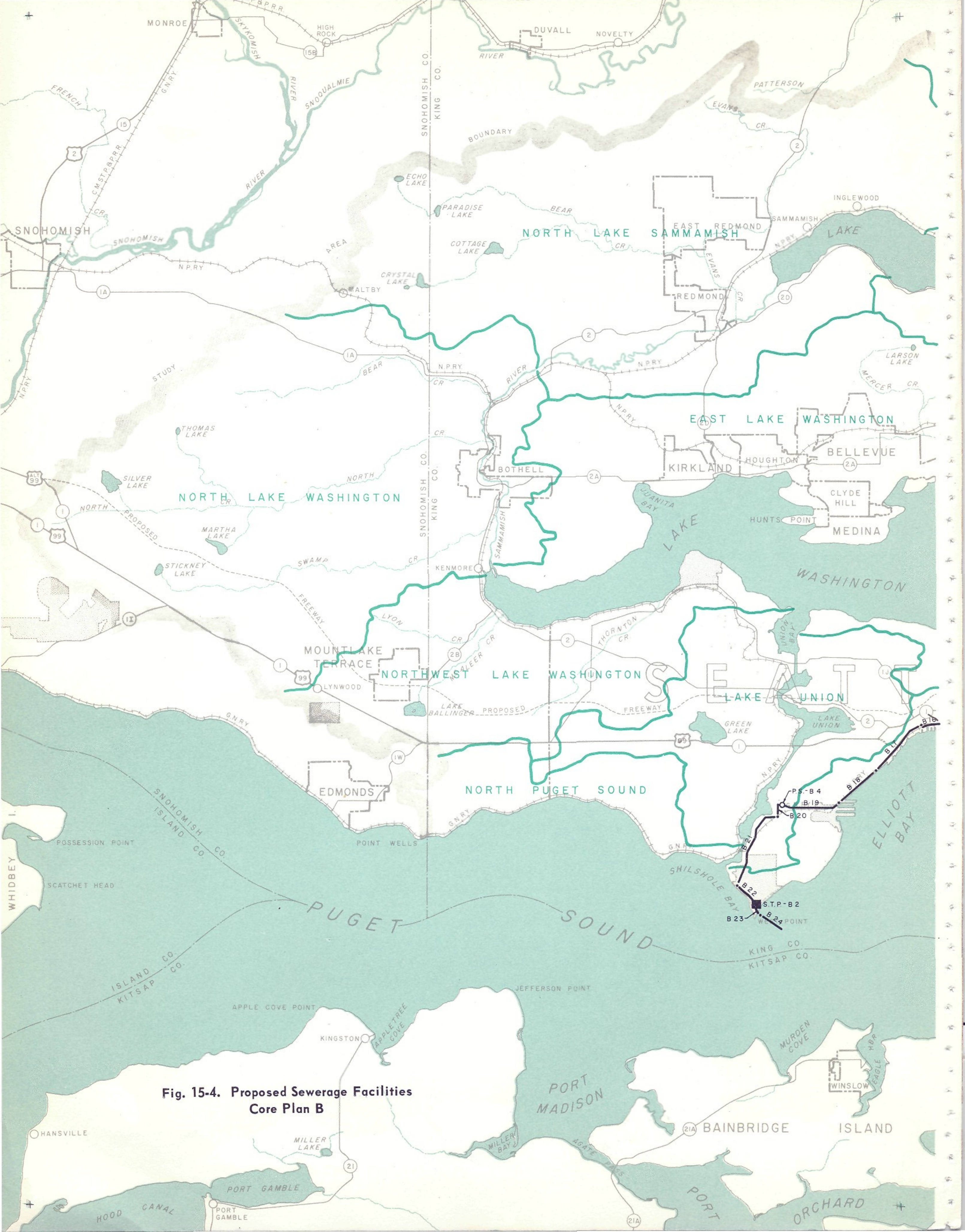
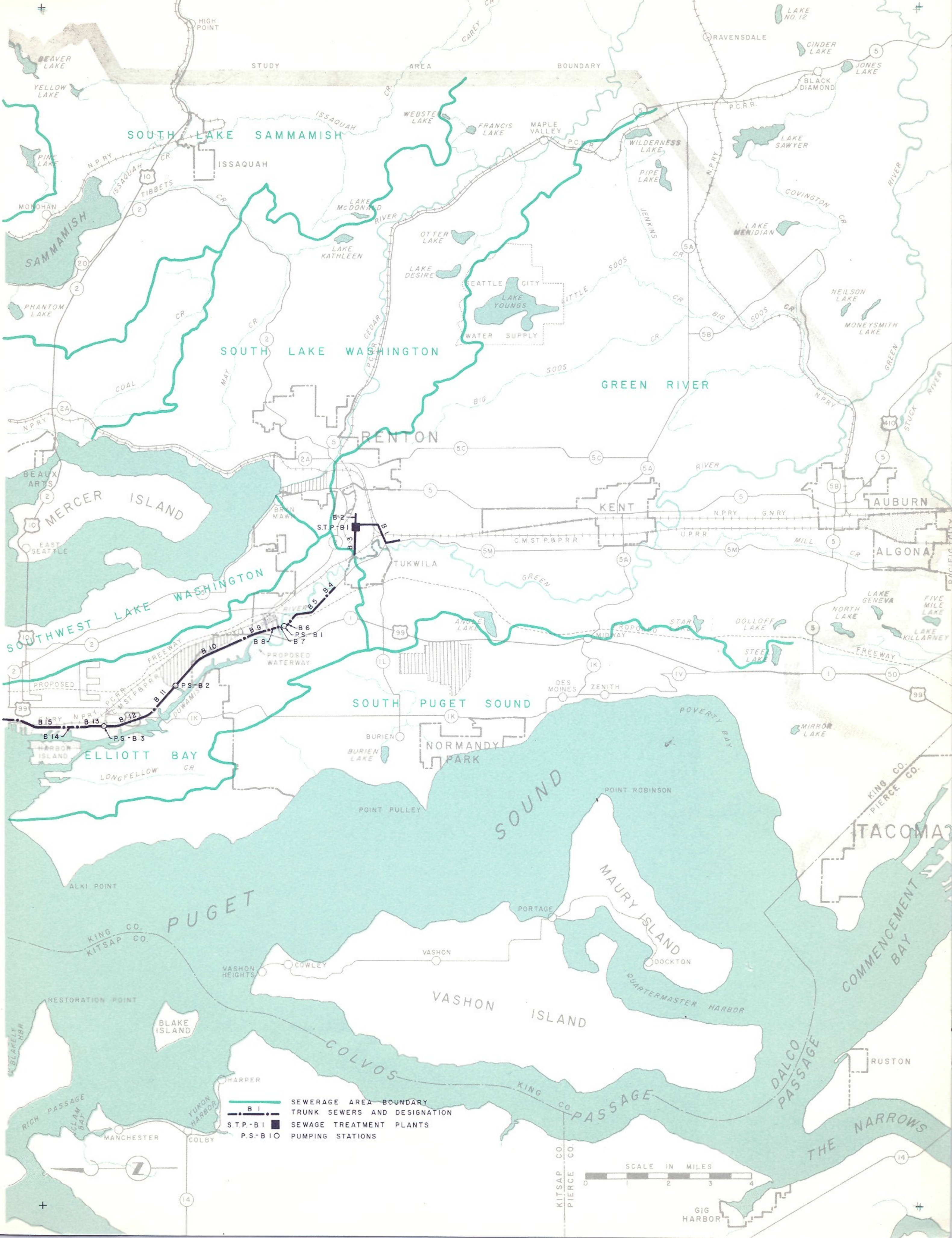


Fig. 15-4. Proposed Sewerage Facilities
Core Plan B



SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
STUDY AREA BOUNDARY
HIGH POINT
RAVENSDALE
LAKE NO. 12
CINDER LAKE
JONES LAKE
BLACK DIAMOND
BEAVER LAKE
YELLOW LAKE
PINE LAKE
ISSAQUAH
WEBSTER LAKE
FRANCIS LAKE
MAPLE VALLEY
WILDERNESS LAKE
LAKE SAWYER
PIPE LAKE
MONOHAN
SAMMAMISH
LAKE MCDONALD
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
LAKE MERIDIAN
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
NEILSON LAKE
MONEYSMITH LAKE
PHANTOM LAKE
RENTON
BEAUX ARTS
MERCER ISLAND
EAST SEATTLE
BRYN MAWR
KENT
AUBURN
ALGONA
TUKWILA
S.T.P.-B1
P.S.-B1
P.S.-B2
P.S.-B3
PROPOSED WATERWAY
LAKE GENEVA
NORTH LAKE
FIVE MILE LAKE
LAKE KILLARNEY
DOLLOFF LAKE
STAR LAKE
STEEL LAKE
ANGLE LAKE
MIDWAY
DES MOINES
ZENITH
BURIEN
BURIEN LAKE
NORMANDY PARK
POVERTY BAY
MIRROR LAKE
ELLIOTT BAY
HARBOR ISLAND
DUWAMISH
POINT ROBINSON
POINT PULLEY
SOUND
TACOMA
KING CO.
PIERCE CO.
ALKI POINT
PUGET
KITSAP CO.
PORTAGE
MAURY ISLAND
VASHON
VASHON HEIGHTS
COWLEY
DOCKTON
RESTORATION POINT
BLAKE ISLAND
VASHON ISLAND
QUARTERMASTER HARBOR
COMMENCEMENT BAY
COLVOS PASSAGE
DALCO PASSAGE
RUSTON
THE NARROWS
HARPER
RICH PASSAGE
CLAM BAY
YUKON HARBOR
MANCHESTER
COLBY
BLAKELY HBR
GIG HARBOR
SEWERAGE AREA BOUNDARY
B1 TRUNK SEWERS AND DESIGNATION
S.T.P.-B1 SEWAGE TREATMENT PLANTS
P.S.-B1 PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

would be used for the generation of all power required in the plant and as a source of heat for sludge heating purposes. Because of the proximity to residential and commercial areas, particular emphasis would be placed on architectural treatment of plant structures and on landscaping of the grounds.

Effluent Disposal. Treated effluent would be chlorinated during the recreational season, May to September, and pumped to the western extremity of West Point, where it would be discharged to Puget Sound approximately 3,900 feet offshore at a water depth of about 210 feet. The outfall would consist of a portion of the existing North Trunk sewer, a land section from the terminus of the North Trunk along the north shoreline of Fort Lawton, a land section across West Point, and a twin 84-inch submarine section. Since the land section along the shoreline of Fort Lawton would be constructed under tidal conditions, cost estimates are adjusted accordingly.

Construction Cost. Estimated construction costs of Core Plan A, including engineering and contingencies, total $73,932,000 (Table 15-3). Of this total, approximately 50 per cent is for intercepting sewers, 8 per cent for pumping stations, 34 per cent for the treatment plant, and 8 per cent for the outfall.

Core Plan B

Locations of sewers, pumping stations, treatment works and outfalls are designated in Fig. 15-4. Descriptions of these facilities are given in Table 15-4.

Intercepting Sewers - Renton System. Intercepting sewers for the Renton system include 2 short branches which join at the treatment plant. One would extend southward 6,700 feet from the plant and would serve the Green River sewerage area. The second branch, serving the North Lake Sammamish, South Lake Sammamish, East Lake Washington and South Lake Washington sewerage areas, would extend 1,300 feet east from the plant.

Sewage Treatment Plant - Renton System. As explained in Chapter 12, the waste receiving capacity of Duwamish River is such that a daily organic load equivalent to 15,000 pounds of 5-day BOD could be discharged at the Renton site while still maintaining satisfactory conditions with respect to dissolved oxygen. Compared to this, the ultimate BOD load which would be delivered to the treatment plant amounts to 332,000 pounds per day, based on an estimated equivalent population of 1,660,000 and a BOD contribution of 0.2 pounds per capita per day. To produce an effluent satisfactory for discharge to the river, the plant would thus have to remove about 95 per cent of the incoming BOD. This high degree of purification would necessitate secondary treatment by the activated sludge process, using a design loading of 35 pounds of BOD per 1,000 cubic feet of aeration tank capacity.

Secondary treatment would be provided at the Renton plant for an ultimate average dry weather flow of 143 mgd and a peak storm flow of 360 mgd. Influent pumping would be required to lift the sewage from both interceptors into the plant. Plant units would consist of preaeration and primary sedimentation tanks, aeration tanks, secondary sedimentation tanks, chlorine contact tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons having an area of 50 acres. When the lagoons are full, the dried sludge would be removed and sold as a soil conditioner or disposed of otherwise.

Effluent Disposal - Renton System. Chlorinated effluent would be discharged through twin 78-inch outfall lines to Duwamish River. Each outfall would be provided with diffusers to obtain effective dilution and dispersion in the river.

Intercepting Sewer - West Point System. The intercepting sewer for the West Point system under Core Plan B would begin north of the city of Tukwila in the Elliott Bay sewerage area and run northward, generally following the route of State Highway 5M and East Marginal Way to the Elliott Bay waterfront. From there, the route would be along Alaskan Way as far as Columbia Street, at which point, as under Core Plan A, a tunnel would be constructed to the intersection of Western and Elliott avenues. From the tunnel exit, the interceptor would be laid along Elliott Avenue to north of Pier 91 and then northward along the Great Northern railroad tracks to a junction with the North Trunk sewer.

Pumping would be required at four locations, one at the North Trunk junction and three others along the route of the interceptor. From the junction, sewage would then be conveyed in the North Trunk along Salmon Bay Waterway and through Fort Lawton to the terminus at Shilshole Bay. From this point, a new sewer would be constructed along the north shore of Fort Lawton to the treatment plant at West Point.

Sewage Treatment Plant - West Point System. Under Core Plan B, the treatment plant at this site would provide primary treatment for an average dry weather flow of 118 mgd, with a peak hydraulic capacity of 302 mgd. Influent pumping would be required to lift the sewage from the interceptor into the plant. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks and other

Table 15-4. Description and Estimated Construction Cost, Core Plan B

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Renton System				
B-1	52	130	6,700 ft of 96-in. RC at 0.05%, average cut 23 - 24 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossings	1,123,000
B-2	89	237	1,300 ft of 108-in. RC at 0.095%, average cut 24 ft, difficult wet, includes sheeting and dewatering	271,000
Subtotal, sewers, Renton system				1,394,000
STP-B-1	143	360	Sewage treatment plant, secondary type, includes influent pumping and facilities for screening and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes purchase of 100 acres of land	16,780,000
B-3	143	360	3,100 ft of twin 78-in. RC outfall sewers, average cut 15 ft, difficult wet, includes sheeting, dewatering and railroad crossing	508,000
Total contract cost, Renton system				18,682,000
Engineering and contingencies, 25 per cent				4,670,000
Total construction cost, Renton system				23,352,000
West Point System				
B-4	1.2	2.8	1,500 ft of 18-in. RC at 0.17%, average cut 14 ft, difficult wet, includes imported backfill, repaving and dewatering	47,000
B-5	2.7	6.9	6,100 ft of 27-in. RC at 0.12%, average cut 22 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	373,000
B-6	4.1	10	900 ft of 33-in. RC at 0.09%, average cut 21 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	70,000
B-7	4.1	10	500 ft of twin 14-in. force mains across Duwamish River	22,000
B-8	4.1	10	1,100 ft of 27-in. RC at 0.25%, average cut 10 ft, difficult wet, includes imported backfill, repaving and dewatering	34,000
B-9	6.7	16	3,700 ft of 36-in. RC at 0.18%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	134,000
B-10	13-14	35-37	10,300 ft of 42-in. RC at 0.1% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	654,000
B-11	15	38-39	4,300 ft of 42-in. RC at 0.12% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	191,000
B-12	17	44	7,000 ft of existing 60-in. at 0.05 - 0.055%. Cost is for reconstruction of existing overflow and regulator	18,000
B-13	19	50	3,500 ft of 72-in. RC at 0.035%, average cut 12 ft, difficult wet, includes imported backfill, repaving and dewatering	287,000
B-14	30	80	1,400 ft of 84-in. RC at 0.04%, average cut 12 ft, difficult wet through congested industrial area, includes imported backfill, repaving, sheeting and dewatering	155,000
B-15	34-36	93-97	6,300 ft of 90-in. RC at 0.04%, average cut 14 ft, difficult wet through congested industrial area, includes imported backfill, repaving, sheeting and dewatering	695,000
B-16	37-38	101-102	4,300 ft of 96-in. RC at 0.032%, average cut 16 - 28 ft, difficult wet through congested industrial area and through debris fill containing wood and cinders at site of old Yesler Mill, includes imported backfill, repaving, sheeting, dewatering and underpinning of structures adjacent to 1st Avenue	679,000

Continued on next page

Table 15-4. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
B-17	38	102	10,100 ft of 102-in. RC tunnel at 0.03%, includes allowance of 20% for uncertainties	4,437,000
B-18	39-40	106	5,100 ft of 102-in. RC at 0.025%, average cut 23 - 24 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering, highway crossing and overflow structure on tributary sewer	989,000
B-19	42-43	113-117	7,900 ft of 108-in. RC at 0.021%, average cut 24 - 31 ft, difficult wet, includes 2,000 ft on piles, sheeting, and dewatering	1,495,000
B-20	113	287	1,600 ft of existing 138-in. at 0.035%. Cost is for overflow structure on tributary sewers	10,000
B-21	113-118	288-302	12,000 ft of existing 144-in. at 0.032 - 0.033%. Cost is for overflow structures on tributary sewers	79,000
B-22	118	302	2,600 ft of 144-in. RC at 0.033%, includes construction of 2,500 ft under tidal conditions and dewatering	760,000
Subtotal, sewers, West Point system				11,129,000
PS-B-1	4.1	10	Pumping station, single stage, motor driven, static lift 19 ft, total head at peak flow 30 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering	133,000
PS-B-2	15	38	Pumping station, single stage, motor driven, static lift 16 ft, total head at peak flow 20 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering. To replace existing city of Seattle pumping station having a total installed capacity of 5.0 mgd, includes connections to existing sewers and diversion of present raw sewage outfall to station	298,000
PS-B-3	19	50	Pumping station, single stage, motor driven, static lift 14 ft, total head at peak flow 20 ft, difficult wet, includes sheeting, dewatering and special foundations. Station at site of existing Diagonal Avenue sewage treatment plant, which is to be abandoned, includes connections to existing sewers and diversion of present raw sewage outfall to station	349,000
PS-B-4	43	117	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 32 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	590,000
Subtotal, pumping stations, West Point system				1,370,000
STP-B-2	118	302	Sewage treatment plant, primary type, includes influent pumping and facilities for screening and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes site preparation and 4,000 ft of 12-in. outfall sludge line to a water depth of 400 ft	9,219,000
B-23	118	302	1,900 ft of 120-in. RC effluent outfall, minimum depth, difficult wet, includes dewatering	288,000
B-24	118	302	3,700 ft of twin 78-in. RC submarine outfalls to a water depth of 150 ft, includes diffuser sections over last 440 ft	2,789,000
Subtotal, outfall, West Point system				3,077,000
Total contract cost, West Point system				24,795,000
Engineering and contingencies, 25 per cent				6,199,000
Total construction cost, West Point system				30,994,000
Total construction cost, Core Plan B				54,346,000

See page 357 for footnotes

Table 15-5. Construction Costs of Alternatives for Elliott Bay Site, Core Plan C

Facility	Construction cost,[a] dollars			
	Alternative 1	Alternative 2	Alternative 3	Alternative 4
Sewers	16,478,000	16,478,000	18,954,000	18,954,000
Pumping stations	3,205,000	3,205,000	3,205,000	3,205,000
Sewage treatment plant	23,205,000	33,772,000	21,224,000	31,804,000
Outfall	8,047,000	409,000	7,065,000	920,000
Total	50,935,000	53,864,000	50,448,000	54,883,000

[a]*Includes engineering and contingencies.*

necessary structures and appurtenances. Chlorine contact tanks would not be required since detention times of about 15 minutes would be available in the outfall sewer at the ultimate peak flow of 302 mgd and of about 30 minutes at the ultimate average flow of 118 mgd. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 4,000 feet offshore to a water depth of 400 feet.

Effluent Disposal - West Point System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged through twin 78-inch outfalls approximately 3,700 feet offshore in water at a depth of about 150 feet.

Construction Cost. Estimated construction costs of Core Plan B, including engineering and contingencies, total $54,346,000 (Table 15-4). Of this total, approximately 29 per cent is for intercepting sewers, 3 per cent for pumping stations, 60 per cent for treatment plants and 8 per cent for outfalls.

Core Plan C

Under Core Plan C, sewage of the metropolitan Seattle area would be treated at two plants, one at the Elliott Bay site and the other at West Point. Four alternatives were considered for the Elliott Bay plant, as follows:

Alternative 1 - A primary type treatment plant situated between First and Fourth Avenues South and Brandon and Front Streets, with effluent disposal to Elliott Bay.

Alternative 2 - A complete type treatment plant at the same location as Alternative 1, with effluent disposal to Duwamish River.

Alternative 3 - A primary type treatment plant situated between Airport Way and Fifth Avenue South and Spokane and Dakota Streets, with effluent disposal to Elliott Bay.

Alternative 4 - A complete type treatment plant at the same location as Alternative 3, with effluent disposal to Duwamish River.

Estimated construction costs for the four alternatives (Table 15-5) show little choice between Alternatives 1 and 3. In view, however, of its somewhat lower cost, Alternative 3 was selected for comparison purposes.

Locations of sewers, pumping stations, treatment works and outfalls included under Core Plan C are shown in Fig. 15-5. Descriptions of these facilities are given in Table 15-6.

Intercepting Sewers - Elliott Bay System. South of Fourth Avenue South, the intercepting sewer would be identical to that for Core Plan A. At the intersection of Fourth Avenue South and East Marginal Way, the south interceptor for Core Plan C would turn north along Fourth Avenue South to the treatment plant where it would join the north interceptor. The north interceptor would begin at the intersection of Alaskan Way and Connecticut Street, and would be routed along Alaskan Way, East Marginal Way, Spokane Street and Fourth Avenue South to the treatment plant.

Sewage Treatment Plant - Elliott Bay System. Primary treatment would be provided at the Elliott Bay plant for an ultimate average dry weather flow of 181 mgd, with a peak hydraulic capacity of 457 mgd. Plan units would consist of influent pumps, preaeration and primary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since a detention time in excess of 20 minutes would be available in the outfall sewer, even at the peak flow of 457 mgd. Because of soil conditions at the site, foundation piles would be required. Digested sludge would be hauled away in tank trucks for disposal elsewhere.

Table 15-4 footnotes

See Fig. 15-4 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

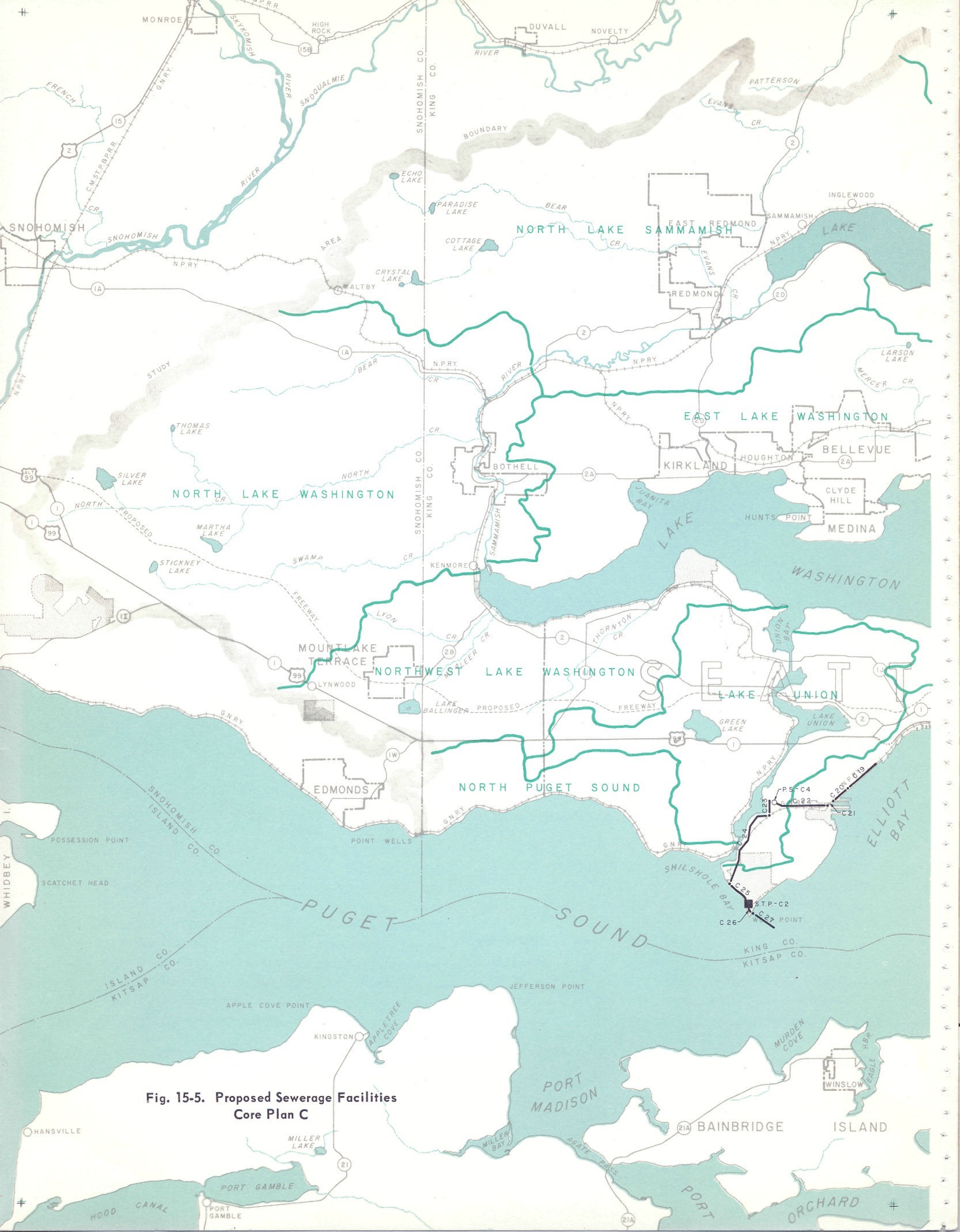
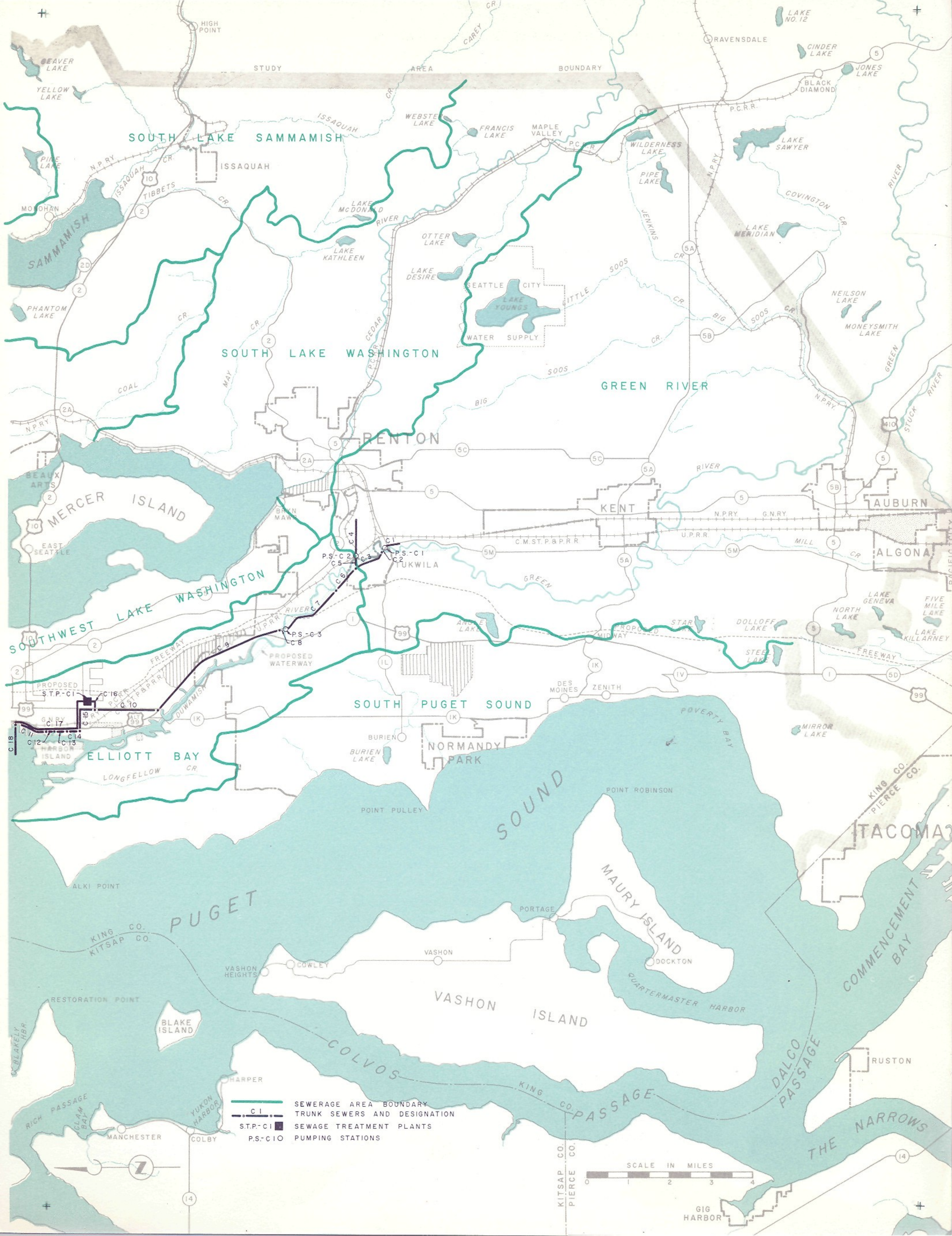


Fig. 15-5. Proposed Sewerage Facilities Core Plan C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SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
STUDY AREA BOUNDARY
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
MERCER ISLAND
ELLIOTT BAY
PUGET SOUND
VASHON ISLAND
MAURY ISLAND
BLAKE ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
TACOMA
RUSTON
NORMANDY PARK
DES MOINES
ZENITH
BURIEN
POINT ROBINSON
POINT PULLEY
ALKI POINT
RESTORATION POINT
VASHON
VASHON HEIGHTS
COWLEY
PORTAGE
DOCKTON
QUARTERMASTER HARBOR
HARPER
MANCHESTER
COLBY
GIG HARBOR
RICH PASSAGE
CLAM BAY
YUKON HARBOR
KING CO.
KITSAP CO.
PIERCE CO.
LAKE YOUNGS
SEATTLE CITY WATER SUPPLY
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
PROPOSED WATERWAY
PROPOSED S.T.P.-C1
P.S.-C1
P.S.-C2
P.S.-C3
C1 C2 C3 C4 C5 C6 C7 C8 C9 C10 C11 C12 C13 C14 C15 C16 C17 C18
SEWERAGE AREA BOUNDARY
C1 TRUNK SEWERS AND DESIGNATION
S.T.P.-C1 SEWAGE TREATMENT PLANTS
P.S.-C10 PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 15-6. Description and Estimated Construction Costs, Core Plan C

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Elliott Bay System				
C-1	52	130	2,300 ft of 96-in. RC at 0.05%, average cut 24 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	395,000
C-2	52	130	600 ft of twin 42-in. force mains across Duwamish River	70,000
C-3	52	130	3,300 ft of 96-in. RC at 0.05%, average cut 15 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	401,000
C-4	90	238	5,400 ft of 114-in. RC at 0.07%, average cut 27 - 31 ft, difficult wet, includes sheeting, dewatering and railroad crossing	1,333,000
C-5	90	238	600 ft of parallel 36-in., 42-in. and 48-in. force mains across Duwamish River	110,000
C-6	143	368	2,400 ft of 114-in. RC at 0.16%, average cut 20 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	446,000
C-7	143-146	370-376	10,900 ft of 120-in. RC at 0.13%, average cut 21 - 32 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	2,948,000
C-8	146	376	500 ft of parallel 48-in. and twin 54-in. force mains across Duwamish River	113,000
C-9	146-160	376-411	18,600 ft of 132-in. RC at 0.075 - 0.086%, average cut 16 - 25 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	5,000,000
C-10	160	411	7,900 ft of 144-in. RC at 0.058%, average cut 27 - 31 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	2,988,000
C-11	2.3	5.9	3,900 ft of 24-in. RC at 0.17%, average cut 15 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	170,000
C-12	2.8	7.2	900 ft of 27-in. RC at 0.14%, average cut 19 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	52,000
C-13	4.4	11	1,400 ft of 33-in. RC at 0.10%, average cut 20 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	91,000
C-14	8.7	24	1,800 ft of 48-in. RC at 0.068%, average cut 21 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	160,000
C-15	19	53	4,700 ft of 54-in. RC at 0.19%, average cut 25 - 31 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	586,000
C-16	178	457	900 ft of 144-in. RC at 0.068%, average cut 34 ft, difficult wet, includes sheeting and dewatering	300,000
Subtotal, sewers, Elliott Bay system				15,163,000
PS-C-1	52	130	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 28 ft, structure about 30 ft below ground, difficult wet, includes sheeting and dewatering	588,000
PS-C-2	90	238	Pumping station, single stage, motor and diesel engine driven, static lift 22 ft, total head at peak flow 34 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	863,000
PS-C-3	146	376	Pumping station, single stage, motor and diesel engine driven, static lift 27 ft, total head at peak flow 37 ft, structure about 35 ft below ground, difficult wet, includes sheeting and dewatering	1,113,000

Continued on next page

Table 15-6. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
Subtotal, pumping stations, Elliott Bay system				2,564,000
STP-C-1	181	457	Sewage treatment plant, primary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes purchase of 45 acres of land and special foundations	16,979,000
C-17	181	457	11,000 ft of 120-in. RC effluent outfall, minimum depth, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	1,993,000
C-18	181	457	3,300 ft of twin 78-in. RC submarine outfall to a water depth of 200 ft, includes diffuser sections over last 350 ft	3,659,000
Subtotal, outfall, Elliott Bay system				5,652,000
Total contract cost, Elliott Bay system				40,358,000
Engineering and contingencies, 25 per cent				10,090,000
Total construction cost, Elliott Bay system				50,448,000
West Point System				
C-19	4.3	11	5,000 ft of 33-in. RC at 0.11%, average cut 10 - 13 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and overflow structures on tributary sewer	268,000
C-20	4.5	12	2,400 ft of 36-in. RC at 0.077%, average cut 16 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering, and highway crossing	152,000
C-21	6.8	19	1,000 ft of 42-in. RC at 0.085%, average cut 18 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	56,000
C-22	7.2-8.0	20-22	6,800 ft of 48-in. RC at 0.047 - 0.055%, average cut 22 - 27 ft, difficult wet, includes 2,000 ft on piles, sheeting and dewatering	538,000
C-23	78	197	1,600 ft of existing 138-in. at 0.035%. Cost is for overflow structure on tributary sewer	10,000
C-24	78-85	197-212	12,000 ft of existing 144-in. at 0.032 - 0.033%. Cost is for overflow structures on tributary sewers	79,000
C-25	85	212	3,000 ft of 120-in. RC at 0.045%, includes construction of 2,500 ft under tidal conditions, and dewatering	670,000
Subtotal, sewers, West Point system				1,773,000
PS-C-4	8.0	22	Pumping station, single stage, motor driven, static lift 23 ft, total head at peak flow 30 ft, structure about 32 ft below ground, difficult wet, includes sheeting and dewatering	208,000
STP-C-2	85	212	Sewage treatment plant, primary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration, and laboratory facilities, includes site preparation and 4,000 ft of 10-in. outfall sludge line to a water depth of 400 ft	6,958,000
C-26	85	212	1,900 ft of 108-in. RC effluent outfall, minimum depth, difficult wet, includes dewatering	247,000
C-27	85	212	3,500 ft of twin 72-in. RC submarine outfalls to a water depth of 120 ft, includes diffuser sections over last 210 ft	2,392,000

Continued on next page

Table 15-6. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Subtotal, outfall, West Point system				2,639,000
Total contract cost, West Point system				11,578,000
Engineering and contingencies, 25 per cent				2,894,000
Total construction cost, West Point system				14,472,000
Total construction cost, Core Plan C				64,920,000

See Fig. 15-5 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

Effluent Disposal - Elliott Bay System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged to Elliott Bay north of Harbor Island approximately 3,300 feet offshore in water at a depth of about 200 feet. The outfall would consist of a land section 11,000 feet in length, extending from the treatment plant along Spokane Street, East Marginal Way and Alaskan Way to Pier 39, and a twin 78-inch submarine section extending 1,400 feet beyond the pier line.

Intercepting Sewers - West Point System. The intercepting sewer for the West Point system would begin at the intersection of Denny Way and Elliott Avenue. From that point, it would be routed along Elliott Avenue to north of Pier 91 and then northward along the Great Northern railroad tracks to a junction with the existing North Trunk sewer of Seattle. Sewage from the new interceptor would be pumped into the North Trunk, and would be conveyed therein, along with sewage from the North Lake Washington, Northwest Lake Washington and Lake Union sewerage areas, to the terminus at Shilshole Bay. From there, a new sewer would be constructed along the north shore of Fort Lawton to the treatment plant at West Point.

Sewage Treatment Plant - West Point System. Except for capacity, the sewage treatment plant at West Point would be identical to that proposed for Core Plan B. Under Core Plan C, the treatment plant would provide primary treatment for an average dry weather flow of 85 mgd, with a peak hydraulic capacity of 212 mgd.

Effluent Disposal - West Point System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged through twin 72-inch outfalls approximately 3,500 feet offshore in water at a depth of about 120 feet.

Construction Cost. As given in Table 15-6, the estimated total construction cost of Core Plan C, including engineering and contingencies, is $64,920,000. Of this total, approximately 33 per cent is for intercepting sewers, 5 per cent for pumping stations, 46 per cent for treatment plants, and 16 per cent for outfalls.

Core Plan D

Of the three treatment plants called for under Core Plan D, one would be at the Renton site, the second at the Elliott Bay site, and the third at the West Point site. Three alternatives were considered for the Elliott Bay site, as follows:

Alternative 1 - A primary type treatment plant situated between First and Fourth Avenues South and Brandon and Front Streets, with effluent disposal to Elliott Bay.

Alternative 2 - A primary type treatment plant situated between Airport Way and Fifth Avenue South and Spokane and Dakota streets, with effluent disposal to Elliott Bay.

Alternative 3 - A primary type treatment plant situated at the site of the existing Diagonal Avenue treatment plant of the city of Seattle, with effluent disposal to Elliott Bay.

Table 15-7. Construction Costs of Alternatives for Elliott Bay Site, Core Plan D

Facility	Construction cost[a], dollars		
	Alternative 1	Alternative 2	Alternative 3
Sewers	4,638,000	4,919,000	3,321,000
Pumping station	166,000	166,000	539,000
Sewage treatment plant	8,706,000	8,081,000	7,259,000
Outfall	2,603,000	1,995,000	2,086,000
Total	16,113,000	15,161,000	13,205,000

[a]*Includes engineering and contingencies.*

Table 15-8. Description and Estimated Construction Costs, Core Plan D

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
Renton System				
D-1 – D-2	–	–	Identical to B-1 – B-2, Core Plan B, see Table 15-4	1,394,000
STP-D-1	143	360	Identical to STP-B-1, Core Plan B, see Table 15-4	16,780,000
D-3	143	360	Identical to B-3, Core Plan B, see Table 15-4	508,000
Total contract cost, Renton system				18,682,000
Engineering and contingencies, 25 per cent				4,670,000
Total construction cost, Renton system				23,352,000
Elliott Bay System				
D-4	1.2	2.8	1,500 ft of 18-in. RC at 0.17%, average cut 14 ft, difficult wet, includes imported backfill, repaving and dewatering	47,000
D-5	2.7	6.9	6,100 ft of 27-in. RC at 0.12%, average cut 22 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	373,000
D-6	4.1	10	900 ft of 33-in. RC at 0.09%, average cut 21 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	70,000
D-7	4.1	10	500 ft of twin 14-in. force mains across Duwamish River	22,000
D-8	4.1	10	1,100 ft of 27-in. RC at 0.25%, average cut 10 ft, difficult wet, includes imported backfill, repaving and dewatering	34,000
D-9	6.7	16	3,700 ft of 36-in. RC at 0.18%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	134,000
D-10	13-14	35-37	10,300 ft of 42-in. RC at 0.1% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	654,000
D-11	15	38-39	4,300 ft of 42-in. RC at 0.12% to parallel existing 42-in. sewer, difficult wet, includes imported backfill, repaving and dewatering	191,000
D-12	17	44	7,000 ft of existing 60-in. RC at 0.05 - 0.055%. Cost is for reconstruction of existing overflow and regulator	18,000
D-13	2.3	5.9	3,900 ft of 24-in. RC at 0.17%, average cut 15 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	170,000
D-14	2.8	7.2	900 ft of 27-in. RC at 0.14%, average cut 19 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	52,000
D-15	4.4	11	1,400 ft of 33-in. RC at 0.10%, average cut 20 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	91,000
D-16	8.7	24	1,800 ft of 48-in. RC at 0.068%, average cut 21 ft, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	160,000
D-17	19	53	3,900 ft of 72-in. RC at 0.04%, average cut 28 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	641,000
Subtotal, sewers, Elliott Bay system				2,657,000
PS-D-1	4.1	10	Pumping station, single stage, motor driven, static lift 19 ft, total head at peak flow 30 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering	133,000
PS-D-2	15	38	Pumping station, single stage, motor driven, static lift 16 ft, total head at peak flow 20 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering. To replace existing city of Seattle pumping station having a total installed capacity of 5.0 mgd, includes connection to existing sewers and diversion of present raw sewage outfall to station	298,000

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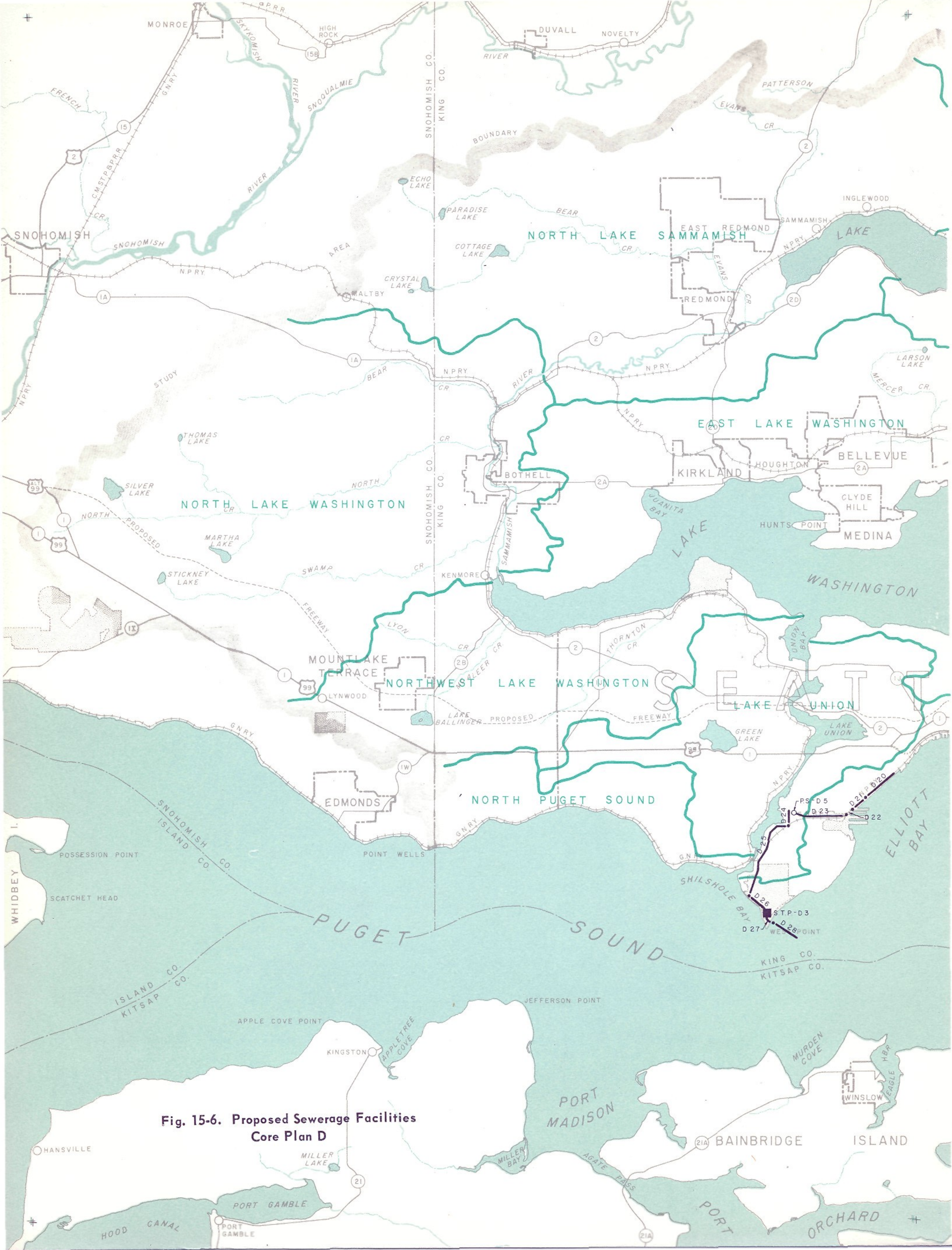


Fig. 15-6. Proposed Sewerage Facilities
Core Plan D


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
MERCER ISLAND
BEAUX ARTS
EAST SEATTLE
ELLIOTT BAY
HARBOR ISLAND
NORMANDY PARK
BURIEN
DES MOINES
ZENITH
TACOMA
RUSTON
GIG HARBOR
MANCHESTER
COLBY
HARPER
VASHON
VASHON HEIGHTS
VASHON ISLAND
MAURY ISLAND
DOCKTON
PORTAGE
BLAKE ISLAND
PUGET SOUND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
QUARTERMASTER HARBOR
POINT ROBINSON
POINT PULLEY
ALKI POINT
RESTORATION POINT
RICH PASSAGE
YUKON HARBOR
KING CO.
KITSAP CO.
PIERCE CO.
STP-D1
STP-D2
PS-D1
PS-D2
PROPOSED WATERWAY
SEWERAGE AREA BOUNDARY
D1 TRUNK SEWERS AND DESIGNATION
S.T.P.-D1 SEWAGE TREATMENT PLANTS
P.S.-D1 PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 15-8. **Continued from Page 363**

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
Subtotal, pumping stations, Elliott Bay system				431,000
STP-D-2	38	102	Sewage treatment plant, primary type, includes influent and effluent pumping and facilities for screening and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities. Plant to be located at site of existing Diagonal Avenue treatment plant, includes purchase of additional land to make total of 25 acres, site preparation, and special foundations	5,807,000
D-18	38	102	11,900 ft of 72-in. RC effluent outfall, minimum depth, difficult wet in congested industrial area, includes imported backfill, repaving, sheeting and dewatering	966,000
D-19	38	102	2,200 ft of 60-in. RC submarine outfall to a water depth of 175 ft, includes diffuser section over last 225 ft	703,000
Subtotal, outfall, Elliott Bay system				1,669,000
Total contract cost, Elliott Bay system				10,564,000
Engineering and contingencies, 25 per cent				2,641,000
Total construction cost, Elliott Bay system				13,205,000
West Point System				
D-20 – D-26	–	–	Identical to C-19 – C-25, Core Plan C, see Table 15-6	1,773,000
PS-D-3	8.0	22	Identical to PS-C-4, Core Plan C, see Table 15-6	208,000
STP-D-3	85	212	Identical to STP-C-2, Core Plan C, see Table 15-6	6,958,000
D-27 – D-28	85	212	Identical to C-26 – C-27, Core Plan C, see Table 15-6	2,639,000
Total contract cost, West Point system				11,578,000
Engineering and contingencies, 25 per cent				2,894,000
Total construction cost, West Point system				14,472,000
Total construction cost, Core Plan D				51,029,000

See Fig. 16-6 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

Estimated construction costs for the three alternatives are presented in Table 15-7. As therein shown, the estimated cost of $13,205,000 for Alternative 3 is $1,956,000 less than the cost for Alternative 2, the next lowest. Alternative 3, therefore, is used in all subsequent discussions relating to Core Plan D.

Locations of sewers, pumping stations, treatment works and outfalls included under Core Plan D are designated in Fig. 15-6. Descriptions of these facilities are given in Table 15-8.

Renton System. All facilities proposed for the Renton system under Core Plan D would be identical to those proposed under Core Plan B for that system.

Intercepting Sewers - Elliott Bay System. Intercepting sewers for the Elliott Bay system would consist of a south branch and a north branch, of which the south branch would be identical to that called for under Core Plan B. The north branch would begin at the intersection of Alaskan Way and Connecticut Street and would be routed south along Alaskan Way and East Marginal Way to the treatment plant.

Sewage Treatment Plant - Elliott Bay System. Except for capacity and location, the sewage treatment plant at Elliott Bay would be identical to that proposed under Core Plan C. Under Core Plan D, the treatment plant would be situated at the site of the existing Diagonal Avenue plant, and would provide primary treatment for an average dry weather flow of 38 mgd, with a peak hydraulic capacity of 102 mgd.

Effluent Disposal - Elliott Bay System. Treated effluent would be chlorinated during the recreational season, May to September, and would be discharged to Elliott Bay north of Harbor Island approximately 2,200 feet offshore in water at a depth of about 175 feet. The outfall would consist of a land section and a submarine section, of which the former would be 11,900 feet in length and would extend from the treatment plant along East Marginal Way and Alaskan Way to Pier 39. The latter would extend 1,200 feet beyond the pier line.

West Point System. All facilities proposed for the West Point system would be identical to those proposed for Core Plan C.

Construction Cost. Total construction costs of Core Plan D, including engineering and contingencies, are estimated to be $51,029,000 (Table 15-8). Of this total, approximately 14 per cent is for intercepting sewers, 2 per cent for pumping stations, 72 per cent for treatment plants, and 12 per cent for outfalls.

COMPARISON OF CORE PLANS

In general, the principal factor to be taken into account when comparing two or more projects to perform a given function is that of cost, both construction and total annual. In some cases, however, particularly where differences in annual costs are not significant, factors other than cost must be considered and evaluated.

Construction Costs

Estimated construction costs for each of the four core plans are summarized in Table 15-9. These estimates, which are based on preliminary layouts of the required facilities, range from $51,029,000 for Core Plan D to $73,932,000 for Core Plan A.

Annual Costs

Assuming that the capital cost can be financed and that all other requirements are fulfilled, the economic merit of any given project is established by determining its total annual cost. This cost is predicated on the anticipated useful life of the required facilities and includes, as set forth in Chapter 13, depreciation, interest on invested capital, and the costs of administration, operation and maintenance.

Depreciation and interest charges used herein are based on the estimated construction costs, as are maintenance and operation costs for sewers and outfalls. In the case of pumping stations and treatment plants, maintenance and operation costs, including those of chlorination, are determined on the basis of estimated average flows during the 70-year design period, 1960 to 2030.

Estimated average annual costs for each of the four core plans (Table 15-10) range from a total of $4,203,000 for Core Plan D to a total of $5,708,000 for Core Plan A. Fixed charges for interest and depreciation vary, of course, in direct proportion to invested capital and range from $3,183,000 per year for Core Plan D to $4,371,000 per year for Core Plan A. From the standpoint of operating costs, Core Plan B is the lowest at $905,000 per year (Table 15-10).

Table 15-9. Comparison of Construction Costs of Core Plans

Facility	Construction cost[a], dollars			
	Core Plan A	Core Plan B	Core Plan C	Core Plan D
Sewers	37,091,000	15,654,000	21,170,000	7,280,000
Pumping stations	6,262,000	1,712,000	3,465,000	798,000
Sewage treatment plants	24,938,000	32,499,000	29,921,000	36,931,000
Outfalls	5,641,000	4,481,000	10,364,000	6,020,000
Total	73,932,000	54,346,000	64,920,000	51,029,000

Plan A proposes one sewage treatment plant of the primary type located at the Government Locks site with discharge of effluent to Puget Sound.

Plan B proposes two sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan C proposes two sewage treatment plants: one of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan D proposes three sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; the second of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the third located at the West Point site with discharge of effluent to Puget Sound.

[a]*From Tables 15-3, 15-4, 15-6, and 15-8; includes allowances for engineering and contingencies.*

Table 15-10. Comparison of Annual Costs of Core Plans

Cost Item	Annual cost, dollars			
	Core Plan A	Core Plan B	Core Plan C	Core Plan D
Fixed costs[a]				
Sewers and outfalls	2,341,000	1,103,000	1,727,000	729,000
Pumping stations and sewage treatment plants	2,030,000	2,225,000	2,172,000	2,454,000
	4,371,000	3,328,000	3,899,000	3,183,000
Maintenance and operation				
Sewers and outfalls[b]	107,000	50,000	79,000	35,000
Pumping stations[c]	314,000	66,000	150,000	14,000
Sewage treatment plants[c]	769,000[d]	634,000[e]	899,000[f]	811,000[f]
Effluent chlorination[c]	147,000[g]	155,000[g]	147,000[g]	160,000[g]
	1,337,000	905,000	1,275,000	1,020,000
Average annual cost[h]	5,708,000	4,233,000	5,174,000	4,203,000

Plan A proposes one sewage treatment plant of the primary type located at the Government Locks site with discharge of effluent to Puget Sound.

Plan B proposes two sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan C proposes two sewage treatment plants: one of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the second of the primary type located at the West Point site with discharge of effluent to Puget Sound.

Plan D proposes three sewage treatment plants: one of the secondary type located at the Renton site with discharge of effluent to the Duwamish River; the second of the primary type located at the Elliott Bay site with discharge of effluent to Elliott Bay; and the third located at the West Point site with discharge of effluent to Puget Sound.

[a] *Includes interest and depreciation calculated by capital recovery method based on five per cent interest and depreciation life of 50 years for sewers and outfalls and of 30 years for pumping stations and sewage treatment plants.*

[b] *0.25 per cent of construction cost.*

[c] *Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[d] *Includes allowance of $10.00 per dry ton for hauling of digested sludge for disposal.*

[e] *Costs reduced for sludge disposal; $2.25 per dry ton at Renton plant (lagooning) and $3.00 per dry ton at West Point plant (disposal to sound).*

[f] *Allowances for sludge disposal as follows: reduction of $2.25 per dry ton at Renton plant (lagooning); increase of $10.00 per dry ton at Elliott Bay plant (hauling); and reduction of $3.00 per dry ton at West Point plant (disposal to sound).*

[g] *Effluent chlorination at plants discharging to Puget Sound or Elliott Bay during period May - September only.*

[h] *During design period, 1960 - 2030.*

Factors Other Than Cost

Assuming the validity of the cost estimates, it appears that the most acceptable plan from an economic standpoint is Core Plan D, under which provision is made for sewage treatment and disposal at three sites, namely, Renton, Elliott Bay and West Point. It will be seen, however, that the annual cost of this plan is but slightly less than that of Core Plan B, which provides for sewage treatment and disposal at two sites, Renton and West Point.

The difference in total annual cost, amounting to $30,000 per year in favor of Core Plan D, is less than one per cent of the average annual cost of that project and undoubtedly is within the accuracy of the cost estimates. In making comparisons, therefore, it is necessary to consider other pertinent factors and to make decisions accordingly. At Seattle, these factors include such items as duplication of operation, interference with business activity during construction, possible future upgrading of disposal requirements, ability to expand facilities in the event that the estimated growth of the tributary area is exceeded, and the simplicity and flexibility of the treatment process. In addition, consideration should be given to possible esthetic objections.

Since both Core Plan B and Core Plan D provide for a sewage treatment plant at the Renton site to serve five of the ten sewerage areas, comparison of the two plans rests on whether one plant (West Point) or two plants (West Point and Elliott Bay) should be provided to serve the remaining five.

Duplication of Operation. In common with many industrial operations, experience with sewage treatment operations has demonstrated that, for a given capacity, more satisfactory over-all performance can be obtained in a single plant rather than in two or more plants. This advantage, though not always evidenced by cost, leads to better quality control, which in sewage treatment plants means a consistently superior effluent. It also means a lesser possibility of plant upsets requiring the bypassing of raw sewage.

Another factor to be considered is the effect of increases in plant maintenance and operation costs. During the past decade, these costs have increased in the order of 25 per cent, primarily due to rising costs of labor and materials. A similar change in the future would increase the indicated maintenance and operation differential from $115,000 per year (Table 15-10) to $143,000 per year in favor of Core Plan B. This change is enough to make the total annual cost of Core Plan D about equal to that of Core Plan B. Further increases would serve to magnify the advantage of Core Plan B.

Interference with Business Activity. The magnitude and complexity of the construction work in the downtown area of Seattle would be less under Core Plan D than Core Plan B. For that reason, interference with existing utilities, such as water mains and underground telephone and electric cables, would be less of a problem. In addition, street excavations would be smaller, construction time would be shorter, and business activity would be less affected.

Quality of Effluent. A possible future upgrading of water quality requirements, though presently unforeseeable, may possibly necessitate a higher degree of treatment at both disposal sites. In such an event, the estimated additional construction cost of providing secondary treatment facilities would be $6,425,000 under Core Plan B. For Core Plan D, the added cost would be $3,045,000 at the Elliott Bay site and $4,925,000 at the West Point site, or a total of $7,970,000. The difference in additional construction costs ($1,545,000) would result in a fixed annual charge of $100,000 more for Core Plan D than for Core Plan B. Similarly, operating costs would increase by $100,000 per year for Core Plan B and $129,000 per year for Core Plan D. If secondary treatment became necessary, the difference in total annual cost would amount to $129,000, thus making Core Plan B superior to Core Plan D from an economic standpoint.

Expansion of Treatment Facilities. Since predictions of future population and industrial development, particularly with respect to their distribution, cannot be regarded as precise, the possibility always exists that presently planned facilities will have to be expanded at some future date to allow for growth in excess of that now estimated.

Future acquisition of additional land at the Elliott Bay site, which is in a densely built-up industrial area, would be both difficult and costly. On the other hand, additional land required at West Point could be obtained by the less costly expedient of filling low-lying tidelands to the north of that site.

Simplicity of Treatment Processes. The simplicity and ease of operation of any sewage treatment process is related directly to the degree of treatment provided. The plants proposed at both the Elliott Bay and West Point sites under Core Plan D, as well as the West Point plant under Core Plan B, would each provide primary treatment. As such, there is no basic difference in the treatment processes. There is, however, a difference in sludge handling requirements between the Elliott Bay and West Point sites.

At the West Point site, digested sludge could be discharged to Puget Sound without producing deleterious effects. This arrangement, in addition to being the least subject to operating complications, would avoid the necessity and cost of truck hauling and would require the least effort on the part of plant attendants.

At the Elliott Bay site, it would not be possible to discharge digested sludge to the bay. This is because current conditions indicate that the sludge would not be adequately dispersed and that, as a consequence, sludge banks would be formed. Accordingly, wet sludge would have to be hauled to remotely located and more suitable disposal points.

Esthetic Considerations. Although conditions at both sites are satisfactory from the standpoint of avoiding nuisance, the isolation provided by the West Point site makes it eminently suitable as a sewage treatment plant location. In contrast, the Elliott Bay site is situated near the center of industrial and commercial activity and is adjacent to a major highway. A treatment plant located at such a site, coupled with the necessity of effluent discharge to Elliott Bay, would possibly meet with serious objection from an esthetic point of view.

SELECTION OF MOST ACCEPTABLE CORE PLAN

As evidenced by the foregoing discussion, it is all too clear that the choice of the most acceptable core plan for central sewerage of the metropolitan area must be more or less arbitrary. With all factors taken into account and evaluated objectively, we are convinced that the most satisfactory plan for central sewerage of the metropolitan Seattle area is that designated herein as Core Plan B. Under this plan, the sewage of the metropolitan area would be conveyed to two treatment plants, one at Renton and the other at

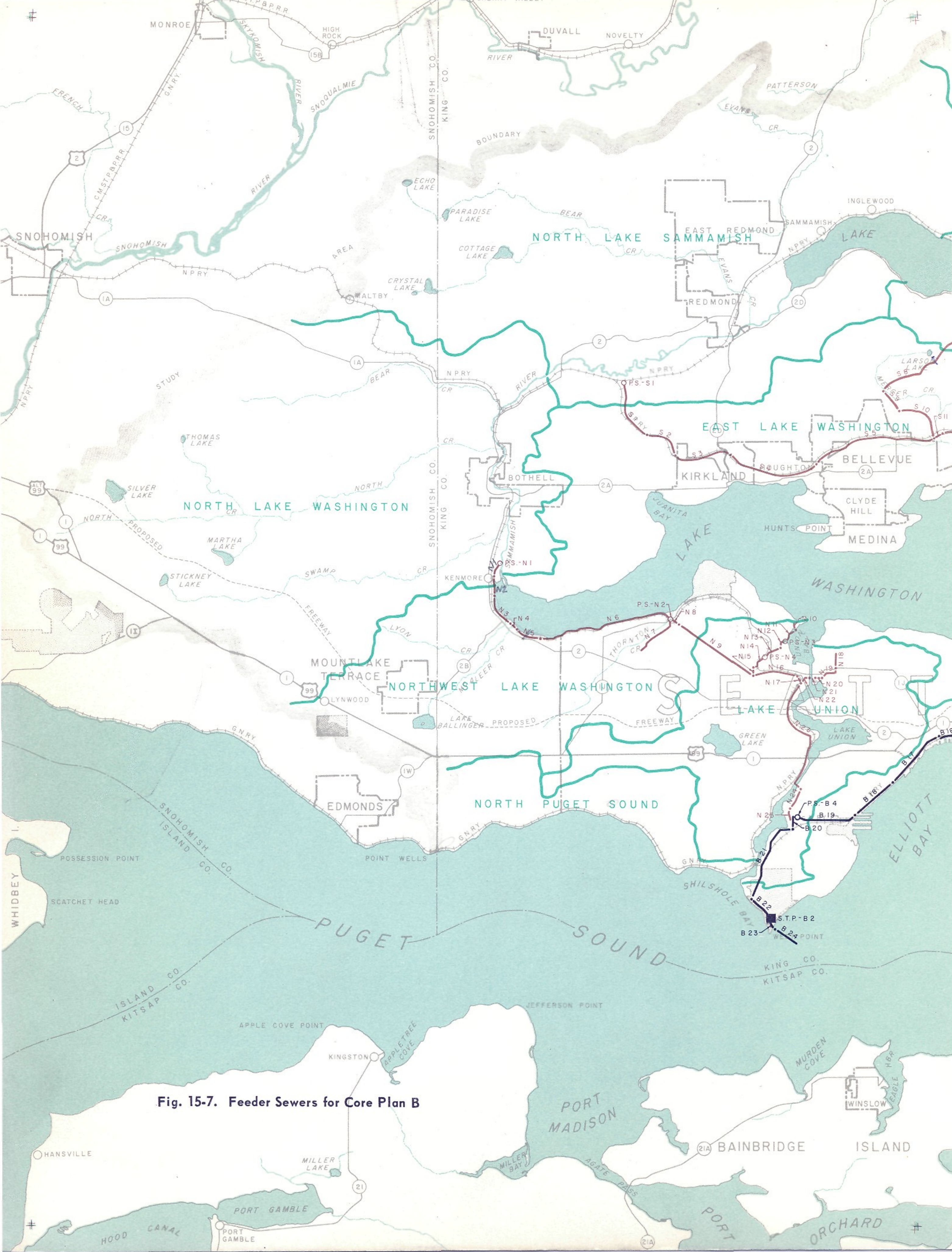
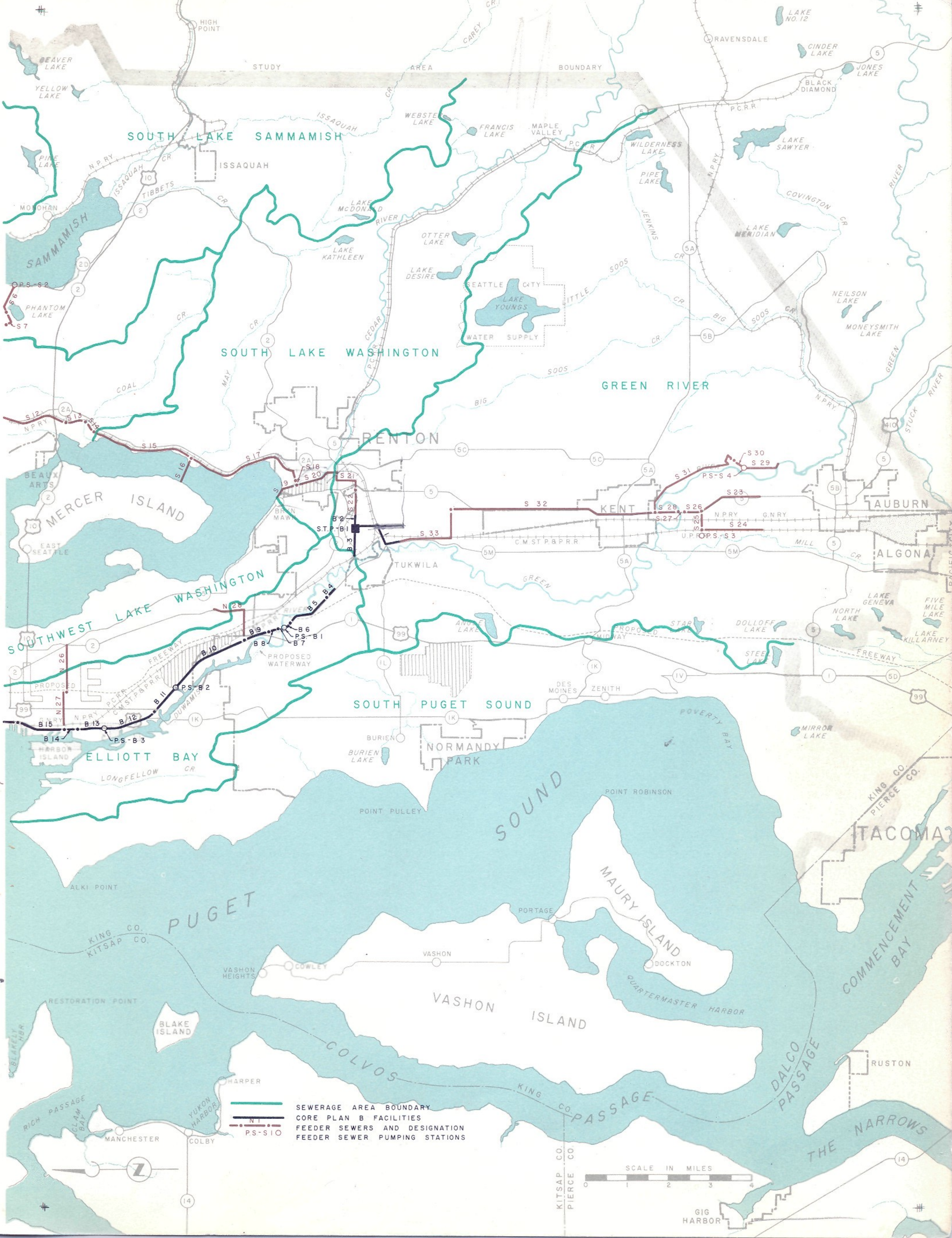


Fig. 15-7. Feeder Sewers for Core Plan B


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
MAPLE VALLEY
RAVENSDALE
BLACK DIAMOND
HIGH POINT
MONOHAN
MERCER ISLAND
BEAUX ARTS
EAST SEATTLE
BRYN MAWR
NORMANDY PARK
BURIEN
DES MOINES
ZENITH
MIDWAY
TACOMA
RUSTON
GIG HARBOR
VASHON ISLAND
VASHON
VASHON HEIGHTS
COWLEY
MAURY ISLAND
PORTAGE
DOCKTON
BLAKE ISLAND
HARPER
COLBY
MANCHESTER
PUGET SOUND
COLVOS PASSAGE
DALCO PASSAGE
THE NARROWS
COMMENCEMENT BAY
QUARTERMASTER HARBOR
POINT ROBINSON
POINT PULLEY
ALKI POINT
RESTORATION POINT
RICH PASSAGE
CLAM BAY
YUKON HARBOR
BLAKELY HBR.
POVERTY BAY
KING CO.
KITSAP CO.
PIERCE CO.
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
PROPOSED WATERWAY
S.T.P.-B1
P.S.-B1
P.S.-B2
P.S.-B3
P.S.-S2
P.S.-S3
P.S.-S4
SEWERAGE AREA BOUNDARY
CORE PLAN B FACILITIES
N1 FEEDER SEWERS AND DESIGNATION
PS-S10 FEEDER SEWER PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

West Point. At the Renton site, complete treatment would be provided and effluent would be disposed of in Duwamish River. At the West Point site, primary treatment would be provided and effluent would be disposed of in Puget Sound.

FEEDER SEWERS FOR CORE PLAN B

Feeder sewers, as defined for purposes of this report, include major trunk sewers and appurtenant pumping stations required to bring sewage from outlying sewerage areas into the core plan system. Since two treatment plants, and hence two points of concentration, are proposed under this plan, feeder sewers are laid out accordingly.

Locations of trunk sewers and pumping stations included in the feeder system for Core Plan B are shown in Fig. 15-7. Descriptions of these facilities appear in Table 15-11.

Renton System

Feeder sewers for the Renton system consist of a south branch and a north branch. The south branch would extend southward through the city of Kent, beyond which it would split into three branches, one to serve the western part of Green River valley and higher areas to the west, the second to serve the central portion of the valley, including the city of Auburn, and the third to serve the eastern portion of the Green River sewerage area. To avoid excessive cuts, two pumping stations would be required on the south branch.

The north branch, or East Side sewer, would extend eastward through the city of Renton and then northward along the Northern Pacific railroad right-of-way through the East Lake Washington sewerage area to the North Lake Sammamish sewerage area. A pumping station would be necessary to pick up the sewage from the latter area.

Sewage collected within the East Lake Washington and South Lake Washington sewerage areas would enter the north branch at various points along its route. In addition, this branch would include a side branch extending generally eastward past Phantom Lake to the South Lake Sammamish sewerage area. A pumping station would be required to serve this area.

Along the railroad right-of-way, the north branch would pass through sections of restricted clearance where the railroad has been constructed in cuts or on fills. In such sections, the sewer would have to be constructed in tunnel or on fill. Additionally, trench support for a Cooper E-60 train load would probably be required where the trench could not be dug farther away from the tracks than about one and one-half times its depth. Because of the possibility of better soil conditions, realignment of a portion of sewer S-20 (Fig. 15-7) to the east of Duwamish River should be investigated during final design.

West Point System

Feeder sewers for the West Point system consist principally of a single main branch which utilizes the existing North Trunk of Seattle to Ravenna Way and East 54th Street. A tunnel would be constructed from this point to a pumping station situated at the mouth of Thornton Creek. Between the pumping station and Sheridan Beach, the route would be offshore in Lake Washington at water depths of 15 to 20 feet. In this section, the sewer would be laid in a trench excavated in the bottom of the lake. The trench would vary from 10 to 19 feet in depth and would be backfilled with rock. From Sheridan Beach, the route would be along the lake front to the North Lake Washington sewerage area, where a pumping station would be necessary to pick up the sewage from that area.

At various points along its route, the main West Point feeder sewer would intercept sewage flows from the Northwest Lake Washington, Lake Union and Southwest Lake Washington sewerage areas. In addition to the main branch, feeder sewers to serve additional areas in the Southwest Lake Washington sewerage area would be provided.

POSSIBLE MODIFICATIONS OF CORE PLAN B

Three basic modifications of Core Plan B were investigated. Of these, two involved possible economies in first cost, while the third involved a determination of the additional cost of providing complete rather than primary treatment should water reclamation be desirable. These modifications are:

1. Conveyance of the sewage, including that from the North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, from the east to the west side of Lake Washington across the lake rather than to the south as proposed under Core Plan B. Three alternatives were considered for this modification, each necessitating a change in the size of the treatment plants proposed at the various sites.
2. Construction of a primary type treatment plant at the Renton site, with an effluent outfall to Puget Sound.
3. Construction of complete type treatment plant at the Government Locks site, with effluent disposal to the Lake Washington Ship Canal above the locks. This would be in lieu of a primary treatment plant at West Point.

Modification of East Side Sewer

By transporting sewage from the east to the west side of Lake Washington, the North Lake Sammamish,

Table 15-11. Description and Estimated Construction Costs, Feeder Sewers for Core Plan B

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Renton System				
S-1	27	76	8,700 ft of twin 36-in. force mains	389,000
S-2	28	79	4,600 ft of 63-in. RC at 0.18%, average cut 9 ft, dry to moderately wet, includes sheeting for train loads where close to tracks	239,000
S-3	34	95	7,500 ft of 78-in. RC at 0.07%, average cut 10 - 15 ft, dry to wet, includes sheeting for train loads where close to tracks	512,000
S-4	36	100	12,300 ft of 78-in. RC at 0.09%, average cut 12 - 16 ft, dry to moderately wet, includes sheeting for train loads where close to tracks and tunneling in areas of restricted clearance	852,000
S-5	38	107	16,500 ft of 78-in. RC at 0.10 - 0.20%, average cut 10 - 19 ft, dry to difficult wet, includes sheeting and dewatering in difficult wet section, sheeting for train loads where close to tracks and tunneling or filling in areas of restricted clearance	1,240,000
S-6	14	42	4,500 ft of twin 30-in. force mains	158,000
S-7	15	44	1,900 ft of 48-in. RC at 0.23%, average cut 10 ft, dry to difficult wet, includes sheeting and dewatering	86,000
S-8	16	45	11,500 ft of 48-in. RC at 0.23 - 1.8%, average cut 12 - 15 ft, dry to wet	459,000
S-9	18	51	3,200 ft of 48-in. RC at 0.32 - 0.45%, average cut 8 - 9 ft, dry to moderately wet, includes imported backfill and repaving	114,000
S-10	19	55	7,200 ft of 57-in. RC at 0.18%, average cut 10 ft, dry to moderately wet, includes railroad crossing	259,000
S-11	58	156	1,600 ft of parallel 36-in, 48-in. and 54-in. inverted siphons, includes inlet and outlet structures	307,000
S-12	60	162	7,600 ft of 78-in. RC at 0.28%, average cut 12 ft, dry to wet, includes sheeting for train loads where close to tracks	549,000
S-13	65	173	5,000 ft of 78-in. RC tunnel at 0.45%, includes allowance of 20% for uncertainties	1,641,000
S-14	67	179	2,400 ft of 78-in. RC at 0.29%, average cut 20 ft, wet, includes tunneling in areas of restricted clearance	378,000
S-15	68	182	13,000 ft of 84-in. RC at 0.27%, average cut 12 ft, wet to difficult wet, includes sheeting and dewatering, sheeting for train loads where close to tracks, and tunneling and filling in areas of restricted clearance	1,735,000
S-16	1.1	2.8	3,500 ft of 12-in. force main across Lake Washington	94,000
S-17	73	195	17,500 ft of 102-in. RC at 0.08%, average cut 15 ft, difficult wet, includes sheeting and dewatering, sheeting for train loads where close to tracks, tunneling and filling in areas of restricted clearance, and piling	2,817,000
S-18	73	195	400 ft of parallel 36-in., 48-in. and 54-in. inverted siphons, includes inlet and outlet structures	77,000
S-19	0.5	1.7	3,800 ft of 15-in. RC at 0.34%, average cut 6 - 8 ft, difficult wet, includes connections to existing sewers at Bryn Mawr - Lake Ridge sewage treatment plant which is to be abandoned	67,000
S-20	73	195	4,900 ft of 102-in. RC at 0.08%, average cut 25 ft, difficult wet, includes sheeting, dewatering and piling	1,020,000
S-21	87	232	4,300 ft of 108-in. RC at 0.085%, average cut 29 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	909,000

Continued on next page

Table 15-11. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
S-22	89	237	3,200 ft of 108-in. RC at 0.095%, average cut 27 ft, difficult wet, includes sheeting, dewatering, and piling	763,000
S-23	5.1	12	7,600 ft of 30-in. RC at 0.19 - 0.5%, average cut 7 - 8 ft, difficult wet, includes imported backfill and repaving	233,000
S-24	7.8	18	7,700 ft of 36-in. RC at 0.16 - 0.22%, average cut 7 - 8 ft, difficult wet	222,000
S-25	11	27	2,500 ft of 42-in. RC at 0.11%, average cut 11 ft, difficult wet, includes imported backfill, repaving, dewatering and railroad crossings	126,000
S-26	14	35	2,500 ft of 42-in. RC at 0.48%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	116,000
S-27	14	35	400 ft of twin 30-in. inverted siphons, includes inlet and outlet structures	35,000
S-28	14	35	3,100 ft of 60-in. RC at 0.045%, average cut 12 - 16 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	249,000
S-29	22	55	5,000 ft of 51-in. RC at 0.26%, average cut 17 - 24 ft, difficult wet, includes sheeting and dewatering	331,000
S-30	22	55	1,200 ft of 36-in. force main	27,000
S-31	22-23	55-58	15,500 ft of 48-in. RC at 0.40%, average cut 11 - 18 ft, dry to difficult wet, includes imported backfill, repaving, sheeting and dewatering	682,000
S-32	37-42	92-105	26,100 ft of 90-in. RC at 0.036 - 0.05%, average cut 12 - 27 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	3,799,000
S-33	45-47	112-118	10,500 ft of 90-in. RC at 0.063 - 0.15%, average cut 15 - 22 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	1,498,000
Subtotal, sewers, Renton system				21,983,000
PS-S-1	27	76	Pumping station, two stage, motor and diesel engine driven, static lift 110 ft, total head at peak flow 140 ft, structure about 25 ft below ground, includes sheeting and dewatering	524,000
PS-S-2	14	42	Pumping station, two stage, motor and diesel engine driven, static lift 200 ft, total head at peak flow 230 ft, structure about 16 ft below ground	342,000
PS-S-3	3.4	8.7	Pumping station, single stage, motor driven, static lift 14 ft, total head at peak flow 20 ft, structure about 20 ft below ground, includes sheeting and dewatering	112,000
PS-S-4	22	55	Pumping station, single stage, motor driven, static lift 56 ft, total head at peak flow 80 ft, structure about 30 ft below ground, includes sheeting and dewatering	357,000
Subtotal, pumping stations, Renton system				1,335,000
Total contract cost, feeder sewers for Renton system				23,318,000
Engineering and contingencies, 25 per cent				5,830,000
Total construction cost, feeder sewers for Renton system				29,148,000
West Point System				
N-1	31	84	2,100 ft of twin 36-in. force mains	94,000
N-2	31	84	2,300 ft of 60-in. RC at 0.26%, average cut 13 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	166,000

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Table 15-11. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
N-3	32	87	4,800 ft of 60-in. RC at 0.28%, average cut 18 - 23 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	496,000
N-4	34	92	1,000 ft of 60-in. RC at 0.31%, average cut 28 ft, difficult wet, includes sheeting and dewatering	104,000
N-5	38	101	3,700 ft of 66-in. RC at 0.24%, average cut 25 ft, difficult wet, includes sheeting and dewatering	375,000
N-6	39	103	16,800 ft of 84-in. RC at 0.067% laid in Lake Washington at water depth of 15-20 ft. Pipe laid in trench 10 - 19 ft deep excavated in lake bottom, includes rock backfill	2,961,000
N-7	10	27	6,100 ft of 42-in. RC at 0.14 - 0.45%, average cut 8 - 10 ft, difficult wet, includes special bedding, railroad crossing and connections to existing sewers at Lake City sewege treatment plant which is to be abandoned	240,000
N-8	49	126	1,100 ft of twin 42-in. force mains, includes railroad crossing	76,000
N-9	49	126	11,600 ft of 90-in. RC tunnel at 0.09%, includes allowance of 20 per cent for uncertainties	4,651,000
N-10	1.5	4.4	1,700 ft of existing 42-in. at 0.06 - 0.07%	Existing
N-11	1.5-2.0	4.4-5.9	2,000 ft of existing 48-in. at 0.045 - 0.07%. Cost is for 1,740,000 gal. holding tank on tributary sewer serving portion of local service area LU-8	160,000
N-12	3.4	9.7	1,300 ft of existing 30-in. at 0.24 - 0.4%	Existing
N-13	3.4	9.7	2,400 ft of existing 42-in. at 0.08%	Existing
N-14	4.3	12	1,700 ft of existing 42-in. at 0.08%	Existing
N-15	5.1	14, 101[c]	900 ft of existing 96-in. at 0.10%	Existing
N-16	57	140, 298[c]	6,900 ft of existing 138-in. at 0.16%	Existing
N-17	57	140.	100 ft of existing 96-in. at 0.074%	Existing
N-18	2.0	5.8	1,200 ft of existing 66-in. at 0.16%	Existing
N-19	5.4	15, 105[c]	2,800 ft of existing 90-in. at 0.17%. Requires partial separation of local service area LU-1	Existing
N-20	6.4	17, 175[c]	800 ft of existing 114-in. at 0.12%	Existing
N-21	6.4	17	500 ft of existing 48-in. inverted siphon	Existing
N-22	6.4	17	1,100 ft of existing 48-in. at 0.15%	Existing
N-23	64-66	159-164	15,300 ft of existing 108-in. at 0.065 - 0.087%. Cost is for overflow from tributary sewers serving local service areas LU-10 and LU-11	126,000
N-24	67	167	400 ft of existing parallel 48-in. and 60-in. inverted siphons	Existing
N-25	70	174	4,600 ft of existing 138-in. at 0.033%	Existing
N-26	4.3	13, 288[c]	6,100 ft of existing 108-in. at 0.4%	Existing
N-27	4.3	13, 288[c]	5,400 ft of existing 100-in. by 150-in. horseshoe section at 0.075%	Existing
N-28	6.7	19	7,400 ft of existing 84-in. at 0.11%, 72-in. at 0.10%, and 60-in. at 0.10 - 0.21%	Existing
Subtotal, sewers, West Point system				9,449,000
PS-N-1	31	84	Pumping station, single stage, motor and diesel engine driven, static lift 33 ft, total head at peak flow 53 ft, structure about 30 ft below ground, includes sheeting, dewatering and special foundations	541,000
PS-N-2	49	126	Pumping station, single stage, motor and diesel engine driven, static lift 75 ft, total head at peak flow 90 ft, structure about 45 feet below ground, includes sheeting and dewatering.	590,000

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Table 15-11. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
PS-N-3	3.4	9.7	Pumping station, single stage, motor driven with gas engine standby, static lift 17 ft, total head at peak flow 22 ft. To replace existing city of Seattle pumping station having 2 pumps and a total installed capacity of 9.0 mgd, includes sheeting, dewatering, connections to existing sewers, and a 4,900,000 gal. holding tank on tributary sewer serving portion of local service area LU-8	528,000
PS-N-4	4.3	12	Pumping station, single stage, motor driven with gas engine standby, static lift 35 ft, total head at peak flow 39 ft. To replace existing city of Seattle pumping station having 2 pumps and a total installed capacity of 10.6 mgd, includes sheeting, dewatering, connections to existing sewers, and a 3,200,000 gal. holding tank on tributary sewer serving portion of local service area LU-9	426,000
Subtotal, pumping stations, West Point system				2,085,000
Total contract cost, feeder sewers for West Point system				11,534,000
Engineering and contingencies, 25 per cent				2,884,000
Total construction cost, feeder sewers for West Point System				14,418,000
Total construction cost, feeder sewers for Core Plan B				43,566,000

See Fig. 15-7 for location of facilities.

[a]Expressed as average dry weather flow and maximum wet weather flow.

[b]Does not include cost of acquiring existing facilities.

[c]Flow resulting from 10-year storm over tributary area.

South Lake Sammamish and East Lake Washington sewerage areas would be tributary to plants located either at the Elliott Bay or the Government Locks sites. Three alternatives were considered in determining the relative economy of this modification as compared with Core Plan B.

Alternative 1. Under this alternative, sewage from the east side of Lake Washington would be concentrated at a point south of Meydenbauer Bay. From that point, it would be conveyed across Lake Washington in parallel twin 42- and twin 54-inch sewers. These lines, each 20,800 feet in length, would be floated in the lake about 50 feet below the water surface and would be provided with necessary buoyancy chambers and concrete anchors. If feasible, the portion of the lines west of Mercer Island, about 7,200 feet in length, could be attached to the floating bridge rather than floated. In either case, the cost of these sewers would approximate the cost of shallow depth submarine outfalls. On the west side of Lake Washington, the sewage would be conveyed by sewer and tunnel to the existing Hanford Street tunnel of the city of Seattle, which has sufficient excess capacity to accommodate expected ultimate flows from the east side. Treatment would be obtained in a primary type plant at the Elliott Bay site, as proposed under Core Plan C, and effluent would be discharged to Elliott Bay. With the removal of east side sewage from the Renton system, this system would serve only the South Lake Washington and Green River sewerage areas.

Alternative 2. Under this alternative, sewage from the east side of Lake Washington would be conveyed to a point south of Meydenbauer Bay, from which an inverted siphon, consisting of parallel 42-, 48- and 60-inch pipes, would be laid across the East Channel of Lake Washington to Mercer Island. Conveyance across Mercer Island would be in tunnel and sewer sections to the west side of the island, from which point Lake Washington would be crossed in parallel twin 42- and twin 54-inch lines. As in Alternative 1, these sections either would be constructed as submerged floating lines or would be attached to the floating bridge. Sewers and treatment and disposal facilities on the west side of Lake Washington and in the Renton system would be identical to those under Alternative 1.

Alternative 3. Sewage concentration and conveyance across Lake Washington under this alternative are identical to Alternative 2. West of the lake, however, sewage from the east side would be conveyed along Elliott Bay waterfront to a treatment plant at the

Government Locks site. Treated effluent would be pumped to Puget Sound for disposal. As in Core Plan A, the treatment plant under this alternative would be situated at the Government Locks site rather than at the West Point site.

Construction Costs. In order to compare the three alternatives with the arrangement called for under Core Plan B, it is necessary to determine the cost of each alternative and to compare these costs with those of the core plan and feeder and service sewer facilities which would be modified or eliminated (Table 15-12). It will be seen that the least costly of the three alternatives, No. 2, would cost $4,496,000 more than the proposed arrangement. It is obvious, therefore, that sewage from the east side of Lake Washington should be conveyed south to the Renton plant rather than across Lake Washington for disposal at either the Elliott Bay site or the Government Locks site.

Modification of Renton Plant

Because both construction and operating costs are less for a primary type treatment plant than for a complete type plant, a modification of Core Plan B was considered whereby primary treatment would be provided at the Renton site and effluent would be disposed of in Puget Sound. Under this modification, the primary effluent, chlorinated during the recreational season, would be pumped through an outfall running westerly to Puget Sound. This outfall would consist of (1) 3,600 feet of parallel 48-, 54-, and 66-inch force mains, (2) 27,000 feet of 132-inch reinforced concrete tunnel, and (3) 3,200 feet of twin 72-inch submarine outfalls to a water depth of about 260 feet.

Estimated construction costs for a primary treatment plant and an effluent pumping station amount to $13,362,000, or $7,613,000 less than that of the secondary plant proposed under Core Plan B. On the other hand, the estimated cost of the effluent outfall for the modification is $18,601,000, or $17,966,000 more than the cost of the Core Plan B outfall, representing a net increase of $10,353,000 for the modification.

As for annual costs, fixed costs for the modified plan would increase by $489,000 per year over Core Plan B, while operating costs would decrease by $36,000 per year. It is apparent, therefore, that secondary treatment at the Renton plant, with effluent disposal to Duwamish River, is a much more satisfactory arrangement than primary treatment with effluent disposal to Puget Sound.

Modification of West Point Plant

As stated earlier (Chapter 10), a need may develop in the future for additional fresh water both to operate the locks on Lake Washington Ship Canal and to prevent salt water intrusion into Lake Washington. In view of that prospect, a study was made to determine the cost of providing secondary treatment at the Government Locks site with effluent disposal to the ship canal above the locks. Estimated construction cost of a secondary plant at the Government Locks site, including land, amounts to $21,631,000, as compared to $11,524,000 for the primary plant at West Point. Under the modification, the sewer along the north shore of Fort Lawton and the outfall to Puget Sound, which are required for the West Point plant, would not be necessary. This means that the cost of these facilities, amounting to $4,940,000, can be deducted from the total construction cost attributable to the Government Locks plant. On that basis, the net increase in construction costs of the Government Locks plant over the West Point plant amounts to $5,167,000.

Fixed costs would be $386,000 per year higher for the Government Locks plant, while operating costs would increase by $287,000 per year, including an allowance of $10 per dry ton for special handling of digested sludge. Hence, the total increase in annual

Table 15-12. Comparison of Construction Costs for Modified East Side Sewer and Core Plan B

Facility	Construction cost,[a] dollars			
	Core Plan B[b]	Modified east side sewer		
		Alternative 1	Alternative 2	Alternative 3
Sewers	28,084,000	29,549,000	26,024,000	38,318,000
Pumping stations	2,225,000	2,485,000	2,485,000	1,825,000
Sewage treatment plants	32,499,000	35,411,000	35,411,000	31,425,000
Outfalls	4,481,000	7,865,000	7,865,000	5,138,000
Total	67,289,000	75,310,000	71,785,000	76,706,000

[a] *Includes engineering and contingencies.*

[b] *Includes, in addition to Core Plan B facilities, the costs of all feeder and service sewers which change because of the possible modification.*

Table 15-13. Apportionment of Construction Costs to Sewerage Areas of Core Plan B and Feeder Sewer Facilities

Sewerage area	Construction cost,[a] dollars				
	Sewers	Pumping stations	Sewage treatment plants	Outfalls	Total
Core Plan B facilities					
North Lake Sammamish	106,000	–	3,964,000	131,000	4,201,000
South Lake Sammamish	60,000	–	2,202,000	74,000	2,336,000
East Lake Washington	92,000	–	3,818,000	115,000	4,025,000
North Lake Washington	321,000	–	3,019,000	1,019,000	4,359,000
Northwest Lake Washington	200,000	–	1,959,000	639,000	2,798,000
South Lake Washington	70,000	–	2,790,000	86,000	2,946,000
Green River	1,411,000	–	8,055,000	225,000	9,691,000
Southwest Lake Washington	4,020,000	554,000	1,414,000	473,000	6,461,000
Elliott Bay	9,024,000	1,133,000	2,927,000	992,000	14,076,000
Lake Union	350,000	25,000	2,351,000	727,000	3,453,000
Total, Core Plan B[b]	15,654,000	1,712,000	32,499,000	4,481,000	54,346,000
Feeder sewer facilities					
North Lake Sammamish	8,371,000	655,000	–	–	9,026,000
South Lake Sammamish	3,891,000	428,000	–	–	4,319,000
East Lake Washington	5,097,000	–	–	–	5,097,000
North Lake Washington	8,200,000	1,145,000	–	–	9,345,000
Northwest Lake Washington	3,254,000	405,000	–	–	3,659,000
South Lake Washington	835,000	–	–	–	835,000
Green River	9,169,000	586,000	–	–	9,755,000
Southwest Lake Washington	116,000	–	–	–	116,000
Elliott Bay	–	–	–	–	–
Lake Union	358,000	1,056,000	–	–	1,414,000
Total, feeder sewers[c]	39,291,000	4,275,000	–	–	43,566,000

[a]*Includes engineering and contingencies.*

[b]*From Table 15-4.*

[c]*From Table 15-11.*

costs amounts to $673,000. Based on an average flow of 99 mgd to the plant during the period 1960-2030, this means that the average cost of providing additional treatment would amount to $18.60 per million gallons, or about $6.20 per acre-foot.

Since the purpose of the present survey is to determine the most economical solution to the sewerage problems of the metropolitan area, it obviously is not within the province of the report to recommend any action in regard to the reclamation modification. Nonetheless, if the needs of the community are such that the additional cost could be justified, then adoption of the modification would be perfectly feasible. In the event of such a program, the additional cost of construction and operation should not be charged against the sewage collection, treatment and disposal function but rather against the agency requiring the reclaimed water.

SEPARATE PROJECTS FOR INDIVIDUAL SEWERAGE AREAS

To determine whether central sewerage would provide the most economical medium for the collection, treatment and disposal of the sewage of the metropolitan Seattle area, it was necessary to compare the cost thereof with the cost of separate or independent projects to serve individual sewerage areas. In each such area, therefore, all independent projects considered to be economical and otherwise feasible were investigated and were compared with participation in the central sewerage project. In some areas, the number of projects was limited by geographical or other conditions and the choice was relatively simple. In others, detailed analyses had to be made of each of several possibilities.

Apportionment of Core Plan B and Feeder Sewer Costs

To determine the relative economy of the central sewerage project as compared to independent projects for each sewerage area, it is necessary to apportion the costs, both construction and annual, of Core Plan B and its feeder sewers to each of the ten tributary areas.

Construction Costs. Apportionment to each sewerage area of the construction costs for central sewerage is based on ultimate flow or capacity requirements. For example, assume the cost of a certain section of trunk sewer is $100,000 and that the design capacity of 10 mgd includes capacity for a flow of 5 mgd from one sewerage area, 3 mgd from a second area and 2 mgd from a third area. On that basis, the total construction cost would be apportioned as follows: $50,000 to the first area, $30,000 to the second and $20,000 to the third.

Apportionment to the tributary sewerage areas of estimated construction costs for Core Plan B facilities and feeder sewers is presented in Table 15-13.

Annual Costs. Apportionment to each sewerage area of fixed costs for the core plan system, including feeder sewers, is based on the apportioned construction costs given in Table 15-13, as are maintenance and operation costs of sewers and outfalls. Apportionment of maintenance and operation costs of pumping stations and treatment plants, including chlorination, is based on the ratio which the average flow from a sewerage area during the design period bears to the total average flow in the particular facility during that period. Average flows, both total and incremental, from each sewerage area were determined from Table 15-1.

Apportionment of estimated annual costs for Core Plan B facilities and feeder sewers is presented in Table 15-14.

North Lake Sammamish, South Lake Sammamish and East Lake Washington Sewerage Areas

Since the sewage generated in these three areas is tributary to a single trunk under the core plan system,

Table 15-14. Apportionment of Annual Costs to Sewerage Areas of Core Plan B and Feeder Sewer Facilities

Cost Item	Total annual cost, $1,000	Sewerage area apportionment, $1,000									
		NLS	SLS	ELW	NLW	NWW	SLW	GR	SWW	EB	LU
Core Plan B facilities											
Fixed costs[a]											
Sewers and outfalls	1,103	13	7	11	73	46	9	90	246	549	59
Pumping stations and treatment plants	2,225	258	143	248	196	127	182	524	128	264	155
	3,328	271	150	259	269	173	191	614	374	813	214
Maintenance and operation											
Sewers[b]	50	1	–	1	3	2	–	4	11	25	3
Pumping stations[c]	66	–	–	–	–	–	–	–	21	44	1
Sewage treatment plants[c]	634	65	31	73	61	51	46	116	40	81	70
Effluent chlorination[c]	155	15	7	17	16	13	11	26	11	21	18
	905	81	38	91	80	66	57	146	83	171	92
Average annual cost,[d] Core Plan B	4,233	352	188	350	349	239	248	760	457	984	306
Feeder sewer facilities											
Fixed costs[a]											
Sewers	2,152	459	213	279	449	178	46	502	6	–	20
Pumping stations	278	43	28	–	74	26	–	38	–	–	69
	2,430	502	241	279	523	204	46	540	6	–	89
Maintenance and operation											
Sewers[b]	98	21	10	13	21	8	2	23	–	–	–
Pumping stations[c]	195	42	38	–	54	28	–	27	–	–	6
	293	63	48	13	75	36	2	50	–	–	6
Average annual cost,[d] feeder sewers	2,723	565	289	292	598	240	48	590	6	–	95

NLS - North Lake Sammamish; SLS - South Lake Sammamish; ELW - East Lake Washington; NLW - North Lake Washington; NWW - Northwest Lake Washington; SLW - South Lake Washington; GR - Green River; SWW - Southwest Lake Washington; EB - Elliott Bay; LU - Lake Union.

[a]*Includes interest and depreciation calculated by the capital recovery method based on 5 per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[b]*0.25 per cent of construction costs.*

[c]*Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[d]*For design period, 1960 - 2030.*

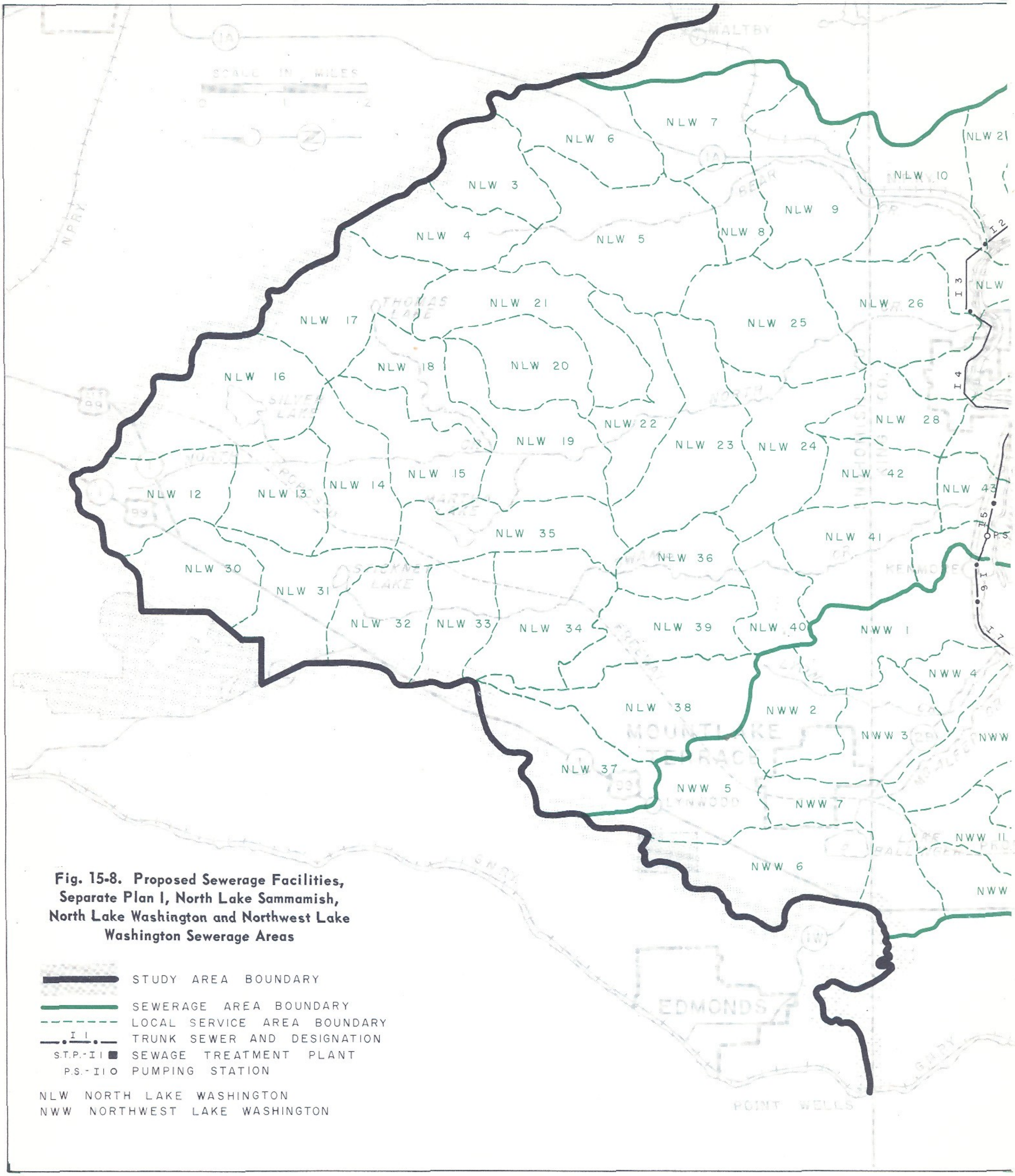

Fig. 15-8. Proposed Sewerage Facilities, Separate Plan I, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

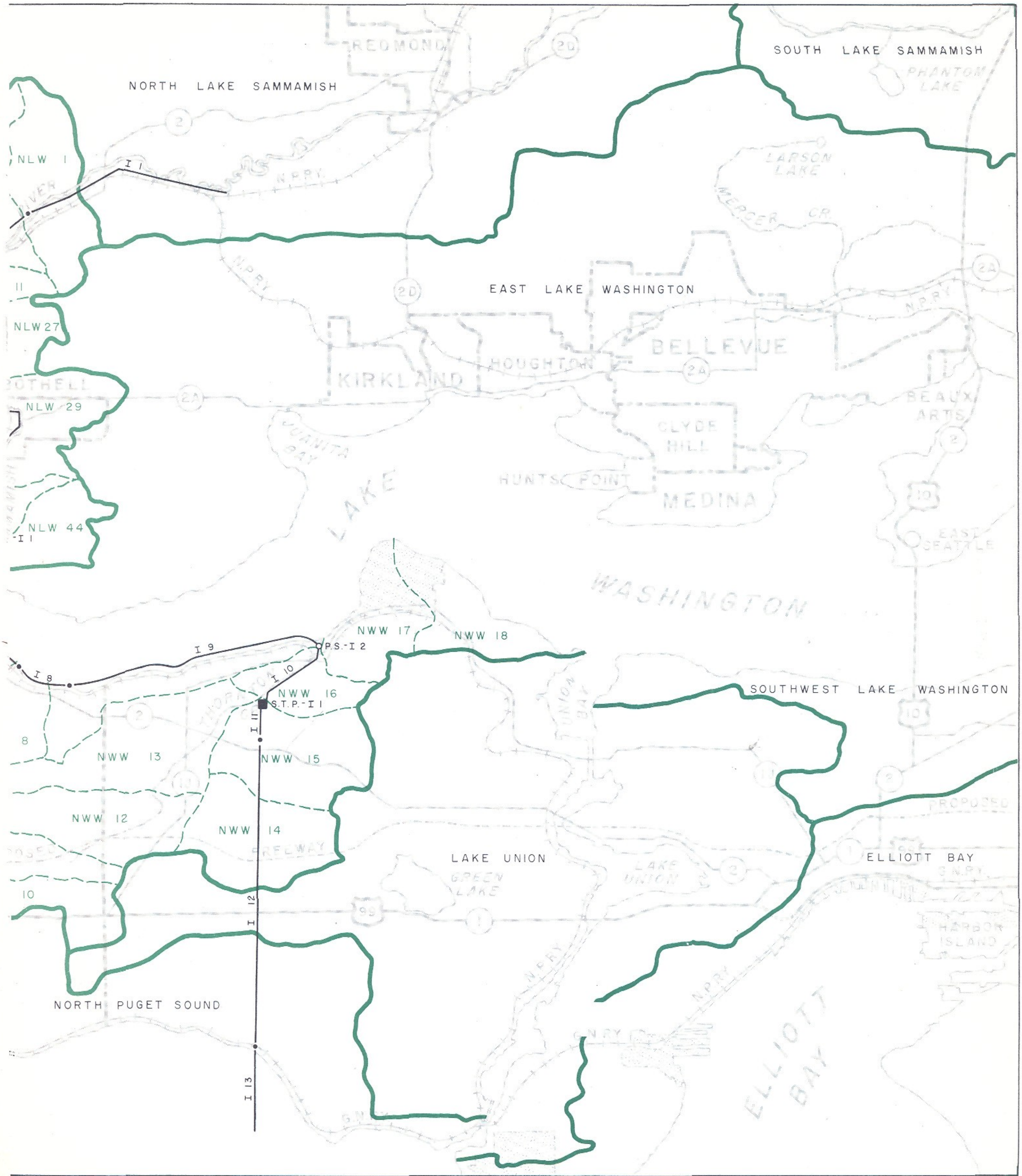
NORTH LAKE SAMMAMISH
SOUTH LAKE SAMMAMISH
REDMOND
PHANTOM LAKE
LARSON LAKE
MERCER CR.
NLW 1
I 1
N.P.R.Y.
NLW 27
NLW 29
NLW 44
EAST LAKE WASHINGTON
KIRKLAND
HOUGHTON
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
BEAUX ARTS
EAST SEATTLE
JUANITA BAY
LAKE
WASHINGTON
I 9
I 8
P.S.-I 2
I 10
S.T.P.-I 1
NWW 17
NWW 18
NWW 16
NWW 13
NWW 15
NWW 12
NWW 14
I 11
I 12
I 13
FREEWAY
UNION BAY
SOUTHWEST LAKE WASHINGTON
LAKE UNION
GREEN LAKE
ELLIOTT BAY
HARBOR ISLAND
NORTH PUGET SOUND
PROPOSED
ELLIOTT BAY

Table 15-15. Description and Estimated Construction Costs, Separate Plan I, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
I-1	27	75-76	13,000 ft of 84-in. RC at 0.035 - 0.036%, average cut 21 - 26 ft, difficult wet, includes sheeting, dewatering and river crossing	1,678,000
I-2	28	78-79	3,900 ft of 84-in. RC at 0.038 - 0.040%, average cut 25 - 27 ft, difficult wet, includes sheeting, dewatering and railroad crossing	524,000
I-3	35-36	96-97	6,900 ft of 84-in. RC at 0.06%, average cut 13 - 21 ft, difficult wet, includes sheeting and dewatering	669,000
I-4	47-49	125-130	18,100 ft of 102-in. RC at 0.028%, average cut 13 - 18 ft, difficult wet, includes sheeting, dewatering and railroad and highway crossings	2,267,000
I-5	58	152	2,100 ft of 102-in. RC at 0.052%, average cut 18 ft, difficult wet, includes sheeting and dewatering	274,000
I-6	58	154	2,100 ft of twin 42-in. force mains	112,000
I-7	58-59	154,157	7,100 ft of 78-in. RC at 0.23%, average cut 16 - 24 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	892,000
I-8	61-65	161-170	4,700 ft of 84-in. RC at 0.23%, average cut 25 - 27 ft, difficult wet, includes sheeting and dewatering	564,000
I-9	65-66	170-172	16,800 ft of 102-in. RC at 0.067%, laid in Lake Washington at water depth of 15 - 20 ft. Pipe laid in trench 10 - 19 feet deep excavated in lake bottom. Cost includes rock backfill	3,797,000
I-10	68	177	5,100 ft of parallel 48-in. and 54-in. force mains	342,000
Subtotal, sewers, Plan I				11,119,000
PS-I-1	58	154	Pumping station, single stage, motor and diesel engine driven, static lift 28 ft, total head at peak flow 55 ft, structure about 25 ft below ground, includes special foundations, sheeting and dewatering	778,000
PS-I-2	68	177	Pumping station, single stage, motor and diesel engine driven, static lift 70 ft, total head at peak flow 100 ft, structure about 45 ft below ground, includes sheeting and dewatering	722,000
Subtotal, pumping stations, Plan I				1,500,000
STP	76	193	Sewage treatment plant, secondary type, includes effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	9,539,000
I-11	76	193	2,400 ft of 72-in. force main, effluent outfall	118,000
I-12	76	193	20,500 ft of 102-in. RC tunnel, effluent outfall, includes allowance of 20% for uncertainties	9,840,000
I-13	76	193	2,800 ft of 96-in. RC submarine outfall to a water depth of 120 ft, includes diffuser sections over last 210 ft	1,948,000
Subtotal, outfall, Plan I				11,906,000
Total contract cost, Plan I				34,064,000
Engineering and contingencies, 25 per cent				8,516,000
Total construction cost, Plan I				42,580,000

See Fig. 15-8 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

it is appropriate to consider them as a unit in developing independent projects. For reasons set forth later, the North Lake Sammamish area is also included with the North Lake Washington and Northwest Lake Washington sewerage areas in another analysis.

Alternative projects available for providing independent sewage collection, treatment and disposal facilities for the three areas include: (1) conveyance of all the sewage to a treatment plant near Yarrow Bay, with effluent pumped across Lake Washington for disposal either in Lake Union or Shilshole Bay; (2) construction of separate plants in each sewerage area, with effluent therefrom pumped to a central point near Yarrow Bay for disposal as under (1); and (3) various combinations of these two alternatives.

Along the route of the Lake Washington crossing,

Table 15-16. Description and Estimated Construction Costs, Separate Plan II, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
North Lake Washington and Northwest Lake Washington Sewerage Areas[b]				
II-1 - II-6	–	–	Same as N-1 - N-6, Core Plan B feeder sewers, see Table 15-11	4,196,000
II-7	40	103	5,100 ft of parallel 36-in. and 48-in. force mains	268,000
Subtotal, sewers				4,464,000
PS-II-1	31	84	Same as PS-N-1, Core Plan B feeder sewers, see Table 15-11	541,000
PS-II-2	41	103	Pumping station, motor and diesel engine driven, single stage, static lift 70 ft, total head at peak flow 100 ft, structure about 45 ft below ground, includes sheeting and dewatering	526,000
Subtotal, pumping stations				1,067,000
STP	49	121	Sewage treatment plant, secondary type, includes effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	6,722,000
II-8	49	121	2,400 ft of 54-in. force main, effluent outfall	83,000
II-9	49	121	20,500 ft of 84-in. RC tunnel at 0.13%, effluent outfall, includes allowance of 20% for uncertainties	7,725,000
II-10	49	121	2,500 ft of 78-in. RC submarine outfall to water depth of 100 ft, includes diffuser section over last 125 ft	799,000
Subtotal, outfalls				8,607,000
Total contract cost, Plan II, North Lake Washington and Northwest Lake Washington Sewerage Areas				20,860,000
Engineering and contingencies, 25 per cent				5,215,000
Total construction cost, Plan II, North Lake Washington and Northwest Lake Washington Sewerage Areas				26,075,000
North Lake Sammamish Sewerage Area[c]				
–	–	–	Sewers, same as Core Plan B and feeder sewers, see Table 15-13	8,477,000[d]
–	–	–	Pumping stations, same as Core Plan B and feeder sewers, see Table 15-13	655,000[d]
–	–	–	Sewage treatment plant, same as Core Plan B, see Table 15-13	3,964,000[d]
–	–	–	Outfall, same as Core Plan B, see Table 15-13	131,000[d]
Total construction cost, Plan II, North Lake Sammamish Sewerage Area				13,227,000[d]
Total construction cost, Plan II				39,302,000

[a] *Expressed as average dry weather flow and maximum wet weather flow.*

[b] *See Fig. 15-9 for location of facilities.*

[c] *See Figs. 15-4 and 15-7 for location of facilities.*

[d] *Apportioned cost to North Lake Sammamish Sewerage Area; includes engineering and contingencies.*

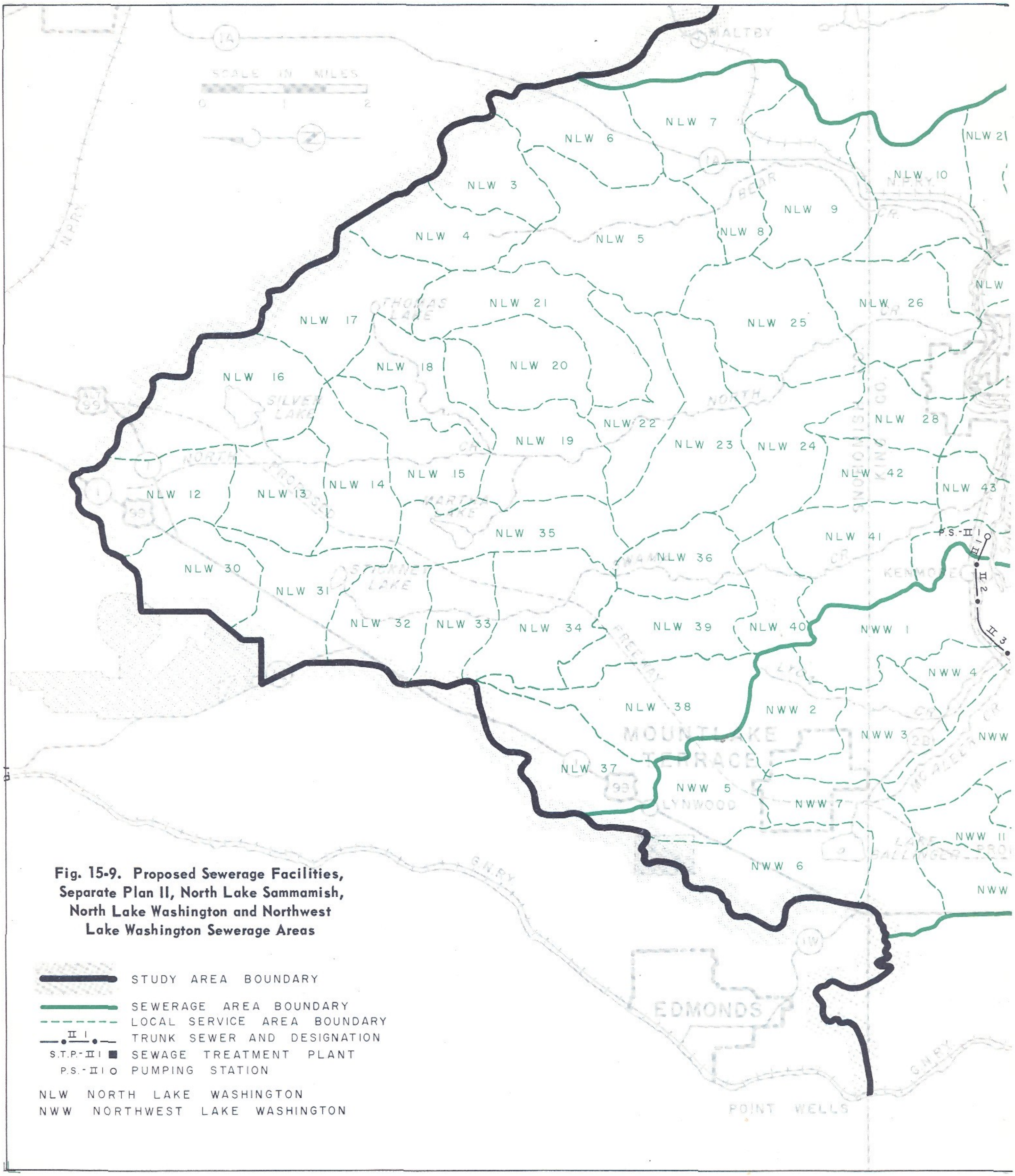

Fig. 15-9. Proposed Sewerage Facilities, Separate Plan II, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

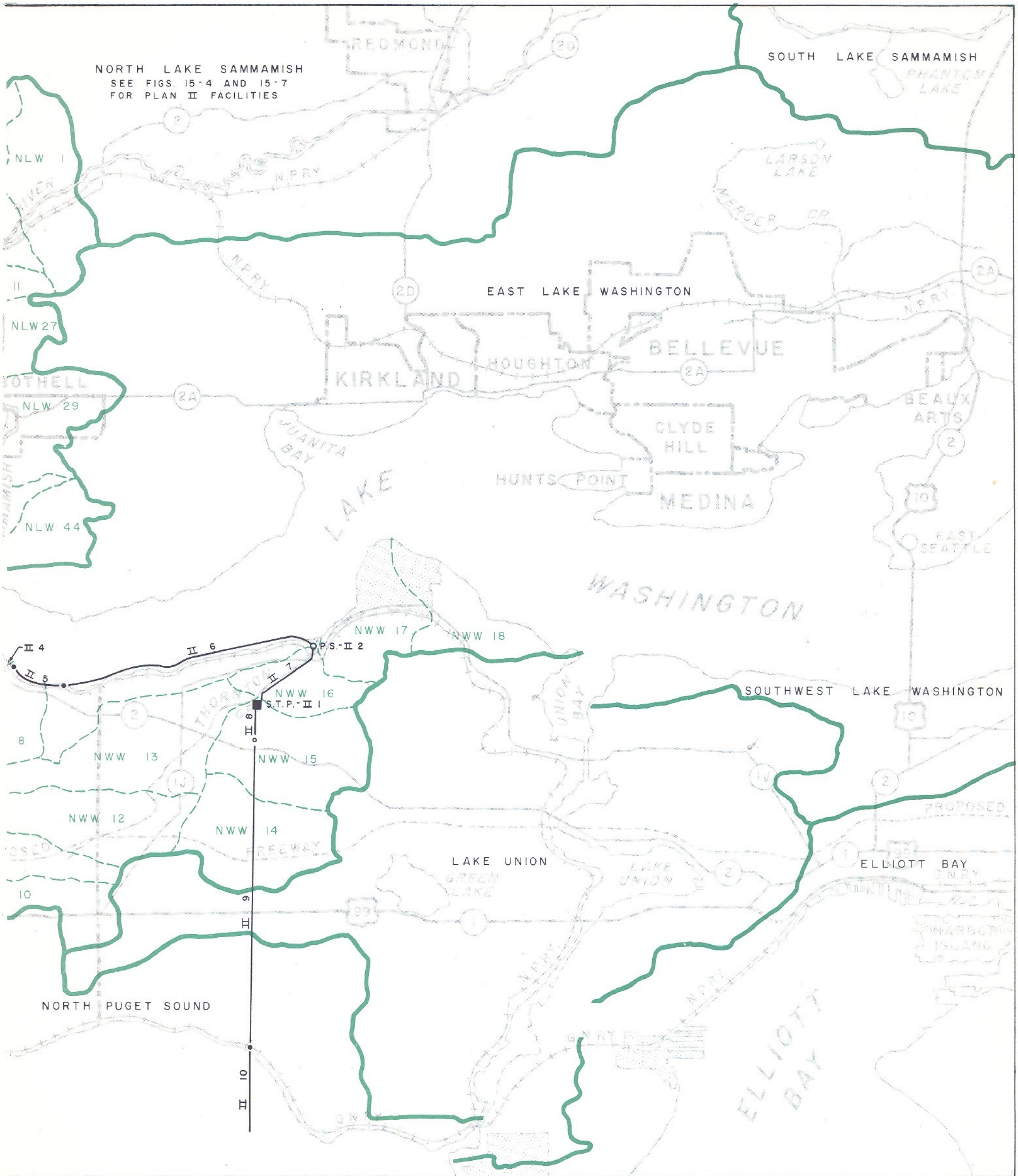
NORTH LAKE SAMMAMISH
SEE FIGS. 15-4 AND 15-7
FOR PLAN II FACILITIES
SOUTH LAKE SAMMAMISH
REDMOND
PHANTOM LAKE
LARSON LAKE
MERCER CR
N.P.RY.
NLW 1
NLW 27
NLW 29
NLW 44
BOTHELL
EAST LAKE WASHINGTON
KIRKLAND
HOUGHTON
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
BEAUX ARTS
EAST SEATTLE
JUANITA BAY
LAKE
WASHINGTON
NWW 17
NWW 18
P.S.-II 2
II 4
II 5
II 6
II 7
II 8
II 9
II 10
S.T.P.-II 1
NWW 16
NWW 13
NWW 15
NWW 12
NWW 14
THORNTON
FREEWAY
UNION BAY
SOUTHWEST LAKE WASHINGTON
LAKE UNION
GREEN LAKE
ELLIOTT BAY
PROPOSED
HARBOR ISLAND
NORTH PUGET SOUND
G.N.RY.

Table 15-17. Description and Estimated Construction Costs, Separate Plan III, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
Northwest Lake Washington Sewerage Area[b]				
III-1	1.1	2.7	4,800 ft of 18-in. RC at 0.16%, average cut 9 - 11 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	134,000
III-2	3.0	7.4	1,000 ft of 24-in. RC at 0.27%, average cut 13 ft, difficult wet, includes sheeting and dewatering	33,000
III-3	6.7	17	3,700 ft of 33-in. RC at 0.22%, average cut 12 ft, difficult wet, includes sheeting and dewatering	136,000
III-4	7.0-7.5	18-19	16,800 ft of 42-in. RC at 0.085%, laid in Lake Washington at water depth of 15-20 ft. Pipe laid in trench 11 - 23 ft deep excavated in lake bottom, includes rock backfill	1,518,000
III-5	9.1	24	5,100 ft of parallel 20-in. and 30-in. force mains	148,000
Subtotal, sewers				1,969,000
PS-III-1	9.1	24	Pumping station, single stage, motor driven with gas engine standby, static lift 75 ft, total head at peak flow 100 ft, structure about 45 ft below ground, includes sheeting and dewatering	221,000
STP-III-1	18	48	Sewage treatment plant, secondary type, includes effluent pumping and facilities for screenings and grit removal, aeration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination	1,988,000
III-6	18	48	2,400 ft of 36-in. force main, effluent outfall	54,000
III-7	18	48	20,500 ft of 72-in. RC tunnel at 0.045%, effluent outfall, includes allowance of 20% for uncertainties	6,150,000
III-8	18	48	1,900 ft of 60-in. RC submarine outfall to water depth of 50 ft, includes diffuser section over last 40 ft	307,000
Subtotal, outfall				6,511,000
Total contract cost, Plan III, Northwest Lake Washington Sewerage Area				10,689,000
Engineering and contingencies, 25 per cent				2,672,000
Total construction cost, Plan III, Northwest Lake Washington Sewerage Area				13,361,000
North Lake Washington Sewerage Area[b]				
STP-III-2	31	78	Sewage treatment plant, secondary type, includes influent and effluent pumping and facilities for screenings and grit removal, pre-aeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes special foundations	5,564,000
III-9	31	78	2,700 ft of 42-in. force main, effluent outfall, includes railroad and highway crossing	72,000
III-10	31	78	32,800 ft of 78-in. RC tunnel at 0.075%, effluent outfall, includes allowance of 20% for uncertainties	11,021,000
III-11	31	78	800 ft of 78-in. RC at 0.055%, effluent outfall, minimum depth, difficult wet, includes sheeting, dewatering and railroad crossing	75,000
III-12	31	78	1,000 ft of 66-in. RC submarine outfall to water depth of 75 ft, includes diffuser section over last 125 ft	257,000
Subtotal, outfall				11,425,000

Continued on next page

Table 15-17. Continued

Facility	Design flow,[a] mgd Average DWF	Design flow,[a] mgd Maximum WWF	Description	Construction cost, dollars
Total contract cost, Plan III, North Lake Washington Sewerage Area				16,989,000
Engineering and contingencies, 25 per cent				4,247,000
Total construction cost, Plan III, North Lake Washington Sewerage Area				21,236,000
North Lake Sammamish Sewerage Area[c]				
–	–	–	Sewers, same as Core Plan B and feeder sewers, see Table 15-13	8,477,000[d]
–	–	–	Pumping stations, same as Core Plan B and feeder sewers, see Table 15-13	655,000[d]
–	–	–	Sewage treatment plant, same as Core Plan B, see Table 15-13	3,964,000[d]
–	–	–	Outfall, same as Core Plan B, see Table 15-13	131,000[d]
Total construction cost, Plan III, North Lake Sammamish Sewerage Area				13,227,000[d]
Total construction cost, Plan III				47,824,000

[a] *Expressed as average dry weather flow and maximum wet weather flow.*

[b] *See Fig. 15-10 for location of facilities.*

[c] *See Figs. 15-4 and 15-7 for location of facilities.*

[d] *Apportioned cost to North Lake Sammamish Sewerage Area; includes engineering and contingencies.*

over 6,000 feet of the effluent outfall would be in water having a depth between 180 and 210 feet. Additionally, the lake bottom is composed of up to 40 feet of soft, almost fluid, organic peat-like sediments which overlie a stratum of compressible glacial blue clay having a depth in excess of 100 feet (Chapter 3). Because of these conditions, the lake crossing would be both extremely difficult and costly to construct. Moreover, the permanence of such a line could not be guaranteed.

Consideration was given also to the possibility of constructing a floating submerged pipeline. This idea was discarded, however, because it is unlikely that approval could be obtained for such an undertaking in this part of the lake. Similarly, the possibility of suspending the line on a proposed new bridge along this route was discarded, since it is highly improbable that such a bridge will be constructed by the time the line would be required.

In view of the problems just mentioned, detailed design and cost estimates were not prepared. A rough estimate, however, indicates that the lake crossing alone would cost in excess of $20 million, or almost the total cost to the three areas for participation in the core plan system. It is evident, therefore, that the interests of the South Lake Sammamish and East Lake Washington sewerage areas will best be served by sewering south to the Renton plant as proposed under Core Plan B. Additional alternatives were considered for the North Lake Sammamish sewerage area and are discussed in the following section.

North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Since the North Lake Sammamish sewerage area can sewer into and through the North Lake Washington sewerage area by gravity, these areas, together with Northwest Lake Washington, should be considered jointly with respect to possible separate sewerage projects. Basically, the alternatives available are the construction either of a single or of several treatment plants, all with effluent disposal to Puget Sound. As indicated in Chapter 11, disposal conditions along the shore of the sound in the northerly part of the study area require that secondary treatment be provided if large volumes of effluent are to be discharged. On that basis, five separate plans were developed as follows:

Plan I - Conveyance of the sewage from all three areas to a secondary type treatment plant at the site of the existing Lake City treatment plant, with effluent disposal to Puget Sound.

Plan II - Conveyance of the sewage from the North Lake Washington and Northwest Lake Washington sewerage areas to a secondary type treatment plant at the site of the Lake City treatment plant, with effluent disposal to Puget Sound. Under this alternative, the North Lake Sammamish sewerage area would be served by the core plan.

Plan III - Construction of separate secondary type treatment plants in the North Lake Washington and Northwest Lake Washington sewerage areas, with each plant discharging its effluent separately to Puget

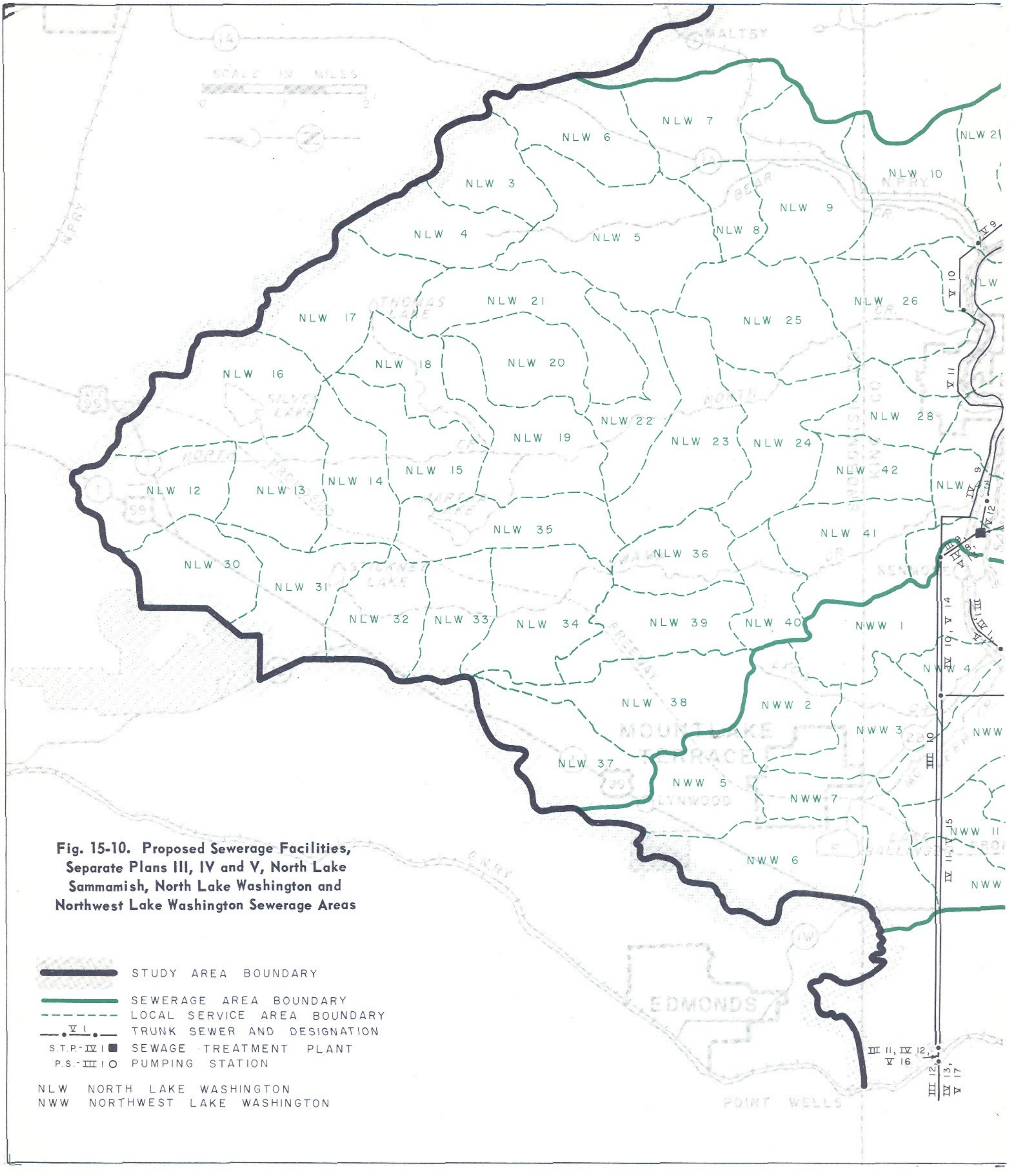

Fig. 15-10. Proposed Sewerage Facilities, Separate Plans III, IV and V, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

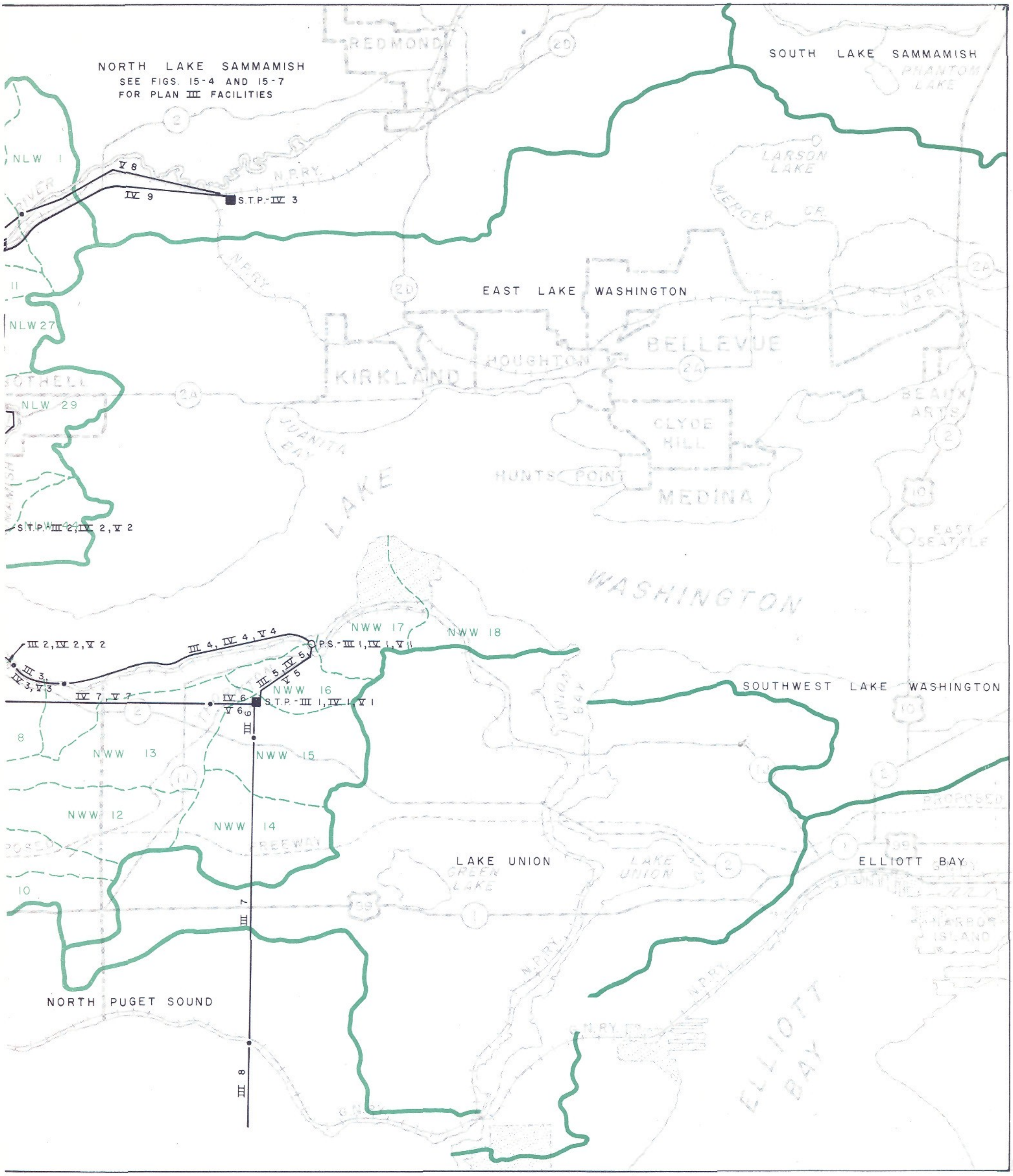
NORTH LAKE SAMMAMISH
SEE FIGS. 15-4 AND 15-7
FOR PLAN III FACILITIES
SOUTH LAKE SAMMAMISH
PHANTOM LAKE
REDMOND
LARSON LAKE
NLW 1
V 8
IV 9
S.T.P.-IV 3
N.P.RY.
EAST LAKE WASHINGTON
NLW 27
NLW 29
KIRKLAND
HOUGHTON
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
BEAUX ARTS
EAST SEATTLE
LAKE
WASHINGTON
S.T.P.-III 2, IV 2, V 2
III 2, IV 2, V 2
III 4, IV 4, V 4
P.S.-III 1, IV 1, V 1
NWW 17
NWW 18
III 5, IV 5, V 5
NWW 16
IV 7, V 7
IV 6
V 6
S.T.P.-III 1, IV 1, V 1
III 6
SOUTHWEST LAKE WASHINGTON
NWW 13
NWW 15
NWW 12
NWW 14
LAKE UNION
GREEN LAKE
ELLIOTT BAY
HARBOR ISLAND
III 7
NORTH PUGET SOUND
III 8
ELLIOTT BAY

Table 15-18. Description and Estimated Construction Costs, Separate Plan IV, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
Northwest Lake Washington Sewerage Area				
IV-1 - IV-5	–	–	Same as III-1 - III-5, Plan III, see Table 15-17	1,969,000
PS-IV-1	–	–	Same as PS-III-1, Plan III, see Table 15-17	221,000
STP-IV-1	–	–	Same as STP-III-1, Plan III, see Table 15-17	1,988,000
IV-6	18	48	2,100 ft of 36-in. force main, effluent outfall	47,000
IV-7	18	48	18,300 ft of 72-in. RC tunnel at 0.045%, effluent outfall, includes allowance of 20% for uncertainties	5,490,000
Subtotal, outfall				5,537,000
Total contract cost, Plan IV, Northwest Lake Washington Sewerage Area				9,715,000
Engineering and contingencies, 25 per cent				2,429,000
Total construction cost, Plan IV, Northwest Lake Washington Sewerage Area				12,144,000
North Lake Washington Sewerage Area				
STP-IV-2	–	–	Same as STP-III-2, Plan III, see Table 15-17	5,564,000
IV-8	–	–	Same as III-9, Plan III, see Table 15-17	72,000
Total contract cost, Plan IV, North Lake Washington Sewerage Area				5,636,000
Engineering and contingencies, 25 per cent				1,409,000
Total construction cost, Plan IV, North Lake Washington Sewerage Area				7,045,000
North Lake Sammamish Sewerage Area				
STP-IV-3	27	72	Sewage treatment plant, secondary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, separate sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	4,495,000
IV-9	27	72	41,800 ft of 66-in. force main, effluent outfall, includes imported backfill, repaving and railroad and river crossings	2,038,000
Total contract cost, North Lake Sammamish Sewerage Area				6,533,000
Engineering and contingencies, 25 per cent				1,633,000
Total construction cost, North Lake Sammamish Sewerage Area				8,166,000
Joint Outfall				
IV-10	58	150	8,700 ft of 96-in. RC tunnel at 0.09%, effluent outfall, includes allowance of 20% for uncertainties	3,863,000
IV-11	76	193	24,100 ft of 102-in. RC tunnel at 0.11%, effluent outfall, includes allowance of 20% for uncertainties	11,568,000
IV-12	76	193	800 ft of 102-in. RC at 0.085%, effluent outfall, minimum depth, difficult wet, includes sheeting, dewatering and railroad crossing	115,000
IV-13	76	193	1,200 ft of 96-in. RC submarine outfall to water depth of 90 ft, includes diffuser section over last 160 ft	727,000
Total contract cost, Plan IV, joint outfall				16,273,000
Engineering and contingencies, 25 per cent				4,068,000
Total construction cost, Plan IV, joint outfall				20,341,000
Total construction cost, Plan IV				47,696,000

See Fig. 15-10 for location of facilities. [a]*Expressed as average dry weather flow and maximum wet weather flow.*

Sound. As in Plan II, the North Lake Sammamish sewerage area would be served by the core plan.

Plan IV - Construction of separate secondary type treatment plants in each of the three areas, with effluent disposal through a joint outfall to Puget Sound.

Plan V - Conveyance of the sewage from the North Lake Sammamish and North Lake Washington sewerage areas to a secondary type treatment plant west of Bothell, and treatment of the sewage from the Northwest Lake Washington sewerage area in a separate secondary type plant, with both effluents discharged through a joint outfall to Puget Sound.

Plan I. Locations of sewers, pumping stations, treatment works and outfall called for under Plan I are shown in Fig. 15-8. Descriptions of these facilities, together with their estimated costs, are given in Table 15-15. For this plan, the estimated construction cost totals $42,580,000.

Under Plan I, the intercepting sewer would begin north of Redmond in the North Lake Sammamish sewerage area and would follow the Sammamish River valley through the North Lake Washington sewerage area to a pumping station east of Bothell. At this station, sewage would be lifted to a sewer on the Northern Pacific Railroad right-of-way and would flow by gravity along the lake front to Sheridan Beach. From that point, the sewer would be laid parallel to the shore in a trench excavated in the bottom of Lake Washington and would extend to a pumping station at the mouth of Thornton Creek.

The treatment plant would be constructed at the site of the existing Lake City plant and would provide secondary treatment for an estimated ultimate average flow of 76 mgd, with a peak hydraulic capacity of 193 mgd. Since the present Lake City plant is not of a size that could be converted economically to handle the anticipated ultimate flows, a completely new plant would have to be constructed. Plant units would include preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks and all other necessary structures and appurtenances. Digested sludge would be hauled away in tank trucks and disposed of elsewhere. Treated effluent would be chlorinated and pumped to south of Piper Creek, at which location it would be discharged to Puget Sound approximately 2,800 feet offshore at a water depth of about 120 feet.

Plan II. Locations of sewers, pumping stations, treatment works and outfall for the North Lake Washington and Northwest Lake Washington sewerage areas are shown in Fig. 15-9, while Core Plan B and feeder sewer facilities for the North Lake Sammamish sewerage area are shown in Figs. 15-4 and 15-7. Descriptions and estimated construction costs of the facilities for Plan II are given in Table 15-16. Total construction costs are estimated to be $39,302,000.

Interception of sewage from the North Lake Washington and Northwest Lake Washington sewerage areas would be accomplished in the same way as interception under the feeder sewer system for Core Plan B as far as the pumping station at the mouth of Thornton Creek. From this point, the sewage would be pumped to a treatment plant at the site of the existing Lake City plant.

As in Plan I, the present Lake City plant, because of its size, is not suitable for incorporation into the plant proposed under Plan II. A new plant would therefore be constructed at this site and would provide secondary treatment for an ultimate average flow of 49 mgd, with a peak hydraulic capacity of 121 mgd. Plant units, except for size, and plant operation would be identical to Plan I. Treated effluent would be disposed of as under Plan I, but the size of the outfall, because of the smaller peak flow, would be reduced accordingly.

Under Plan II, the North Lake Sammamish sewerage area would be served by the central sewerage project. All facilities for this area would thus be identical to those required for participation in that project.

Plan III. Locations of sewers, pumping stations, treatment works and outfalls for the North Lake Washington and Northwest Lake Washington sewerage areas are shown in Fig. 15-10, while Core Plan B and feeder sewer facilities for the North Lake Sammamish sewerage area are shown in Figs. 15-4 and 15-7. These facilities are described in Table 15-17, which also gives their estimated construction costs. As indicated therein, facilities called for under Plan III are estimated to cost a total of $47,824,000.

The intercepting sewer for the Northwest Lake Washington sewerage area would start at the eastern boundary of the area and would be laid along the lake front to Sheridan Beach. From there, the sewer would be laid parallel to shore in a trench excavated in the bottom of Lake Washington and would extend to a pumping station at the mouth of Thornton Creek. The sewage would then be pumped to a treatment plant at the site of the existing Lake City plant.

Under Plan III, the present Lake City plant could be enlarged to provide secondary treatment for the expected ultimate average flow of 18 mgd. This phase of the project, including the provision of necessary new facilities and structures, is estimated to cost $1,988,000, not including engineering and contingencies. Digested sludge would be hauled away in tank trucks for disposal elsewhere. Treated effluent would be pumped through a force main and tunnel to a submarine outfall discharging to Puget Sound south of Piper Creek. Discharge would be approximately

1,900 feet offshore in water at a depth of about 50 feet.

No intercepting sewers would be required in the North Lake Washington area. This is because the sewage treatment plant serving that area would be situated at a site about two miles west of Bothell, the point to which all sewage would be conveyed by the service sewer system.

Treatment would be obtained in a secondary type plant capable of accommodating an ultimate average dry weather flow of 31 mgd, with a peak hydraulic capacity of 78 mgd. Plant units would include preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks and all other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons. Because of soil conditions at the site, foundation piles would be required. Treated effluent would be chlorinated and pumped to south of Richmond Beach, where it would be discharged to Puget Sound approximately 1,000 feet offshore in water at a depth of about 75 feet.

Under Plan III, the North Lake Sammamish sewerage area would be served by the central sewerage project. All facilities for this area would thus be identical to those required for participation in that project.

Plan IV. Sewers, pumping stations, treatment works and outfalls called for under Plan IV are shown in Fig. 15-10 and are described in Table 15-18. For these facilities, the estimated construction cost amounts to $47,696,000.

The intercepting sewer and the sewage treatment

Table 15-19. Description and Estimated Construction Costs, Separate Plan V, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd: Average DWF	Design flow,[a] mgd: Maximum WWF	Description	Construction cost, dollars
Northwest Lake Washington Sewerage Area				
V-1 - V-5	–	–	Same as III-1 - III-5, Plan III, see Table 15-17	1,969,000
PS-V-1	–	–	Same as PS-III-1, Plan III, see Table 15-17	221,000
STP-V-1	–	–	Same as STP-III-1, Plan III, see Table 15-17	1,988,000
V-6 - V-7	–	–	Same as IV-6 - IV-7, Plan IV, see Table 15-18	5,537,000
Total contract cost, Plan V, Northwest Lake Washington Sewerage Area				9,715,000
Engineering and contingencies, 25 per cent				2,429,000
Total construction cost, Plan V, Northwest Lake Washington Sewerage Area				12,144,000
North Lake Sammamish and North Lake Washington Sewerage Areas				
V-8 - V-12	–	–	Same as I-1 - I-5, Plan I, see Table 15-15	5,412,000
STP-V-2	58	150	Sewage treatment plant, secondary type, includes influent and effluent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes special foundations	9,577,000
V-13	58	150	2,700 ft of 60-in. force main, effluent outfall, includes railroad and highway crossing	109,000
Total contract cost, Plan V, North Lake Sammamish and North Lake Washington Sewerage Areas				15,098,000
Engineering and contingencies, 25 per cent				3,774,000
Total construction cost, Plan V, North Lake Sammamish and North Lake Washington Sewerage Areas				18,872,000
Joint Outfall				
V-14 - V-17	–	–	Same as IV-10 - IV-13, Plan IV, see Table 15-18	20,341,000[b]
Total construction cost, Plan V				51,357,000

See Fig. 15-10 for location of facilities.

[a]Expressed as average dry weather flow and maximum wet weather flow.

[b]Includes allowance for engineering and contingencies.

plant for the Northwest Lake Washington sewerage area are identical to those provided under Plan III. Plant effluent would be pumped through a force main and tunnel to the joint outfall serving the three areas.

The treatment plant and effluent force main for the North Lake Washington sewerage area are also identical to those proposed under Plan III.

In the North Lake Sammamish sewerage area, the

Table 15-20. Comparison of Construction Costs, Core Plan B and Separate Plans, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Construction cost,[a] $1,000							
	Core Plan B[b]				Plan I			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Sewers	20,452	8,477	8,521	3,454	13,898	8,316	4,980	602
Pumping stations	2,205	655	1,145	405	1,875	844	914	117
Sewage treatment plants	8,942	3,964	3,019	1,959	11,924	4,236	4,864	2,824
Outfalls	1,789	131	1,019	639	14,883	5,552	6,015	3,316
Total	33,388	13,227	13,704	6,457	42,580	18,948	16,773	6,859

Facility	Construction cost,[a] $1,000							
	Plan II				Plan III			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Sewers	14,057	8,477	4,709	871	10,938	8,477	–	2,461
Pumping stations	1,989	655	1,186	148	931	655	–	276
Sewage treatment plants	12,366	3,964	5,316	3,086	13,404	3,964	6,955	2,485
Outfalls	10,890	131	6,936	3,823	22,551	131	14,281	8,139
Total	39,302	13,227	18,147	7,928	47,824	13,227	21,236	13,361

Facility	Construction cost,[a] $1,000							
	Plan IV				Plan V			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Sewers	2,461	–	–	2,461	9,226	5,178	1,587	2,461
Pumping stations	276	–	–	276	276	–	–	276
Sewage treatment plants	15,059	5,619	6,955	2,485	14,456	5,573	6,398	2,485
Outfalls	29,900	10,653[c]	8,870[c]	10,377[c]	27,399	8,171[c]	8,851[c]	10,377[c]
Total	47,696	16,272	15,825	15,599	51,357	18,922	16,836	15,599

Plan I proposes concentration of the sewage from all three areas in one secondary type treatment plant with effluent disposal to Puget Sound.

Plan II proposes concentration of the sewage from the North Lake Washington and Northwest Lake Washington Sewerage Area in one secondary type treatment plant with effluent disposal to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan III proposes construction of separate secondary type treatment plants in the North Lake Washington and Northwest Lake Washington Sewerage Areas with each plant discharging its effluent separately to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan IV proposes construction of separate secondary type treatment plants in each of the three areas with effluent disposal through a joint outfall to Puget Sound.

Plan V proposes concentration of the sewage from the North Lake Sammamish and North Lake Washington Sewerage Areas in one secondary type treatment plant and concentration of the sewage from the Northwest Lake Washington Sewerage Area in a separate secondary type treatment plant with effluent disposal through a joint outfall to Puget Sound.

[a]*Includes engineering and contingencies.*

[b]*Apportioned cost to the three areas, see Table 15-13; includes feeder sewers.*

[c]*Includes apportioned cost of joint outfall.*

Table 15-21. Comparison of Annual Costs, Core Plan B and Separate Plans, North Lake Sammamish, North Lake Washington and Northwest Lake Washington Sewerage Areas

Cost Item	Average annual cost, $1,000							
	Core Plan B[a]				Plan I			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Fixed costs[b]								
Sewers and outfalls	1,218	472	522	224	1,577	760	602	215
Pumping stations and treatment plants	724	301	270	153	897	330	376	191
	1,942	773	792	377	2,474	1,090	978	406
Maintenance and operation								
Sewers[c]	56	22	24	10	72	35	27	10
Pumping stations[d]	124	42	54	28	108	41	53	14
Sewage treatment plants[d]	177	65	61	51	342	105[e]	135[e]	102[e]
Effluent chlorination[d]	44	15	16	13	50	15	20	15
	401	144	155	102	572	196	235	141
Total annual cost	2,343	917	947	479	3,046	1,286	1,213	547

Cost Item	Average annual cost, $1,000							
	Plan II				Plan III			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Fixed costs[b]								
Sewers and outfalls	1,367	472	638	257	1,835	472	782	581
Pumping stations and treatment plants	934	301	423	210	933	301	452	180
	2,301	773	1,061	467	2,768	773	1,234	761
Maintenance and operation								
Sewers[c]	63	22	29	12	85	22	36	27
Pumping stations[d]	114	42	58	14	59	42	–	17
Sewage treatment plants[d]	329	65	151[e]	113[e]	358	65	146[f]	147[e]
Effluent chlorination[d]	50	15	20	15	50	15	20	15
	556	144	258	154	552	144	202	206
Total annual cost	2,857	917	1,319	621	3,320	917	1,436	967

Cost Item	Average annual cost, $1,000							
	Plan IV				Plan V			
	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington	Total	North Lake Sammamish	North Lake Washington	Northwest Lake Washington
Fixed costs[b]								
Sewers and outfalls	1,773	584	486	703	2,006	731	572	703
Pumping stations and treatment plants	998	366	452	180	959	363	416	180
	2,771	950	938	883	2,965	1,094	988	883
Maintenance and operation								
Sewers[c]	81	27	22	32	91	33	26	32
Pumping stations[d]	17	–	–	17	17	–	–	17
Sewage treatment plants[d]	419	126[f]	146[f]	147[e]	355	92[f]	116[f]	147[e]
Effluent chlorination[d]	50	15	20	15	50	15	20	15
	567	168	188	211	513	140	162	211
Total annual cost	3,338	1,118	1,126	1,094	3,478	1,234	1,150	1,094

Continued on next page

treatment plant would be constructed at a site about two miles north of Redmond. This plant would be of the secondary type and would be capable of treating an ultimate average flow of 27 mgd, with a peak hydraulic capacity of 72 mgd. Both influent and effluent pumping would be required. Plant units would consist of preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks, and all other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons. Chlorinated effluent would be pumped to the joint outfall. This outfall would consist of tunnels and pipe to the shoreline south of Richmond Beach and of a 96-inch submarine section. Discharge to Puget Sound would be approximately 1,200 feet offshore in water at a depth of about 90 feet.

Plan V. Locations of sewers, pumping stations, treatment works and outfalls proposed under Plan V are shown in Fig. 15-10. These facilities are described in Table 15-19 and are estimated to cost $51,357,000.

The intercepting sewer and the sewage treatment plant for the Northwest Lake Washington sewerage area are identical to those proposed under Plan III. Effluent disposal would be achieved in the same manner as under Plan IV.

The intercepting sewer for the North Lake Sammamish and North Lake Washington sewerage areas would consist of a gravity sewer in the Sammamish River valley which would extend from the point of concentration north of Redmond through the North Lake Washington sewerage area to a treatment plant approximately two miles west of Bothell. Except for size, the proposed treatment plant would be the same as that proposed under Plan IV. Treated and chlorinated effluent would be pumped to the joint outfall, which would be identical to the outfall called for in Plan IV.

Comparison of Plans. Apportioned costs to the three sewerage areas of the estimated construction cost of Core Plan B and feeder sewer facilities are given in Table 15-20, as are the estimated construction costs of the five separate plans just considered. Total costs to the three sewerage areas are estimated to range from $33,388,000 for participation in the central project to $51,357,000 for Plan V. It is shown also that the cost to each individual area is lower for participation in the central project than it would be under any of the five plans.

Estimated average annual costs for the central project and for each of the five separate plans are given in Table 15-21. As there indicated, the lowest total annual cost to the three areas, amounting to $2,343,000, is for participation in the central sewerage project. It will be seen also that the annual cost to the individual areas is the lowest for participation in the central project.

Selection of Most Acceptable Plan. From the foregoing discussion of costs, both construction and annual, it is apparent that the most economical means of sewage collection, treatment and disposal for the North Lake Sammamish, North Lake Washington and Northwest Lake Washington sewerage areas would be obtained by

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Table 15-21 footnotes continued from page 394

Plan I proposes concentration of the sewage from all three areas in one secondary type treatment plant with effluent disposal to Puget Sound.

Plan II proposes concentration of the sewage from the North Lake Washington and Northwest Lake Washington Sewerage Area in one secondary type treatment plant with effluent disposal to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan III proposes construction of separate secondary type treatment plants in the North Lake Washington and Northwest Lake Washington Sewerage Areas with each plant discharging its effluent separately to Puget Sound. The North Lake Sammamish Sewerage Area would be served by the core plan.

Plan IV proposes construction of separate secondary type treatment plants in each of the three areas with effluent disposal through a joint outfall to Puget Sound.

Plan V proposes concentration of the sewage from the North Lake Sammamish and North Lake Washington Sewerage Areas in one secondary type treatment plant and concentration of the sewage from the Northwest Lake Washington Sewerage Area in a separate secondary type treatment plant with effluent disposal through a joint outfall to Puget Sound.

[a] *Apportioned cost to the three areas, see Table 15-14; includes feeder sewers.*

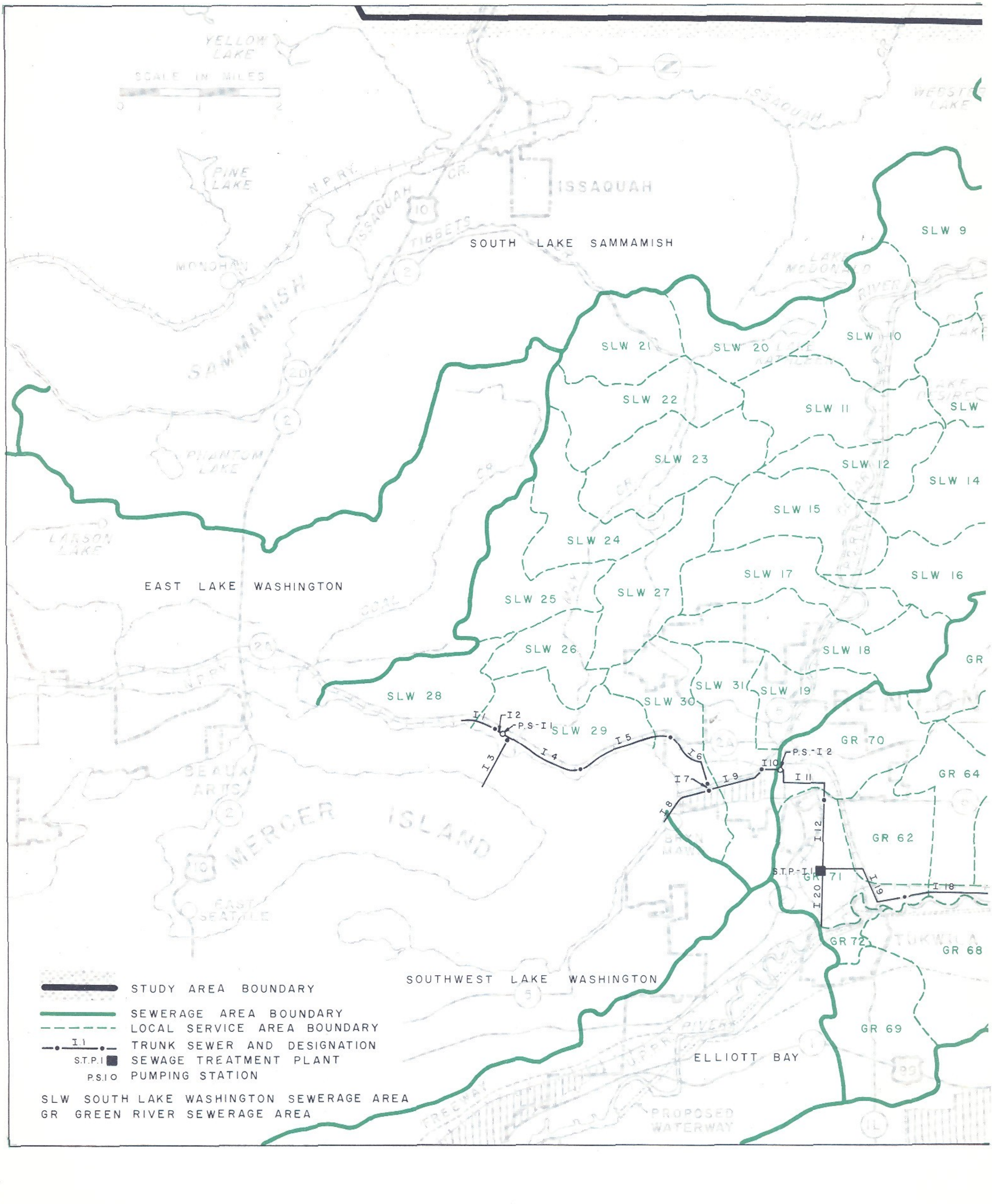
[b] *Includes interest and depreciation calculated by the capital recovery method based on five per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[c] *0.25 per cent of construction costs.*

[d] *Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[e] *Includes allowance of $10.00 per dry ton for hauling of digested sludge for disposal.*

[f] *Includes reduction of $2.25 per dry ton for lagooning of digested sludge.*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SOUTHWEST LAKE WASHINGTON
ELLIOTT BAY
PROPOSED WATERWAY
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STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
I 1 TRUNK SEWER AND DESIGNATION
S.T.P. I SEWAGE TREATMENT PLANT
P.S. I O PUMPING STATION
SLW SOUTH LAKE WASHINGTON SEWERAGE AREA
GR GREEN RIVER SEWERAGE AREA

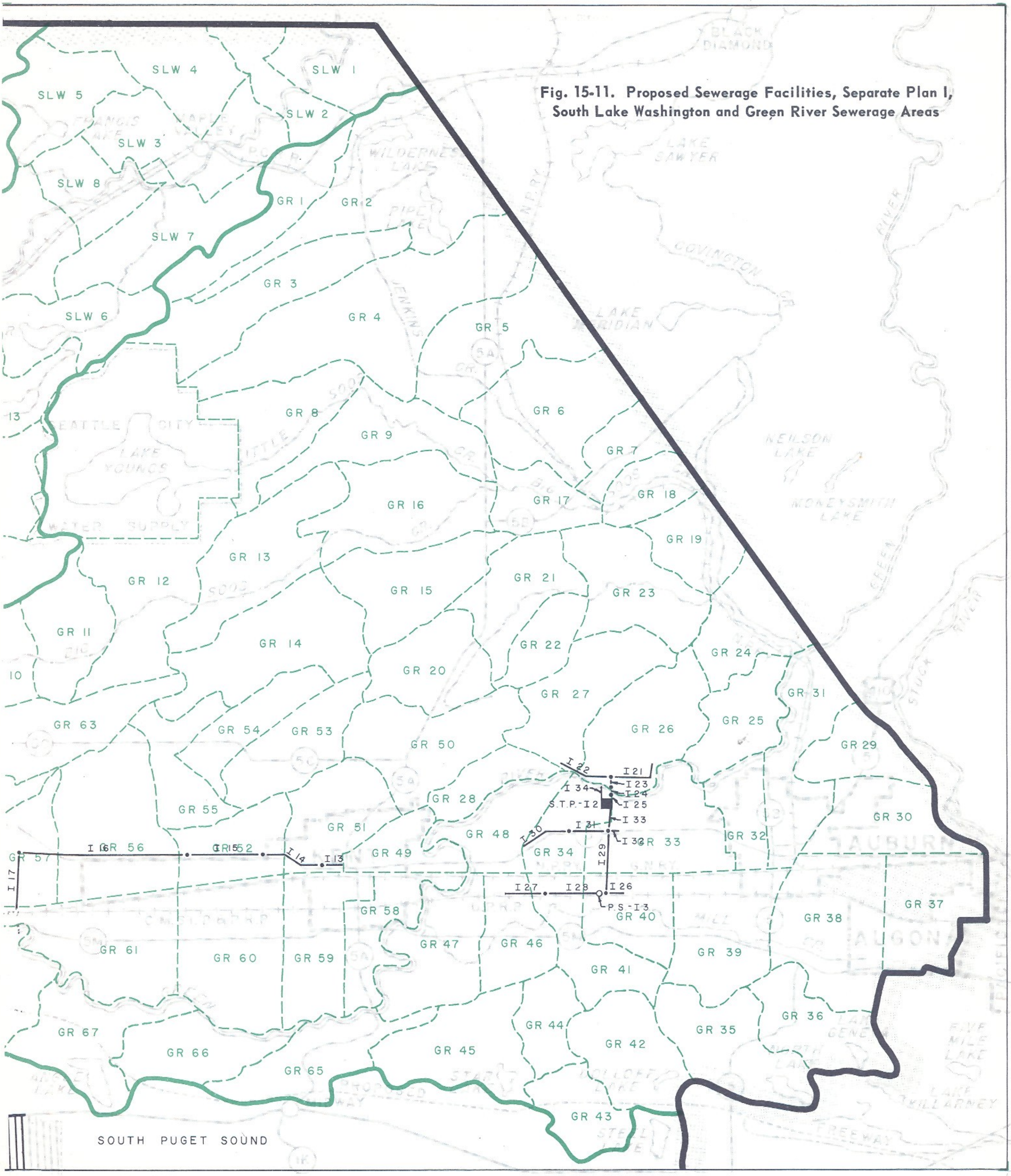

Fig. 15-11. Proposed Sewerage Facilities, Separate Plan I, South Lake Washington and Green River Sewerage Areas

Table 15-22. Description and Estimated Construction Costs, Separate Plan I, South Lake Washington and Green River Sewerage Areas

Facility	Design flow,[a] mgd Average DWF	Design flow,[a] mgd Maximum WWF	Description	Construction cost, dollars
Renton System				
I-1	0.5	1.5	2,400 ft of 12-in. RC at 0.45%, average cut 10 ft, difficult wet, includes sheeting and dewatering	60,000
I-2	1.2	2.8	600 ft of 15-in. RC at 0.45%, average cut 16 ft, difficult wet, includes sheeting and dewatering	19,000
I-3	1.1	2.8	3,500 ft of 12-in. force main across Lake Washington	94,000
I-4	7.5	20	5,400 ft of 39-in. RC at 0.14%, average cut 7 - 8 ft, difficult wet, includes sheeting and dewatering	215,000
I-5	7.7	20	6,200 ft of 42-in. RC at 0.095%, average cut 8 - 10 ft, difficult wet, includes sheeting and dewatering	274,000
I-6	9.4	24	4,400 ft of 48-in. RC at 0.068%, average cut 10 ft, difficult wet, includes sheeting, dewatering and piling	228,000
I-7	9.4	24	400 ft of twin 20-in. inverted siphons, includes inlet and outlet structures	23,000
I-8	0.5	1.7	3,800 ft of 15-in. RC at 0.34%, average cut 6 - 8 ft, difficult wet, includes connections to existing sewers at Bryn Mawr - Lake Ridge sewage treatment plant which is to be abandoned	67,000
I-9	9.9	26	3,800 ft of 54-in. RC at 0.043%, average cut 21 ft, difficult wet, includes sheeting, dewatering and piling	317,000
I-10	12	30	1,600 ft of 54-in. RC at 0.06%, average cut 25 ft, difficult wet, includes sheeting, dewatering and piling	193,000
I-11	23	61	4,000 ft of 57-in. RC at 0.20%, average cut 20 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	391,000
I-12	24	64	5,300 ft of 57-in. RC at 0.20%, average cut 22 - 25 ft, difficult wet, includes sheeting, dewatering and piling	482,000
I-13	1.2	3.4	1,200 ft of 21-in. RC at 0.11%, average cut 9 ft, difficult wet, includes imported backfill, repaving and dewatering	30,000
I-14	1.9	5.5	4,200 ft of 24-in. RC at 0.15%, average cut 9 ft, difficult wet, includes imported backfill, repaving and dewatering	116,000
I-15	2.3	6.4	5,000 ft of 27-in. RC at 0.12%, average cut 11 ft, difficult wet, includes imported backfill, repaving and dewatering	168,000
I-16	4.5-6.5	12-16	10,900 ft of 36-in. RC at 0.072 - 0.14%, average cut 9 - 12 ft, difficult wet, includes imported backfill, repaving and dewatering	360,000
I-17	9.8	24	4,000 ft of 39-in. RC at 0.21%, average cut 14 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	234,000
I-18	13	33	6,500 ft of 57-in. RC at 0.062%, average cut 22 ft, difficult wet, includes imported backfill, repaving, sheeting and dewatering	675,000
I-19	17	41	6,700 ft of 63-in. RC at 0.045%, average cut 23 - 25 ft, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	757,000
Subtotal, sewers, Renton system				4,703,000
PS-I-1	1.2	2.8	Pumping station, single stage, motor driven with gas engine standby, static lift 13 ft, total head at peak flow 20 ft, structure about 20 ft below ground, includes sheeting and dewatering	60,000

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Table 15-22. Continued

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
PS-I-2	12	30	Pumping station, single stage, motor and engine driven, static lift 18 ft, total head at peak flow 25 ft, structure about 35 ft below ground, includes sheeting and dewatering	248,000
Subtotal, pumping stations, Renton system				308,000
STP-I-1	41	101	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities	6,234,000
I-20	41	101	3,100 ft of 60-in. RC effluent outfall to Duwamish River, difficult wet, includes sheeting, dewatering and railroad crossing	217,000
Total contract cost, Renton system				11,462,000
Engineering and contingencies, 25 per cent				2,866,000
Total construction cost, Renton system				14,328,000
Auburn System				
I-21	22	55	3,700 ft of 51-in. RC at 0.26%, average cut 17 ft, difficult wet, includes sheeting and dewatering	234,000
I-22	0.6	1.6	3,200 ft of 12-in. RC at 0.55%, average cut 6 - 7 ft, difficult wet, includes dewatering	49,000
I-23	23	57	200 ft of 51-in. RC at 0.28%, average cut 23 ft, difficult wet, includes sheeting and dewatering	16,000
I-24	23	57	400 ft of twin 30-in. inverted siphons, includes inlet and outlet structures	35,000
I-25	23	57	800 ft of 51-in. RC at 0.28%, average cut 28 ft, difficult wet, includes sheeting and dewatering	73,000
I-26	7.1	17	1,100 ft of 36-in. RC at 0.16%, average cut 9 ft, difficult wet, includes dewatering	33,000
I-27	3.4	8.7	2,700 ft of 33-in. RC at 0.067%, average cut 20 ft, difficult wet, includes sheeting and dewatering	140,000
I-28	4.1	11	3,900 ft of 36-in. RC at 0.06%, average cut 29 ft, difficult wet, includes sheeting and dewatering	292,000
I-29	11	27	4,300 ft of 36-in. RC at 0.42%, average cut 21 ft, difficult wet, includes sheeting, dewatering and railroad crossings	245,000
I-30	0.4	1.4	3,300 ft of 12-in. RC at 0.38%, average cut 15 ft, difficult wet, includes imported backfill, repaving and dewatering	97,000
I-31	0.6	1.8	2,500 ft of 15-in. RC at 0.19%, average cut 29 ft, difficult wet, includes imported backfill, repaving and dewatering	137,000
I-32	4.8	11	600 ft of 30-in. RC at 0.19%, average cut 8 ft, difficult wet, includes imported backfill, repaving and dewatering	18,000
I-33	16	36	2,100 ft of 42-in. RC at 0.38%, average cut 30 ft, difficult wet, includes sheeting and dewatering	173,000
Subtotal, sewers, Auburn system				1,542,000
PS-I-3	4.1	11	Pumping station, single stage, motor driven with gas engine standby, static lift 29 ft, total head at peak flow 35 ft, structure about 35 ft below ground, includes sheeting and dewatering	140,000

Continued on next page

Table 15-22. Continued

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
STP-I-2	36	85	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation, administration and laboratory facilities, includes special foundation	6,651,000
I-34	36	85	500 ft of 54-in. RC outfall to Green River	26,000
Total contract cost, Auburn system				8,359,000
Engineering and contingencies, 25 per cent				2,090,000
Total construction cost, Auburn system				10,449,000
Total construction cost, Plan I				24,777,000

See Fig. 15-11 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

participation in the central sewerage project. Under this project, sewage from the North Lake Sammamish sewerage area would be pumped into the East Lake Washington area, through which it would flow by gravity to the Renton plant of Core Plan B. Sewage from the North Lake Washington and Northwest Lake Washington areas would be conveyed to the West Point plant of Core Plan B for treatment and disposal.

South Lake Washington and Green River Sewerage Areas

Since all sewage from the South Lake Washington sewerage area can be conveyed to a point in the city of Renton at the southwest corner of the area, and since flow can be by gravity from that point into the Green River sewerage area, it is evident that both areas should be combined for the planning of a separate sewerage project. Only one basic alternative is available for such a project. This would involve construction of (1) a complete type treatment plant about one and one-half miles north of Auburn to serve the southern and eastern portions of the Green River sewerage area, and (2) construction of a complete type treatment plant at a site west of Renton, as proposed under Core Plan B, to serve the remainder of the Green River area, as well as the South Lake Washington area.

Two variations in the basic alternative were considered and are designated in the following discussion as Plan I and Plan II. Plan I provides capacity in the Renton plant for the South Lake Washington and the northern portion of the Green River areas only; Plan II provides capacity in the Renton plant for the North Lake Sammamish, South Lake Sammamish and East Lake Washington sewerage areas, as well as for the South Lake Washington and the northern portion of the Green River areas.

Plan I. Locations of sewers, pumping stations, treatment works and outfalls proposed under this plan are shown in Fig. 15-11. These facilities are described in Table 15-22 and are estimated to cost $24,777,000.

Intercepting sewers for the Renton system include two branches, a north and a south. Starting at Hazelwood, the north branch would follow the route of the Northern Pacific Railroad to Renton and would go through Renton to the treatment plant west of the city. In addition to picking up sewage from the South Lake Washington sewerage area, this branch would intercept flow from the south half of Mercer Island in the East Lake Washington area and from the southern portion of the Southwest Lake Washington area. To avoid excessive cuts, two pumping stations would be required.

The south branch would extend southward from the treatment plant and would provide service to that portion of the Green River valley north of where the river flows from east to west. It would also serve areas to the east and west which drain directly into the valley.

Secondary treatment by the activated sludge process would be provided at the Renton plant for an ultimate average flow of 41 mgd, with a peak hydraulic capacity of 101 mgd. Except for capacity, and consequently the size of the units, this plant would be identical to the Renton plant proposed under Core Plan B. Chlorinated effluent would be discharged to Duwamish River through a 60-inch diffuser-equipped outfall.

Intercepting sewers for the Auburn plant were laid out to serve the Green River valley south of where the river flows from east to west. They were laid out also to serve areas on the east and west slopes of the valley, as well as the entire eastern portion of the Green River sewerage area, which portion is drained by Big

Soos Creek. Secondary treatment by the activated sludge process would be provided at the Auburn plant for an ultimate average dry weather flow of 36 mgd, with a peak hydraulic capacity of 85 mgd. Plant units would consist of preaeration and primary sedimentation tanks, aeration tanks, secondary sedimentation tanks, chlorine contact tanks, separate sludge digestion tanks and other necessary structures and appurtenances. Digested sludge would be disposed of in sludge lagoons. Chlorinated effluent would be discharged to Green River.

Plan II. Descriptions of facilities proposed under Plan II are given in Table 15-23. As indicated in this table, the apportioned construction cost of Plan II to the South Lake Washington and Green River sewerage areas amounts to a total of $20,343,000.

Two interceptor sewers, a north and a south, are required for the Renton system under Plan II. Of these, the north interceptor would be identical to that proposed for Core Plan B (Fig. 15-4), while the south interceptor would be identical to that proposed under Plan I (Fig. 15-11). Except for capacity, the Renton

Table 15-23. Description and Estimated Construction Costs, Separate Plan II, South Lake Washington and Green River Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
Renton System				
–	–	–	Sewers, same as B-2, Core Plan B, see Fig. 15-4 and Table 15-4	62,000[b]
–	–	–	Sewers, same as S-15, S-17, S-18, S-20, S-21, S-22, feeder sewers for Core Plan B, see Fig. 15-7 and Table 15-11	685,000[c]
–	–	–	Sewers, same as I-13 - I-19, Plan I, see Fig. 15-11 and Table 15-22	2,340,000
Subtotal, sewers, Renton system				3,087,000
STP	107	275	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as necessary operation, administration and laboratory facilities	4,674,000[d]
Outfall	107	275	3,100 ft of twin 72-in. RC outfall sewers, difficult wet, includes sheeting, dewatering and railroad crossing	154,000[e]
Total contract cost,[f] Renton system				7,915,000
Engineering and contingencies, 25 per cent				1,979,000
Total construction cost,[f] Renton system				9,894,000
Auburn System				
–	–	–	Sewers, same as I-21 - I-33, Plan I, see Fig. 15-11 and Table 15-22	1,542,000
–	–	–	Pumping station, same as PS-I-3, Plan I, see Fig. 15-11 and Table 15-22	140,000
–	–	–	Sewage treatment plant, same as STP-I-2, see Fig. 15-11 and Table 15-22	6,651,000
–	–	–	Outfall, same as I-34, Plan I, see Fig. 15-11 and Table 15-22	26,000
Total contract cost, Auburn system				8,359,000
Engineering and contingencies, 25 per cent				2,090,000
Total construction cost, Auburn system				10,449,000
Total construction cost,[f] Plan II				20,343,000

[a] *Expressed as average dry weather flow and maximum wet weather flow.*

[b] *Apportioned cost to the two sewerage areas; total cost of facilities $271,000.*

[c] *Apportioned cost to the two sewerage areas; total cost of facilities $7,936,000.*

[d] *Apportioned cost to the two sewerage areas; total cost of facility $13,127,000.*

[e] *Apportioned cost to the two sewerage areas; total cost of facility $467,000.*

[f] *Cost to the two sewerage areas.*

Table 15-24. Comparison of Construction Costs, Core Plan B and Separate Plans, South Lake Washington and Green River Sewerage Areas

Facility	Construction cost,[a] $1,000								
	Core Plan B[b]			Plan I[c]			Plan II[c]		
	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River
Sewers	11,485	905	10,580	7,272	2,391	4,881	5,786	905	4,881
Pumping stations	586	–	586	508	333	175	175	–	175
Sewage treatment plants	10,845	2,790	8,055	15,529	3,608	11,921	14,157	2,921	11,236
Outfalls	311	86	225	290	145	145	225	104	121
Totals	23,227	3,781	19,446	23,599	6,477	17,122	20,343	3,930	16,413

Plan I and II propose concentration of the sewage of the South Lake Washington and Green River Sewerage Areas in two secondary type treatment plants, one to be located north of Auburn and the second at the Renton site. The two plans differ only in the size of plant required at the Renton site.

[a] *Includes engineering and contingencies.*

[b] *Apportioned cost to the two areas, see Table 15-13; includes feeder sewers.*

[c] *Apportioned cost to the two areas.*

treatment plant would be identical to that proposed at this site under Core Plan B. Similarly, chlorinated effluent would be discharged to Duwamish River.

For the Auburn plant, intercepting sewers, pumping stations, treatment units and effluent disposal facilities would be identical to those proposed under Plan I.

Comparison of Plans. Apportioned construction costs to the two sewerage areas of the central sewerage project are given in Table 15-24, as are the estimated construction costs of the two separate plans just considered. Estimates of the total cost to the two areas range from $20,343,000 for Plan II to $23,599,000 for Plan I. For the South Lake Washington area, the costs range from $3,781,000 for participation in the central sewerage project to $6,477,000 for Plan I. For the Green River area, the costs range from $16,413,000 for Plan II to $19,446,000 for participation in the central project.

Estimated average annual costs for the core plan system and for each of two separate plans are given in Table 15-25. As there indicated, the total annual cost to the two areas is $1,586,000 for Plan II, $1,646,000 for the central project, and $1,841,000 for Plan I. For the South Lake Washington area, the cost for participation in the central sewerage project is the lowest and amounts to $296,000 per year. For the Green River area, Plan II is the least costly at $1,279,000 per year.

Selection of Most Acceptable Plan. On the basis of the cost estimates just given, it appears that Plan II is the most acceptable from the standpoint of over-all economy. Actually, however, the annual cost of this plan is only slightly lower than that for participation in the central sewerage project. This is true with respect not only to the total cost to the two sewerage areas but to the cost to the Green River area individually.

It will be seen that the difference in costs between Plan II and the central project amounts to $60,000 per year, or less than 4 per cent of the annual cost of Plan II. In the case of the Green River area, the difference is $71,000 per year, or less than 6 per cent of the Plan II cost. With annual costs close enough to minimize them as a decisive element, other pertinent factors must be taken into account and decisions made accordingly.

As shown in Table 15-25, operation and maintenance costs would be $69,000 per year lower for the central sewerage project than for Plan II. This difference is attributable to the higher cost of operating two plants rather than one. Furthermore, as the costs of labor and materials increase, operating costs for two plants would increase at a rate higher than that for a single plant and thus would tend to reduce the presently indicated difference in total annual costs.

In addition to lower operating costs, experience has shown generally that a single plant is more likely to produce a consistently satisfactory effluent and is less subject to plant upsets of a degree requiring the bypassing of raw sewage. These advantages, as they relate to the problem in question, are of particular importance.

Disposal conditions in Green River at the site of the Auburn plant are much less favorable than they are at Renton (Chapter 12). This means that plant upsets, as well as changed conditions with respect to anticipated loading, would cause a more serious impact at the Auburn location. In addition, discharge of plant effluent at two locations would require a higher degree of BOD removal than would be the case if all the sewage were conveyed to a single plant at Renton (Chapter 12).

In conclusion, it is evident first that the choice between central sewerage and Plan II for the South Lake Washington and Green River sewerage areas tends to favor Plan II from the standpoint of total annual cost. On the other hand, it is equally evident that the advantages of more uniform operation and better receiving water conditions obtainable under the central project are sufficient to outweigh its slight disadvantage costwise. For that reason, it is recommended that the two areas be sewered to the Renton plant in the manner called for under Core Plan B.

Southwest Lake Washington, Elliott Bay and , and Lake Union Sewerage Areas

These three areas are now served for the most part by the city of Seattle. As such, the Lake Union and a part of the Southwest Lake Washington area are tributary to the North Trunk sewer of the city and thus would be tributary to the West Point plant proposed under Core Plan B. As a consequence, there are no feasible alternatives for the independent sewerage of these areas.

Since Elliott Bay and most of the remainder of the Southwest Lake Washington sewerage areas are sewered to the waterfront along Duwamish River and Elliott Bay, they are tributary to the required waterfront interceptor. On that basis, the only alternative to Core Plan B for these areas would be the provision of an independent plant at a site along the Duwamish River or Elliott Bay waterfront. This alternative was considered earlier as a part of Core Plan D, whereby it was determined that treatment at the West Point plant under Core Plan B would provide the most satisfactory solution.

South Puget Sound Sewerage Area

Because the topography of the South Puget Sound sewerage area requires high head pumping to con-

Table 15-25. Comparison of Annual Costs, Core Plan B and Separate Plans, South Lake Washington and Green River Sewerage Areas

Cost item	Average annual cost, $1,000								
	Core Plan B[a]			Plan I[b]			Plan II[b]		
	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River	Total	South Lake Washington	Green River
Fixed costs[c]									
Sewers and outfalls	647	55	592	414	139	275	330	55	275
Pumping stations and treatment plants	744	182	562	1,043	256	787	932	190	742
	1,391	237	1,154	1,457	395	1,062	1,262	245	1,017
Maintenance and operation									
Sewers[d]	29	2	27	19	6	13	15	2	13
Pumping stations[e]	27	–	27	12	8	4	4	–	4
Sewage treatment plants[e]	162	46	116	308[f]	72	236	260[f]	49	211
Effluent chlorination[e]	37	11	26	45	11	34	45	11	34
	255	59	196	384	97	287	324	62	262
Total annual cost	1,646	296	1,350	1,841	492	1,349	1,586	307	1,279

Plans I and II propose concentration of the sewage of the South Lake Washington and Green River Sewerage Areas in two secondary type treatment plants, one to be located north of Auburn and the second at the Renton site. The two plans differ only in the size of plant required at the Renton site.

[a] *Apportioned cost to the two areas, see Table 15-14; includes feeder sewers.*

[b] *Apportioned cost to the two areas.*

[c] *Includes interest and depreciation calculated by the capital recovery method based on five per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[d] *0.25 per cent of construction cost.*

[e] *Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

[f] *Includes reduction of $2.25 per dry ton for lagooning of digested sludge.*

vey sewage out of the area, no capacity was provided for it in the core plan system. Various plans for providing independent sewerage were studied and the plan found to be most feasible was compared with that of the central sewerage project for the area as a whole.

As stated in Chapter 14, the South Puget Sound sewerage area is divided topographically into five major subareas, namely, Redondo Beach, Des Moines, Miller Creek, Southwest Suburban and West Seattle. In the Redondo Beach and West Seattle subareas, the only possible independent project is that of providing for sewage collection, treatment and disposal within each subarea itself. For the other three subareas, studies were made of five possible alternatives. These are:

Plan I - Conveyance of sewage to a plant in each subarea, with effluent disposal in Puget Sound.

Plan II - Conveyance of sewage from the Southwest Suburban and Miller Creek subareas to a plant in the Miller Creek subarea and treatment of the sewage from the Des Moines subarea at a separate plant in that area. Effluent from both plants would be disposed of in Puget Sound.

Table 15-26. Summary of Construction and Annual Costs, Alternative Sewerage Plans, Des Moines, Miller Creek and Southwest Suburban Subareas, South Puget Sound Sewerage Area

Facility	Construction cost,[a] $1,000				
	Plan I	Plan II	Plan III	Plan IV	Plan V
Sewers	1,210	1,636	2,205	1,556	1,782
Pumping stations	–	325	970	594	510
Sewage treatment plants	3,767	3,598	3,914	3,675	4,035
Outfalls	651	659	275	270	308
Total construction cost	5,628	6,218	7,364	6,095	6,635
Cost Item	Average annual cost, $1,000				
	Plan I	Plan II	Plan III	Plan IV	Plan V
Fixed costs[b]					
Sewers and outfalls	102	126	136	100	114
Pumping stations and treatment plants	245	255	318	278	296
	347	381	454	378	420
Maintenance and operation					
Sewers[c]	5	6	6	5	5
Pumping stations[d]	–	26	79	35	53
Sewage treatment plants[d]	142	114	98	132	127
Effluent chlorination[d]	18	18	18	18	18
	165	164	201	190	203
Total annual cost	512	545	655	568	623

Plan I proposes concentration of the sewage in a separate plant in each subarea with effluent disposal to Puget Sound.

Plan II proposes concentration of the sewage of the Southwest Suburban and Miller Creek subareas in a plant located in the Miller Creek subarea and concentration of the sewage of the Des Moines subarea in a separate plant in that area.

Plan III proposes concentration of the sewage of all three subareas in a plant located in the Des Moines subarea with effluent disposal to Puget Sound.

Plan IV proposes concentration of the sewage of the northern portion of the Miller Creek subarea, along with the sewage of the Southwest Suburban subarea, in the existing plant of the Southwest Suburban Sewer District and concentration of the sewage of the southern portion of the Miller Creek subarea, along with the sewage of the Des Moines subarea, in a plant located in the Des Moines subarea.

Plan V proposes concentration of the sewage of the Southwest Suburban subarea in the existing plant of the Southwest Suburban Sewer District and concentration of the sewage of the Miller Creek and Des Moines subareas in a plant in the Des Moines subarea.

[a] *Includes engineering and contingencies.*

[b] *Includes interest and depreciation calculated by the capital recovery method based on 5 per cent interest and depreciation life of 50 years for sewers and outfalls and 30 years for pumping stations and sewage treatment plants.*

[c] *0.25 per cent of construction cost.*

[d] *Based on average flow during design period, 1960 - 2030, as determined from Table 15-1.*

Plan III - Conveyance of sewage from all three subareas to a plant in the Des Moines subarea, with effluent disposal to Puget Sound.

Plan IV - Conveyance of sewage from the northern portion of the Miller Creek subarea, along with that from the Southwest Suburban subarea, to the existing plant of the Southwest Suburban Sewer District and treatment of sewage from the southern portion of the Miller Creek subarea, together with that from the Des Moines subarea, at a plant in the Des Moines subarea. Effluent from both plants would be disposed of to Puget Sound. Existing facilities of the Southwest Suburban Sewer District in the vicinity of Lake Burien in the Miller Creek subarea would continue in use as at present.

Plan V - Conveyance of sewage from the Southwest Suburban subarea to the present plant of the Southwest Suburban Sewer District, and treatment of sewage from the Miller Creek and Des Moines subareas at a plant in the Des Moines subarea. Effluent from both plants would be disposed of in Puget Sound.

Total construction costs for the five plans range from $5,628,000 for Plan I to $7,364,000 for Plan III (Table 15-26). Total annual costs range from $512,000 for Plan I to $655,000 for Plan III (Table 15-26). Based on these costs, it is apparent that the most satisfactory independent sewerage plan for the three subareas is that proposed under Plan I whereby sewage collection, treatment, and disposal facilities would be provided in each subarea.

Locations of all trunk sewers, pumping stations, treatment works and outfalls required to provide independent sewerage for the five subareas are shown in Fig. 15-12. Descriptions of the facilities and their estimated construction costs are given in Table 15-27.

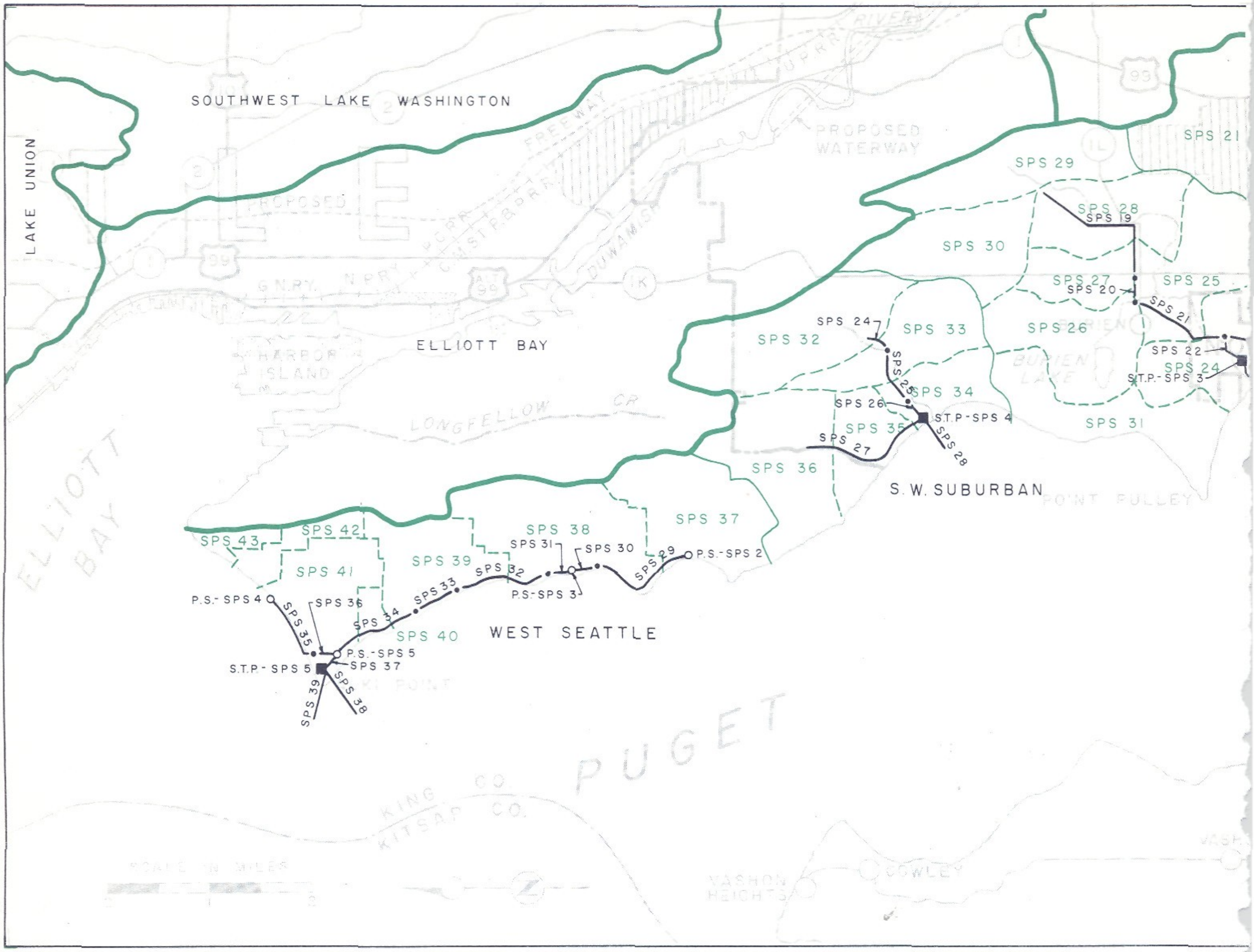
Redondo Beach Subarea. Treatment and disposal of the sewage of the Redondo Beach subarea would be obtained in a plant located about one and one-half miles from Lakota. Trunk sewers would consist of three branches, west, central and east. Of these, the west branch would be a high elevation sewer from the plant to Lakota, from which point it would turn south to serve the area south and west of Lakota. Because of the relatively high elevation of the sewer, local pumping would be required along the waterfront to bring in sewage from areas which cannot be served by gravity. The central branch would run generally southward from the plant and would serve Mirror Lake and surrounding areas. The east branch would consist of a waterfront interceptor to serve Redondo Beach and Woodmont Beach, a pumping station, and a high elevation interceptor to the plant. This branch would serve Steel Lake and surrounding areas. As in the west branch, some local pumping would be required.

Treatment would be provided in a primary type plant capable of accommodating an ultimate average dry weather flow of 5.0 mgd, with a peak hydraulic capacity of 14 mgd. Plant units would consist of influent pumps, preaeration and primary sedimentation tanks, separate sludge digestion tanks, chlorine contack tanks, and other necessary structures and appurtenances. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 3,000 feet offshore to a water depth of 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 1,500 feet offshore in water at a depth of about 120 feet.

Des Moines Subarea. Treatment and disposal of the sewage of the Des Moines subarea would be obtained in a plant located south of Des Moines. Trunk sewers would consist of a north and south branch, of which the south branch would extend southward from the plant through the community of Zenith and then eastward to U.S. Highway 99. Along much of its route, this branch would be laid at a high elevation, with the result that local pumping would be required in some areas. The north branch would be routed northward through the community of Des Moines and then northeastward to a point south of the Seattle-Tacoma International Airport. This branch would serve the airport and all its facilities.

Because of disposal conditions offshore from Des Moines, complete treatment would be required for an ultimate average flow of 6.5 mgd, with a peak hydraulic capacity of 16 mgd. Plant units would consist of influent pumps, preaeration and primary sedimentation tanks, trickling filters, secondary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since a detention time of over 20 minutes would be available in the outfall at ultimate average flow. Digested sludge, after passing through a washer, would be disposed of in Puget Sound through a submarine line extending approximately 2,700 feet offshore to a water depth of 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 1,300 feet offshore at a depth of about 60 feet.

Miller Creek Subarea. Treatment and disposal of the sewage of the Miller Creek subarea would be obtained in a plant located in the northern part of the city of Normandy Park. Two trunk sewers, a north and a south, would be required. Of these, the south trunk would serve the southern and central portions of Normandy Park as well as adjacent tributary areas. The north trunk would be laid generally along Miller Creek to beyond Five Corners and would serve the highly developed area around Lake Burien plus areas



to the north as far as White Center. It would also serve the northern portion of the city of Normandy Park. Some local pumping would be required to serve low areas along the routes of both trunks.

Treatment would be provided in a primary type plant laid out for an ultimate average flow of 7.5 mgd, with a peak hydraulic capacity of 18 mgd. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks, and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since a detention time of over 20 minutes would be available in the outfall at the ultimate average flow. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 3,300 feet offshore to a water depth of 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 2,900 feet offshore at a depth of about 200 feet.

Southwest Suburban Subarea. Treatment and disposal of the sewage of the Southwest Suburban subarea would continue to be obtained in the present plant of the Southwest Suburban Sewer District. Existing trunks extending eastward from the plant are of adequate capacity for expected future flows and would be fully utilized under the program herein proposed. A new trunk laid northward from the plant would be required to serve the Roxbury Heights area. The present primary type treatment plant has sufficient capacity for the expected ultimate average flow of 4.0 mgd and peak flow of 8.5 mgd.

Chlorinated effluent is presently being discharged to Puget Sound through a 36-inch submarine outfall terminating 600 feet offshore at a depth of 60 feet. As pointed out in Chapter 11, this method of disposal is inadequate in that it fails to provide the required degree of protection along the adjacent shoreline. That being the case, it appears that the present plant

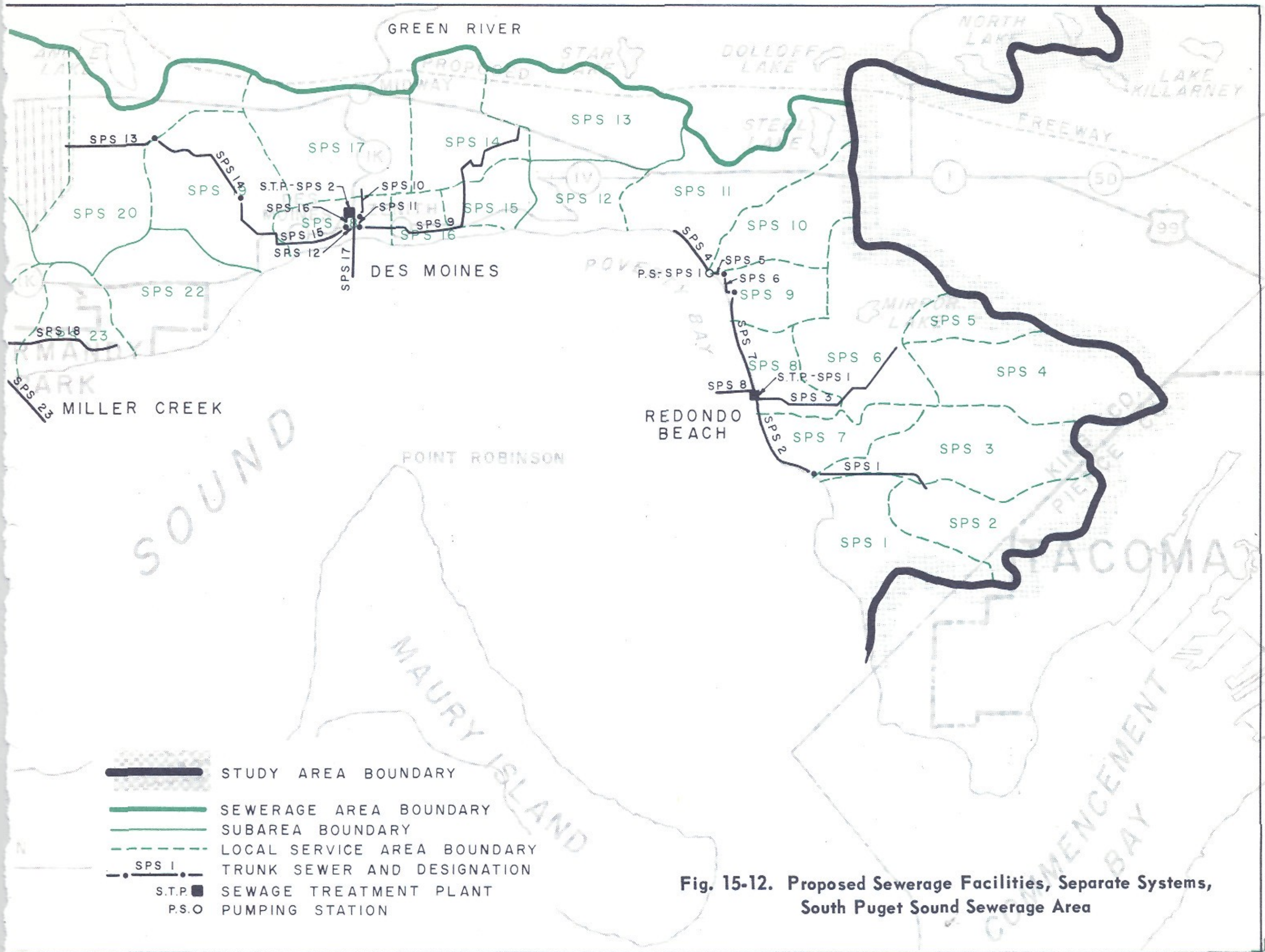


Fig. 15-12. Proposed Sewerage Facilities, Separate Systems, South Puget Sound Sewerage Area

will have to be expanded and modified to provide for secondary treatment. Before embarking on such a project, however, consideration should be given to the possibility of extending the outfall to a depth suitable for disposal of disinfected primary effluent.

In the event that an outfall extension is found to be impracticable or uneconomical, secondary treatment would be the only alternative and would require the addition of trickling filters and secondary sedimentation tanks. In addition, certain alterations to the existing plant would be required to permit the discharge of digested sludge to Puget Sound and the utilization of sludge gas for sludge heating purposes. These alterations would consist of piping changes, installation of necessary equipment, and construction of a sludge outfall line extending 3,300 feet offshore to a water depth of 400 feet. It would be necessary also to extend the effluent outfall an additional 200 feet to a water depth of 85 feet. Estimated costs of the required secondary facilities and the plant alterations amount to $568,000 exclusive of engineering and contingencies (Table 15-27).

West Seattle Subarea. Treatment and disposal of the sewage of the West Seattle subarea will be obtained in the Alki Point plant of the city of Seattle. This plant is now under construction and is scheduled to go into operation late in 1958. Two waterfront interceptors, one north and the other south of the plant, will serve this subarea. In addition, the north interceptor will receive sewage from the most westerly local service area of the Elliott Bay sewerage area. Three pumping stations will be required along the routes of the interceptors, two on the south and one on the north. In addition, a pumping station at 63rd Avenue S.W. will lift all sewage into the treatment plant. Since the West Seattle subarea is served by combined sewers, interceptor design is based on an overflow frequency of 12

Table 15-27. Description and Estimated Construction Costs, Separate Systems for South Puget Sound Sewerage Area

Facility	Design flow,[a] mgd Average DWF	Design flow,[a] mgd Maximum WWF	Description	Construction cost,[b] dollars
Redondo Beach Subarea				
SPS-1	1.1	3.1	5,500 ft of 12-in. RC at 2.7%, average cut 6 ft, dry to moderately wet, includes access road	62,000
SPS-2	1.8	4.9	5,200 ft of 21-in. RC at 0.25%, average cut 6 ft, dry to wet, includes imported backfill, repaving and access road	86,000
SPS-3	0.9-1.6	2.4-4.3	8,400 ft of 18-in. RC at 0.3 - 4.9%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	111,000
SPS-4	0.9	2.5	3,400 ft of 18-in. RC at 0.18%, average cut 6 ft, dry to wet, includes imported backfill and repaving	53,000
SPS-5	1.3	3.4	800 ft of 12-in. force main	5,000
SPS-6	1.5	3.8	1,500 ft of 15-in. RC at 1.3%, average cut 6 ft, dry to moderately wet, includes imported backfill	19,000
SPS-7	1.5-1.8	4.0-4.8	6,000 ft of 18-in. RC at 0.43 - 2.5%, average cut 6 ft, dry to wet, includes imported backfill, repaving and access road	85,000
Subtotal, sewers				421,000
PS-SPS-1	1.3	3.4	Pumping station, single stage, motor driven, static lift 70 ft, total head at peak flow 90 ft, structure about 10 ft below ground	88,000
STP-SPS-1	5.0	14	Sewage treatment plant, primary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 3,000 ft of 6-in. outfall sludge line to a water depth of 400 ft	821,000
SPS-8	5.0	14	1,900 ft of 30-in. RC effluent outfall to water depth of 120 ft, includes 400 ft land section, and diffuser section over last 90 ft	217,000
Total contract cost, Redondo Beach subarea				1,547,000
Engineering and contingencies, 25 per cent				387,000
Total construction cost, Redondo Beach subarea				1,934,000
Des Moines Subarea				
SPS-9	0.7-1.8	1.8-4.3	14,900 ft of 15-in. RC at 0.24 - 4.8%, average cut 6 ft, dry to moderately wet, includes imported backfill, repaving and access road	189,000
SPS-10	1.0	2.5	1,700 ft of 12-in. RC at 2.2%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	19,000
SPS-11	1.2	2.8	600 ft of 15-in. RC at 0.5%, average cut 8 ft, dry to moderately wet	7,000
SPS-12	2.9	7.1	1,000 ft of 15-in. RC at 4.2%, average cut 6 ft, wet	15,000
SPS-13	1.3	3.2	5,200 ft of 15-in. RC at 2.0%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	62,000
SPS-14	2.5-3.0	6.1-7.2	5,400 ft of 21-in. RC at 0.74 - 3.2%, average cut 6 ft, dry to moderately wet	72,000
SPS-15	3.2-3.6	7.6-8.6	7,300 ft of 24-in. RC at 0.43 - 4.1%, average cut 6 - 8 ft, dry to wet, includes imported backfill and repaving	134,000
SPS-16	6.4	16	600 ft of 30-in. RC at 0.36%, average cut 12 ft, wet	15,000
Subtotal, sewers				513,000

Continued on next page

Table 15-27. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
STP-SPS-2	6.5	16	Sewage treatment plant, secondary type, includes influent pumping and facilities for screenings and grit removal, preaeration and primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 2,700 ft of 6-in. outfall sludge line to a water depth of 400 ft	1,479,000
SPS-17	6.5	16	2,500 ft of 30-in. RC effluent outfall to water depth of 60 ft, includes 1,200 ft land section, and diffuser section over last 45 ft	153,000
Total contract cost, Des Moines subarea				2,145,000
Engineering and contingencies, 25 per cent				536,000
Total construction cost, Des Moines subarea				2,681,000
Miller Creek Subarea				
SPS-18	1.2	2.8	7,000 ft of 12-in. RC at 1.5%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	79,000
SPS-19	2.0-2.6	4.5-6.0	8,400 ft of 21-in. RC at 0.22 - 0.9%, average cut 6 - 9 ft, dry to moderately wet, includes imported backfill and repaving	151,000
SPS-20	2.9	6.8	1,000 ft of existing 24-in. RC at 0.3%	Existing
SPS-21	4.1-5.9	9.4 - 14	5,300 ft of 24-in. RC at 0.65 - 2.1%, average cut 6 - 8 ft, dry to wet, includes imported backfill, repaving and connections to existing sewers at Southwest Suburban Sewer District pumping station No. 6 which is to be abandoned	109,000
SPS-22	7.4	17	1,700 ft of 27-in. RC at 1.2%, average cut 7 ft, wet, includes imported backfill and repaving	42,000
Subtotal, sewers				381,000
STP-SPS-3	7.5	18	Sewage treatment plant, primary type, includes facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as necessary operation and maintenance facilities, includes 3,300 ft of 6-in. outfall sludge line to a water depth of 400 ft	966,000
SPS-23	7.5	18	4,100 ft of 27-in. RC outfall to water depth of 200 ft, includes 1,200 ft land section, and diffuser section over last 150 ft	323,000
Total contract cost, Miller Creek subarea				1,670,000
Engineering and contingencies, 25 per cent				418,000
Total construction cost, Miller Creek subarea				2,088,000
Southwest Suburban Subarea				
SPS-24	1.4	3.2	1,400 ft of existing 18-in. and 24-in. RC at 0.67 - 6.3%	Existing
SPS-25	1.9	4.2	2,000 ft of existing 30-in. RC at 5.2%	Existing
SPS-26	2.1	4.6	800 ft of existing 36-in. RC at 1.5%	Existing
SPS-27	1.0-1.5	2.1-3.3	6,600 ft of 12-in. RC at 1.5 - 2.4%, average cut 6 - 8 ft, dry to moderately wet, includes imported backfill, repaving and access road.	74,000
Subtotal, sewers				74,000
STP-SPS-4[c]	4.0	8.5	Sewage treatment plant, existing primary type to be converted to secondary type, includes facilities for preaeration and grit removal, comminution, primary sedimentation, trickling filtration, secondary sedimentation, sludge digestion and effluent chlorination, as well as facilities for operation and maintenance functions, includes 3,300 ft of 6-in. outfall sludge line to water depth of 400 ft	568,000

Continued on next page

Table 15-27. Continued

Facility	Design flow,[a] mgd Average DWF	Design flow,[a] mgd Maximum WWF	Description	Construction cost,[b] dollars
SPS-28	4.0	8.5	1,900 ft of 36-in. RC effluent outfall to water depth of 85 ft, includes 1,100 ft existing land section, 600 ft existing submarine section to water depth of 60 ft, and 200-ft extension with diffuser sections over last 90 ft	45,000
Total contract cost, Southwest Suburban subarea				687,000
Engineering and contingencies, 25 per cent				172,000
Total construction cost, Southwest Suburban subarea				859,000
West Seattle Subarea[d]				
SPS-29	1.6	23	6,200 ft of existing 30-in. force main	Existing
SPS-30	1.6	23	900 ft of existing 42-in. at 0.12%	Existing
SPS-31	3.5	39	1,400 ft of existing twin 27-in. force mains	Existing
SPS-32	3.5	39	5,000 ft of existing 36-in. pressure sewer	Existing
SPS-33	4.5	42	2,000 ft of existing 54-in. pressure sewer	Existing
SPS-34	4.5-4.7	42	4,800 ft of existing 54-in. at 0.11%	Existing
SPS-35	1.3	17	4,400 ft of existing 24-in. force main	Existing
SPS-36	1.3	17	1,400 ft of existing 30-in. at 0.45%	Existing
SPS-37	7.2	60	1,200 ft of existing parallel 24-in. and 42-in. force mains	Existing
PS-SPS-2	1.6	23	Pumping station, existing, static lift 30 ft, total head at peak flow 70 ft	Existing
PS-SPS-3	3.5	39	Pumping station, existing, static lift 43 ft, total head at peak flow 70 ft	Existing
PS-SPS-4	1.3	17	Pumping station, existing, static lift 17 ft, total head at peak flow 64 ft	Existing
PS-SPS-5	7.2	60	Pumping station, existing, static lift 26 ft, total head at peak flow 38 ft	Existing
STP-SPS-5[e]	7.2	60	Sewage treatment plant, existing primary type, includes facilities for preaeration and grit removal, comminution, primary sedimentation, sludge digestion, and effluent chlorination, as well as facilities for operation and maintenance functions. Cost is for 3,300 ft of 6-in. outfall sludge line to a water depth of 400 ft	66,000
SPS-38	7.2	60	1,400 ft of existing 42-in. RC effluent outfall to water depth of 85 ft	Existing
SPS-39	7.2	20	2,500 ft of 36-in. RC effluent outfall to water depth of 210 ft, includes diffuser section over last 500 ft	329,000
Total contract cost, West Seattle subarea				395,000
Engineering and contingencies, 25 per cent				99,000
Total construction cost, West Seattle subarea				494,000

See Fig. 15-12 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*No construction cost allowed for facilities already constructed or for facilities for which money has been allocated.*

[c]*Plant designed on basis of 1.3 hours detention in sedimentation tanks at 8.2 mgd flow. Detention time at ultimate average dry weather flow of 4.0 mgd will be 2.6 hours.*

[d]*West Seattle subarea presently served by combined sewers. Interceptor capacity is based on 12 overflows per summer.*

[e]*Plant designed on basis of 0.5 hours detention in sedimentation tanks at 60 mgd flow. Detention time at ultimate average dry weather flow of 7.2 mgd will be 5 hours.*

times per summer. If this frequency proves to be unsatisfactory because of beach contamination, additional interceptor capacity will have to be provided.

The treatment plant at Alki Point is a primary type with a capacity sufficient for an ultimate average flow of 7.2 mgd, with a peak hydraulic capacity of 60 mgd. Although present plans call for digested sludge to be hauled away, it is believed that this material could be safely disposed of in Puget Sound (Chapter 11). For that reason, the plan herein proposed calls for construction of a sludge outfall line which would extend approximately 3,300 feet into Puget Sound to a water depth of 400 feet.

Disinfected effluent is to be discharged through an outfall, presently under construction, which will terminate 1,400 feet offshore at a depth of 85 feet. As reported in Chapter 11, a chlorinated primary effluent discharged in water at that depth probably will not prevent some contamination of adjacent beaches. This implies that the outfall now going in should be extended to a depth sufficient to assure adequate dispersion at the point of discharge. Unfortunately, however, not enough head is available at the treatment plant to permit such an extension. It is proposed, therefore, that protection of the beaches be obtained by putting in a second outfall, which would be designed to carry the peak dry weather flow, and by utilizing the original outfall for wet weather overflows. Under this plan, the second outfall would extend directly west of Alki Point and would terminate approximately 1,100 feet offshore at a depth of about 210 feet.

Comparison of Independent Systems with Central Sewerage Project. High head pumping would be required to convey sewage of the four southern subareas in the South Puget Sound sewerage area into facilities of the core plan system. On the other hand, sewage from the northern subarea, West Seattle, could be conveyed by gravity northward from Alki Point along the waterfront through the Elliott Bay sewerage area and into the core plan system.

Table 15-28. Comparison of Construction and Annual Costs of Separate Plan and Central Sewerage Project, Redondo Beach Subarea, South Puget Sound Sewerage Area

	Separate plan, \$1,000	Central sewerage[a], \$1,000
Construction cost[b]	1,934[c]	1,209
Annual cost[d]	156	101

[a]*Does not include apportioned cost to subarea of any sewers within Green River Sewerage Area or of core plan facilities.*

[b]*Includes engineering and contingencies.*

[c]*From Table 15-27.*

[d]*Includes fixed costs and maintenance and operation costs.*

Facilities required in the Redondo Beach subarea for participation in the central project would include a high elevation interceptor to the south, a short high elevation interceptor to the north, and a pumping station at Redondo Beach. This station would have a capacity of 14 mgd at a total head of 260 feet and would pump through 2,700 feet of 24-inch force main to a second pumping station. The latter would operate against a total head of 290 feet and would pump through 4,400 feet of 24-inch force main to the crest of the ridge dividing the South Puget Sound and Green River sewerage areas. From there, sewage would flow by gravity through the Green River area to the Renton plant proposed under Core Plan B.

Table 15-28 gives estimated construction and annual costs both for the independent project and for participation in the central sewerage project. In the latter case, the only costs shown are those involved in collection of the sewage and pumping it to the crest of the ridge. They do not include the proportionate share of the Redondo Beach subarea either in the Renton treatment plant or in any sewers within the Green River sewerage area.

It will be seen that both construction and annual costs for the independent system are about 50 per cent higher than those for pumping to the crest of the ridge. No detailed estimates were made of Redondo Beach's share in central facilities beyond that point. A rough check indicates, however, that its share in Core Plan B treatment costs would amount to \$712,000 for construction and to \$57,000 per year for operation and maintenance and fixed charges. These costs, when added to those given in Table 15-28, bring the costs of the central sewerage project to about the same totals estimated for the independent system. If the proportionate share of sewers and pumping stations within the Green River sewerage area were added, the cost of central sewerage would be considerably higher. It is concluded, therefore, that the Redondo Beach subarea should be served by an independent system.

Two alternatives were considered for possible inclusion of the Des Moines, Miller Creek and Southwest Suburban subareas in the central sewerage project. Under Alternative 1, sewage from the Southwest Suburban subarea would be pumped into the Miller Creek subarea, through which it would flow by gravity to the mouth of Miller Creek. At that point, sewage from the two subareas would be pumped into the Des Moines subarea. Beyond Des Moines, flow from the three subareas would be pumped eastward to the crest of the ridge separating the South Puget Sound and Green River sewerage areas. From that point, flow would be by gravity through the Green River area to the Renton plant proposed under Core Plan B.

Under Alternative 2, sewage from each subarea would be pumped independently into sewers connecting with core plan facilities. Sewage from the Southwest Suburban subarea would be conveyed to the existing sewage treatment plant, from which it would be pumped eastward through three pumping stations to the crest of the ridge separating the South Puget Sound and Elliott Bay areas. Flow from that point would be through service sewers in the Elliott Bay area to the West Point treatment plant of Core Plan B. Sewage from the Miller Creek subarea would be pumped into and would flow through the Green River sewerage area to the Renton plant of Core Plan B, as would sewage from the Des Moines subarea.

Estimated construction and annual costs for these two alternatives are given in Table 15-29, as are the costs for the independent systems. For the two alternatives, the indicated costs are limited to those involved in conveyance and pumping. They do not include the apportioned cost to the subareas of any facilities in the core plan system, nor do they include the cost of any service sewers within other sewerage areas necessary to convey the sewage to the core plan system.

Both the construction and annual costs for collection and pumping facilities required under the second of the two alternatives involving participation in the central sewerage project are substantially lower than those for the independent systems (Table 15-29). A preliminary estimate was made, therefore, of the costs which would be borne by the three subareas for the treatment plants involved in the central sewerage project. Distributions thus determined were as follows:

	Construction cost $1,000	Annual cost $1,000
Southwest Suburban	375	35
Miller Creek	997	88
Des Moines	865	75
Total	2,237	198

These costs, when added to those for Alternative 2 (Table 15-29), show that participation in the central sewerage project would cost about 10 per cent more than the independent systems. It is evident, therefore, that greater economy will be achieved by providing each of the three subareas with independent facilities for sewage collection, treatment and disposal.

In the West Seattle subarea, facilities for independent sewage collection, treatment and disposal are presently under construction by the city of Seattle. To determine whether detailed studies should be made of the possibility of participation by this subarea in the central sewerage project, estimates were made of the costs of sharing in the West Point treatment plant. These estimates amount to $624,000 for construction cost, and to $64,000 for annual cost, including fixed and operating charges. Additional costs which would be incurred for conveying sewage to West Point were not estimated. They would, however, increase the total annual cost by a substantial sum.

Table 15-29. Comparison of Construction and Annual Costs of Separate Plan and Central Sewerage Project, Southwest Suburban, Miller Creek and Des Moines Subareas, South Puget Sound Sewerage Area

	Separate plan[a], $1,000	Central sewerage[b], $1,000	
		Alternative 1	Alternative 2
Construction cost[c]	5,628	4,118	4,057
Annual cost[d]	512	422	350

[a]From Table 15-26, Plan I.

[b]Does not include apportioned cost to subareas of any sewers within Green River and Elliott Bay Sewerage Areas or of core plan facilities.

[c]Includes engineering and contingencies.

[d]Includes fixed costs and maintenance and operation costs.

Under either the central or the independent project, fixed costs for facilities presently under construction would have to be paid by the city. Such charges, therefore, need not be considered in comparing costs of the two possibilities. For comparison purposes, therefore, the only cost applicable to the independent project is that of plant maintenance and operation. This is estimated to be $63,000 per year.

Because participation in the central system at this time would involve higher annual costs than those applicable to the independent system, it is concluded that the West Seattle subarea can be served more economically by the independent system.

North Puget Sound Sewerage Area

The North Puget Sound sewerage area is topographically divided into three major subareas, namely, Seaview, Piper Creek and Boeing Creek. Since Seaview encompasses only 230 acres, its sewerage is a matter for local consideration. It should be noted, however, that construction of facilities to connect this area with the North Trunk sewer of the city of Seattle is now in progress. As a result, the Seaview subarea will be tributary to the West Point plant proposed under Core Plan B.

In the case of the Piper Creek and Boeing Creek subareas, plans were laid out under which each one would be served by independent facilities for sewage collection, treatment and disposal. Locations of all trunk sewers, pumping stations, treatment works and outfalls required for this purpose are shown in Fig. 15-13. Descriptions of these facilities and their estimated construction costs are given in Table 15-30.

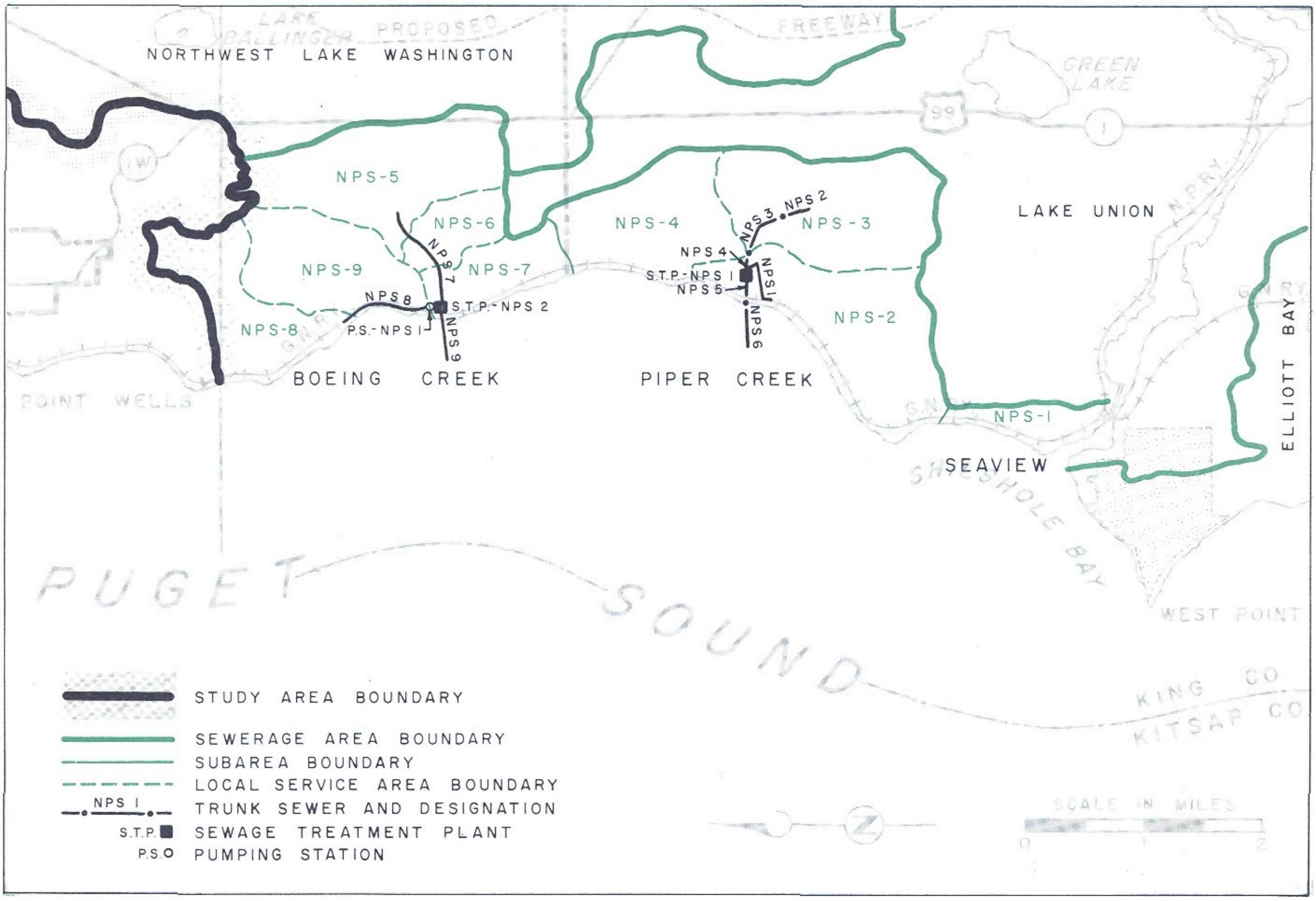


Fig. 15-13. Proposed Sewerage Facilities, Separate Systems, North Puget Sound Sewerage Area

Piper Creek Subarea. Treatment of sewage from the Piper Creek subarea is to be obtained in a plant constructed at the site of the present treatment installation of the Greenwood Sewer District. Trunk sewers would consist (1) of the existing sewer of the Greenwood Sewer District, which extends southeastward from the plant along Piper Creek to the Greenwood area, and (2) a high elevation interceptor to the south to serve the North Beach-Blue Ridge area. Some local pumping would be required in the south branch.

At the treatment plant, primary treatment would be provided for an ultimate average flow of 3.7 mgd, with a peak hydraulic capacity of 12 mgd. Plant units would consist of preaeration and primary sedimentation tanks, separate sludge digestion tanks and other necessary structures and appurtenances. Chlorine contact tanks would not be required, since over 25 minutes detention time would be available in the outfall at ultimate average flow. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately 3,300 feet offshore to a water depth of about 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 2,400 feet offshore at a depth of about 265 feet.

Boeing Creek Subarea. Treatment of the sewage of the Boeing Creek subarea would be obtained in a plant at the mouth of Boeing Creek. Trunk sewers would consist of two branches, an east and a north. The east branch would be laid eastward from the plant along Boeing Creek to the Ronald area, and the north would be laid northward along the waterfront to Richmond Beach. A pumping station would be required on the north branch to lift its flow into the treatment plant. The Highlands area to the south would be served by local sewers connecting to the plant.

The treatment plant would be a primary type having an ultimate average capacity of 2.6 mgd and a peak hydraulic capacity of 7.5 mgd. Plant units would consist of preparation and primary sedimentation tanks, separate sludge digestion tanks, and all other necessary structures and appurtenances. Chlorine contact tanks would not be required, since about 25 minutes detention time would be available in the outfall at ultimate average flow. Digested sludge, after passing through a washer, would be discharged to Puget Sound through a submarine line extending approximately

2,000 feet offshore to a water depth of about 400 feet. Chlorinated effluent would be discharged to Puget Sound approximately 1,400 feet offshore at a depth of 180 feet.

Comparison of Independent Systems with Central Sewerage Project. In determining the cost to the two subareas for participation in the central sewerage project, detailed layouts and cost estimates were made only for facilities required to collect the sewage and pump it to the crest of the ridge separating the North Puget Sound sewerage area from the Northwest Lake Washington and Lake Union sewerage areas. Costs thus obtained, both construction and annual, were compared with corresponding costs for the independent sewerage systems. As indicated in Table 15-31, esti-

Table 15-30. Description and Estimated Construction Costs, Separate Systems for North Puget Sound Sewerage Area

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Piper Creek Subarea				
NPS-1	1.1	3.6	2,000 ft of 15-in. RC at 0.75%, average cut 6 ft, wet	35,000
NPS-2	1.0	3.1	1,400 ft of existing 21-in. at 0.28 - 3.7%	Existing
NPS-3	1.4	4.4	400 ft of existing 15-in. at 4.3 - 28% and 2,200 ft of existing 18-in. at 2.0 - 5.6%	Existing
NPS-4	2.3	7.3	300 ft of existing 21-in. at 2.5% and 500 ft of 24-in. at 1.6%	Existing
Subtotal, sewers				35,000
STP-NPS-1	3.7	12	Sewage treatment plant, primary type, includes facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 3,300 ft of 6-in. outfall sludge line to a water depth of 400 ft	576,000
NPS-5	3.7	12	2,000 ft of existing 27-in. effluent outfall, land section	Existing
NPS-6	3.7	12	2,400 ft of 33-in. RC submarine outfall to water depth of 265 ft, includes diffuser section over last 200 ft	188,000
Total contract cost, Piper Creek Subarea				799,000
Engineering and contingencies				200,000
Total construction cost, Piper Creek subarea				999,000
Boeing Creek Subarea				
NPS-7	0.7-1.0	1.8-2.9	4,900 ft of 12-in. RC at 0.65 - 10%, average cut 6 - 10 ft, wet	66,000
NPS-8	1.1-1.2	3.1-3.4	4,000 ft of 18-in. RC at 0.21 - 0.26%, average cut 10 - 15 ft, difficult wet	98,000
Subtotal, sewers				164,000
PS-NPS-1	1.2	3.4	Pumping station, single stage, motor driven, static lift 29 ft, total head at peak flow 35 ft, structure about 23 ft below ground	66,000
STP-NPS-2	2.6	7.5	Sewage treatment plant, primary type, includes facilities for screenings and grit removal, preaeration and primary sedimentation, sludge digestion and disposal, and effluent chlorination, as well as all necessary operation and maintenance facilities, includes 2,000 ft of 6-in. outfall sludge line to a water depth of 400 ft	452,000
NPS-9	2.6	7.5	1,600 ft of 24-in. RC effluent outfall to water depth of 180 ft, includes 200 ft land section, and diffuser section over last 135 ft	149,000
Total contract cost, Boeing Creek subarea				831,000
Engineering and contingencies, 25 per cent				208,000
Total construction cost, Boeing Creek subarea				1,039,000

See *Fig. 15-13* for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.* [b]*Does not include cost of acquiring existing facilities.*

mates for the independent systems are more than 40 per cent higher than those for pumping to the crest of the ridge. Because of these differences, an estimate was made of the proportionate costs to the two subareas of the central sewage treatment plant at West Point, proposed under Core Plan B. This indicated that the apportioned construction cost of the central plant would be $372,000 for the Piper Creek subarea and $279,000 for the Boeing Creek subarea, or a total of $651,000. The addition of these costs to the figures given in Table 15-31 makes the construction cost of central sewerage greater than that of the independent systems.

As for annual costs, the apportioned cost of the central treatment plant would be $36,000 per year to the Piper Creek subarea and $26,000 per year to the Boeing Creek subarea. It can thus be seen that addition of the apportioned costs of the West Point treatment plant only makes the annual costs for central sewerage about the same as those for the independent systems. By adding facilities required to convey sewage from the two subareas to West Point the cost of the central project would become considerably higher. It is apparent, therefore, that the more economical of the two plans for the Piper Creek and Boeing Creek subareas will be to provide independently in each area for sewage collection, treatment and disposal.

Table 15-31. Comparison of Construction and Annual Costs of Separate Plan and Central Sewerage Project, Piper Creek and Boeing Creek Subareas, North Puget Sound Sewerage Area

	Construction cost,[a] $1,000	Annual cost,[b] $1,000
Separate plan[c]		
Piper Creek	999	103
Boeing Creek	1,039	98
Total	2,038	201
Central sewerage[d]		
Piper Creek	637	68
Boeing Creek	796	70
Total	1,433	138

[a] *Includes engineering and contingencies.*

[b] *Includes fixed costs and maintenance and operation costs.*

[c] *From Table 15-30.*

[d] *Does not include apportioned cost to subarea of any sewers within Northwest Lake Washington and Lake Union Sewerage Areas or of core plan facilities.*

SERVICE SEWERS

In each sewerage area, sewers were laid out to serve a minimum tributary area of 1,000 acres. These sewers, designated herein as service sewers, will convey sewage from each local service area to the point of concentration in the area, at which point the sewage will be discharged to facilities provided under the central sewerage project.

In the case of the South Puget Sound and North Puget Sound areas, where independent sewerage projects were found to be the more satisfactory, service sewers required to convey sewage to the individual treatment plants were included in the layout of those systems. These sewers are shown in Figs. 15-12 and 15-13.

North Lake Sammamish Sewerage Area

Service sewers for the North Lake Sammamish sewerage area are laid out to convey the sewage of that area to a pumping station about two miles north of the city of Redmond (Fig. 15-14). Descriptions and estimated costs of these facilities are given in Table 15-32. Major elements of the system are as follows:

1. A waterfront interceptor (NLS 1-NLS 7) to serve areas draining directly into the lake from the east. A pumping station (PS-NLS 1) would be required to lift sewage from this interceptor into the trunk sewer serving areas to the north and east.
2. A trunk (NLS 8-NLS 11) to serve the Crystal and Cottage Lake areas to the north. A pumping station (PS-NLS 2) would be required at Cottage Lake to lift sewage from the east side of the lake into the main trunk.
3. A trunk (NLS 12-NLS 13) along Cottage Lake Creek to its confluence with Bear Creek. Sewage from NLS 11 would be discharged to this trunk.
4. A trunk (NLS 14-NLS 25) along Bear Creek to its confluence with Cottage Lake Creek. This trunk would serve areas north of Paradise Lake and, by means of branch trunks, would also serve areas along Stuve Creek and Seidel Creek. A pumping station (PS-NLS 3) would be required at Paradise Lake to avoid excessive cuts.
5. A trunk (NLS 26-NLS 28) along Bear Creek to its confluence with Evans Creek. This trunk would serve areas draining to Bear Creek.
6. A trunk (NLS 29-NLS 32) along Evans Creek to its confluence with Bear Creek. This trunk would serve areas to the east.
7. A trunk (NLS 33) along Bear Creek from the confluence of Bear and Evans Creek to the intersection with the east waterfront interceptor.
8. A trunk (NLS 34) along the south city limit of Redmond.
9. A waterfront interceptor (NLS 35-NLS 38) along the west shore of Lake Sammamish to serve areas draining directly into the lake.
10. A main trunk (NLS 39-NLS 41) along the Sammamish River valley to the pumping station (PS-S1) included in the feeder sewer system.

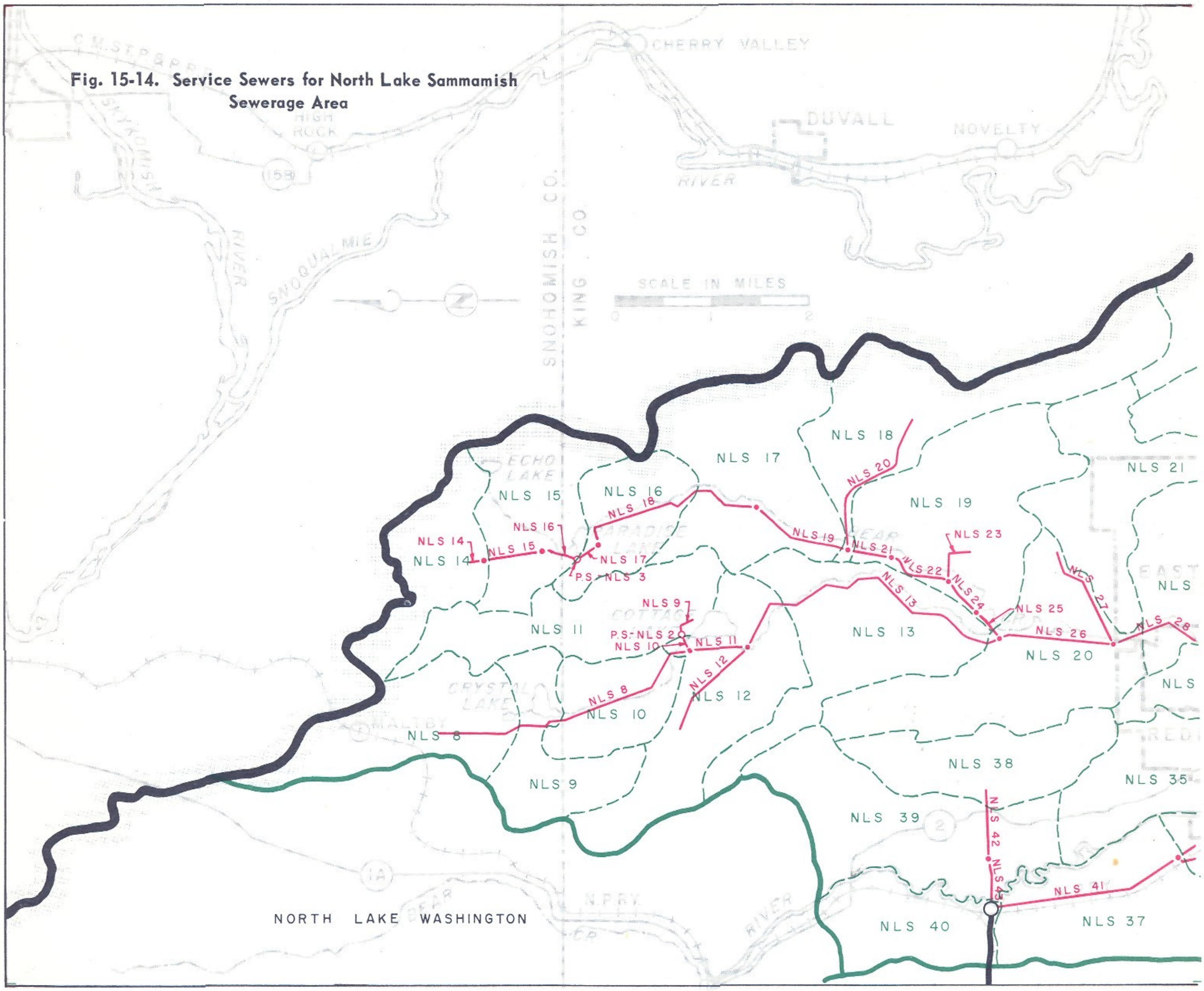


Fig. 15-14. Service Sewers for North Lake Sammamish Sewerage Area

11. A trunk (NLS 42-NLS 43) east of the pumping station (PS-S1) to serve areas along the eastern slope of Sammamish River valley.

South Lake Sammamish Sewerage Area

Service sewers for the South Lake Sammamish sewerage area are laid out to convey the sewage of that area to a pumping station situated on the waterfront of the lake east of Phantom Lake (Fig. 15-15). Descriptions and estimated costs of these facilities are given in Table 15-33. Major elements of the system are as follows:

1. A trunk (SLS 1-SLS 9) along Issaquah Creek to its confluence with the East Fork of Issaquah Creek. This trunk would serve the south portion of the area and most of the city of Issaquah.

2. A trunk (SLS 10) along the East Fork of Issaquah Creek to its confluence with Issaquah Creek.

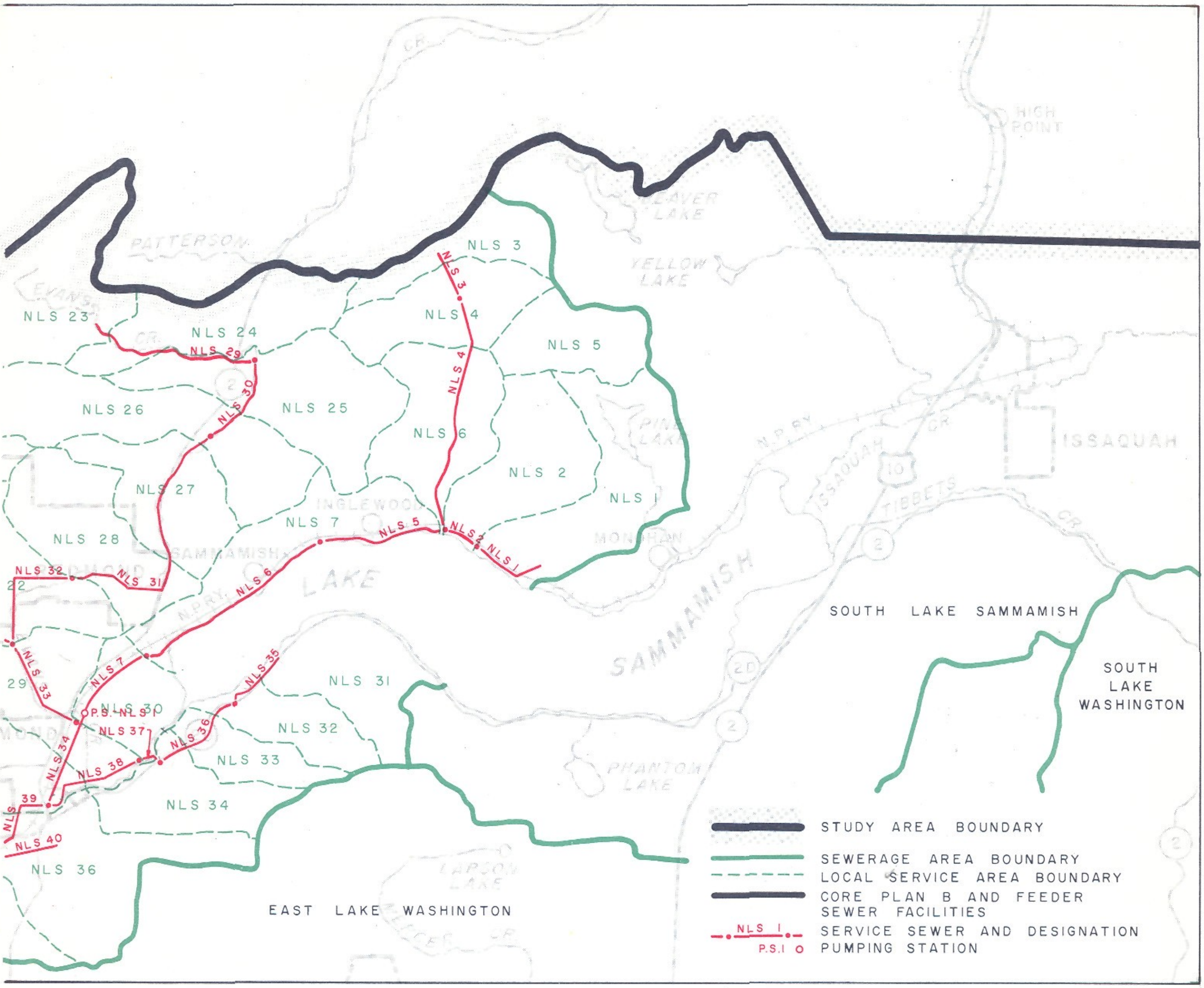
3. A trunk (SLS 11) along U.S. Highway 10.

4. A trunk (SLS 12-SLS 15) along Tibbetts Creek. Sewer SLS 11 would discharge into this trunk.

5. A trunk (SLS 16-SLS 20) to the north and east to serve the Beaver Lake area, as well as areas draining directly to Lake Sammamish.

6. A trunk (SLS 21) along the shore of Lake Sammamish to a pumping station (PS-SLS 1) situated on the lake front. The pumping station would discharge through a force main (SLS 22) to a high elevation waterfront interceptor (SLS 23).

7. A high elevation waterfront interceptor (SLS 23-SLS 24) to the pumping station (PS-S2) included in the feeder sewer system. This sewer would serve high



areas west of Lake Sammamish. Local pumping would be required for waterfront areas which could not sewer by gravity to the two pumping stations.

East Lake Washington Sewerage Area

Service sewers for the East Lake Washington sewerage area are laid out to convey the sewage of that area into the feeder sewer system (Fig. 15-15). Descriptions and estimated costs of these facilities are given in Table 15-33. Major elements of the system are as follows:

1. A high level waterfront interceptor (ELW 1-ELW 8) along the shore of Lake Washington and north of Juanita Bay to a pumping station (PS-ELW 2), which would discharge through a force main (ELW 9) into the feeder system. A pumping station (PS-ELW 1) would be required along the route of the sewer. Some local pumping would also be required.
2. A pumping station (PS-ELW 3) at the site of the existing sewage treatment plant of the city of Kirkland. This station would discharge through a force main (ELW 10) to the feeder sewers and would serve areas sewering to the Kirkland treatment plant.
3. A high elevation waterfront interceptor (ELW 11-ELW 13) to serve areas in Hunts Point, Medina and Clyde Hill. This interceptor would discharge to a pumping station (PS-ELW 5), which in turn would discharge through a force main ELW 14) into the feeder sewers. A pumping station (PS-ELW 4) would be required along the route of the sewer. Local pumping would be required for areas which could not be served by gravity.

Table 15-32. Description and Estimated Construction Costs, Service Sewers, North Lake Sammamish Sewerage Area

Facility	Design flow,[a] mgd		Description	Construction cost, dollars
	Average DWF	Maximum WWF		
NLS-1	0.6	2.0	3,900 ft of 15-in. RC at 0.64%, average cut 6 ft, wet, includes railroad and highway crossings	62,000
NLS-2	0.9-1.2	2.5-3.4	2,700 ft of 18-in. RC at 0.14 - 0.25%, average cut 7 - 8 ft, wet	44,000
NLS-3	0.3	0.9	2,900 ft of 12-in. RC at 0.16%, average cut 8 ft, difficult wet, includes sheeting and special bedding	66,000
NLS-4	0.6-1.5	2.1-4.5	12,900 ft of 15-in. RC at 0.26 - 5.2%, average cut 9 - 24 ft, dry to difficult wet in swampy material, includes sheeting and special bedding in swamp and railroad and highway crossings	321,000
NLS-5	2.6	8.0	7,100 ft of 30-in. RC at 0.09%, average cut 10 - 12 ft, wet	182,000
NLS-6	3.0	9.0	12,200 ft of 33-in. RC at 0.064 - 0.075%, average cut 13 - 15 ft, wet to difficult wet in peat, includes sheeting and special bedding in peat	395,000
NLS-7	4.4	12	5,300 ft of 36-in. RC at 0.07%, average cut 18 ft, wet to difficult wet in peat, includes sheeting and special bedding in peat	260,000
NLS-8	0.9-1.8	2.4-4.9	15,700 ft of 18-in. RC at 0.13 - 1.5%, average cut 6 - 11 ft, wet to difficult wet in swampy material, includes sheeting and special bedding in swamp	349,000
NLS-9	0.5	1.5	1,400 ft of 12-in. RC at 0.45%, average cut 10 ft, difficult wet	26,000
NLS-10	0.8	1.9	400 ft of 8-in. force main, difficult wet	4,000
NLS-11	2.5	7.0	2,900 ft of 21-in. RC at 0.5%, average cut 7 - 8 ft, wet	52,000
NLS-12	0.7	2.0	5,300 ft of 12-in. RC at 1.1%, average cut 6 ft, dry to difficult wet	63,000
NLS-13	3.2-4.0	9.2 - 12	17,400 ft of 24-in. RC at 0.4 - 1.1%, average cut 7 - 10 ft, dry to difficult wet	325,000
NLS-14	0.6	1.6	800 ft of 12-in. RC at 0.62%, average cut 6 ft, difficult wet	12,000
NLS-15	0.8	2.1	3,400 ft of 15-in. RC at 0.26%, average cut 9 ft, difficult wet in swampy material, includes sheeting and special bedding	95,000
NLS-16	1.0	2.8	1,300 ft of 18-in. RC at 0.17%, average cut 14 ft, difficult wet in swampy material, includes sheeting and special bedding	47,000
NLS-17	1.5	4.0	1,100 ft of 12-in. force main	8,000
NLS-18	1.6-2.4	4.5-6.8	12,500 ft of 18-in. RC at 0.54 - 1.0%, average cut 8 - 11 ft, dry to moderately wet, includes imported backfill and repaving	171,000
NLS-19	2.7	7.6	4,300 ft of 21-in. RC at 0.68%, average cut 6 ft, dry to moderately wet	58,000
NLS-20	0.5-0.9	1.7-2.7	9,500 ft of 12-in. RC at 2.1 - 2.4%, average cut 6 ft, dry to moderately wet	88,000
NLS-21	3.4	10	2,600 ft of 21-in. RC at 1.2%, average cut 6 ft, dry to moderately wet.	35,000
NLS-22	3.6	10	4,000 ft of 24-in. RC at 0.5%, average cut 7 ft, dry to moderately wet	65,000
NLS-23	0.3-0.6	1.0-1.9	3,400 ft of 12-in. RC at 1.3 - 2.0%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	34,000
NLS-24	4.2	12	2,500 ft of 24-in. RC at 0.8%, average cut 7 ft, dry to moderately wet.	41,000
NLS-25	4.3	12	1,900 ft of 27-in. RC at 0.53%, average cut 9 ft, dry to moderately wet	36,000
NLS-26	8.3-9.2	24-27	6,700 ft of 36-in. RC at 0.31 - 0.38%, average cut 10 - 11 ft, dry to moderately wet, includes imported backfill and repaving	166,000
NLS-27	0.6	2.0	6,400 ft of 12-in. RC at 2.0%, average cut 6 ft, dry to moderately wet	60,000
NLS-28	10	29	5,000 ft of 39-in. RC at 0.29 - 0.41%, average cut 7 - 9 ft, dry to wet	141,000
NLS-29	0.7	2.2	9,800 ft of 12-in. RC at 4.3%, average cut 6 ft, dry to moderately wet, includes access roads	98,000
NLS-30	0.9-1.6	2.9-4.9	5,200 ft of 18-in. RC at 0.25 - 0.5%, average cut 6 - 10 ft, difficult wet in peaty material, includes sheeting and special bedding	130,000

Continued on next page

Table 15-32. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost, dollars
NLS-31	2.6	7.8	14,400 ft of 27-in. RC at 0.14 - 0.19%, average cut 10 - 12 ft, dry to difficult wet in peaty material, includes sheeting and special bedding in peat, imported backfill and repaving	360,000
NLS-32	3.4	10	7,600 ft of 30-in. RC at 0.15 - 0.35%, average cut 7 - 9 ft, wet, includes imported backfill and repaving	205,000
NLS-33	14-16	41-43	5,800 ft of 48-in. RC at 0.21 - 0.33%, average cut 8 ft, wet, includes railroad and highway crossings	228,000
NLS-34	19	56	5,300 ft of 72-in. RC at 0.045%, average cut 11 ft, difficult wet	360,000
NLS-35	0.6	2.2	3,400 ft of 15-in. RC at 0.2 - 0.37%, average cut 6 - 8 ft, wet, includes highway crossing	54,000
NLS-36	1.0-1.5	2.8-3.8	5,800 ft of 18-in. RC at 0.17 - 0.31%, average cut 10 - 11 ft, wet, includes imported backfill and repaving	114,000
NLS-37	2.0	5.2	1,100 ft of 21-in. RC at 0.26%, average cut 10 ft, difficult wet, includes Sammamish River crossing	27,000
NLS-38	2.8-3.0	6.7-7.5	6,100 ft of 30-in. RC at 0.065 - 0.082%, average cut 8 - 10 ft, difficult wet, includes sheeting and highway crossing	210,000
NLS-39	22-23	63-66	4,900 ft of 84-in. RC at 0.024 - 0.028%, average cut 12 - 13 ft, difficult wet, includes sheeting and railroad, highway and Sammamish River crossings	445,000
NLS-40	0.8	2.0	3,300 ft of 12-in. RC at 1.9%, average cut 6 ft, wet, includes railroad and highway crossings	48,000
NLS-41	24-25	68-70	11,300 ft of 84-in. RC at 0.03%, average cut 14 - 19 ft, difficult wet, includes sheeting	1,100,000
NLS-42	0.6	1.7	3,800 ft of 15-in. RC at 0.4%, average cut 6 ft, difficult wet, includes sheeting	90,000
NLS-43	1.5	4.3	2,800 ft of 18-in. RC at 0.4%, average cut 12 ft, difficult wet, includes sheeting and Sammamish River crossing	125,000
Subtotal, sewers				6,800,000
PS-NLS-1	4.4	12	Pumping station, single stage, motor driven with engine standby, static lift 16 ft, total head at peak flow 25 ft, structure about 25 ft below ground, includes sheeting and dewatering	148,000
PS-NLS-2	0.7	1.9	Pumping station, single stage, motor driven with engine standby, static lift 22 ft, total head at peak flow 40 ft, structure about 15 ft below ground, includes dewatering	47,000
PS-NLS-3	1.4	4.0	Pumping station, single stage, motor driven with engine standby, static lift 31 ft, total head at peak flow 60 ft, structure about 18 ft below ground, includes dewatering	73,000
Subtotal, pumping stations				268,000
Total contract cost				7,068,000
Engineering and contingencies, 25 per cent				1,767,000
Total construction cost				8,835,000

See Fig. 15-14 for location of facilities. [a]*Expressed as average dry weather flow and maximum wet weather flow.*

4. A trunk (ELW 17-ELW 19) along Richie Road and U. S. Highway 10 to a pumping station (PS-ELW 6), which would discharge through a force main (ELW 20) into the feeder sewers. This trunk would serve Factorial and areas to the east.

5. A waterfront interceptor (ELW 21-ELW 22) along the shore of Meydenbauer Bay to a pumping station (PS-ELW 7) at the site of the existing sewage treatment plant of the city of Bellevue. The pumping station would discharge through a force main and

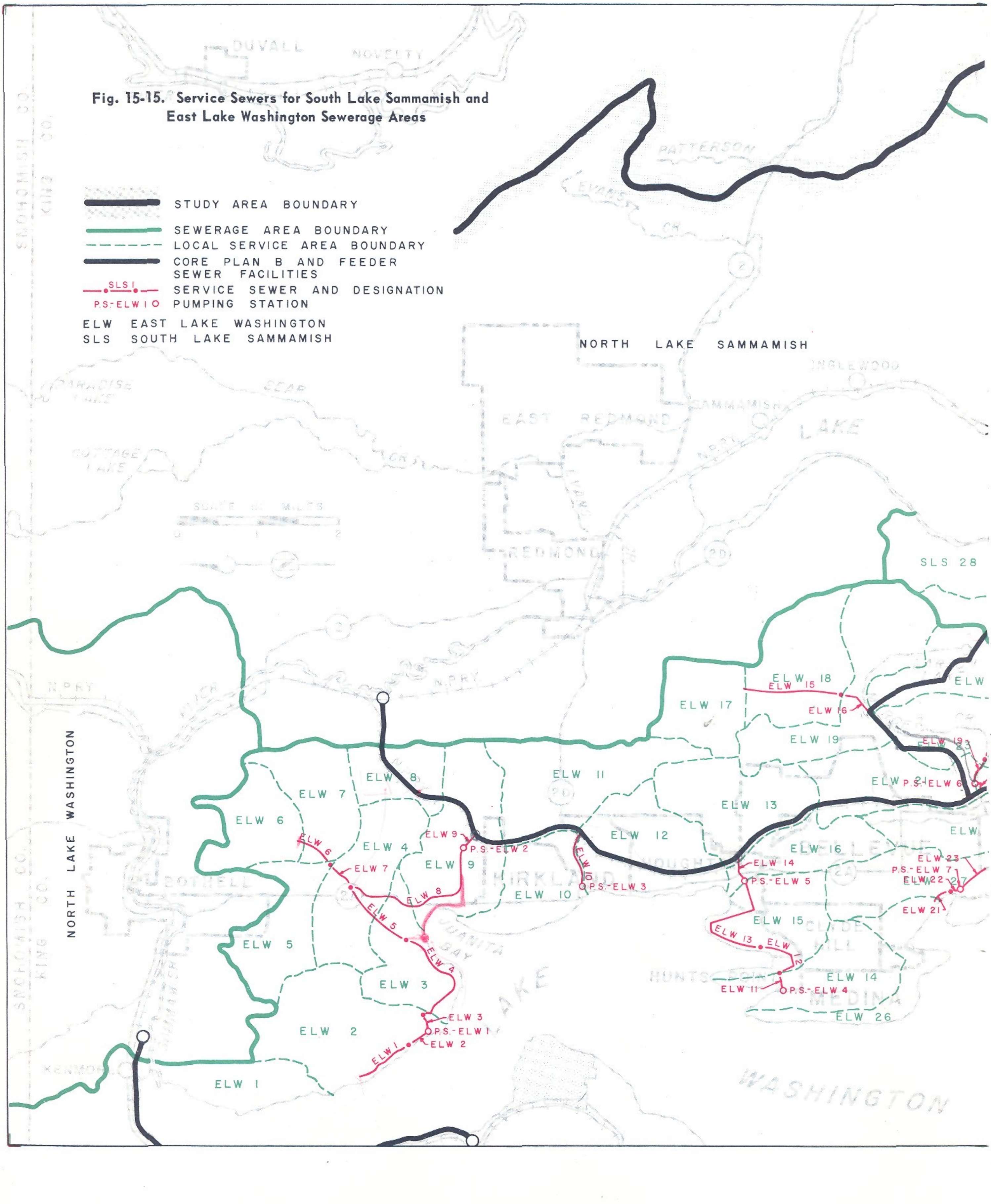


Fig. 15-15. Service Sewers for South Lake Sammamish and East Lake Washington Sewerage Areas

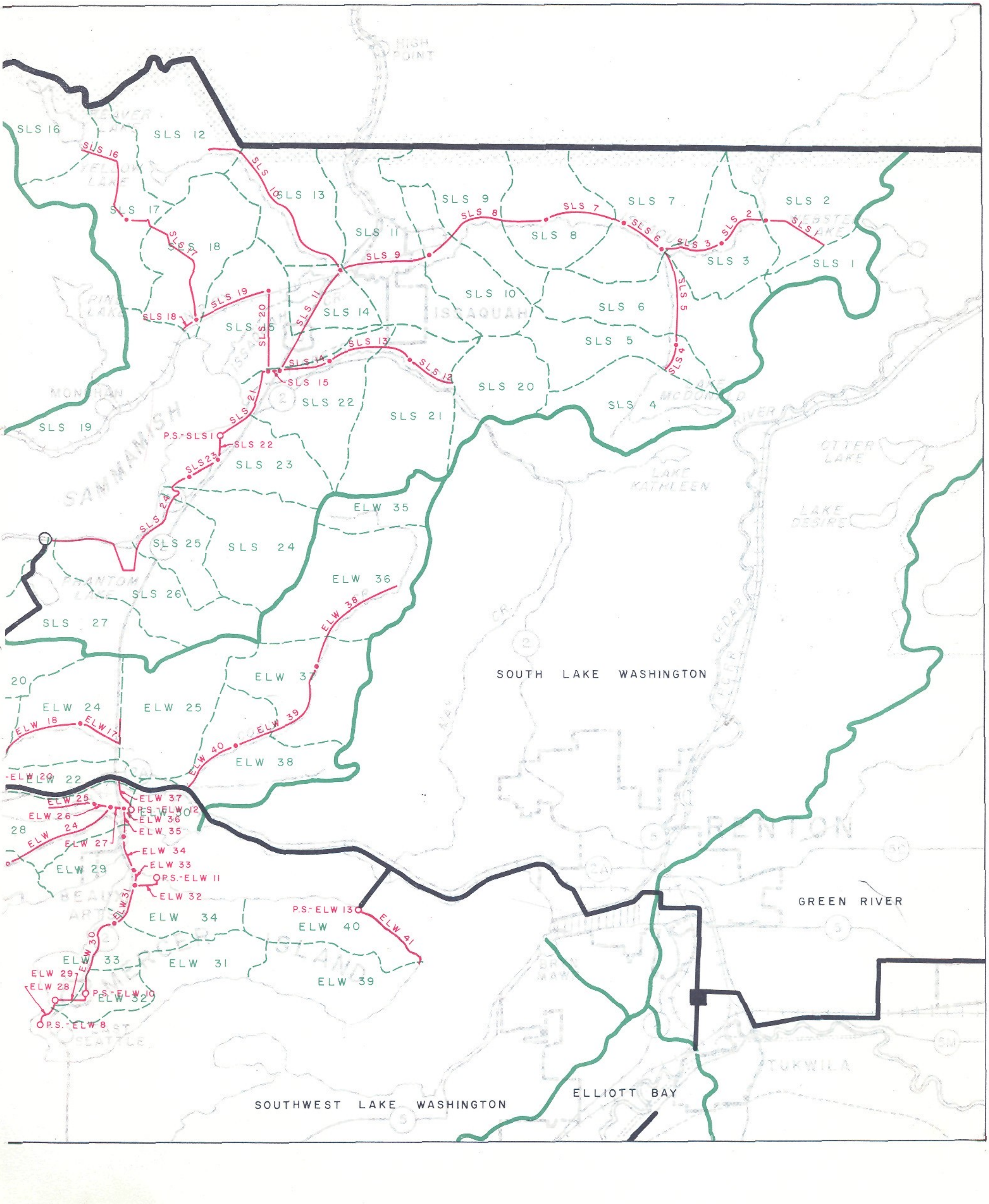

HIGH POINT
SLS 16
SLS 12
SLS 13
SLS 11
SLS 9
SLS 7
SLS 2
SLS 8
SLS 3
SLS 1
SLS 17
SLS 18
SLS 10
SLS 6
SLS 14
SLS 15
SLS 5
ISSAQUAH
SLS 20
SLS 4
SLS 22
SLS 21
SLS 19
SAMMAMISH
P.S.-SLS 1
SLS 23
ELW 35
SLS 25
SLS 24
ELW 36
PHANTOM LAKE
SLS 26
SLS 27
OTTER LAKE
LAKE KATHLEEN
LAKE DESIRE
ELW 37
ELW 24
ELW 25
ELW 38
ELW 22
ELW 29
P.S.-ELW 12
P.S.-ELW 11
P.S.-ELW 13
ELW 34
ELW 40
ELW 33
ELW 31
ELW 39
MERCER ISLAND
P.S.-ELW 10
P.S.-ELW 8
ELW 32
SOUTH LAKE WASHINGTON
RENTON
GREEN RIVER
TUKWILA
ELLIOTT BAY
SOUTHWEST LAKE WASHINGTON

Table 15-33. Description and Estimated Construction Costs, Service Sewers, South Lake Sammamish and East Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
South Lake Sammamish Sewerage Area				
SLS-1	0.4	1.3	4,200 ft of 12-in. RC at 1.2%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	48,000
SLS-2	1.1	3.1	3,500 ft of 15-in. RC at 1.4%, average cut 6 ft, dry to moderately wet	38,000
SLS-3	1.6	4.3	4,700 ft of 18-in. RC at 0.85%, average cut 6 ft, dry to moderately wet	57,000
SLS-4	0.5	1.5	1,600 ft of 12-in. RC at 0.45%, average cut 7 ft, dry to moderately wet	15,000
SLS-5	0.8-1.6	2.8-4.5	6,300 ft of 15-in. RC at 0.45 - 1.8%, average cut 8 - 9 ft, dry to moderately wet	70,000
SLS-6	3.2-3.6	9.1-10	3,600 ft of 21-in. RC at 1.1%, average cut 7 - 9 ft, dry to moderately wet	50,000
SLS-7	3.8	11	5,200 ft of 24-in. RC at 0.6%, average cut 9 ft, dry to moderately wet, includes imported backfill and repaving	97,000
SLS-8	4.6	13	6,300 ft of 27-in. RC at 0.47%, average cut 7 - 8 ft, dry to wet, includes imported backfill and repaving	126,000
SLS-9	4.8-5.2	14-17	8,500 ft of 30-in. RC at 0.32-0.57%, average cut 7 - 8 ft, wet to difficult wet, includes imported backfill and repaving	251,000
SLS-10	0.4-0.8	1.7-2.8	13,200 ft of 12-in. RC at 0.55 - 4.4%, average cut 6 - 9 ft, dry to difficult wet, includes railroad and highway crossings	150,000
SLS-11	7.2	21	7,900 ft of 33-in. RC at 0.4%, average cut 7 - 10 ft, difficult wet, includes imported backfill and repaving	255,000
SLS-12	0.4-0.8	1.1-2.5	3,700 ft of 12-in. RC at 1.2 - 3.8%, average cut 6 ft, dry to moderately wet, includes repaving	38,000
SLS-13	1.4	3.9	5,100 ft of 15-in. RC at 2.5%, average cut 6 ft, wet, includes imported backfill and repaving	87,000
SLS-14	1.8	5.3	3,500 ft of 18-in. RC at 0.86%, average cut 7 ft, difficult wet	64,000
SLS-15	8.9	26	600 ft of 36-in. RC at 0.37%, average cut 13 ft, difficult wet, includes sheeting	24,000
SLS-16	0.4-0.6	1.1-2.1	7,300 ft of 12-in. RC at 0.37 - 0.87%, average cut 6 ft, dry to wet	97,000
SLS-17	0.9-1.4	2.7-4.4	10,700 ft of 15-in. RC at 0.43 - 6.0%, average cut 6 - 8 ft, dry to difficult wet, includes railroad and highway crossings	170,000
SLS-18	0.4	1.3	700 ft of 12-in. RC at 0.33%, average cut 9 ft, difficult wet	12,000
SLS-19	1.8	5.6	5,200 ft of 27-in. RC at 0.08%, average cut 13 ft, difficult wet, includes sheeting	185,000
SLS-20	2.4	7.3	5,200 ft of 30-in. RC at 0.07%, average cut 14 ft, difficult wet, includes sheeting and Issaquah Creek crossing	204,000
SLS-21	12	34	5,600 ft of 48-in. RC at 0.10%, average cut 14 - 15 ft, difficult wet, includes sheeting	312,000
SLS-22	12	36	1,900 ft of 30-in. force main	33,000
SLS-23	12	36	2,100 ft of 42-in. RC at 0.48%, average cut 7 ft, dry to moderately wet	60,000
SLS-24	13-14	38-40	15,300 ft of 48-in. RC at 0.18 - 0.27%, average cut 9 - 11 ft, dry to wet, includes imported backfill and repaving	621,000
Subtotal, sewers				3,064,000
PS-SLS-1	12	36	Pumping station, single stage, motor driven with engine standby, static lift 86 ft, total head at peak flow 115 ft, structure about 20 ft below ground, includes sheeting and dewatering	291,000

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Table 15-33. Continued

Facility	Design flow,[a] mgd Average DWF	Design flow,[a] mgd Maximum WWF	Description	Construction cost,[b] dollars
Total contract cost, South Lake Samamish Sewerage Area				3,355,000
Engineering and contingencies, 25 per cent				839,000
Total construction cost, South Lake Sammamish Sewerage Area				4,194,000
East Lake Washington Sewerage Area				
ELW-1	0.7	1.8	4,000 ft of 12-in. RC at 0.5 - 2.5%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	38,000
ELW-2	1.3	3.4	1,700 ft of 18-in. RC at 0.25%, average cut 8 ft, dry to moderately wet, includes imported backfill and repaving	27,000
ELW-3	1.3	3.4	1,400 ft of 12-in. force main, includes imported backfill and repaving	11,000
ELW-4	1.8	4.7	8,600 ft of 18-in. RC at 0.43 - 0.54%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	111,000
ELW-5	2.0-2.3	5.2-6.0	4,800 ft of 21-in. RC at 0.27 - 0.35%, average cut 7 - 11 ft, dry to moderately wet	67,000
ELW-6	0.6	1.4	2,400 ft of 12-in. RC at 1.2%, average cut 6 ft, dry to moderately wet	22,000
ELW-7	1.9	4.7	2,300 ft of 15-in. RC at 1.5 - 2.3%, average cut 6 - 9 ft, dry to moderately wet	25,000
ELW-8	4.1-5.0	11-13	10,800 ft of 33-in. RC at 0.10 - 0.15%, average cut 9 - 13 ft, dry to moderately wet, includes imported backfill and repaving	275,000
ELW-9	5.4	14	1,100 ft of 20-in. force main	12,000
ELW-10	0.9	3.3	4,100 ft of 12-in. force main, includes imported backfill and repaving	36,000
ELW-11	0.7	1.8	1,100 ft of 8-in. force main	7,000
ELW-12	0.8	2.0	3,900 ft of 15-in. RC at 0.28%, average cut 7 - 9 ft, wet	59,000
ELW-13	0.9-1.3	2.5-3.2	9,300 ft of 18-in. RC at 0.20%, average cut 6 - 9 ft, wet to difficult wet	158,000
ELW-14	1.9	4.9	1,500 ft of 14-in. force main	11,000
ELW-15	0.5	1.4	6,600 ft of 12-in. RC at 1.4%, average cut 6 ft, dry to moderately wet	61,000
ELW-16	1.4-1.9	3.8-5.0	2,600 ft of 15-in. RC at 0.85 - 1.8%, average cut 7 ft, dry to difficult wet, includes sheeting and highway crossing	42,000
ELW-17	0.4-0.8	1.1-2.3	4,400 ft of 12-in. RC at 1.0 - 4.7%, average cut 6 - 8 ft, dry to difficult wet, includes imported backfill, repaving and highway crossing	68,000
ELW-18	1.5	4.0	6,500 ft of 18-in. RC at 0.37%, average cut 8 ft, wet to difficult wet, includes sheeting	133,000
ELW-19	1.9	5.2	2,100 ft of 21-in. RC at 0.26%, average cut 8 ft, wet	38,000
ELW-20	2.2	5.8	700 ft of 14-in. force main, includes railroad crossing	9,000
ELW-21	0.6	1.7	1,600 ft of 12-in. RC at 0.55%, average cut 10 ft, difficult wet	29,000
ELW-22	1.4	4.2	500 ft of 18-in. RC at 0.38%, average cut 14 ft, difficult wet	13,000
ELW-23	1.5	4.4	2,500 ft of 14-in. force main, includes imported backfill and repaving	24,000
ELW-24	1.5	4.4	8,400 ft of 18-in. RC at 1.1%, average cut 7 ft, dry to difficult wet in peaty material, includes sheeting and special bedding in peat, imported backfill, repaving, and slough crossing	175,000
ELW-25	0.6	1.7	3,300 ft of 12-in. RC at 0.55%, average cut 13 ft, difficult wet in peaty material, includes sheeting, special bedding and piling	139,000
ELW-26	0.9	2.6	800 ft of 15-in. RC at 0.38%, average cut 22 ft, difficult wet in peaty material, includes sheeting, special bedding and piling	46,000
ELW-27	2.5	6.5	400 ft of 27-in. RC at 0.11%, average cut 23 ft, difficult wet in peaty material, includes sheeting, special bedding and piling	31,000

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Table 15-33. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
ELW-28	1.1	3.3	2,800 ft of existing 12-in. force main	Existing
ELW-29	1.5	4.5	2,700 ft of 10-in. force main to parallel existing 10-in. force main, includes imported backfill and repaving	22,000
ELW-30	1.8	5.6	6,400 ft of 12-in. force main to parallel existing 12-in. force main, includes imported backfill, repaving and highway crossing	64,000
ELW-31	1.8	5.6	2,500 ft of 18-in. RC at 0.22 - 2.4%, to parallel existing 12-in. and 18-in. sewers, includes imported backfill and repaving	36,000
ELW-32	0.5	1.8	2,100 ft of 10-in. force main, includes highway crossing	20,000
ELW-33	2.3	7.2	800 ft of existing 16-in. at 4.8 - 10.9%	Existing
ELW-34	2.3	7.2	1,600 ft of existing 16-in. inverted siphon across Lake Washington	Existing
ELW-35	2.3	7.2	2,500 ft of 18-in. RC at 1.2 - 1.8%, average cut 6 - 9 ft, wet to difficult wet in peaty material, includes sheeting, special bedding and piling in peat, imported backfill, repaving, slough crossing and connections to existing sewers at Mercer Island Sewer District pumping station No. 8 which is to be abandoned	74,000
ELW-36	5.0	15	1,600 ft of 39-in. RC at 0.08%, average cut 23 - 25 ft, difficult wet in peaty material, includes sheeting, special bedding, piling and slough crossing	158,000
ELW-37	5.5	16	1,100 ft of 24-in. force main, difficult wet in peaty material, includes sheeting, piling and special bedding	49,000
ELW-38	0.4-1.3	1.1-3.6	11,000 ft of 12-in. RC at 2.0 - 6.7%, average cut 6 ft, dry to moderately wet, includes access road	106,000
ELW-39	1.8	4.9	5,100 ft of 15-in. RC at 2.0%, average cut 6 ft, dry to moderately wet, includes access road	58,000
ELW-40	2.2	6.4	5,000 ft of 18-in. RC at 1.0 - 1.5%, average cut 6 - 8 ft, dry to wet, includes railroad crossing	76,000
ELW-41	0.5	1.4	4,300 ft of 12-in. RC at 0.4%, average cut 9 ft, wet	59,000
Subtotal, sewers				2,389,000
PS-ELW-1	1.3	3.4	Pumping station, single stage, motor driven with engine standby, static lift 80 ft, total head at peak flow 105 ft, structure about 14 ft below ground	63,000
PS-ELW-2	3.4	14	Pumping station, two stage, motor driven with engine standby, static lift 110 ft, total head at peak flow 135 ft, structure about 15 ft below ground	180,000
PS-ELW-3	0.9	3.3	Pumping station, two stage, motor driven with engine standby, static lift 110 ft, total head at peak flow 175 ft, structure about 20 ft below ground. Station located at site of existing Kirkland sewage treatment plant which is to be abandoned, includes sheeting, dewatering and connections to existing sewers	88,000
PS-ELW-4	0.7	1.8	Pumping station, single stage, motor driven with engine standby, static lift 40 ft, total head at peak flow 80 ft, structure about 15 ft below ground, difficult wet construction, includes sheeting and dewatering	50,000
PS-ELW-5	1.9	4.9	Pumping station, two stage, motor driven with engine standby, static lift 130 ft, total head at peak flow 155 ft, structure about 15 ft below ground, difficult wet construction, includes sheeting and dewatering	105,000
PS-ELW-6	2.2	5.8	Pumping station, single stage, motor driven with engine standby, static lift 92 ft, total head at peak flow 110 ft, structure about 14 ft below ground, includes sheeting and dewatering	100,000

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Table 15-33. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
PS-ELW-7	1.4	4.4	Pumping station, two stage, motor driven with engine standby, static lift 100 ft, total head at peak flow 135 ft, difficult wet construction. Station located at site of existing Bellevue sewage treatment plant which is to be abandoned, includes sheeting, dewatering and connections to existing sewers	111,000
PS-ELW-8	1.0	3.3	Pumping station, single stage, motor driven with engine standby, static lift 20 ft, total head at peak flow 65 ft. To replace existing Mercer Island Sewer District pumping station No. 3 which has a total installed capacity of 1.1 mgd, includes connection to existing sewers	62,000
PS-ELW-9	1.5	4.5	Pumping station, two stage, motor driven with engine standby, static lift 100 ft, total head at peak flow 150 ft. To replace existing Mercer Island Sewer District pumping station No. 2 which has a total installed capacity of 1.7 mgd, includes connections to existing sewers	90,000
PS-ELW-10	1.7	5.6	Pumping station, single stage, motor driven with engine standby, static lift 10 ft, total head at peak flow 80 ft. To replace existing Mercer Island Sewer District pumping station No. 6 which has a total installed capacity of 1.9 mgd, includes connections to existing sewers	85,000
PS-ELW-11	0.6	1.8	Pumping station, single stage, motor driven with engine standby, static lift 50 ft, total head at peak flow 80 ft, structure about 15 ft below ground. Station located at site of existing East Mercer Island Sewer District sewage treatment plant which is to be abandoned, includes connections to existing sewers	48,000
PS-ELW-12	5.6	16	Pumping station, single stage, motor and engine driven, static lift 80 ft, total head at peak flow 95 ft, structure about 30 ft below ground, difficult wet construction in peaty material, includes special foundations, sheeting and dewatering	212,000
PS-ELW-13	1.1	2.8	Pumping station, single stage, motor driven with engine standby, static lift 15 ft, total head at peak flow 55 ft, structure about 15 ft below ground	56,000
Subtotal, pumping stations				1,250,000
Total contract cost, East Lake Washington Sewerage Area				3,639,000
Engineering and contingencies, 25 per cent				910,000
Total construction cost, East Lake Washington Sewerage Area				4,549,000

See Fig. 15-15 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

gravity sewers (ELW 23, 24, 27 and 36) to a second pumping station (PS-ELW 12), which in turn would discharge through a force main (ELW 37) into the feeder sewers.

6. A trunk (ELW 25-ELW 26) in Mercer Slough to serve areas draining directly into the slough.

7. A trunk system (ELW 28-ELW 35) on Mercer Island, including the crossing to the east side of Lake Washington. This system would serve the northern half of Mercer Island and would incorporate existing sewers of the Mercer Island Sewer District. Three pumping stations (PS-ELW 8, PS-ELW 9 and PS-ELW 10) would be required along the routes of the trunk sewers. In addition a fourth pumping station (PS-ELW 11) would be required at the site of the existing sewage treatment plant of the East Mercer Sewer District to pump the sewage from that point into the trunk.

8. A trunk (ELW 38-ELW 40) along Coal Creek to serve areas to the south and east.

9. A waterfront interceptor (ELW 41) to a pumping station (PS-ELW 13) at the southeastern end of Mercer

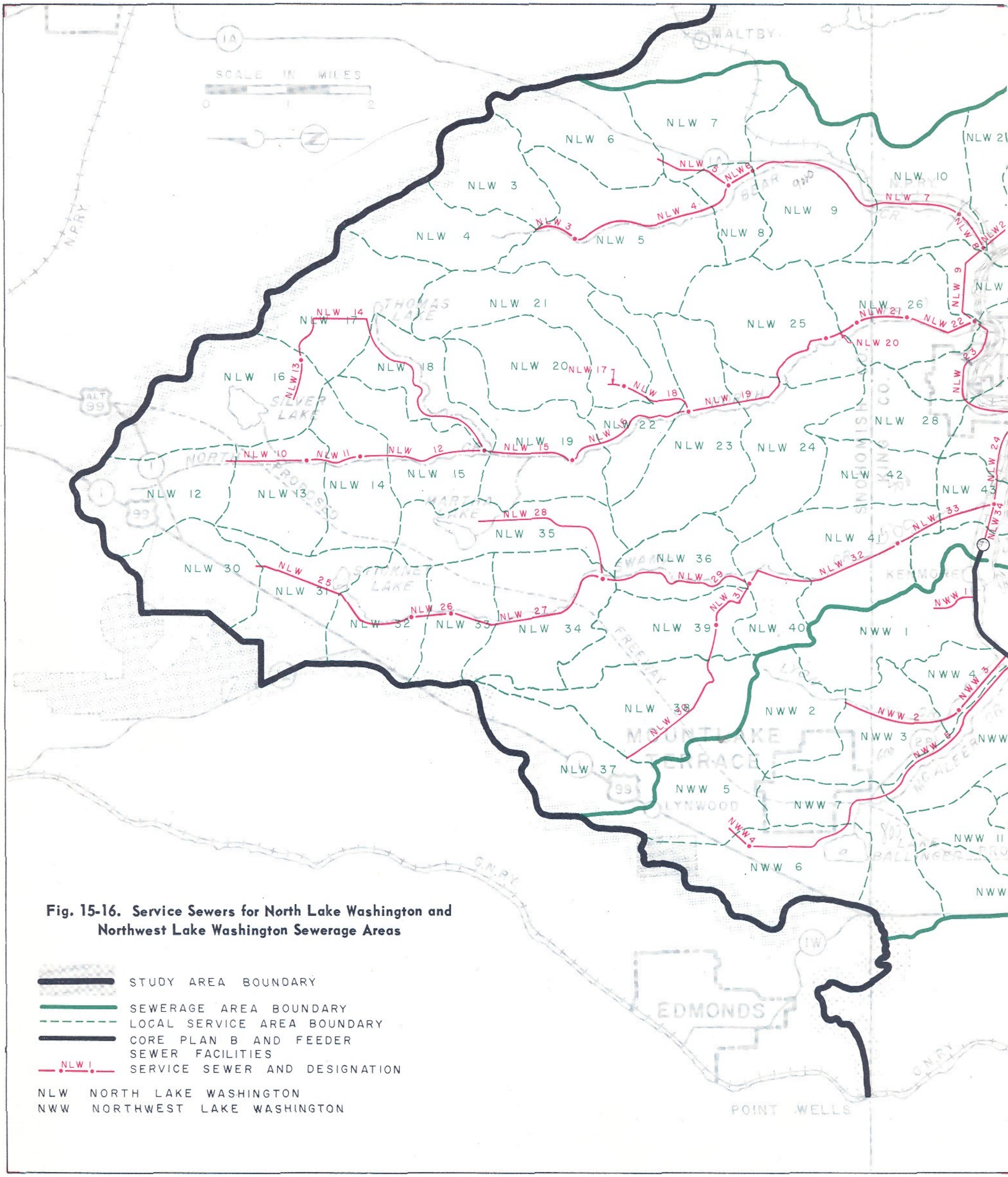
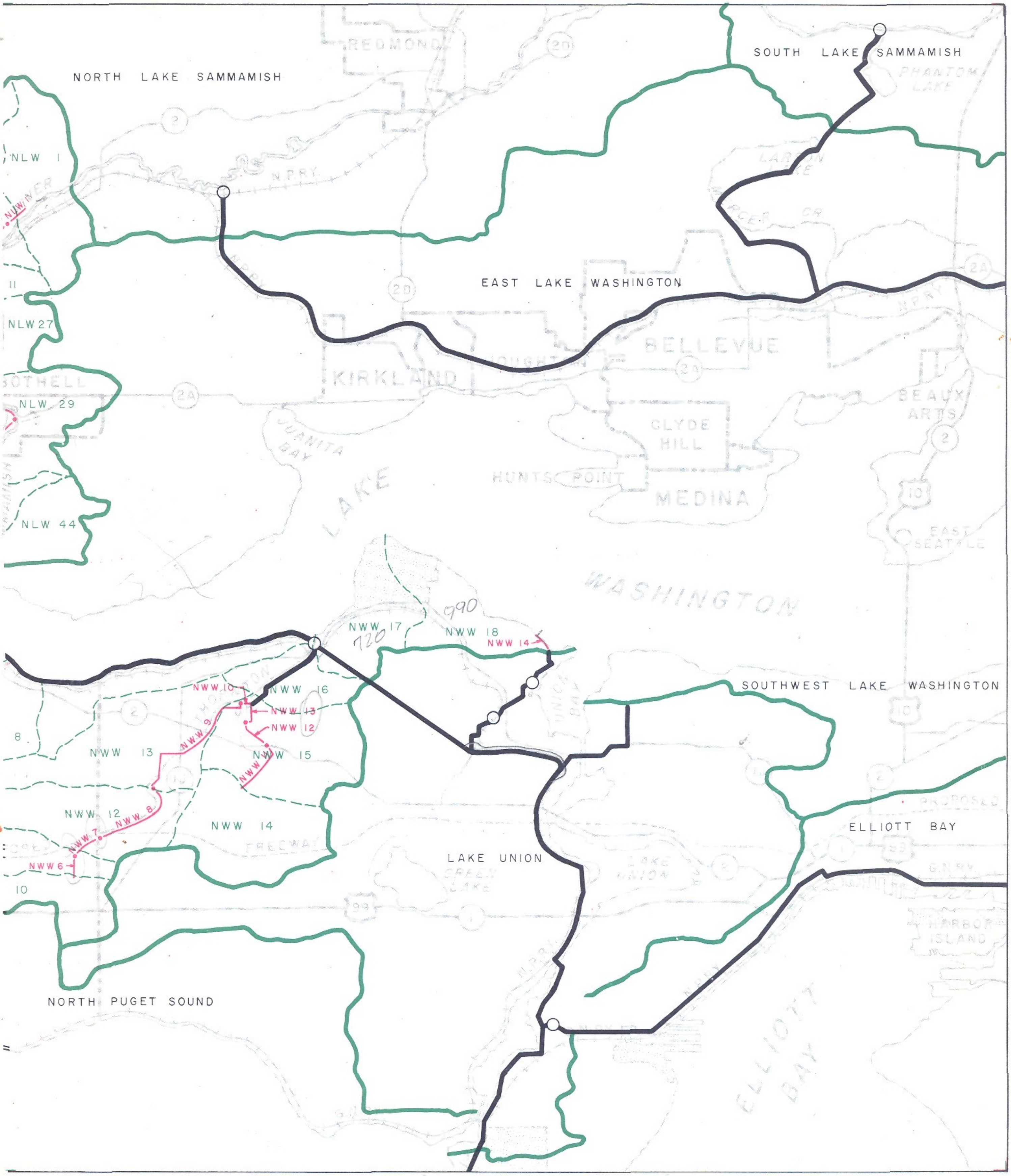


Fig. 15-16. Service Sewers for North Lake Washington and Northwest Lake Washington Sewerage Areas



NORTH LAKE SAMMAMISH
SOUTH LAKE SAMMAMISH
PHANTOM LAKE
REDMOND
EAST LAKE WASHINGTON
KIRKLAND
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
BEAUX ARTS
EAST SEATTLE
JUANITA BAY
LAKE WASHINGTON
NLW 1
NLW 27
NLW 29
NLW 44
NWW 17
NWW 18
NWW 14
NWW 16
NWW 13
NWW 12
NWW 15
NWW 10
NWW 9
NWW 8
NWW 7
NWW 6
NWW 14
SOUTHWEST LAKE WASHINGTON
LAKE UNION
GREEN LAKE
ELLIOTT BAY
HARBOR ISLAND
NORTH PUGET SOUND
N.P.RY.
G.N.RY.
PROPOSED
FREEWAY
990
720

Table 15-34. Description and Estimated Construction Costs, Service Sewers, North Lake Washington and Northwest Lake Washington Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
North Lake Washington Sewerage Area				
NLW-1	0.7	2.0	1,600 ft of 15-in. RC at 0.24%, average cut 11 ft, difficult wet, includes sheeting	46,000
NLW-2	1.2	3.4	2,300 ft of 18-in. RC at 0.25%, average cut 13 ft, difficult wet, includes sheeting	84,000
NLW-3	1.2	3.0	2,200 ft of 15-in. RC at 0.52%, average cut 7 ft, difficult wet	37,000
NLW-4	1.9-2.7	5.1-7.2	12,100 ft of 24-in. RC at 0.12-1.0%, average cut 7 ft, difficult wet	270,000
NLW-5	1.0	2.7	5,100 ft of 12-in. RC at 2.3 - 3.4%, average cut 6 ft, dry to moderately wet	47,000
NLW-6	3.7	9.9	2,000 ft of 24-in. RC at 0.60%, average cut 7 ft, dry to moderately wet	33,000
NLW-7	4.0-6.7	11-18	15,900 ft of 27-in. RC at 0.33 - 1.5%, average cut 7 ft, wet to difficult wet	~~365,000~~
NLW-8	7.2	20	2,800 ft of 30-in. RC at 0.57 - 0.70%, average cut 9 - 12 ft, difficult wet, includes sheeting	92,000
NLW-9	8.8	24	6,900 ft of 39-in. RC at 0.20%, average cut 10 - 12 ft, difficult wet, includes sheeting	297,000
NLW-10	0.8	2.0	5,500 ft of 12-in. RC at 1.2%, average cut 8 ft, dry to moderately wet	53,000
NLW-11	1.3	3.6	3,900 ft of 15-in. RC at 1.5%, average cut 6 ft, dry to moderately wet	42,000
NLW-12	2.0-2.6	5.2-6.9	9,300 ft of 18-in. RC at 0.83 - 1.2%, average cut 6 ft, wet, includes special bedding	173,000
NLW-13	0.8	2.0	2,300 ft of 12-in. RC at 1.5%, average cut 8 ft, dry to moderately wet	30,000
NLW-14	1.0-2.3	2.6-6.1	21,600 ft of 18-in. RC at 0.15 - 1.4%, average cut 7 - 17 ft, wet to difficult wet in peaty material, includes special bedding in peat, imported backfill and repaving	464,000
NLW-15	5.1	14	5,800 ft of 24-in. RC at 0.90%, average cut 7 - 12 ft, wet, includes special bedding	143,000
NLW-16	5.9	16	8,900 ft of 27-in. RC at 0.70 - 1.1%, average cut 7 - 12 ft, wet	211,000
NLW-17	0.5	1.4	900 ft of 12-in. RC at 1.1%, average cut 6 ft, wet	12,000
NLW-18	1.6	4.3	5,300 ft of 15-in. RC at 1.5 - 1.9%, average cut 6 ft, wet	69,000
NLW-19	7.5-9.7	20-26	12,700 ft of 33-in. RC at 0.34 - 0.86%, average cut 7 - 8 ft, wet to difficult wet	354,000
NLW-20	10	27	2,400 ft of 36-in. RC at 0.40%, average cut 10 ft, difficult wet	74,000
NLW-21	11	28	3,000 ft of 39-in. RC at 0.27%, average cut 9 ft, difficult wet	101,000
NLW-22	11	30	4,900 ft of 42-in. RC at 0.21%, average cut 8-10 ft, difficult wet	178,000
NLW-23	20-21	54-57	11,600 ft of 66-in. RC at 0.065 - 0.07%, average cut 12 - 17 ft, difficult wet, includes sheeting, piling and highway crossing	911,000
NLW-24	21-22	58-59	6,500 ft of 72-in. RC at 0.047%, average cut 20 - 22 ft, difficult wet, includes sheeting, piling and railroad crossing	684,000
NLW-25	0.6-1.6	1.6-4.1	11,200 ft of 15-in. RC at 0.28 - 1.0%, average cut 6 - 12 ft, dry to difficult wet	195,000
NLW-26	1.9	5.1	3,300 ft of 18-in. RC at 0.79%, average cut 7 ft, difficult wet	61,000
NLW-27	2.3-3.1	6.2-8.3	11,800 ft of 21-in. RC at 0.37 - 0.76%, average cut 7-12 ft, wet to difficult wet, includes sheeting	292,000
NLW-28	0.4-0.8	1.1-2.1	10,200 ft of 12-in. RC at 1.5-1.6%, average cut 7 - 8 ft, wet to difficult wet	151,000
NLW-29	4.2	11	11,600 ft of 24-in. RC at 0.61 - 1.0%, average cut 7 ft, wet	235,000

Continued on next page

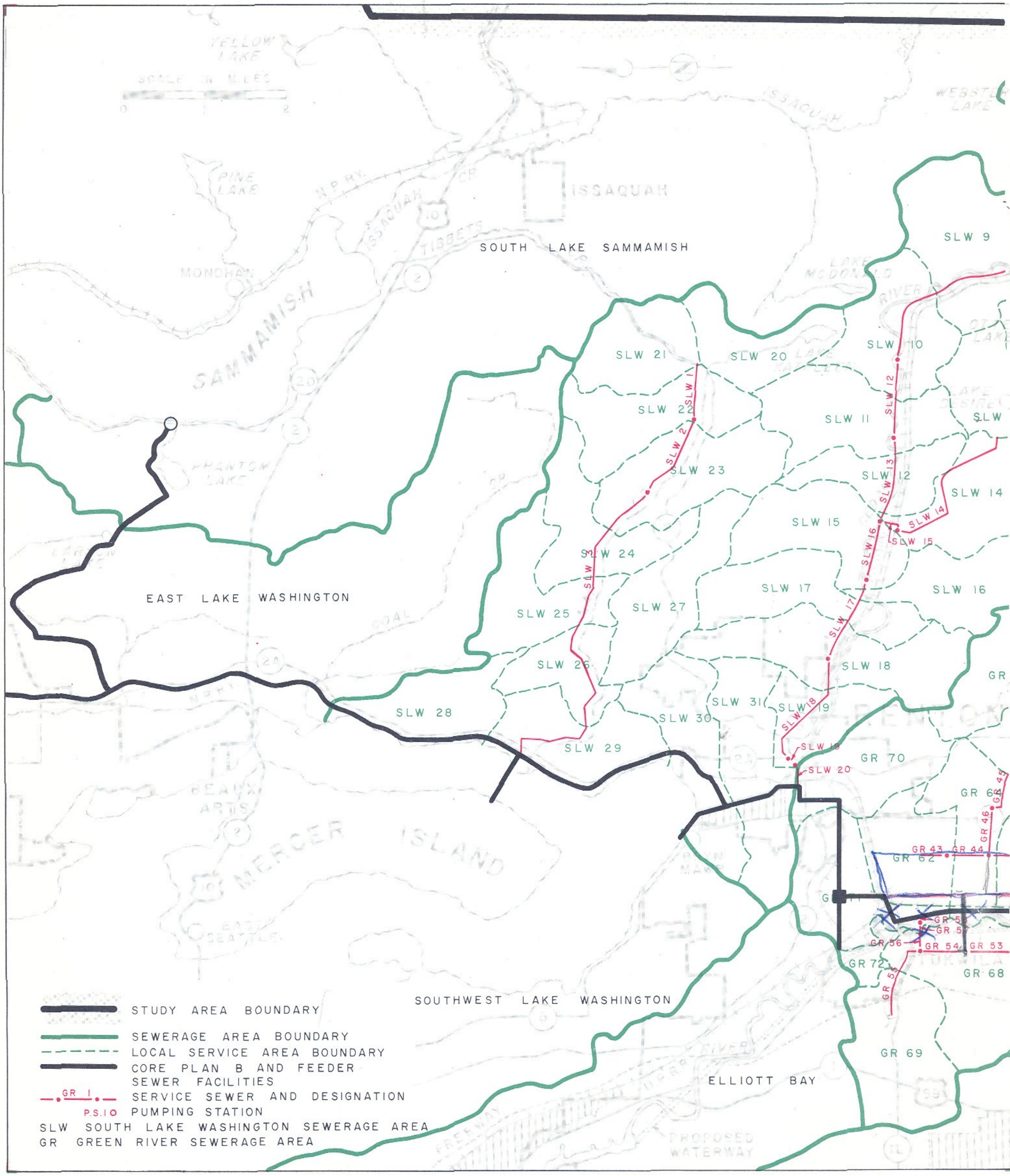
Table 15-34. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
NLW-30	0.8-1.7	2.1-4.6	13,100 ft of 15-in. RC at 0.29 - 1.3%, average cut 6 - 7 ft, difficult wet in peaty material, includes special bedding	245,000
NLW-31	2.4	6.3	4,700 ft of 18-in. RC at 1.2%, average cut 7 ft, wet to difficult wet in peaty material, includes special bedding	95,000
NLW-32	7.0	19	12,000 ft of 27-in. RC at 0.85 - 1.3%, average cut 11 ft, wet to difficult wet	303,000
NLW-33	8.0-8.8	22-23	7,500 ft of 30-in RC at 0.65 - 0.80%, average cut 9 - 14 ft, difficult wet, includes railroad and highway crossings	232,000
NLW-34	31	82	2,100 ft of 72-in. RC at 0.095%, average cut 23 ft, difficult wet, includes sheeting and piling	228,000
Total contract cost, North Lake Washington Sewerage Area				6,807,000
Engineering and contingencies, 25 per cent				1,702,000
Total construction cost, North Lake Washington Sewerage Area				8,509,000
Northwest Lake Washington Sewerage Area				
NWW-1	0.9	2.3	2,000 ft of 12-in. RC at 3.5%, average cut 8 ft, wet, includes railroad and highway crossings	35,000
NWW-2	0.6-1.4	1.6-3.6	7,500 ft of 12-in. RC at 1.6 - 3.1%, average cut 6 - 7 ft, wet, includes imported backfill and repaving	113,000
NWW-3	1.5-1.8	4.0-4.6	4,700 ft of 15-in. RC at 1.6%, average cut 6-7 ft, wet to difficult wet, includes railroad and highway crossings	80,000
NWW-4	0.6	1.6	2,500 ft of 12-in. RC at 0.56%, average cut 6 ft, wet	33,000
NWW-5	1.3-3.7	3.6-9.3	24,000 ft of 18-in. RC at 0.43 - 1.9%, average cut 6 - 19 ft, wet to difficult wet in peaty material, includes sheeting and special bedding in peat, imported backfill, repaving and railroad and highway crossings	568,000
NWW-6	0.9	2.2	1,500 ft of 15-in. RC at 0.33%, average cut 6 ft, difficult wet	24,000
NWW-7	1.7	4.0	2,700 ft of 18-in. RC at 0.56%, average cut 6 ft, difficult wet	48,000
NWW-8	2.1-2.6	5.4-6.8	6,100 ft of existing 30-in. RC at 0.5 - 3.6%	Existing
NWW-9	3.4-5.7	9.1-16	9,800 ft of existing 27-in. and 30-in. RC at 0.6 - 13.5%	Existing
NWW-10	5.9	17	1,000 ft of existing 42-in. RC at 0.3%	Existing
NWW-11	1.0-1.4	2.5-3.4	4,700 ft of 12-in. RC at 1.8 - 2.2%, average cut 6 - 7 ft, dry to moderately wet, includes highway crossing	48,000
NWW-12	2.2	5.6	2,700 ft of existing 30-in. RC at 0.9 - 2.7%	Existing
NWW-13	2.7	7.3	1,600 ft of existing 36-in. RC at 0.45%	Existing
NWW-14	1.5	4.4	700 ft of existing 42-in. at 0.05%. Cost is for 1,100,000, 400,000, 1,400,000, 2,150,000 and 640,000 gal. holding tanks on tributary local sewers	549,000
Total contract cost, Northwest Lake Washington Sewerage Area				1,498,000
Engineering and contingencies, 25 per cent				374,000
Total construction cost, Northwest Lake Washington Sewerage Area				1,872,000

See Fig. 15-16 for location of facilities.

[a] *Expressed as average dry weather flow and maximum wet weather flow.*

[b] *Does not include cost of acquiring existing facilities.*


SOUTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
SOUTHWEST LAKE WASHINGTON
ISSAQUAH
MERCER ISLAND
ELLIOTT BAY
SLW 9
SLW 10
SLW 11
SLW 12
SLW 14
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 68
GR 69
GR 70
GR 72
STUDY AREA BOUNDARY
SEWERAGE AREA BOUNDARY
LOCAL SERVICE AREA BOUNDARY
CORE PLAN B AND FEEDER SEWER FACILITIES
SERVICE SEWER AND DESIGNATION
P.S.10 PUMPING STATION
SLW SOUTH LAKE WASHINGTON SEWERAGE AREA
GR GREEN RIVER SEWERAGE AREA

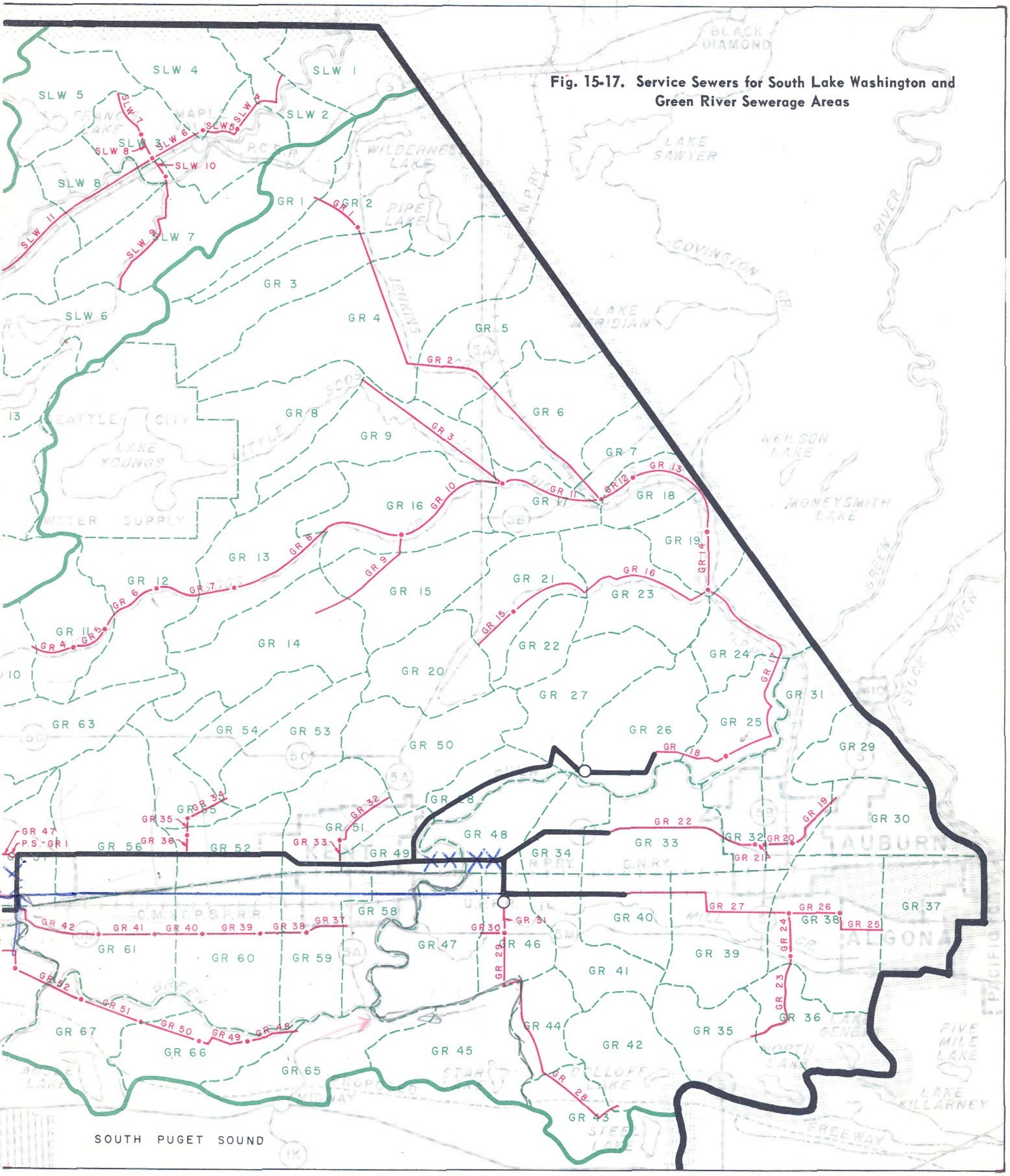


Fig. 15-17. Service Sewers for South Lake Washington and Green River Sewerage Areas

Island, which would discharge into a force main included under the feeder sewer system. The location of the pumping station may have to be changed during final design if it is decided that the route of the crossing to the east side of Lake Washington must avoid the submerged forests in the lake. This possibility does not affect the basic layouts either of the service sewers or of the central sewerage system. ELW 41 and PS-ELW 13 serve the southern portion of Mercer Island.

North Lake Washington Sewerage Area

Service sewers for the North Lake Washington sewerage area are laid out to convey the sewage of that area to a pumping station about two miles west of Bothell (Fig. 15-16). Descriptions and estimated costs of these facilities are given in Table 15-34. Major elements of the system are as follows:

1. A main trunk (NLW 1, 2, 9, 23, 24, and 34) along Sammamish River valley from about 4,000 feet southeast of Woodinville to the pumping station west of Bothell (PS-N 1). The latter is included in the feeder sewer system.
2. A trunk (NLW 3-NLW 8) along Bear Creek to serve the Bear Creek basin.
3. A trunk (NLS10-NLW22) along North and Penny creeks. This trunk would serve the Intercity and Silver Lake areas.
4. A trunk (NLW 25-NLW 33) along Swamp Creek. This trunk would serve the Fairmount, Mirror Lake and Lynwood areas.

Northwest Lake Washington Sewerage Area

Service sewers for the Northwest Lake Washington sewerage area consist of two systems (Fig. 15-16). Sewers for the first system are laid out to convey sewage of the northern portion of this area into the feeder sewers which are routed along the shore of Lake Washington. Sewers for the second system serve a small area bordering Lake Washington south of Sand Point Naval Air Station and discharge into the feeder sewers in the Lake Union sewerage area. Descriptions and estimated costs of the facilities in both systems are given in Table 15-34. Major elements are as follows:

1. A trunk (NWW 1) west of Kenmore.
2. A trunk (NWW 2-NWW 3) along Lyons Creek to serve a portion of Mountlake Terrace and surrounding areas.
3. A trunk (NWW 4-NWW 5) along McAleer and Hall creeks to serve the Lake Ballinger and surrounding areas, including the remainder of Mountlake Terrace.
4. A trunk (NWW 6-NWW 10) north from the existing Lake City sewage treatment plant of the city of Seattle. This trunk would incorporate existing sewers of the city of Seattle and would serve the northern part of the city, as well as the Ronald area and other areas to the north.
5. A trunk (NWW 11-NWW 13) west and south from the existing Lake City treatment plant. This trunk would incorporate existing sewers of the city of Seattle.
6. A trunk serving the waterfront area south of Sand Point Naval Air Station (NWW 14). This trunk would incorporate existing sewers of the city of Seattle and would receive sewage from the areas served by combined sewers. Storm water holding tanks sized to allow one overflow per summer would be provided on local sewers tributary to the trunk.

South Lake Washington Sewerage Area

Service sewers for the South Lake Washington sewerage area are laid out to convey the sewage of that area into the feeder sewer system (Fig. 15-17). Descriptions and estimated costs of these facilities are given in Table 15-35. Major elements of the system are as follows:

1. A trunk (SLW 1-SLW 3) along May Creek.
2. A trunk (SLW 4-SLW 20) along Cedar River to serve Maple Valley and other areas to the east, as well as all local areas draining to Cedar River.

Green River Sewerage Area

Service sewers for the Green River sewerage area are laid out to convey the sewage of that area into the feeder sewer system (Fig. 15-17). Descriptions and estimated costs of these facilities are given in Table 15-35. Major elements of the system are as follows:

1. A trunk (GR 1-GR 2) along Jenkins Creek to serve the Piper and Wilderness Lake areas. This trunk would also serve the major portion of the proposed Covington industrial area.
2. A trunk (GR 3-GR 14) along Big and Little Soos creeks. This trunk would serve areas to the north almost to the south city limit of Renton, as well as a portion of the proposed Covington industrial area.
3. A trunk (GR 15-GR 16) to serve areas west of Lake Meridian.
4. A trunk (GR 17-GR 18) along Green River which would discharge to the feeder sewer system. This trunk would serve areas east of Green river valley.
5. A trunk (GR 19-GR 22) along State Highway 5 to serve the city of Auburn and areas to the south which lie between Green River and the Great Northern railroad. This trunk would incorporate existing sewers of the city of Auburn.
6. A trunk (GR 23-GR 27) to serve areas west of the Great Northern railroad, including the city of Algona.
7. A trunk (GR 28-GR 31) to serve the Steel and Star Lake areas, as well as areas south of Green River.
8. Trunks (GR 32-GR 36) to serve part of the city of Kent and areas to the east of that city.

Table 15-35. Description and Estimated Construction Costs, Service Sewers, South Lake Washington and Green River Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
South Lake Washington Sewerage Area				
SLW-1	1.0-1.5	2.7-4.3	3,600 ft of 18-in. RC at 0.16 - 0.4%, average cut 7-11 ft, difficult wet, includes sheeting	95,000
SLW-2	1.8	5.2	5,400 ft of 21-in. RC at 0.26%, average cut 15 - 19 ft, difficult wet, includes sheeting	227,000
SLW-3	2.2-5.1	6.2-14	24,000 ft of 24-in. RC at 0.18 - 2.4%, average cut 6 - 24 ft, dry to difficult wet, includes sheeting and railroad and highway crossings	602,000
SLW-4	0.5-0.9	1.5-2.5	4,700 ft of 12-in. RC at 1.1 - 1.5%, average cut 6 - 9 ft, dry to moderately wet	45,000
SLW-5	0.9	2.5	2,900 ft of 15-in. RC at 0.36%, average cut 17 - 20 ft, dry to moderately wet	40,000
SLW-6	2.0	5.4	3,900 ft of 18-in. RC at 0.80%, average cut 12 ft, dry to moderately wet, includes imported backfill and repaving	77,000
SLW-7	0.5	1.5	2,800 ft of 12-in. RC at 3.8%, average cut 6 ft, dry to moderately wet	26,000
SLW-8	1.1	3.1	1,900 ft of 15-in. RC at 1.1%, average cut 6 ft, dry to moderately wet	21,000
SLW-9	0.6-0.9	1.6-2.7	8,300 ft of 12-in. RC at 1.0 - 3.0%, average cut 6 ft, dry to moderately wet, includes access road and Cedar River crossing	86,000
SLW-10	0.9	2.7	1,400 ft of 18-in. RC at 0.16%, average cut 7 ft, dry to moderately wet, includes railroad crossing	20,000
SLW-11	3.9-5.7	11-16	23,100 ft of 27-in. RC at 0.44 - 0.66%, average cut 7 - 8 ft, dry to wet, includes imported backfill, repaving, and Cedar River crossing	523,000
SLW-12	6.2	18	5,100 ft of 30-in. RC at 0.45%, average cut 8 ft, wet	125,000
SLW-13	7.0	20	5,800 ft of 33-in. RC at 0.35%, average cut 7 - 8 ft, wet	149,000
SLW-14	0.5-1.2	1.5-3.4	11,700 ft of 12-in. RC at 0.74 - 4.4%, average cut 6 ft, dry to moderately wet, includes access road	117,000
SLW-15	1.2	3.4	1,700 ft of 15-in. RC at 1.1%, average cut 7 ft, wet, includes railroad crossing	28,000
SLW-16	9.0	26	5,000 ft of 33-in. RC at 0.57 - 0.64%, average cut 7 ft, wet	127,000
SLW-17	10-11	28-31	5,100 ft of 36-in. RC at 0.4 - 0.5%, average cut 8 - 11 ft, wet, includes imported backfill, repaving, and Cedar River crossing	169,000
SLW-18	12-13	33-35	8,300 ft of 42-in. RC at 0.27 - 0.59%, average cut 10 - 13 ft, wet to difficult wet, includes imported backfill and repaving	364,000
SLW-19	13	35	400 ft of twin 24-in. inverted siphons, includes inlet and outlet structures	28,000
SLW-20	13	35	1,700 ft of 42-in. RC at 0.29%, average cut 15 ft, difficult wet, includes sheeting, dewatering, imported backfill and repaving	104,000
Total contract cost, South Lake Washington Sewerage Area				2,973,000
Engineering and contingencies, 25 per cent				743,000
Total construction cost, South Lake Washington Sewerage Area				3,716,000
Green River Sewerage Area				
GR-1	1.0	2.4	2,700 ft of 18-in. RC at 0.18%, average cut 6 ft, dry to moderately wet	33,000
GR-2	4.8-9.6	11-22	26,900 ft of 36-in. RC at 0.08 - 2.4%, average cut 7 ft, dry to difficult wet, includes railroad crossings	676,000
GR-3	2.2-2.9	4.5-6.3	10,400 ft of 18-in. RC at 0.44 - 1.4%, average cut 6 ft, dry to moderately wet	126,000

Continued on next page

Table 15-35. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
GR-4	0.7	1.8	2,500 ft of 12-in. RC at 0.72%, average cut 6 ft, dry to moderately wet	23,000
GR-5	1.2	3.0	2,300 ft of 18-in. RC at 0.22%, average cut 6 ft, difficult wet	41,000
GR-6	1.3	3.6	4,200 ft of 21-in. RC at 0.13%, average cut 8 ft, difficult wet	85,000
GR-7	2.0	5.3	5,200 ft of 27-in. RC at 0.075%, average cut 9 ft, wet to difficult wet	127,000
GR-8	2.4-2.8	6.3-7.3	11,600 ft of 30-in. RC at 0.06 - 0.24%, average cut 8 - 10 ft, wet to difficult wet	286,000
GR-9	0.9	2.6	9,400 ft of 24-in. RC at 1.3%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	94,000
GR-10	3.7-5.3	9.8-14	8,300 ft of 36-in. RC at 0.055 - 0.16%, average cut 7 ft, dry to moderately wet	183,000
GR-11	8.2-8.5	20-21	7,300 ft of 36-in. RC at 0.41 - 0.86% average cut 7 ft, dry to moderately wet	161,000
GR-12	18	44	3,100 ft of 36-in. RC at 1.6%, average cut 7 ft, dry to moderately wet	68,000
GR-13	18	44	8,100 ft of 39-in. RC at 0.86%, average cut 7 ft, dry to moderately wet	211,000
GR-14	19	46	4,200 ft of 48-in. RC at 0.25%, average cut 9 ft, dry to moderately wet	145,000
GR-15	0.6	1.6	2,900 ft of 12-in. RC at 0.5%, average cut 8 ft, dry to moderately wet	28,000
GR-16	1.0-2.0	2.9-5.3	15,000 ft of 15-in. RC at 0.7 - 1.9%, average cut 6 - 8 ft, dry to moderately wet, includes access road	176,000
GR-17	21-22	51-53	15,800 ft of 48-in. RC at 0.32 - 0.84%, average cut 12 - 23 ft, dry to moderately wet, includes access road	623,000
GR-18	22	54	6,500 ft of 51-in. RC at 0.25 - 0.53%, average cut 14 - 16 ft, wet, includes imported backfill and repaving	406,000
GR-19	0.3	0.8	3,500 ft of 12-in. RC at 0.57%, average cut 8 ft, difficult wet, includes imported backfill and repaving	45,000
GR-20	3.3	7.4	2,400 ft of existing 24-in. at 0.24%	Existing
GR-21	4.0	9.0	600 ft of existing 30-in. at 0.13%	Existing
GR-22	4.0-4.6	9.0-11	9,700 ft of 30-in. RC at 0.14 - 0.67%, average cut 7 - 8 ft, difficult wet, includes imported backfill and repaving	288,000
GR-23	0.8-1.4	2.0-3.6	6,500 ft of 12-in. RC at 2.5 - 5.0%, average cut 6 ft, dry to moderately wet, includes imported backfill and repaving	73,000
GR-24	1.4	3.6	2,900 ft of 21-in. RC at 0.21%, average cut 8 ft, difficult wet, includes sheeting	81,000
GR-25	2.4-2.9	4.9-6.1	3,900 ft of 24-in. RC at 0.13 - 0.18%, average cut 7 - 9 ft, difficult wet, includes imported backfill, repaving and sheeting	126,000
GR-26	4.5	9.8	3,500 ft of 30-in. RC at 0.15%, average cut 10 ft, difficult wet, includes imported backfill, repaving and sheeting	145,000
GR-27	5.9-6.6	13-15	12,300 ft of 36-in. RC at 0.10 - 0.13%, average cut 8 - 10 ft, difficult wet, includes imported backfill and repaving	376,000
GR-28	0.7-1.5	1.8-4.0	13,900 ft of 12-in. RC at 2.0 - 4.5%, average cut 6 ft, dry to difficult wet, includes imported backfill, repaving and access road	156,000
GR-29	2.4	6.0	3,500 ft of 21-in. RC at 0.49%, average cut 10 ft, difficult wet	77,000
GR-30	0.6	1.6	1,500 ft of 15-in. RC at 0.16%, average cut 11 ft, difficult wet, includes imported backfill and repaving	38,000
GR-31	3.4	8.7	2,500 ft of 30-in. RC at 0.12%, average cut 15 ft, difficult wet, includes imported backfill, repaving and sheeting	126,000

Continued on next page

Table 15-35. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
GR-32	0.6	1.8	5,000 ft of 12-in. RC at 3.1%, average cut 6 ft, dry to moderately wet, includes access road	52,000
GR-33	1.1	2.9	1,500 ft of 15-in. RC at 0.55%, average cut 7 ft, difficult wet, includes imported backfill and repaving	30,000
GR-34	0.8	2.4	2,500 ft of 12-in. RC at 2.4%, average cut 6 ft, wet	33,000
GR-35	1.2	3.4	1,000 ft of 15-in. RC at 0.75%, average cut 7 ft, difficult wet	17,000
GR-36	2.1	5.3	1,200 ft of 18-in. RC at 0.65%, average cut 10 ft, difficult wet	25,000
GR-37	0.4	1.0	2,300 ft of 12-in. RC at 0.20%, average cut 10 ft, difficult wet, includes imported backfill and repaving	50,000
GR-38	0.8	2.0	2,900 ft of 15-in. RC at 0.24%, average cut 13 ft, difficult wet, includes imported backfill, repaving and sheeting	97,000
GR-39	1.5	4.1	3,700 ft of 21-in. RC at 0.17%, average cut 17 ft, difficult wet, includes imported backfill, repaving and sheeting	170,000
GR-40	1.9	5.0	2,900 ft of 24-in. RC at 0.12%, average cut 19 ft, difficult wet, includes imported backfill, repaving and sheeting	153,000
GR-41	2.1	5.7	4,000 ft of 27-in. RC at 0.085%, average cut 19 ft, difficult wet, includes imported backfill, repaving and sheeting	231,000
GR-42	2.4-2.8	6.8-7.4	6,100 ft of 30-in. RC at 0.067 - 0.085%, average cut 22 ft, difficult wet, includes imported backfill, repaving and sheeting	414,000
GR-43	1.5	3.2	1,700 ft of 18-in. RC at 0.22%, average cut 8 ft, difficult wet	32,000
GR-44	1.9	4.3	2,700 ft of 21-in. RC at 0.18%, average cut 11 ft, difficult wet	62,000
GR-45	0.6	1.6	2,300 ft of 12-in. RC at 4.3%, average cut 6 ft, wet	31,000
GR-46	0.8	2.2	3,100 ft of 15-in. RC at 0.42%, average cut 6 ft, difficult wet	50,000
GR-47	3.2	7.9	2,500 ft of 24-in. RC at 0.30%, average cut 20 ft, difficult wet, includes sheeting	110,000
GR-48	0.6	1.6	3,300 ft of 12-in. RC at 1.5%, average cut 6 ft, dry to moderately wet	31,000
GR-49	0.8	2.1	3,500 ft of 15-in. RC at 0.51%, average cut 12 ft, dry to moderately wet, includes imported backfill and repaving	62,000
GR-50	1.2	3.0	4,400 ft of 18-in. RC at 0.20%, average cut 12 ft, difficult wet, includes sheeting	123,000
GR-51	1.3	3.4	4,000 ft of 21-in. RC at 0.12%, average cut 8 ft, difficult wet	82,000
GR-52	1.8	4.9	5,000 ft of 24-in. RC at 0.12%, average cut 13 ft, difficult wet, includes sheeting	163,000
GR-53	3.1	7.5	5,000 ft of 30-in. RC at 0.08%, average cut 17 ft, difficult wet, includes sheeting	220,000
GR-54	4.5	10	2,700 ft of 36-in. RC at 0.06%, average cut 17 ft, difficult wet, includes sheeting	130,000
GR-55	0.5-0.8	1.4-2.2	5,600 ft of 12-in. RC at 1.7-2.3%, average cut 6 ft, dry to difficult wet, includes imported backfill and repaving	76,000
GR-56	6.5	15	1,200 ft of 42-in. RC at 0.055%, average cut 18 ft, difficult wet, includes sheeting	67,000
GR-57	6.5	15	400 ft of twin 18-in. inverted siphons across Duwamish Waterway, includes inlet and outlet structures	21,000
GR-58	6.5	15	800 ft of 42-in. RC at 0.055%, average cut 24 ft, difficult wet, includes sheeting	57,000
Subtotal, sewers				7,612,000

Continued on next page

Table 15-35. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
PS-GR-1	3.2	7.9	Pumping station, single stage, motor driven, static lift 24 ft, total head at peak flow 30 ft, structure about 25 ft below ground, difficult wet, includes sheeting and dewatering	111,000
Total contract cost, Green River Sewerage Area				7,723,000
Engineering and contingencies, 25 per cent				1,931,000
Total construction cost, Green River Sewerage Area				9,654,000

See Fig. 15-17 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

9. A trunk (GR 37–GR 42) to serve Green River valley west of the Great Northern railroad and east of Green River.

10. A trunk (GR 43–GR 47) to serve areas south of the city of Renton. A pumping station (PS-GR 1) would be required to lift the sewage from the trunk to the feeder sewer system.

11. A trunk (GR 48–GR 58) to serve areas west of Green River.

Southwest Lake Washington Sewerage Area

Service sewers for the Southwest Lake Washington sewerage area consist of three existing systems, all with combined sewers which serve the portions of this area lying within the city of Seattle (Fig. 15-18). Service to the southern portion, including the Bryn Mawr-Lake Ridge area, would be obtained by the feeder sewers. Descriptions and estimated costs of the required facilities are given in Table 15-36.

The first service sewer system serves lower Rainier valley from Seward Park on the north to the city limit of Seattle on the south. This system would incorporate existing facilities of the city of Seattle and would include the following major elements:

1. A waterfront interceptor (SWW 1-SWW 2) serving areas draining directly to Lake Washington. Storm water holding tanks sized to allow one overflow per summer would be provided on tributary local sewers.

2. A pumping station (PS-SWW 1), which would discharge through a force main (SWW 3) and gravity sewer (SWW 4) to the feeder sewer system.

3. A trunk (SWW 5-SWW 7) serving Rainier valley and discharging to the feeder sewer system. Since this trunk does not have sufficient capacity for the storm flow resulting from a rainfall with a recurrence interval of once in 10 years, partial separation of the tributary area (local service areas SWW 6 and SWW 5) would be required.

4. An overflow line (SWW 8-SWW 9) which receives storm water overflow both from sewer SWW 6 and from local sewers tributary to sewer SWW 4. A storm water holding tank sized to allow one overflow per summer would be provided at the discharge end of this line.

The second system serves upper Rainier valley from about Yesler Way on the north to Seward Park on the south. This system would incorporate existing facilities of the city of Seattle and would include the following major elements:

1. A trunk (SWW 10) serving areas draining directly to Lake Washington. Storm water holding tanks sized to allow one overflow per summer would be provided on tributary local sewers.

2. A pumping station (PS-SWW 2) which discharges through a gravity sewer (SWW 11) to the feeder sewer system. Partial separation of the tributary area (local service area SWW 10) would be required.

3. A trunk serving northern Rainier valley (SWW 12-SWW 13) and discharging to the feeder sewer system.

The third system serves the narrow waterfront strip bordering Lake Washington from Edgewater Park on the north to Coleman Park on the south. This system would incorporate existing facilities of the city of Seattle and would include the following major elements:

1. A pumping station (PS-SWW 3) which discharges to the feeder sewer system through a force main (SWW 14) and a high elevation gravity sewer (SWW 15, SWW 17-SWW 20). Storm water holding tanks sized to allow one overflow per summer would be provided on local sewers tributary to the pumping station. Partial separation of the tributary area (local service area SWW 15) would be required.

2. A pumping station (PS-SWW 4) which discharges through a force main (SWW 16) to the high elevation gravity sewer. Storm water holding tanks sized to

allow one overflow per summer would be provided on local sewers tributary to the pumping station.

Elliott Bay Sewerage Area

Service sewers for the Elliott Bay sewerage area are laid out to convey the sewage of that area into the central sewerage system. Descriptions and estimated costs of the service sewers are given in Table 15-36. Major elements are shown in Fig. 15-18 and are as follows:

1. A trunk (EB 1-EB 2) to serve industrial and residential areas east of West Duwamish.
2. A trunk system (EB 3-EB 18) to serve industrial and residential areas west of Duwamish River. To avoid excessive cuts, two pumping stations (PS-EB 1 and PS-EB 2) would be required. This system will include the Delridge trunk sewer (EB 4-EB 6), for which final designs have been prepared by the engineering department of the city of Seattle. Lengths, sizes and slopes of sewers included in this section of the trunk (Table 15-36) are exactly as designed by the city and are considered to be adequate.
3. A waterfront interceptor (EB 19) to serve the Magnolia Bluff area.

Lake Union Sewerage Area

Service sewers for the Lake Union sewerage area consist essentially of existing combined sewers of the city of Seattle which are tributary to the North Trunk (Fig. 15-18). The latter is to be incorporated in the central sewerage project and would convey sewage of the Lake Union area to the West Point treatment plant. Descriptions and estimated costs of the required facilities are given in Table 15-36. Major components are as follows:

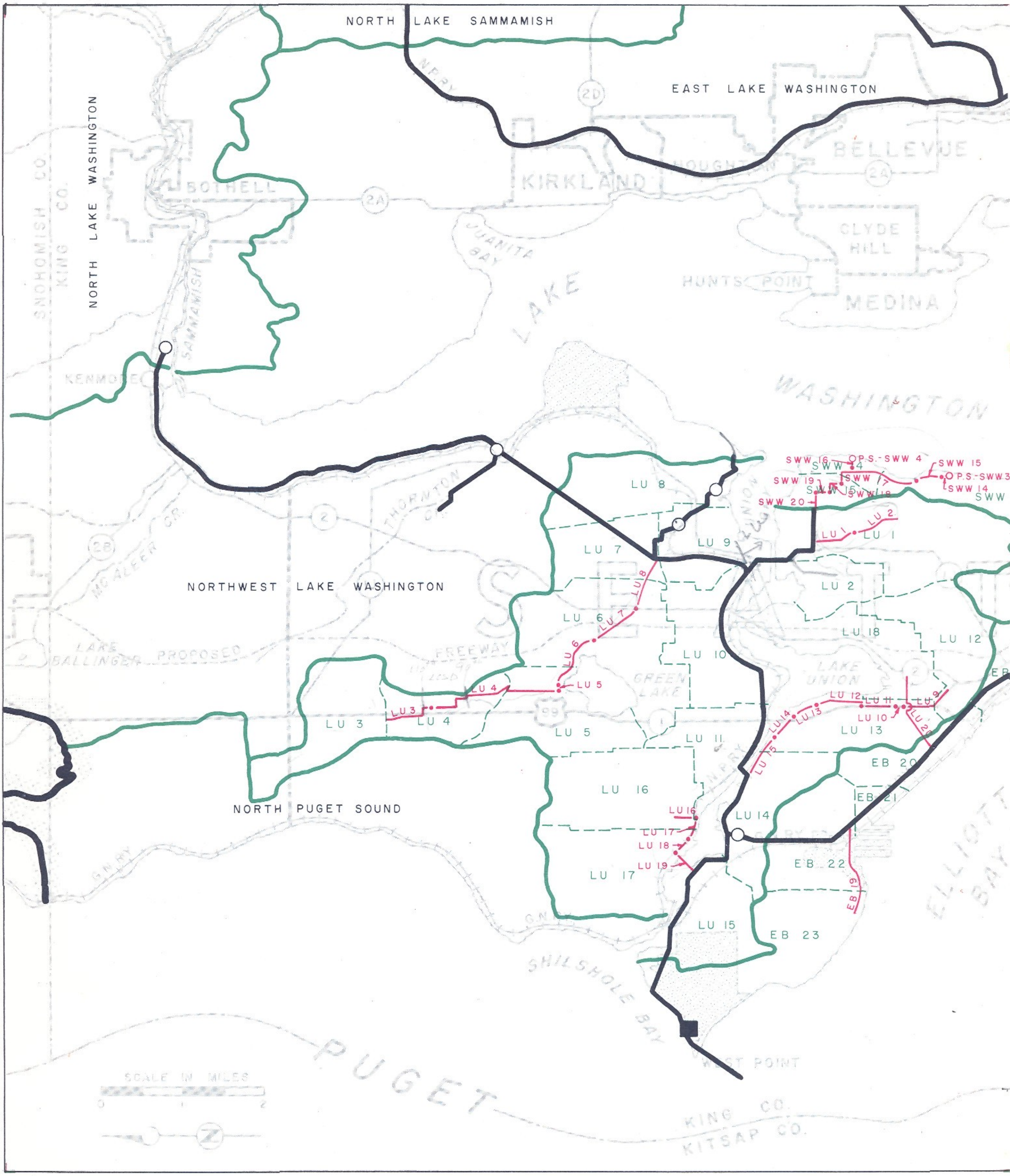
1. A trunk (LU 1-LU 2) to serve the Broadmoor and Madrona areas. Partial separation of the tributary area (local service area LU 1) would be required.
2. A trunk (LU 3-LU 4) to serve the Haller Lake area.
3. A trunk (LU 5-LU 6) to serve the area north of Green Lake.
4. A trunk (LU 7-LU 8) to serve all the area draining to Green Lake. Partial separation of a portion of the tributary area (local service area LU 6) would be required. Trunk LU 7-LU 8 includes the existing sewer of the city of Seattle (LU 8) which was damaged recently by a cave-in of overlying material. If inspection indicates that the damage is severe enough to require that this section be abandoned, the replacement sewer then would serve as a portion of the service sewer.
5. A high elevation trunk (LU 9-LU 11) to serve the higher areas of Capitol Hill. Partial separation of the tributary area (local service area LU 12) would be required.
6. A high elevation waterfront interceptor (LU 12-LU 15) to serve the higher areas draining to Lake Union and the Lake Washington Ship Canal.
7. A trunk and interceptor system (LU 16-LU 18) to serve the Ballard area. This system discharges through an inverted siphon (LU 19) to the North Trunk sewer. The existing inverted siphon consists of parallel 36-inch wood stave pipes, which undoubtedly will require replacement at some future date. Estimated costs presented in Table 15-36 allow for such a possibility. Partial separation of the tributary area (local service area LU 16) would be required.
8. A trunk (LU 20) which serves the lower areas draining to Lake Union and discharges to Core Plan B facilities in the Elliott Bay sewerage area. Partial separation of the tributary area (local service area LU 18) would be required.

OUTLINE OF PROPOSED SEWERAGE PROJECTS

It is evident from the foregoing analyses of costs and other controlling factors that collection, treatment and disposal of the sewage of the metropolitan Seattle area can be achieved most effectively by means of the central sewerage project designated herein as Core Plan B. Under this plan (Fig. 15-19), sewage from ten of the twelve individual sewerage areas would be conveyed to two plants for treatment and disposal.

Of the two treatment plants, one would be situated at Black River junction west of Renton and would provide complete treatment for sewage from the North Lake Sammamish, South Lake Sammamish, East Lake Washington, South Lake Washington and Green River sewerage areas. Chlorinated effluent from this plant would be discharged to Duwamish River. The second plant would be situated at West Point at the western extremity of Fort Lawton and would provide primary treatment for sewage from the North Lake Washington, Northwest Lake Washington, Southwest Lake Washington, Elliott Bay and Lake Union sewerage areas. Chlorinated effluent produced at this plant would be discharged to Puget Sound approximately 3,700 feet offshore at a water depth of 150 feet. Feeder and service sewers necessary to convey sewage from the ten sewerage areas to the Core Plan B facilities are described and their estimated costs are given in foregoing sections of this chapter.

In the South Puget Sound sewerage area, participation in the central sewerage project would not be economically feasible. It is proposed instead that sewage generated in this area be conveyed to and treated at five independent plants (Fig. 15-19). Each of these plants would serve a major sewerage subarea and would provide either primary treatment or secondary treatment, depending on receiving water requirements


NORTH LAKE SAMMAMISH
EAST LAKE WASHINGTON
NORTH LAKE WASHINGTON
SNOHOMISH CO.
KING CO.
BOTHELL
KIRKLAND
BELLEVUE
CLYDE HILL
HUNTS POINT
MEDINA
KENMORE
JUANITA BAY
LAKE WASHINGTON
NORTHWEST LAKE WASHINGTON
NORTH PUGET SOUND
THORNTON CR.
McALEER CR.
LAKE BALLINGER
PROPOSED FREEWAY
GREEN LAKE
LAKE UNION
SHILSHOLE BAY
WEST POINT
ELLIOTT BAY
PUGET
KING CO.
KITSAP CO.
SCALE IN MILES
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LU 1
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LU 3
LU 4
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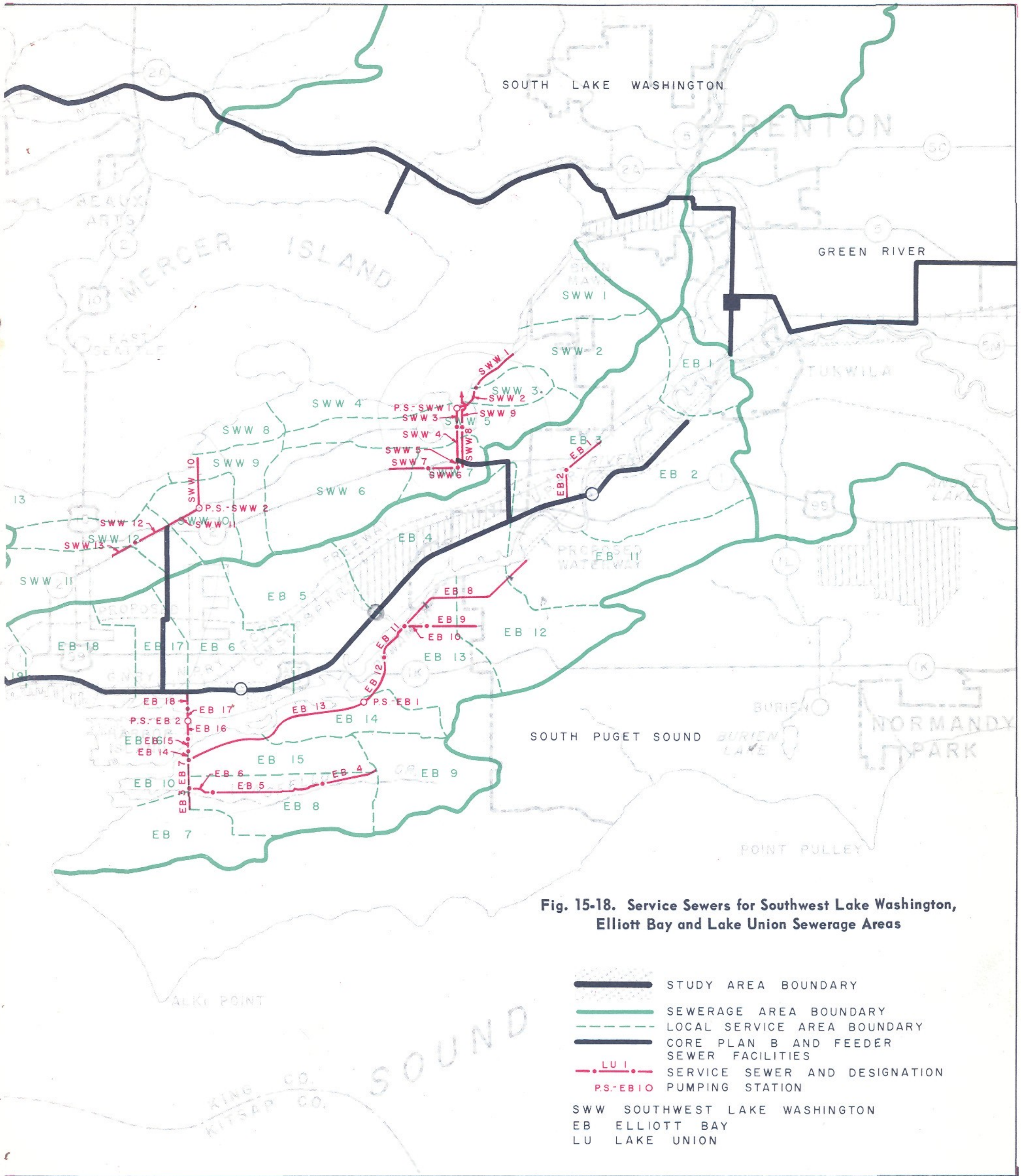

Fig. 15-18. Service Sewers for Southwest Lake Washington, Elliott Bay and Lake Union Sewerage Areas

Table 15-36. Description and Estimated Construction Costs, Service Sewers, Southwest Lake Washington, Elliott Bay and Lake Union Sewerage Areas

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
Southwest Lake Washington Sewerage Area				
SWW-1	1.1	3.6	3,300 ft of 18-in. RC at 0.19%, wet, includes imported backfill and repaving. To parallel existing 18-in. sewer of insufficient capacity for peak wet weather sanitary flow, includes 4,800,000 gal. holding tank and overflow structure with overflow pipe and 1,000,000 gal. holding tank on tributary local sewers	609,000
SWW-2	1.5	4.5	1,100 ft of 18-in. RC at 0.19%, wet, includes imported backfill and repaving. To parallel existing 18-in. sewer of insufficient capacity for peak wet weather sanitary flow, includes 1,300,000 gal. holding tank on overflow from tributary local sewer	146,000
SWW-3	2.6	7.8	1,300 ft of existing twin 15-in. force mains	Existing
SWW-4	4.0	12	2,300 ft of existing 48-in. at 0.12%, 42-in. at 0.20%, and parallel 24-in. at 0.87% and 30-in. at 0.35%. Requires partial separation of a portion of local service area SWW-5	Existing
SWW-5	2.1	5.8, 85[c]	2,000 ft of existing 60-in. at 0.5%. Requires partial separation of local service area SWW-6	Existing
SWW-6	2.5	7.1, 107[c]	2,900 ft of existing 60-in. at 0.9%. Requires partial separation of a portion of local service area SWW-5	Existing
SWW-7	2.5	7.1	500 ft of existing 18-in.	Existing
SWW-8	-	307[d]	2,600 ft of existing 60-in. to 72-in. overflow pipe. Receives storm water overflow from sewer SWW-6	Existing
SWW-9	-	548[d]	1,300 ft of existing 84-in. overflow pipe. In addition to flow from sewer SWW-8, receives storm water overflow from local sewers tributary to sewer SWW-4. Cost is for 4,400,000 gal. holding tank at discharge end	370,000
SWW-10	1.2	3.6	3,500 ft of existing 42-in. at 0.1%. Cost is for 520,000, 350,000, 120,000, 520,000 and 3,400,000 gal. holding tanks on tributary local sewers	494,000
SWW-11	2.1	6.1, 50[c]	2,500 ft of existing 66-in. at 0.3%. Requires partial separation of local service area SWW-10	Existing
SWW-12	1.5	4.2, 173[e]	1,400 ft of existing 75-in. at 0.55%	Existing
SWW-13	2.3	6.5, 256[e]	2,700 ft of existing 102-in. at 0.24 - 0.3%	Existing
SWW-14	1.2	3.4	400 ft of existing 12-in. force main.	Existing
SWW-15	1.2	3.4	2,600 ft of existing 24-in. by 36-in. egg shaped sewer at 2.1% and 24-in. by 48-in. egg shaped sewer at 0.54%	Existing
SWW-16	0.4	1.2	1,000 ft of existing parallel 12-in. and 16-in. force mains	Existing
SWW-17	1.6	4.6	5,400 ft of existing 48-in. at 0.25%	Existing
SWW-18	1.6	4.6	1,900 ft of existing 54-in. at 0.15%	Existing
SWW-19	2.0	5.8, 30[d]	800 ft of existing 54-in. at 0.20%. Requires partial separation of local service area SWW-15	Existing
SWW-20	2.0	5.8, 30[d]	1,800 ft of existing 60-in. at 0.19%	Existing
			Holding tanks on local sewers tributary to PS-SWW-1. Tank capacities: 900,000, 1,400,000, 800,000 and 1,000,000 gals.	321,000
			Holding tanks on local sewers tributary to PS-SWW-3. Tank capacities: 230,000, 640,000, 230,000, 460,000, 520,000, 350,000, 350,000, 460,000, 870,000 and 290,000 gals.	506,000
			Holding tanks on local sewers tributary to PS-SWW-4. Tank capacities: 290,000, 640,000 and 520,000 gals.	178,000
Subtotal, sewers and holding tanks				2,624,000

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Table 15-36. Continued

Facility	Design flow,[a] mgd Average DWF	Maximum WWF	Description	Construction cost,[b] dollars
PS-SWW-1	2.6	7.8	Pumping station, single stage, motor driven with engine standby, static lift 10 ft, total head at peak flow 25 ft. To replace existing sewage pumping station which has a total installed capacity of 5.2 mgd, includes connections to existing sewers	110,000
PS-SWW-2	1.2	3.6	Existing pumping station having total installed capacity of 10.7 mgd (3 pumps of 1.0, 4.7 and 5.0 mgd capacity)	Existing
PS-SWW-3	1.2	3.4	Pumping station, single stage, motor driven with engine standby, static lift 80 ft, total head at peak flow 90 ft. To replace existing sewage pumping station which has a total installed capacity of 2.65 mgd, includes connection to existing sewers	68,000
PS-SWW-4	0.4	1.2	Existing pumping station having total installed capacity of 3.9 mgd (2 pumps of 1.2 and 2.7 mgd capacity)	Existing
Subtotal, pumping stations				178,000
Total contract cost, Southwest Lake Washington Sewerage Area				2,802,000
Engineering and contingencies, 25 per cent				700,000
Total construction cost, Southwest Lake Washington Sewerage Area				3,502,000
Elliott Bay Sewerage Area				
EB-1	2.1	4.6	1,700 ft of 21-in. RC at 0.21%, average cut 8 ft, difficult wet	35,000
EB-2	4.9	10	2,000 ft of 33-in. RC at 0.09%, average cut 10 ft, difficult wet	60,000
EB-3	1.8	5.1	1,200 ft of 18-in. RC at 1.0%, average cut 15 ft, difficult wet, includes imported backfill, repaving, sheeting and railroad crossing	62,000
EB-4[f]	1.0	3.2	700 ft of 15-in. RC at 1.3 - 8.0%, average cut 9 - 14 ft, and 2,700 ft of 18-in. RC at 1.1 - 1.25%, average cut 9 - 11 ft, wet, includes repaving	60,000
EB-5[f]	1.9	6.3	1,200 ft of 18-in. RC at 3.0 - 4.5%, average cut 12 - 14 ft, 3,600 ft of 21-in. RC at 0.6 - 4.0%, average cut 9-16 ft, 2,100 ft of 24-in. RC at 0.4 - 0.8%, average cut 13 - 17 ft, and 1,300 ft of 30-in. RC at 0.15%, average cut 18 ft, wet to difficult wet, includes repaving and sheeting	234,000
EB-6[f]	2.3	7.4	200 ft of 24-in. RC at 1.0%, average cut 10 ft, and 900 ft of 30-in. RC at 0.54 - 0.85%, average cut 8 - 9 ft, difficult wet	30,000
EB-7	4.1	12	2,100 ft of 30-in. RC at 0.2%, average cut 19 - 20 ft, difficult wet, includes imported backfill, repaving and sheeting	130,000
EB-8	2.0	5.8	8,800 ft of 24-in. RC at 0.17 - 0.20%, average cut 10 - 11 ft, difficult wet, includes imported backfill and repaving	252,000
EB-9	2.3	5.9	3,200 ft of 21-in. RC at 0.5%, average cut 14 ft, wet, includes imported backfill and repaving	93,000
EB-10	3.6	8.4	1,500 ft of existing 42-in. at 0.11%. Requires complete separation of a portion of local service area EB-13	Existing
EB-11	5.5	14	2,700 ft of 39-in. RC at 0.07%, average cut 13 ft, difficult wet, includes imported backfill, repaving and sheeting	157,000
EB-12	6.4-6.7	16-17	3,200 ft of 42-in. RC at 0.07%, average cut 15 - 19 ft, difficult wet, includes imported backfill, repaving, sheeting and slough crossing	202,000
EB-13	6.9-7.4	17-19	12,600 ft of 39-in. RC at 0.13 - 0.24%, average cut 8 - 16 ft, difficult wet, includes imported backfill, repaving, sheeting and railroad crossing	532,000
EB-14	11	25	500 ft of 48-in. RC at 0.075%, average cut 18 ft, difficult wet, includes imported backfill, repaving and sheeting	39,000

Continued on next page

Table 15-36. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
EB-15	11	25	800 ft of parallel 16-in. and 30-in. inverted siphons across West Channel of Duwamish Waterway, includes inlet and outlet structures...	54,000
EB-16	12	27	1,500 ft of 48-in. RC at 0.085%, average cut 15 ft, difficult wet, includes imported backfill, repaving and sheeting	103,000
EB-17	12	27	700 ft of parallel 16-in. and 24-in. force mains across East Channel of Duwamish Waterway	42,000
EB-18	12	27	900 ft of 36-in. RC at 0.39%, average cut 10 ft, difficult wet, includes imported backfill, repaving, sheeting and railroad crossing	42,000
EB-19	2.0-2.3	6.1-7.0	5,700 ft of 21-in. RC at 0.38 - 0.5%, average cut 7 - 11 ft, difficult wet, includes imported backfill, repaving, railroad crossing and overflow structures on tributary local sewers	177,000
Subtotal, sewers				2,304,000
PS-EB-1	6.7	17	Pumping station, single stage, motor driven, static lift 39 ft, total head at peak flow 45 ft, structure about 30 ft below ground, difficult wet, includes sheeting and dewatering	183,000
PS-EB-2	12	27	Pumping station, single stage, motor driven, static lift 10 ft, total head at peak flow 22 ft, structure about 20 ft below ground, difficult wet, includes sheeting, dewatering and special foundations	246,000
Subtotal, pumping stations				429,000
Total contract cost, Elliott Bay Sewerage Area				2,733,000
Engineering and contingencies, 25 per cent				683,000
Total construction cost, Elliott Bay Sewerage Area				3,416,000
Lake Union Sewerage Area				
LU-1	2.4	6.1, 86[c]	4,300 ft of existing 60-in. at 0.5%. Requires partial separation of local service area LU-1.	Existing
LU-2	2.7	7.0, 86[c]	1,500 ft of existing 72-in. at 0.2%	Existing
LU-3	1.2	3.1	3,200 ft of 18-in. RC at 0.74%, average cut 8 ft, wet, includes imported backfill and repaving	60,000
LU-4	1.3-2.6	3.4-7.2	4,500 ft of existing 24-in. at 0.6 - 3.2% and 5,000 ft of existing 30-in. at 0.6 - 3.5%	Existing
LU-5	3.3	9.2, 103[e]	300 ft of existing 72-in. at 0.11%	Existing
LU-6	3.8	11, 167[e]	4,000 ft of existing 90-in. at 0.11-0.15%	Existing
LU-7	5.7	16, 230[c]	3,400 ft of existing 90-in. at 0.45%. Requires partial separation of local service area LU-6	Existing
LU-8	6.1	17, 230[c]	3,700 ft of existing 72-in. at 2.6%	Existing
LU-9	1.6	4.3, 70[c]	3,900 ft of existing 60-in. at 0.61%. Requires partial separation of local service area LU-12	Existing
LU-10	2.1	5.5, 99[c]	200 ft of existing parallel 24-in. and 66-in. inverted siphons	Existing
LU-11	2.1	5.5, 99[c]	2,600 ft of existing 84-in. at 0.21%	Existing
LU-12	2.4	6.5	2,900 ft of existing 48-in. at 0.2 - 0.26%	Existing
LU-13	2.6	6.9	1,000 ft of existing 54-in. at 0.18%	Existing
LU-14	2.8	7.6	2,800 ft of existing 60-in. at 0.13 - 0.14%	Existing
LU-15	3.1	8.4	2,700 ft of existing 72-in. at 0.19%	Existing
LU-16	2.5	6.4, 84[c]	1,400 ft of existing 69-in. by 116-in. rectangular shape at 0.075 - 0.09 0.09%. Requires partial separation of local service area LU-16	Existing

Continued on next page

Table 15-36. Continued

Facility	Design flow,[a] mgd		Description	Construction cost,[b] dollars
	Average DWF	Maximum WWF		
LU-17	2.5	6.4	2,100 ft of existing 54-in. at 0.04%	Existing
LU-18	2.5	6.4	1,000 ft of existing 66-in. at 0.035%	Existing
LU-19	4.7	12	1,300 ft of parallel 36-in. inverted siphons to replace existing wood stave inverted siphon, includes inlet and outlet structures	135,000
LU-20	3.0	7.8, 51[c]	5,600 ft of existing 72-in. at 0.08%. Requires partial separation of local service area LU-18	Existing
Total contract cost, Lake Union Sewerage Area				195,000
Engineering and contingencies, 25 per cent				49,000
Total construction cost, Lake Union Sewerage Area				244,000

See Fig. 15-18 for location of facilities.

[a]*Expressed as average dry weather flow and maximum wet weather flow.*

[b]*Does not include cost of acquiring existing facilities.*

[c]*Flow for 10-year storm from partially separated tributary area.*

[d]*Storm flow in excess of trunk capacity.*

[e]*Flow for 10-year storm from tributary area.*

[f]*Lengths, sizes and slopes of sewers are based on final design of trunk by the Seattle engineering department.*

at each site. Chlorinated effluent from the five plants would be discharged through submarine outfalls to Puget Sound.

Similarly, it would not be economically feasible for the North Puget Sound to participate in the central sewerage project. It is proposed, therefore, that sewage from this area be conveyed to and treated at two independent plants. These plants would serve the Piper Creek and Boeing Creek subareas and would provide primary treatment plus effluent disinfection. Effluent from each plant would be discharged through submarine outfalls to Puget Sound.

ADMINISTRATION OF PROPOSED SEWERAGE FACILITIES

It seems appropriate at this point to touch briefly on what are believed to be minimum administrative requirements for construction, maintenance and operation of the proposed sewerage facilities. Provision of an adequate administrative organization and enactment of proper administrative regulations will assure orderly and efficient development of the various projects.

Central Control

Sewerage problems of the metropolitan Seattle area are area-wide in scope. As such, they are the responsibility not only of the city of Seattle but of the area as a whole. Corrective measures must be formulated and carried out accordingly. This survey is the first step in that direction and is but one of the many which still remain to be taken.

Because of the physical and economic magnitude of the required sewerage projects, it is highly unlikely that any single existing political body would be in a position to assume responsibility for both their financing and their construction and operation. Further, it is equally unlikely that existing political bodies acting individually would be able to finance and construct the required facilities. Some means will have to be developed, therefore, whereby the necessary burdens can be equitably assumed, necessary funds can be raised, and necessary work can be undertaken on a systematic and truly economic basis.

To achieve the three objectives just noted and to assure area-wide participation in the solution of what is most assuredly an area-wide problem, it appears advisable to undertake the formation of a central agency encompassing the entire metropolitan area. Such an agency would be responsible for all administrative and engineering duties related to the financing, design, construction, maintenance and operation of all sewerage facilities herein recommended. It seems advisable also that this agency should take over and become responsible for the maintenance and operation of all existing sewage treatment plants in the metropolitan area. With full responsibility for major sewerage facilities vested in one agency, effective protection of all waters of the area will be assured.

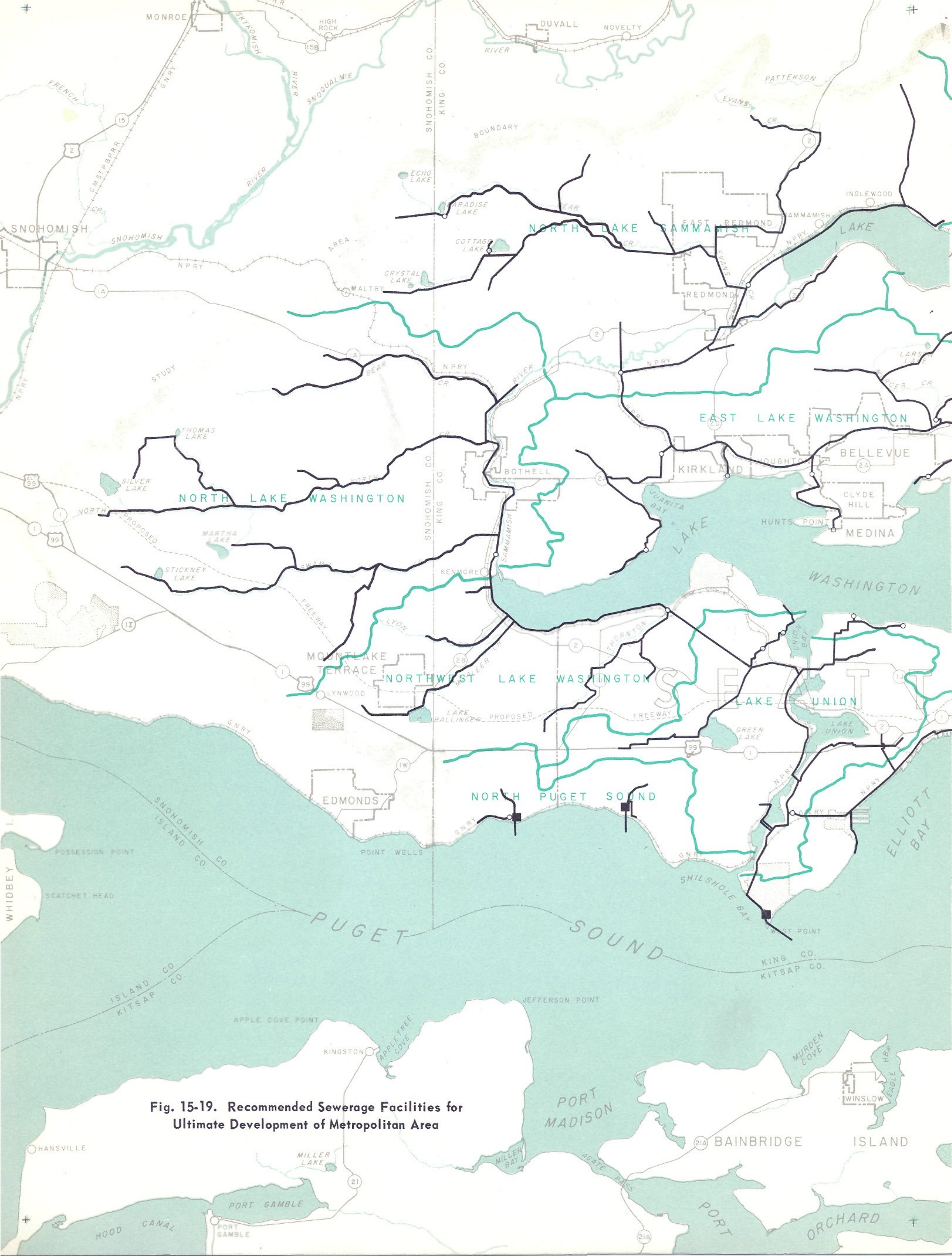

Fig. 15-19. Recommended Sewerage Facilities for Ultimate Development of Metropolitan Area


STUDY AREA BOUNDARY
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
ELLIOTT BAY
HIGH POINT
RAVENSDALE
LAKE NO 12
CINDER LAKE
BLACK DIAMOND
JONES LAKE
BEAVER LAKE
YELLOW LAKE
PINE LAKE
ISSAQUAH
MONOHAN
SAMMAMISH
PHANTOM LAKE
WEBSTER LAKE
FRANCIS LAKE
MAPLE VALLEY
WILDERNESS LAKE
LAKE SAWYER
PIPE LAKE
LAKE MCDONALD
LAKE KATHLEEN
OTTER LAKE
LAKE DESIRE
LAKE MERIDIAN
SEATTLE CITY WATER SUPPLY
LAKE YOUNGS
NEILSON LAKE
MONEYSMITH LAKE
RENTON
KENT
AUBURN
ALGONA
TUKWILA
BEAUX ARTS
MERCER ISLAND
EAST SEATTLE
BRYN MAWR
LAKE GENEVA
NORTH LAKE
FIVE MILE LAKE
LAKE KILLARNEY
DOLLOFF LAKE
STEEL LAKE
ANGLE LAKE
PROPOSED WATERWAY
DES MOINES
ZENITH
BURIEN
BURIEN LAKE
NORMANDY PARK
POVERTY BAY
MIRROR LAKE
HARBOR ISLAND
LONGFELLOW CR
ALKI POINT
POINT PULLEY
POINT ROBINSON
KING CO. PIERCE CO.
TACOMA
PUGET SOUND
MAURY ISLAND
PORTAGE
KING CO. KITSAP CO.
VASHON
VASHON HEIGHTS
COWLEY
DOCKTON
RESTORATION POINT
VASHON ISLAND
QUARTERMASTER HARBOR
BLAKE ISLAND
COMMENCEMENT BAY
COLVOS PASSAGE
DALCO PASSAGE
RUSTON
HARPER
RICH PASSAGE
CLAM BAY
YUKON HARBOR
MANCHESTER
COLBY
KITSAP CO. PIERCE CO.
THE NARROWS
GIG HARBOR
SCALE IN MILES
0 1 2 3 4
SEWERAGE AREA BOUNDARY
TRUNK SEWERS
SEWAGE TREATMENT PLANTS
PUMPING STATIONS

Predesign Investigations

In undertaking the design and construction of major sewerage facilities, it is necessary at the outset to know what soil conditions and what other conditions, particularly subsurface obstructions, are likely to be encountered during construction. This requirement is particularly applicable to the metropolitan Seattle area where geological formations are variable and major sewers will have to be routed through highly developed districts. One of the functions, therefore, of the central sewerage agency should be that of assembling essential information concerning soil conditions along the routes of all proposed intercepting sewers and tunnels, and at the sites of all treatment plants and pumping stations. Experience indicates that such information is useful not only in developing a sound and economical design but in obtaining favorable bids on construction.

Engineering Design and Control of Construction

In addition to administrative control, the central agency should have an engineering staff capable of performing both design work and inspection during construction. Design work will fall in two general categories, one more or less continuous and the other intermittent or occasional. Continuous work would include design of all trunk sewers and logically could be performed by the engineering staff of the central agency. Intermittent work would include design of the waterfront intercepting sewers, treatment plants, and major pumping stations, all of which require a design staff having specialized experience in this particular field. For that reason, and because the work load would be periodic rather than continuous, the central agency would possibly find it difficult to undertake all of the necessary engineering.

Rigid inspection will be required during construction of the proposed facilities. This will assure effective compliance with all specifications relating to the work and will guarantee construction in full accordance with the design requirements. Inspection generally should be performed by the engineering staff of the central agency. In the case, however, of facilities designed under contract by consulting engineering firms, it may be advisable to have such firms assume responsibility for resident engineering and inspection during construction.

Enforcement of Design Criteria

Successful operation of the facilities designed and constructed under the authority and control of the central agency will depend upon the cooperation of the many smaller agencies which are actually responsible for the local collection systems. Cooperation will be required in particular in two phases. Of these, the first is concerned with infiltration and storm inflow, and the second with industrial wastes.

Infiltration and Storm Inflow. As stated earlier in Chapter 13, present day construction materials and methods, coupled with adequate inspection, are such that infiltration and storm inflow quantities can be kept well within the allowances provided herein. Some method will have to be set up whereby the central agency can be assured that these allowances will not be exceeded in local systems where construction and operation are not within its direct jurisdiction.

A question may arise as to the necessity or economy of minimizing inflow from these sources. Benefits thus achieved may be illustrated by the following examples. Two areas, each of 1,000 acres, are to be served by separate sanitary sewerage systems. Design criteria for the first area include an allowance for infiltration and storm inflow of 1,000 gpad. For the second area, the allowance is four times that amount, or 4,000 gpad. Assuming a peak sanitary sewage flow from each area of 1.0 mgd, the total design flow for which sewer capacity must be provided is 2.0 mgd in the first case and 5.0 mgd in the second. Assuming also that the slope of the trunk sewers serving the areas is 0.7 per cent in both cases, the sewer required to serve the first area would be 12 inches in diameter, while that for the second area would be 18 inches in diameter. The difference in cost between these relatively small diameter sewers amounts to approximately three dollars per foot of sewer length. With larger tributary areas and similar infiltration ratios, the cost difference per foot of trunk sewer would be correspondingly greater.

As a further example, if roof leaders and foundation drains were permitted to be connected, maximum flows would be two to fifteen times those which would otherwise occur. In such an event, either sufficient sewer capacity would have to be provided or more frequent overflows would have to be allowed.

Obviously, it would not be feasible economically in a central sewerage project to provide collection, treatment and disposal facilities of a capacity sufficient to accommodate the excess flows illustrated by the two examples. Although the central agency cannot directly control construction of local sewers, it can exercise control by indirect means. First, it can require the adoption of standard specifications regarding the construction of local sewers, including house connections. These specifications should stipulate the class of construction required to maintain infiltration within the limits called for in this report. Second, the central agency should require complete infiltration tests prior to the connection of any new local sewerage facilities. Should it be found that the infiltration limits are exceeded, a connection to the central system should be refused until the local system has been brought up to standard. And third, the central agency should maintain constant flow measurements to determine whether inputs from the local systems are within allowable limits. In cases where excess flows are consistently recorded, the local agency should be subject to an additional assessment and the charge thus imposed should be based on the capitalized cost of providing for the measured excess.

Industrial Wastes. A regulatory ordinance defining the characteristics of wastes acceptable to the sewerage systems should be adopted by the central agency. This should set forth allowable physical and chemical limits of all wastes which would produce undue loadings or would lead to deleterious effects either on collection and treatment facilities or on treatment and disposal functions.

Chapter 16
STAGE CONSTRUCTION OF SEWERAGE FACILITIES

As a final step in the development of a long-range program of sewerage improvements, it is necessary to establish a logical and orderly schedule for their construction on a stage or incremental basis. This in turn requires a determination of the most urgent immediate needs, as well as a determination of the times in the future when the recommended additions are likely to be required.

As stated earlier, the most urgent immediate needs include the removal of sewage and sewage effluents from the Lake Washington watershed, the interception of raw sewage and industrial waste outfalls discharging to Duwamish River, Elliott Bay and Puget Sound, and the provision of service to highly developed areas presently without public sewerage facilities. Scheduling of the many urgent projects must recognize the fact that the volume of work which can be undertaken in any one year is necessarily limited by the engineering and construction force which can be obtained and effectively utilized. Scheduling must recognize also the problems of financing and the limitations imposed by the time involved in obtaining necessary funds.

BASIS OF CONSTRUCTION PROGRAM

Forecasts of population and of average sewage flows for the various sewerage areas (Tables 14-1 and 15-1), coupled with estimated population distributions (Fig. 5-9), were used both to determine construction timing and to estimate treatment plant loadings. It should be realized, however, that urban development in specific areas may take place either sooner or later than here anticipated, and that metropolitan growth as a whole may occur at a slower rate. For that reason, later stages of the proposed construction program will be subject to adjustment.

Construction of the proposed improvements is to be undertaken in three stages. Stage I, includes facilities required to alleviate the most serious problems and assumes initial construction in 1960, with completion by 1970. This scheduling allows time for organization of a central sewerage agency to administer financing, design, construction, and operation of the recommended facilities. Stage II includes extension of facilities to serve additional areas and assumes construction during the period 1970 to 1980. Stage III includes all remaining facilities, the construction of which would be undertaken as required some time after 1980.

Because of limited development in adjoining areas and also because of financial considerations, construction in some areas presently in need of sewerage service has been deferred to Stages II or III. Such deferments require that local sewers and, in most cases, temporary sewage treatment plants be constructed and utilized until such a time as a connection to the central sewerage system becomes financially possible. Likewise, some of the existing treatment plants which serve small, individual areas will have to be retained until central sewerage can be made available.

STAGE I CONSTRUCTION, 1960-1970

Relief of the most urgent sewerage needs of the metropolitan area will be achieved upon completion of the Stage I construction program. Scheduled to begin by 1960 and to end by 1970, this program calls for construction of facilities (Fig. 16-1) estimated to cost a total of $83,215,000 (Table 16-1). Three basic systems are to be constructed, the first designated as the Renton system, the second as the West Point system, and the third as independent systems. Essential features of each are described in the following presentation.

Renton System Sewers

Renton system sewers to be constructed under Stage I include (1) the Core Plan B interceptor extending east from the proposed sewage treatment plant, (2) the north branch feeder sewer along the Northern Pacific railroad right-of-way to the point at which sewage presently discharged to the city of Kirkland treatment plant will be intercepted, and (3) service sewers required to intercept major sewage discharges to Lake Washington. Preliminary plans and profiles of all Core Plan B sewers and feeder sewers to be constructed under Stage I are presented in Appendix E.

Although initial flows in the north branch feeder sewer will be low as related to its design capacity, provision for ultimate flow requirements is necessary because of the difficult construction conditions along much of its route. These conditions are such that it would not be economically feasible to install a parallel line at some future date. Moreover, a large part of the cost of laying a sewer along this route is for preliminary items, such as the provision of adequate access and working space, and for contingencies as-

Table 16-1. Stage I Construction, Recommended Sewerage Facilities

Facility	Construction cost,[a] dollars
Core Plan B[b]	
Sewers	
B-2	339,000
B-4 – B-22[c]	13,911,000
Pumping stations	
PS-B-1 – PS-B-4	1,713,000
Treatment plants	
STP-B-1[d]	11,848,000
STP-B-2	11,524,000
Outfalls	
B-3[e]	318,000
B-23 – B-24	3,846,000
Total, Core Plan B	43,499,000
Core Plan B feeder sewers[f]	
S-4 – S-5	2,615,000
S-11 – S-15	5,762,000
S-17 – S-22	7,066,000
N-7 – N-9	6,209,000
N-10 – N-28[c]	358,000
PS-N-2[g]	738,000
PS-N-3 – PS-N-4[h]	–
Total, feeder sewers	22,748,000
Service sewers	
East Lake Washington[i]	
ELW-10	45,000
ELW-11 – ELW-14	294,000
ELW-21 – ELW-24	301,000
ELW-27	39,000
ELW-28 – ELW-31[j]	–
ELW-32	25,000
ELW-33 – ELW-34[j]	–
ELW-35 – ELW-37	351,000
PS-ELW-3 – PS-ELW-5	304,000
PS-ELW-7	139,000
PS-ELW-8 – PS-ELW-10[j]	–
PS-ELW-11 – PS-ELW-12	325,000
	1,823,000
Northwest Lake Washington[k]	
NWW-6 – NWW-7	90,000
NWW-8 – NWW-10[c]	–
NWW-11	60,000
NWW-12 – NWW-14[c]	686,000
	836,000
South Lake Washington[l]	
SLW-18 – SLW-20	620,000
	620,000
Green River[l]	
GR-20 – GR-21[c]	–
Southwest Lake Washington[m]	
SWW-1 – SWW-2[h, n]	770,000
SWW-3 – SWW-9[c, n]	462,000
SWW-10 – SWW-13[c, n]	618,000
SWW-14 – SWW-20[c]	–
PS-SWW-1[h,n]	401,000
PS-SWW-2[c]	–
PS-SWW-3[h, n]	632,000
PS-SWW-4[c, n]	222,000
	3,105,000
Elliott Bay[m]	
EB-1 – EB-19	2,880,000
PS-EB-1 – PS-EB-2	536,000
	3,416,000
Lake Union[m]	
LU-1 – LU-18[c]	75,000
LU-19[j]	–
LU-20[c]	–
	75,000
Total, service sewers	9,875,000
Separate systems	
South Puget Sound[o]	
Des Moines subarea	
SPS-15 – SPS-16	186,000
STP-SPS-2[p]	1,241,000
SPS-17	191,000
	1,618,000
Miller Creek subarea	
SPS-22	52,000
STP-SPS-3[q]	855,000
SPS-23	404,000
	1,311,000
Southwest Suburban subarea	
SPS-24 – SPS-27[c]	92,000
STP-SPS-4[c]	160,000
SPS-28[r]	–
	252,000
West Seattle subarea	
SPS-29 – SPS-37[c]	–
PS-SPS-2 – PS-SPS-5[c]	–
STP-SPS-5[c]	80,000
SPS-38[r]	–
	80,000
North Puget Sound[s]	
Piper Creek subarea	
NPS-1 – NPS-4[c]	44,000
STP-NPS-1	720,000
NPS-5 – NPS-6[c]	235,000
	999,000

(Continued on Page 452.)

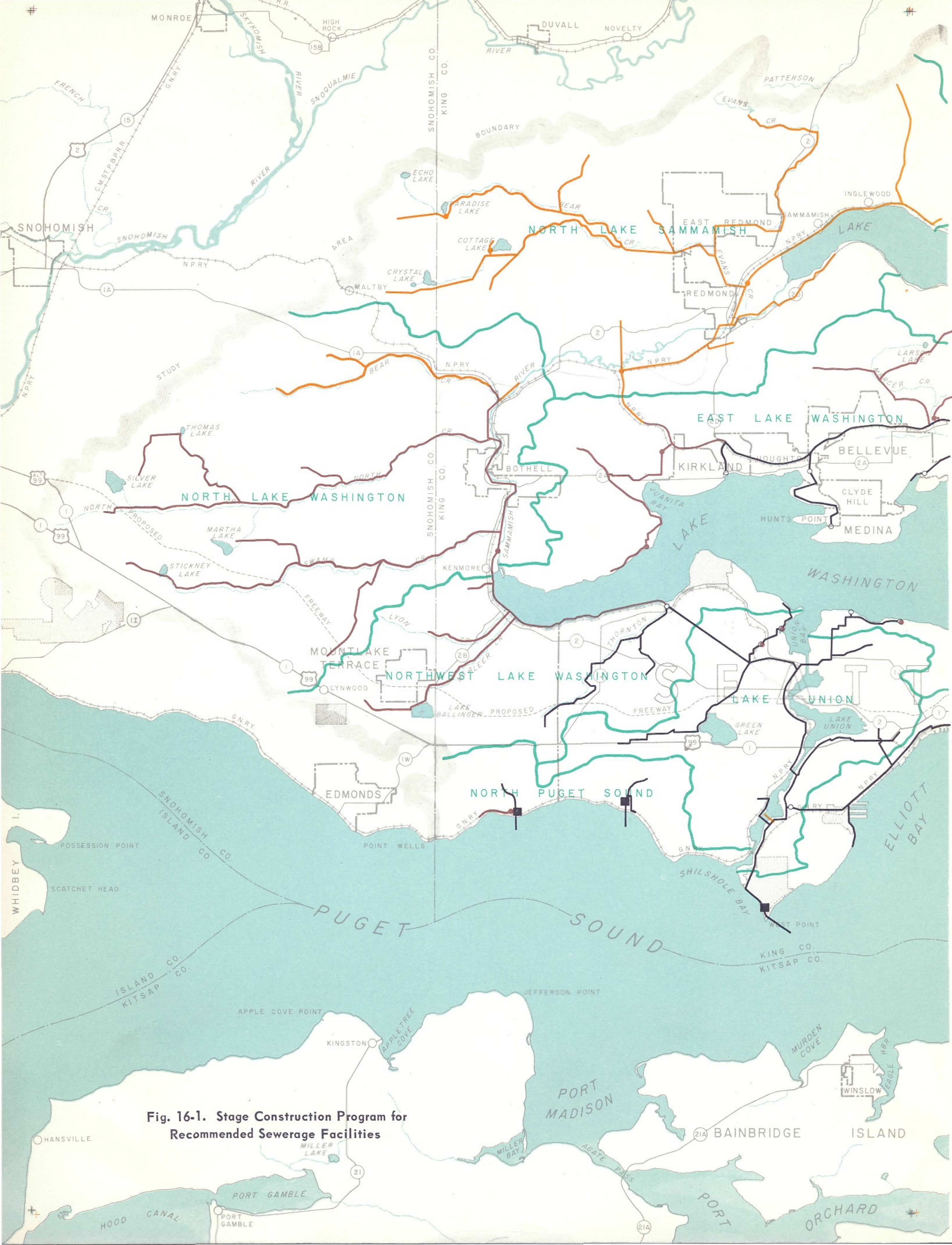

Fig. 16-1. Stage Construction Program for Recommended Sewerage Facilities

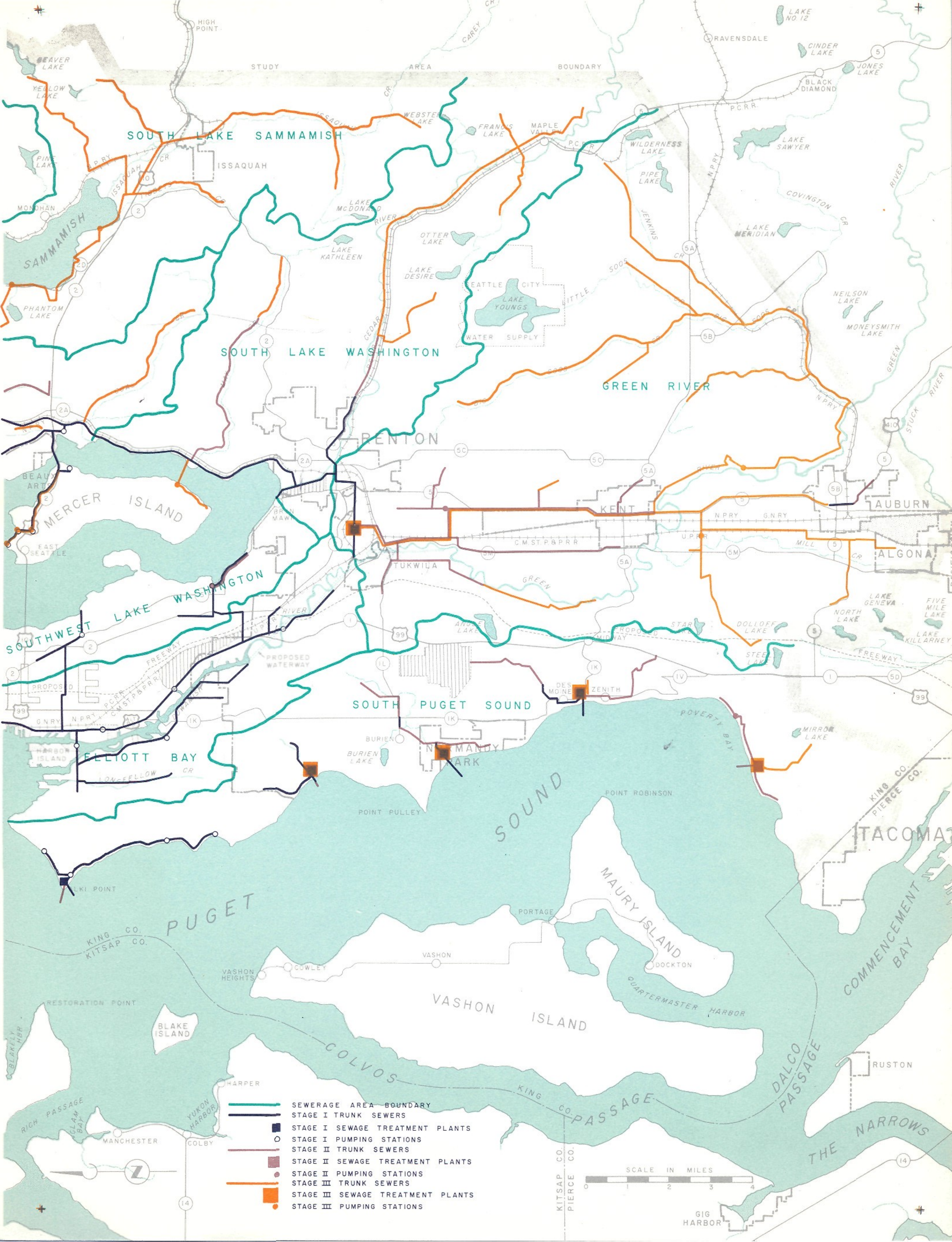
SOUTH LAKE SAMMAMISH
SOUTH LAKE WASHINGTON
GREEN RIVER
SOUTHWEST LAKE WASHINGTON
SOUTH PUGET SOUND
STUDY AREA BOUNDARY
ELLIOTT BAY
MERCER ISLAND
RENTON
KENT
AUBURN
ALGONA
TUKWILA
ISSAQUAH
TACOMA
RUSTON
PUGET SOUND
VASHON ISLAND
MAURY ISLAND
COLVOS PASSAGE
DALCO PASSAGE
COMMENCEMENT BAY
THE NARROWS
QUARTERMASTER HARBOR
POINT ROBINSON
POINT PULLEY
RESTORATION POINT
BLAKE ISLAND
KING CO.
KITSAP CO.
PIERCE CO.
SEWERAGE AREA BOUNDARY
STAGE I TRUNK SEWERS
STAGE I SEWAGE TREATMENT PLANTS
STAGE I PUMPING STATIONS
STAGE II TRUNK SEWERS
STAGE II SEWAGE TREATMENT PLANTS
STAGE II PUMPING STATIONS
STAGE III TRUNK SEWERS
STAGE III SEWAGE TREATMENT PLANTS
STAGE III PUMPING STATIONS
SCALE IN MILES
0 1 2 3 4

Table 16-1. Continued

Facility	Construction cost[a] dollars	Facility	Construction cost[a] dollars
Boeing Creek subarea		Total, separate systems	5,093,000
NPS-7	82,000	Temporary sewage treatment plants	2,000,000
STP-NPS-2	565,000		
NPS-9	186,000		
	833,000	Total, Stage I construction	83,215,000

See Fig. 16-1 for location of facilities to be constructed under Stage I.

[a] *Includes engineering and contingencies.*

[b] *See Fig. 15-4 for location and Table 15-4 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[c] *Includes existing facilities.*

[d] *Initial construction: preaeration and primary sedimentation tanks – 24 mgd capacity (16.7% of ultimate); aeration tanks – 24 mgd capacity (16.7% of ultimate); secondary sedimentation tanks – 24 mgd capacity (16.7% of ultimate); and sludge digesters – 36 mgd capacity (25% of ultimate); includes cost of 100 acres of land.*

[e] *Initial construction, single 78-inch outfall.*

[f] *See Fig. 15-7 for location and Table 15-11 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[g] *Includes structure for ultimate requirements and equipment for 60 per cent of ultimate.*

[h] *Existing sewers and pumping stations; paralleling or replacing of inadequate sewers or pumping stations under Stage II construction.*

[i] *See Fig. 15-15 for location and Table 15-33 for description of facilities.*

[j] *Existing sewers and pumping stations; paralleling or replacing of inadequate sewers or pumping stations, Stage III construction.*

[k] *See Fig. 15-16 for location and Table 15-34 for description of facilities.*

[l] *See Fig. 15-17 for location and Table 15-35 for description of facilities.*

[m] *See Fig. 15-18 for location and Table 15-36 for description of facilities.*

[n] *Includes holding tanks on tributary local sewers.*

[o] *See Fig. 15-12 for location and Table 15-27 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[p] *Initial construction, primary plant for 3.25 mgd capacity (50 per cent of ultimate).*

[q] *Initial construction for 3.75 mgd capacity (50 per cent of ultimate.*

[r] *Existing outfall; extension Stage II construction.*

[s] *See Fig. 15-13 for location and Table 15-30 for description of facilities.*

sociated with difficult working conditions. Since these costs would remain more or less constant regardless of the size of line to be installed, about the only saving that would accrue in laying a smaller sewer initially would be the relatively small difference in pipe costs. It would thus be false economy to start with a sewer smaller than that required for ultimate development of the tributary area.

Intercepting sewers to serve the Green River sewerage area south of the proposed treatment plant are not included in the Stage I construction program. This is because developed areas to the south, namely, Kent and Auburn, are presently served by independent treatment and disposal works which should remain adequate or can feasibly be enlarged during the period covered by this program. Sewers necessary to connect these areas to the central system are provided for under subsequent stages and would be constructed and extended to keep pace with industrial and residential development of Green River valley and adjoining areas. Similarly, feeder sewers to serve the North Lake Sammamish and South Lake Sammamish sewerage areas are not included in Stage I because those areas are not yet developed to an extent sufficient to justify such a project.

Renton System Treatment Plant

Secondary treatment by the activated sludge process will be provided at the Renton plant. Initially, this plant will serve the Southwest Lake Washington, East Lake Washington and South Lake Washington sewerage areas. Ultimately, it will receive sewage also from the North Lake Sammamish, South Lake Sammamish and Green River sewerage areas.

Stage I construction of treatment plant facilities calls for provision of a capacity sufficient to last for approximately 10 to 15 years. Following that period, the capacity will be increased by adding parallel units.

Based on an analysis of predicted average flows to the Renton plant (Table 16-2), it appears feasible to plan construction in three increments, with each increment providing for a capacity of 48 mgd, or one-third of the ultimate requirement. Since the first increment would provide sufficient capacity until 1985-1990, a further division of construction is required in order to keep the initial outlay to the lowest practi-

Table 16-2. Estimated Average Sewage Flows, Renton Sewage Treatment Plant

Year	Average flow, mgd
1965	7
1970	13
1975	19
1980	30
1985	43
1990	60
1995	74
2000	87

cable level. Initial construction, therefore, will consist of providing sedimentation and aeration capacity for 24 mgd and sludge digestion capacity for a plant flow of 36 mgd. These facilities will be adequate until 1975-1980, at which time additions will be made to increase the total capacity of the plant to 48 mgd.

Flow Diagram and Design Data. A diagrammatic arrangement of the necessary treatment facilities is shown in Fig. 16-2. Basic data relating to the various structures and items of equipment are presented in Table 16-3. A detailed layout cannot be made at this time because orientation and final arrangement of the various units will depend on the physical characteristics of the plant site. Nevertheless, a tentative layout is shown in Fig. 16-3 and assumes construction on a 50-acre site about 1,000 feet in width by 2,200 feet in length. A perspective view of the Renton plant based on this layout is shown in Fig. 16-4.

Required structures at the plant will include sedimentation tanks, aeration tanks, sludge digestion tanks, power building, administration building, chlorination building, sludge control building, and miscellaneous items such as passageways, pipe chases, and meter boxes. The power building will contain mechanical bar screens, influent pumping equipment, aeration blowers, power generation equipment, lavatory and locker room, day room, store rooms, workshops and garage, offices, and miscellaneous mechanical equipment. The administration building will contain offices, laboratories and other necessary facilities. It is possible that the administration building may be eliminated, or its construction delayed to some later stage, by combining it initially with the power building. For preliminary estimating purposes, however, the administration building is included under Stage I construction. The sludge control building will contain all sludge heating and sludge handling equipment. The chlorination building will contain all chlorination equipment and a bulk chlorine storage tank.

The estimated initial cost of the treatment plant provides for the purchase of 100 acres of land, or sufficient for ultimate development. Of this total, approximately 50 acres will be used for plant structures and facilities, while the remaining 50 will be used for sludge lagoons. While the latter need not necessarily be contiguous to the plant, they should at least be within an economical pumping distance. Only a portion of the total land will be used under initial construction. Nevertheless, the entire 100 acres should be purchased immediately to allow for economic and orderly expansion of plant facilities as the need arises.

Pretreatment Facilities. Prior to pumping and treatment, raw sewage will be screened through bar racks to remove large objects and rags which otherwise would damage or clog pumps and interfere with operation.

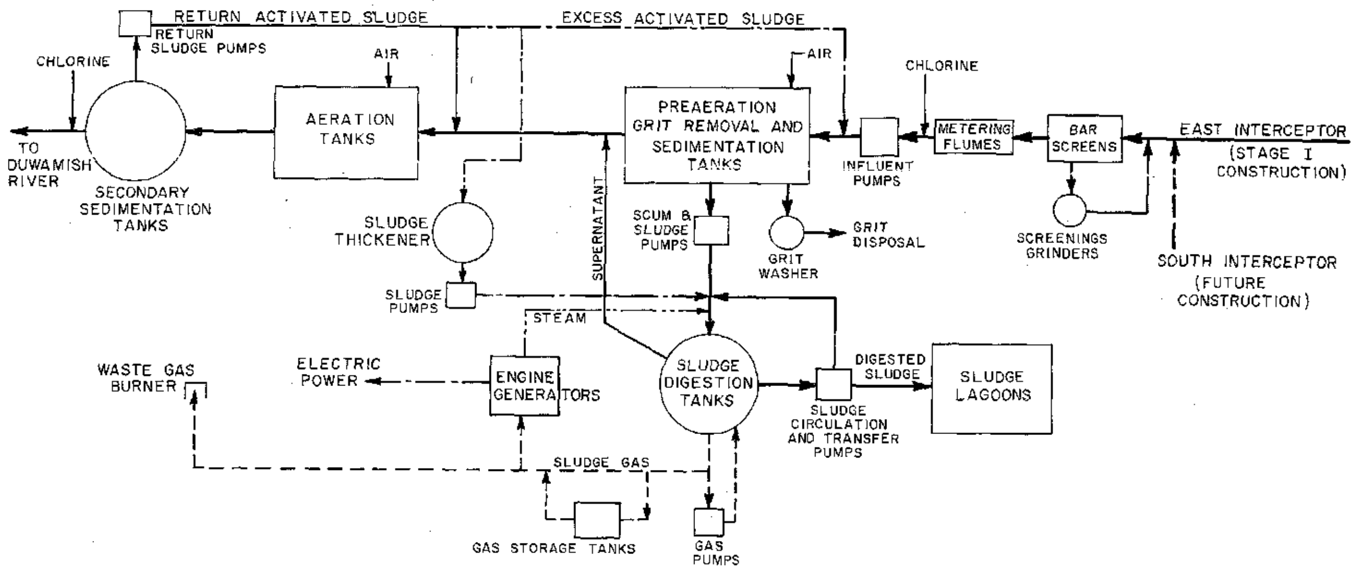

Fig. 16-2. Flow Diagram, Renton Sewage Treatment Plant

Table 16-3. Design Factors, Renton Sewage Treatment Plant

Population in thousands		Length, feet	130
Initial design	170	Width, feet	38
Ultimate	995	Average water depth, feet	9
Industrial, population equivalent in thousands		Detention time at average DWF, minutes	60
Initial design	80	Overflow rate at average DWF, gal. per sq ft per day	1,600
Ultimate	665	Hydraulic capacity per tank, mgd	35
Total design population in thousands		Primary treatment efficiency	
Initial design	250	Assumed BOD reduction, per cent	25
Ultimate	1,660	Assumed suspended solids reduction, per cent	45
Loading		Aeration tanks	
Average dry weather flow, mgd		Number	
Initial design	24	Initial design	3
Ultimate	143	Ultimate	18
Peak wet weather flow, mgd		Length, feet	700
Initial design	100	Width, feet	38
Ultimate	360	Average water depth, feet	15
Present minimum flow, mgd	3.5	Detention time at average DWF, hours	8.9
BOD, 1,000 pounds per day		Return activated sludge, per cent of average DWF	25
Initial design	50	Air supplied, cu ft per gal. at average DWF	1.0
Ultimate	332	Hydraulic capacity per tank, mgd	35
Suspended solids, 1,000 pounds per day		Secondary sedimentation tanks	
Initial design	60	Number	
Ultimate	415	Initial design	8
Bar screens		Ultimate	48
Number		Diameter, feet	70
Initial design	2	Average effective water depth, feet	8.5
Ultimate	4	Detention time at average DWF, hours	2.0
Clear spacing, inches	0.75	Overflow rate at average DWF, gal. per sq ft per day	800
Influent pumps		Hydraulic capacity per tank, mgd	12.5
Number		Secondary treatment efficiency	
Initial design	3	Assumed BOD reduction, per cent	93
Ultimate	5	Assumed suspended solids reduction, per cent	82
Installed capacity, mgd		Total treatment efficiency	
Initial design	150	Assumed BOD reduction, per cent	95
Ultimate	450	Assumed suspended solids reduction, per cent	90
Aeration blowers		Sludge digestion tanks	
Number		Number	
Initial design	2	Initial design	2
Ultimate	5	Ultimate	8
Installed capacity, 1,000 cfm		Diameter, feet	100
Initial design	50	Side water depth, feet	30
Ultimate	125	Volume per tank, 1,000 cubic feet	233
Preaeration and grit removal tanks		Loading, pounds per cubic foot per day	0.20
Number		Sludge lagoons	
Initial design	3	Volume, acre-feet	
Ultimate	18	Initial design	100
Length, feet	26	Ultimate	600
Width, feet	38	Storage, years (ultimate construction at design loading, 1960 – 2030, 80 per cent moisture)	10
Average water depth, feet	15		
Detention time at average DWF, minutes	20		
Hydraulic capacity per tank, mgd	35		
Primary sedimentation tanks			
Number			
Initial design	3		
Ultimate	18		

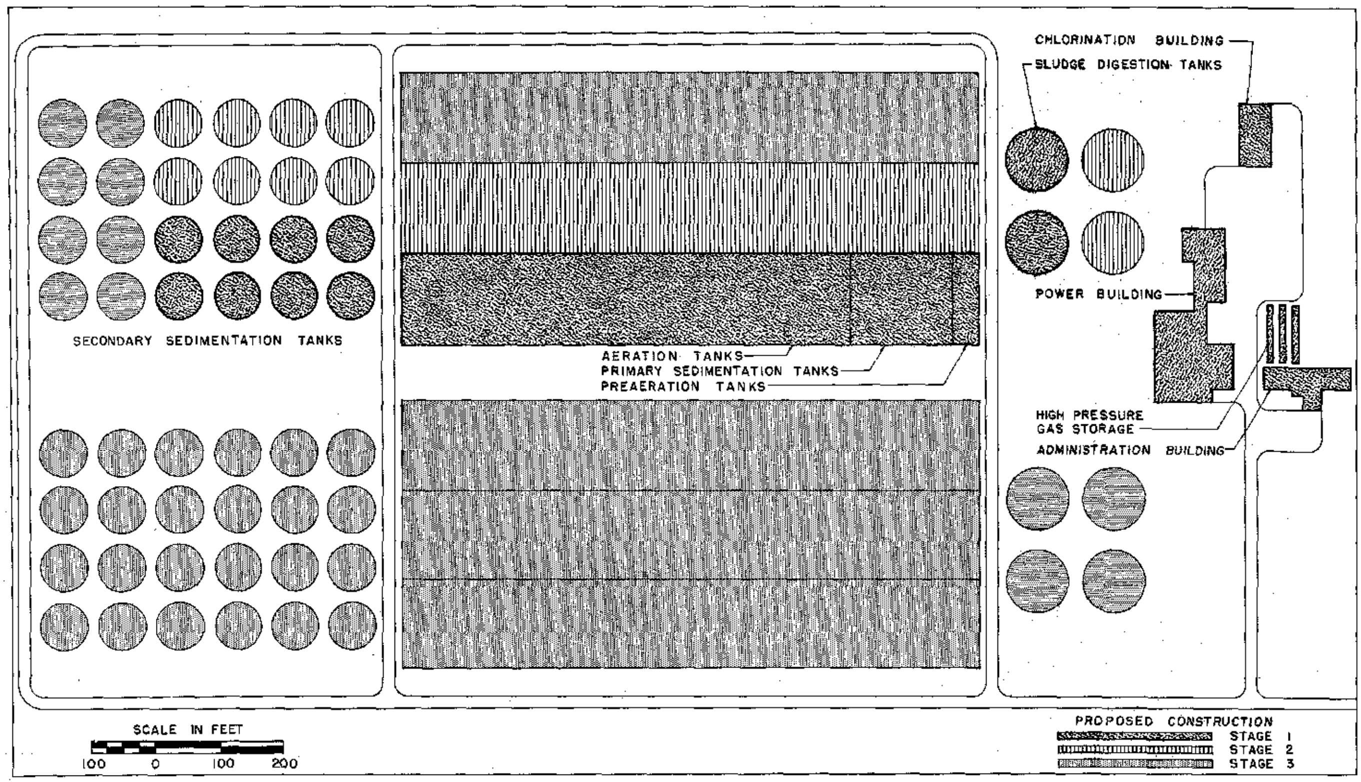

Fig. 16-3. Possible Layout of Sewage Treatment Facilities, Renton Plant

of the sedimentation tanks. Equipment for this purpose will consist of four mechanically cleaned bar screens, two of which will be installed initially. Material removed by the screens will be ground by one of two grinders and returned to the sewage flow upstream from the screens. Each screen will be installed in a suitable influent channel. Water surface elevations downstream from the bar screens will be controlled by parshall flumes, which will act also as primary metering elements for flow measurements.

Influent Pumping. Screened raw sewage will be lifted about 30 feet by engine driven pumps. Pump speed and discharge rate will be controlled by flow-responsive pneumatic equipment. This arrangement permits elimination of the usually costly raw sewage sump and results in high sedimentation efficiency through prevention of pumping surges in the sedimentation tanks.

Stage I construction calls for the installation of three pumps, two of which will be capable of accommodating the estimated design peak flow, with the third unit reserved as a standby. Subsequent stages of construction call for replacement of the three initial units with pumps of larger capacity, as well as the installation of two additional units.

Power Generation. Engines for driving the raw sewage pumps and aeration blowers, and for supplying other major power demands, will be either dual-fuel or spark ignition type, using as a source of fuel sludge gas produced in the sludge digestion process. Waste heat rejected through the cooling system and in the exhaust gases will be used to heat the digester and plant buildings.

Preaeration and Grit Removal. Combination preaeration and grit removal will be obtained in reinforced concrete tanks, each with a detention period of 20 minutes at the design flow of 8 mgd, and a maximum hydraulic capacity of 35 mgd. Three tanks will be provided initially, with 18 required for ultimate capacity. Under initial design, all tanks will have to be in operation during peak flows. At ultimate development, however, it will be possible to take one or more tanks out of service at any time. Grit will be collected in hoppers within the tanks, pumped to a grit washer by water ejectors, and finally disposed of by filling low areas around the plant.

Primary Sedimentation. Primary sedimentation will be obtained in reinforced concrete tanks, which structurally, will be a continuation of the preaeration tanks. As in the preaeration system, three tanks will be constructed initially and 18 ultimately. Rectangular in plan, each tank will provide a detention period of 60 minutes at design flow and a maximum hydraulic capacity of 35 mgd. All of the tanks will be equipped

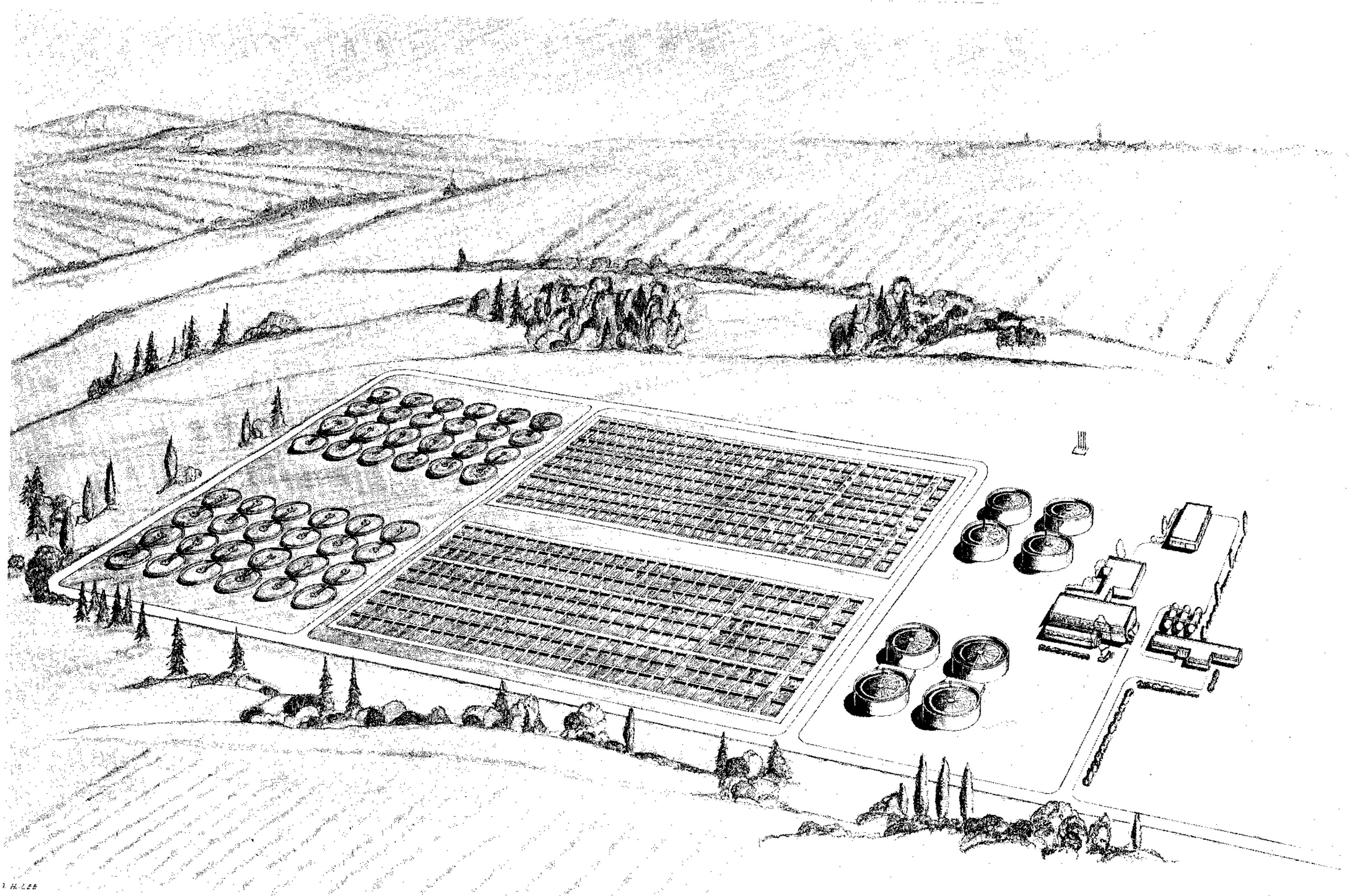

Fig. 16-4. Perspective View of Renton Sewage Treatment Plant

with mechanical sludge collecting and scum removing mechanisms.

Aeration. Aeration of the mixture consisting of primary settled sewage and activated sludge will be obtained in reinforced concrete tanks which will provide a detention period of 8.9 hours at design flow and a maximum hydraulic capacity of 35 mgd. Three of these tanks will be constructed initially and 18 ultimately.

Aeration capacity will be provided at a rate of 1.0 cubic foot of air per gallon of sewage, average dry weather flow. Five engine driven blowers will be installed, two of them under Stage I construction. One blower will be available as a standby unit, both initially and in the future.

Return activated sludge will be introduced at the inlet to each aeration tank. Input rates will be regulated automatically and the flow will be distributed to each tank, using pneumatic controllers. Excess activated sludge either will be returned to the plant influent upstream from the raw sewage metering flumes, or will be discharged directly to the digesters after passing through a sludge thickener.

Secondary Sedimentation. Secondary sedimentation will be obtained in circular reinforced concrete tanks having a detention time of two hours at design flow and a maximum hydraulic capacity of 12.5 mgd. These tanks will be constructed in batteries of four, of which two will be installed under Stage I and twelve ultimately. A distribution structure will be provided at each battery to distribute the flow equally to each tank. The rate of withdrawal of sludge from each tank will be regulated by pneumatic controllers.

Effluent Chlorination. Effluent will be chlorinated to an extent sufficient to obtain adequate disinfection. Chlorine doses will be regulated automatically, using flow-responsive pneumatic control equipment. Provision will be made for bulk chlorine storage and a railroad siding will be constructed to enable purchase of chlorine in tank car lots. Since adequate detention time will be available in the outfall, chlorine contact tanks will not be required during initial operation. Contact tanks to provide an additional detention time of 10 minutes will be required in the future.

Raw Sludge Pumping. Combined raw sludge and scum removed from the primary sedimentation tanks will be conveyed to an external sump, from which it will be pumped directly to the digesters by means of heavy duty pumps. Operation of these pumps will be automatically regulated by means of pneumatic control equipment and will depend on the level of sludge in the sump.

Sludge Digestion and Disposal. Sludge digestion will be obtained in circular reinforced concrete, brick veneered tanks. Eight tanks will be required for ultimate capacity, with two being constructed under the Stage I program. Each tank will be equipped with a floating steel cover and will be designed for utmost flexibility of operation. No pipes or equipment of any kind will be suspended in the tanks. Sludge inlets and outlets, as well as the gas take-offs, will be carried outside the tanks either in the ground or over the tops in suitable flexible conduits. All outdoor lines will be adequately protected against freezing. Although the two initial tanks will be identical, sludge piping will be so arranged that either one may be used for primary digestion and the other for secondary.

Temperature of the digesting sludge will be maintained at 95° F, using an automatic control system and circulation of the sludge through an external heat exchanger. In addition to sludge circulation, a separate gas circulating system will be provided to break up scum formations.

Supernatant liquor removed from the digestion tanks will be returned to the inlet of the aeration tanks. Digested sludge will be removed from the digesters periodically and discharged to the lagoons. A lagoon capacity of 100 acre-feet is expected to be sufficient for approximately 10 years under initial loading.

Effluent Outfall. Effluent from the treatment plant will be discharged to Duwamish River through a 78-inch reinforced concrete outfall. Ultimately, a parallel 78-inch line will be required.

West Point System Sewers

Sewers to be constructed in the West Point system under the Stage I program include all those of Core Plan B, plus feeder sewers to the point at which flow to the Lake City treatment plant can be intercepted. Stage I construction provides also for service sewers which are required in the Elliott Bay, Northwest Lake Washington and Lake Union sewerage areas both to intercept existing raw sewage and industrial waste outfalls, and to provide service to

Table 16-4. Estimated Average Sewage Flows, West Point Sewage Treatment Plant

Year	Average flow, mgd
1965	57
1970	67
1975	78
1980	83
1985	89
1990	95
1995	100
2000	104

presently developed areas. Preliminary plans and profiles of Core Plan B sewers and feeder sewers to be constructed under Stage I are presented in Appendix E.

West Point System Treatment Plant

Primary treatment will be provided in the West Point plant. Initially, this plant will serve the Southwest Lake Washington, Elliott Bay, Lake Union and

Table 16-5. Design Factors, West Point Sewage Treatment Plant

Population in thousands	958	Average water depth, feet	15
Industrial, population equivalent in thousands	532	Detention time at average DWF, minutes	30
Total design population in thousands	1,490	Hydraulic capacity per tank, mgd	30
Loading		Sedimentation tanks	
Average dry weather flow, mgd	~~118~~ [illegible]	Number	12
Peak wet weather flow, mgd	~~302~~ 322	Length, feet	240
Present minimum flow, mgd	28	Width, feet	38
BOD, 1,000 pounds per day	298	Average water depth, feet	9
Suspended solids, 1,000 pounds per day	372	Detention time at average DWF, hours	1.5
Bar screens		Overflow rate at average DWF, gal. per sq ft per day	1,100
Number	4	Hydraulic capacity per tank, mgd	30
Clear spacing, inches	0.75	Treatment efficiency	
Influent pumps		Assumed BOD reduction, per cent	30
Number	4	Assumed suspended solids reduction, per cent	60
Installed capacity, mgd	400	Sludge digestion tanks	
Preaeration blowers		Number	4
Number	3	Diameter, feet	100
Installed capacity, 1,000 cfm	12	Side water depth, feet	36
Preaeration and grit removal tanks		Volume per tank, 1,000 cubic feet	280
Number	12	Loading, pounds per cubic foot per day	0.2
Length, feet	48		
Width, feet	38		

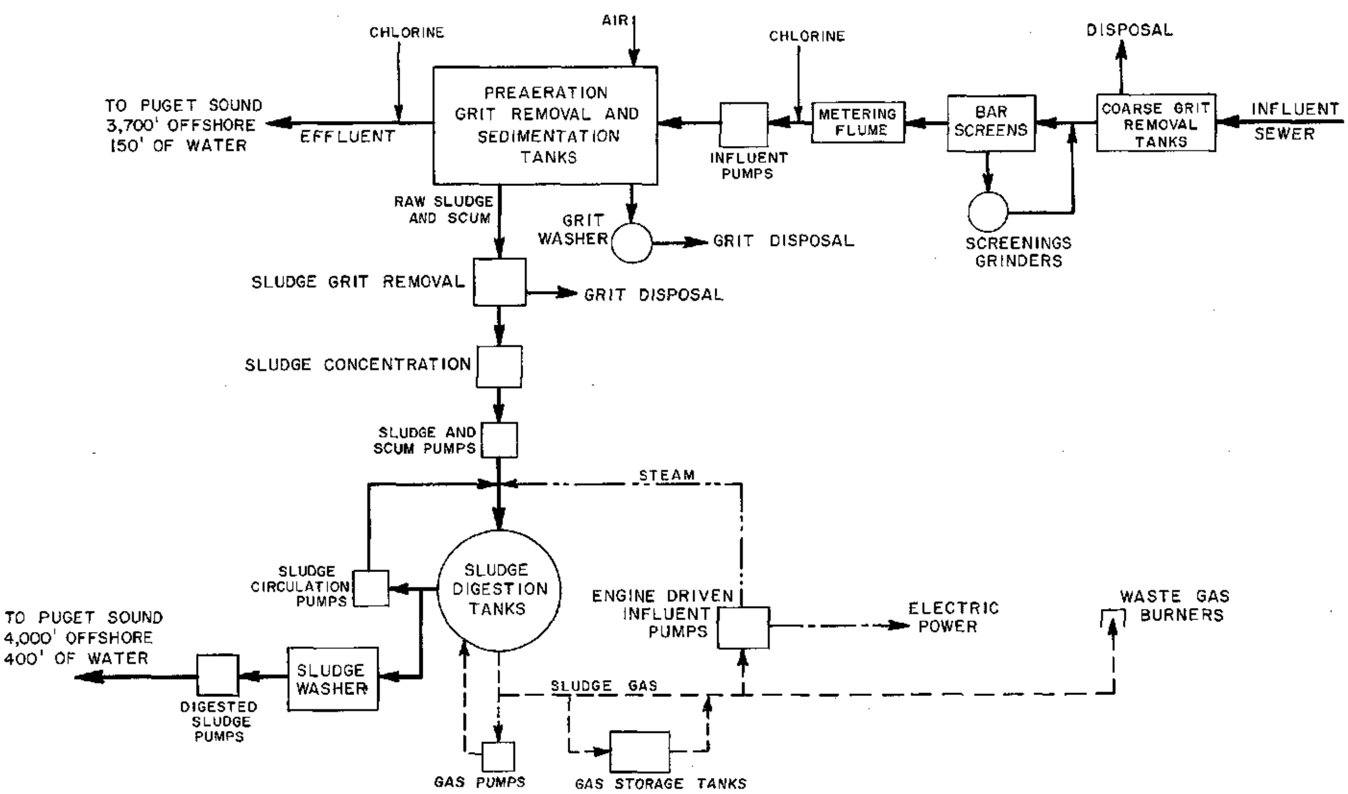

Fig. 16-5. Flow Diagram, West Point Sewage Treatment Plant

Northwest Lake Washington sewerage areas. At some future date, probably in about 10 to 15 years, it will receive sewage also from the North Lake Washington area.

An analysis of predicted average flows to the West Point plant (Table 16-4) indicates that the initial flow will be 57 mgd, or about 50 per cent of the ultimate dry weather capacity of 118 mgd. By 1975, the flow will increase to 78 mgd, or about 70 per cent of the ultimate. It is obvious, therefore, that any program designed for stage or incremental construction would require enlargement of the plant within a relatively short period. On that basis, the plant should be constructed initially to meet ultimate needs.

Flow Diagram and Design Data. Basic design data relating to the various structures and items of equipment to be incorporated in the West Point treatment plant are presented in Table 16-5. A flow diagram is shown in Fig. 16-5, a tentative layout is shown in Fig. 16-6, and a sketch of the treatment works as it might appear when completed is shown in Fig. 16-7.

Required structures will include sedimentation tanks, sludge digestion tanks, plant control building, chlorination building, sludge control building, and miscellaneous units such as passageways, pipe chases, and necessary in the future, planning should be such that a 50-acre site could be developed.
power generation equipment, lavatory and locker room, day room, store rooms, workshops and garage, offices, laboratories, and miscellaneous mechanical equipment. The sludge control building will contain all sludge heating and handling equipment. The chlorination building will contain all chlorination equipment and chlorine storage facilities.

An area of 20 acres will be required for the units to

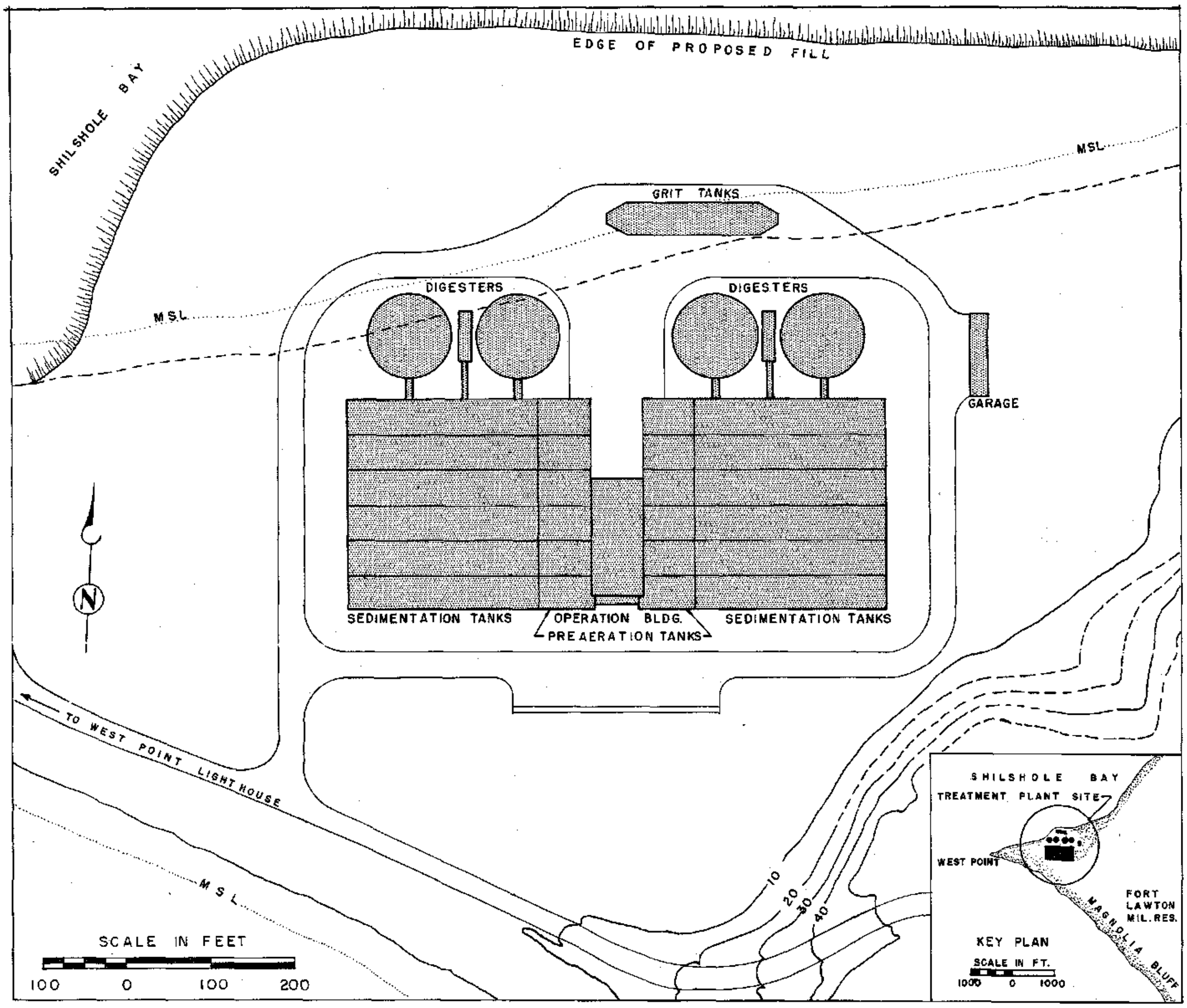

Fig. 16-6. Suggested Layout of Sewage Treatment Facilities, West Point Plant

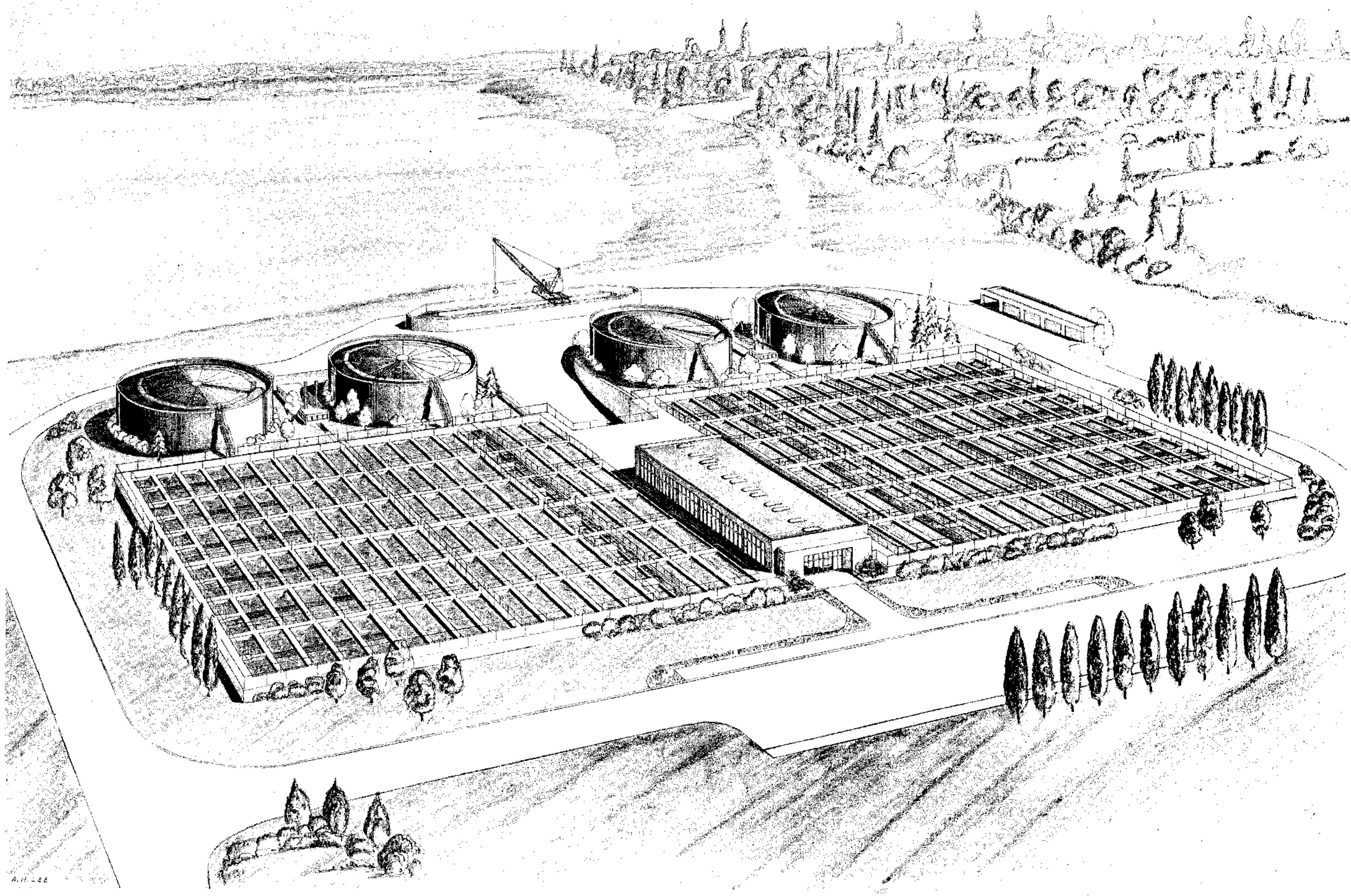

Fig. 16-7. Perspective View of West Point Sewage Treatment Plant

be constructed initially. In view, however, of the possibility that secondary treatment could become necessary in the future planning should be such that a 50-acre site could be developed.

Coarse Grit Removal. Because much of the area tributary to the plant is served by combined sewers, the incoming sewage will contain large particles of grit. This material is capable of damaging mechanical equipment and must therefore be removed. For that purpose, two rectangular channels will be provided in which gravel and other objects heavier than water and larger than 1/2 inch in diameter will be removed. A clam shell bucket, operating from a top-riding, electric crane, will dredge accumulated coarse grit from the channels for hauling away by dump truck.

Pretreatment Facilities. Prior to pumping and treatment, raw sewage will be screened through bar racks to removal large objects and rags which otherwise would damage or clog pumps and interfere with operation of the sedimentation tanks. Equipment for this purpose will consist of four mechanically cleaned bar screens, each installed in a suitable influent channel. Material removed by the screens will be ground by one of two grinders and returned to the sewage flow upstream from the screens. Water surface elevations downstream from the screens will be controlled by parshall flumes, which will act also as the primary metering elements for flow measurement.

Influent Pumping. After passing through the metering flumes, the incoming sewage will be lifted to the sedimentation tanks by four identical engine-driven pumps. Each pump will be designed to discharge a maximum flow of 100 mgd against a total dynamic head of approximately 30 feet. Thus, at the peak capacity of 302 mgd, one pump will be available as a standby unit. Pump speeds and discharge rates will be controlled by pneumatic equipment to equal exactly the rate of raw sewage input.

Power Generation. Four gas-burning diesel or spark ignition engines will be provided to drive the raw sewage pumps and preaeration blowers and to supply other major power demands. Gas produced in the sludge digestion process will be used as a source of fuel for the engines, and waste heat rejected through the cooling system and in the exhaust gases will be used to heat the digesters and plant buildings.

Preaeration and Grit Removal. Combination preaeration and grit removal will be obtained in 12 reinforced concrete tanks, which will provide a detention period of 30 minutes at design flow and a maximum hydraulic capacity of 30 mgd. It will be possible for at least one tank to be out of service at all times. Grit will be collected in hoppers within the tanks, pumped to a grit washer by water ejectors, and finally disposed of by filling low areas around the plant.

Sedimentation. Sedimentation will be provided in 12 reinforced concrete tanks which, structurally, will be a continuation of the preaeration tanks. Rectangular in plan, each tank will provide a detention period of 90 minutes at design flow and a maximum hydraulic capacity of 30 mgd. All of the tanks will be equipped with mechanical sludge collection and scum removing mechanisms.

Effluent Chlorination. Effluent will be chlorinated during the recreational season, May to September, to insure adequate protection of beaches. Chlorine doses will be regulated automatically, using flow-responsive pneumatically controlled equipment. Provision will be made for storage and handling of chlorine in ton containers. Chlorine contact tanks will not be required since adequate detention times will be available in the outfall.

Raw Sludge Handling. It is expected that a fairly large amount of fine sand not readily removable by conventional grit removal methods will be carried to the plant and will settle with organic sludge in the sedimentation tanks. To prevent deposition of this material in the digestion tanks, provision will be made for its separation from the sludge before the sludge enters the tanks. For that purpose, sludge will be pumped continuously from the sedimentation tanks by means of air lift units and will pass through a constant velocity channel of such design that organic solids will be kept in suspension and grit will separate and deposit. Material thus deposited will be removed from the channel and disposed of by filling low areas around the plant site. Thin sludge will flow to a sump where it will be concentrated by a thickening operation to 5 or 6 per cent solids. From the sump, the thickened material will be delivered to the digesters by means of heavy duty pumps.

Sludge Digestion and Disposal. Sludge digestion will be obtained in four circular reinforced concrete, brick veneered tanks, each equipped with a floating cover. Although all tanks will be identical, sludge piping will be so designed that any of them may be used for either primary or secondary stage digestion. Each tank will be designed for utmost flexibility and no pipes or equipment of any kind will be suspended inside. Sludge inlets and outlets, as well as gas take-offs, will be carried outside the tanks either in the ground or over the tops in suitable flexible conduits. All outdoor lines will be adequately protected against freezing.

Table 16-6. Estimated Average Sewage Flows, Sewage Treatment Plants, South Puget Sound Sewerage Area

Year	Average flow, mgd				
	Redondo Beach	Des Moines	Miller Creek	Southwest Suburban	West Seattle
1965	–	0.8	0.6	2.5	6.7
1970	–	1.6	1.0	3.0	6.8
1975	0.6	2.7	3.1	2.2	6.9
1980	1.0	3.4	3.8	2.5	6.9
1985	1.4	4.0	4.6	2.7	7.0
1990	1.9	4.5	5.2	3.0	7.0
1995	2.4	4.9	5.7	3.2	7.0
2000	3.0	5.2	6.1	3.4	7.1

Temperature of the digesting sludge will be maintained at 95° F, using an automatic control system and circulating of the sludge through an external heat exchanger. In addition to sludge circulation, a separate gas circulating system will be provided to break up scum formations.

Digested sludge will be removed from the digesters periodically and, after passing through a washer, will be pumped to Puget Sound through a sludge outfall line extending 4,000 feet offshore to a water depth of 400 feet. The sludge washer will be of the counterflow type and will be designed to remove any floating material remaining in the digested material. Wash water will be returned to the plant influent. Supernatant liquor removed from the digestion tanks will be disposed of to Puget Sound through the sludge outfall line.

Effluent Outfall. Effluent from the treatment plant will be discharged through a gravity outfall sewer terminating 3,700 feet from shore at a depth of 150 feet. This line will consist of 1,900 feet of 120-inch reinforced concrete pipe land section and 3,700 feet of twin 78-inch reinforced concrete pipe submarine section. Each of the latter will have a diffuser section with a total of twelve 18-inch diameter outlets.

Independent Systems

Independent systems to be constructed under the Stage I program include facilities required both to intercept raw sewage discharges to Puget Sound and to serve areas in which sewerage service is presently needed. Units to be constructed include trunk sewers, pumping stations, treatment works and outfalls.

Des Moines Subarea, South Puget Sound Sewerage Area. Construction of a sewage treatment plant, submarine outfall and trunk sewers to serve the rapidly developing area in the immediate vicinity of the community of Des Moines is scheduled under Stage I. Because of financial considerations, construction of the sewer connecting the Seattle-Tacoma International Airport to the plant is deferred to Stage II. This presupposes continued operation of the small treatment plant at the airport. Should it prove desirable to abandon the plant at an earlier date, the construction date of the connecting sewer would have to be advanced.

A primary type treatment plant would be constructed initially. Since it will not be possible under ultimate flow conditions to maintain adequate protection of the shoreline with a primary effluent, the plant will be so designed that secondary units can be added in the future.

To determine the required initial capacity as well as construction dates of subsequent enlargements, an analysis was made of predicted flows to the treat-

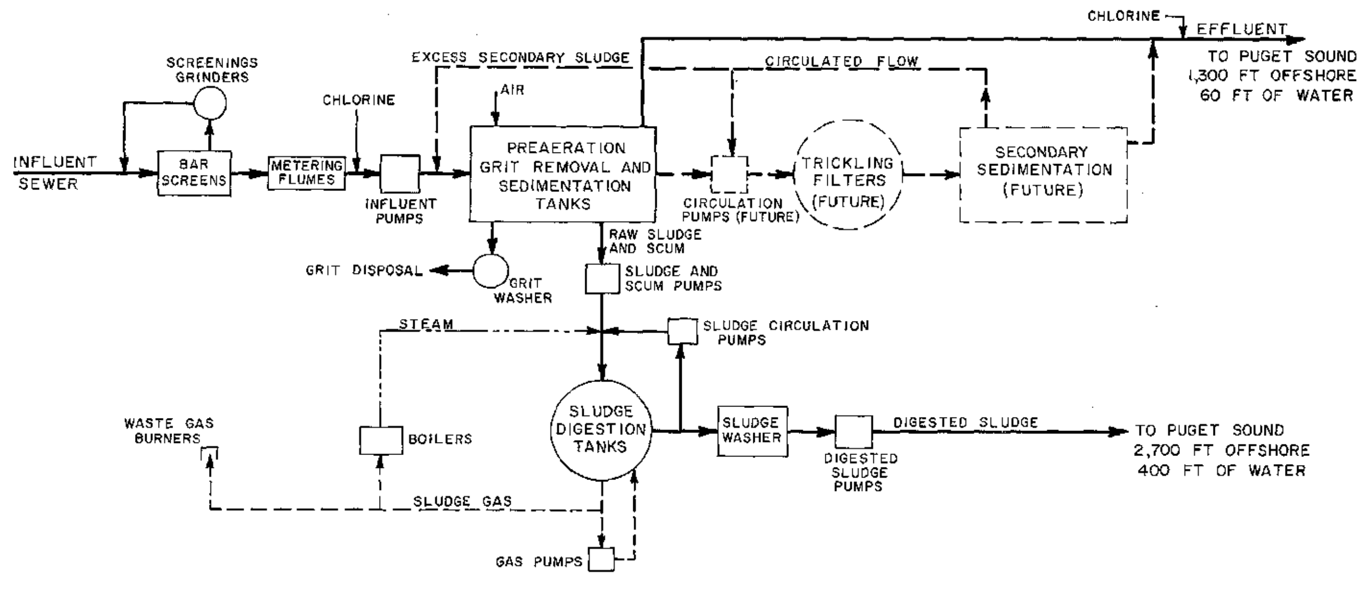

Fig. 16-8. Flow Diagram, Des Moines Sewage Treatment Plant

ment plant (Table 16-6). From this it appears that construction of a primary treatment plant with an initial capacity of 3.25 mgd will be satisfactory. Expansion to an ultimate capacity of 6.5 mgd will be required between 1975 and 1980. Construction of secondary units will also be required at that time.

As shown schematically in Fig. 16-8, the plant to be constructed initially includes facilities for pretreatment, influent pumping, preaeration and grit removal, primary sedimentation, separate sludge digestion, sludge disposal and effluent chlorination. Ultimately, facilities for trickling filtration and secondary sedimentation will have to be provided. To facilitate orderly future development, approximately 10 acres of land should be obtained for the plant site. Design factors are presented in Table 16-7.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 1,300 feet from shore at a depth of 60 feet. This sewer will consist of 1,200 feet of 30-inch reinforced concrete pipe land section and 1,300 feet of 30-inch reinforced concrete pipe submarine section.

Miller Creek Subarea, South Puget Sound Sewerage Area. Construction of a sewage treatment plant, submarine outfall and trunk sewers to serve the area in the immediate vicinity of the city of Normandy Park is scheduled under Stage I. Because of financial considerations, construction of the sewer connecting the Burien Lake area to the plant is deferred to Stage II. This presupposes continued operation of the sewerage system in that area, with discharge of the sewage therefrom to the existing sewage treatment plant of the Southwest Suburban Sewer District. Should it prove advisable to modify the existing system at an earlier date, the construction date of the connecting sewer would have to be advanced.

Treatment works to serve the Miller Creek subarea will be of the primary type and will be designed with an initial capacity of 3.75 mgd, or 50 per cent of the ultimate requirement. As determined from Table 16-6, expansion to its ultimate capacity of 7.5 mgd will be required between 1975 and 1980.

Facilities will be provided for pretreatment, preaeration and grit removal, sedimentation, separate sludge digestion, sludge disposal, and effluent chlor-

Table 16-7. Design Factors, Des Moines Sewage Treatment Plant

Population in thousands		Preaeration and grit removal tanks	
Initial design	28	Number	
Ultimate	60	Initial design	2
Industrial, population equivalent in thousands		Ultimate	4
		Detention, minutes	30
Initial design	6	Hydraulic capacity per tank, mgd	6
Ultimate	9	Primary sedimentation tanks	
Total design population in thousands		Number	
Initial design	34	Initial design	2
Ultimate	69	Ultimate	4
Loading		Detention, minutes	90
Average dry weather flow, mgd		Overflow rate, gallons per square foot per day	1,080
Initial design	3.25	Hydraulic capacity per tank, mgd	6
Ultimate	6.5	Trickling filters (future)	
Peak wet weather flow, mgd		Number	2
Initial design	8.0	Rock volume per filter, cubic yard	1,550
Ultimate	16	Circulation ratio	1.5:1
Present minimum flow, mgd	0.4	Volumetric loading per filter, mgd	8.0
BOD, 1,000 pounds per day		BOD loading, pounds per cubic yard	2.5
Initial design	7	Secondary sedimentation tanks (future)	
Ultimate	14	Number	4
Suspended solids, 1,000 pounds per day		Detention, minutes	120
Initial design	8	Overflow rate, gallons per square foot per day	800
Ultimate	17	Hydraulic capacity per tank, mgd	6
Bar screens		Sludge digestion tanks	
Number		Number	
Initial design	1	Initial design	2
Ultimate	2	Ultimate	3
Clear spacing, inches	0.75	Volume per tank, 1,000 cubic feet	25
Influent pumps		Loading, pounds per cubic foot per day	0.2
Number	3		
Installed capacity, mgd	24		

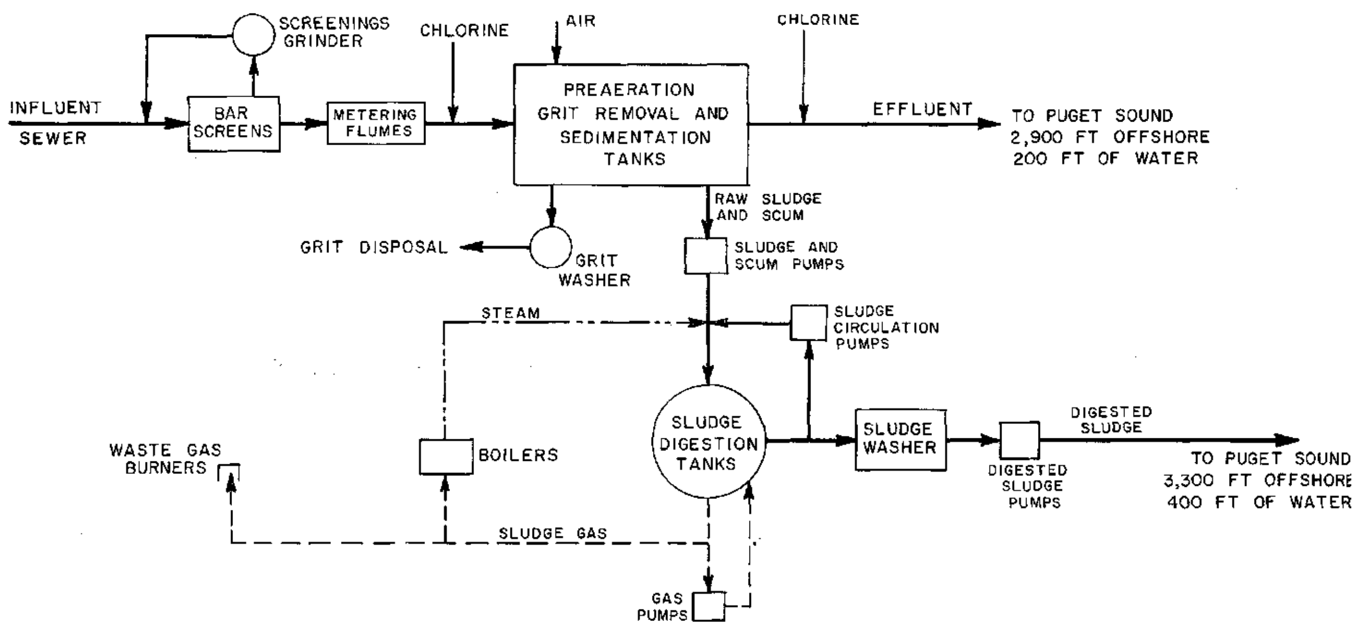

Fig. 16-9. Flow Diagram, Miller Creek Sewage Treatment Plant

ination. Approximately five acres of land will be required for ultimate development. A schematic flow diagram of the recommended plant is shown in Fig. 16-9, while design factors are given in Table 16-8.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 2,900 feet from shore at a depth of 200 feet. This line will consist of a land section comprising 1,200 feet of 27-inch reinforced concrete pipe and a submarine section comprising 2,900 feet of 27-inch reinforced concrete pipe.

Southwest Suburban Subarea, South Puget Sound Sewerage Area. The Stage I program calls for acquisition by the central agency of existing sewerage facilities of the Southwest Suburban Sewer District. Also called for is construction of a trunk sewer to serve the Roxbury Heights area.

Since the Stage I schedule for the Miller Creek subarea does not provide for construction of the sewer from the Miller Creek treatment plant to the Burien Lake area, the existing system serving the latter will

Table 16-8. Design Factors, Miller Creek Sewage Treatment Plant

Population in thousands		Bar screens	
Initial design	42	Number	
Ultimate	85	Initial design	1
		Ultimate	2
		Clear spacing, inches	0.75
Loading		Preaëration and grit removal tanks	
Average dry weather flow, mgd		Number	
Initial design	3.75	Initial design	2
Ultimate	7.5	Ultimate	4
Peak wet weather flow, mgd		Detention, minutes	30
Initial design	9.0	Hydraulic capacity per tank, mgd	6
Ultimate	18	Sedimentation tanks	
Present minimum flow, mgd	0.3	Number	
BOD, 1,000 pounds per day		Initial design	2
Initial design	8	Ultimate	4
Ultimate	17	Detention, minutes	90
		Overflow rate, gal. per sq ft per day	1,150
Suspended solids, 1,000 pounds per day		Hydraulic capacity per tank, mgd	6
		Sludge digestion tanks	
Initial design	10	Number	2
Ultimate	21	Volume per tank, 1,000 cubic feet	18
		Loading, pounds per cubic foot per day	0.2

continue to function as presently laid out. It is expected that this sewer will be constructed under the Stage II program, at which time the entire Lake Burien area will become tributary to the Miller Creek plant.

As reported earlier, the existing treatment plant of the Southwest Suburban Sewer District is of the primary type and has ample capacity for the predicted ultimate flow from the tributary area. On the other hand, receiving water conditions are such that a disinfected primary effluent cannot be safely discharged offshore from the plant. It will be necessary, therefore, to add facilities for secondary treatment. This improvement, though presently scheduled for Stage II construction, may be required at an earlier date if conditions in the receiving waters are found to be unsatisfactory.

For the treatment plant, the only work scheduled under Stage I is that of revamping sludge handling and heating facilities to enable ocean disposal of sludge and use of digester gas for heating purposes. Boilers utilizing digester gas will be installed, and sludge heating will be accomplished by external heat exchangers, using hot water produced in the boilers. Sludge will be circulated through the heat exchangers, using nonclog centrifugal pumps, and hot water will be circulated by means of a centrifugal pump. Heating rates will be controlled by regulating the temperature of the hot water in the heat exchanger. With these alterations, the existing heat pump presently used for sludge heating purposes will be abandoned, but all useable parts will be salvaged. Digested sludge, after passing through a washer, will be pumped to Puget Sound through a sludge outfall line extending 3,300 feet offshore to a water depth of 400 feet. The sludge washer will be of the counterflow type and will be designed to remove any floating material remaining in the digested sludge. Wash water will be returned to the plant influent. A schematic flow diagram of the existing plant is presented in Fig. 6-11.

Effluent is presently discharged through a 36-inch submarine outfall sewer terminating 600 feet offshore at a depth of 60 feet. As shown in Chapter 11, disposal this near shore and at the relatively shallow depth will not provide the required degree of shoreline protection, even with secondary treatment. It is proposed, therefore, to extend the outfall an additional 200 feet to a water depth of 85 feet, which would be sufficient to assure safe disposal of secondary effluent. Although extension of the outfall is not scheduled until Stage II, earlier construction may be required if bacterial contamination of the shoreline occurs in the meanwhile.

West Seattle Subarea, South Puget Sound Sewerage Area. The Stage I program calls for acquisition by the central agency of existing sewerage facilities of the city of Seattle, including intercepting sewers, pumping stations and treatment and disposal works.

Facilities now under construction will have ample capacity for the predicted ultimate flows from the tributary area. Under the Stage I program, the only work to be undertaken will be the construction of an outfall sludge line to a distance of 3,300 feet offshore and a water depth of 400 feet. Addition of the sludge line, together with a sludge washer, will enable disposal of digested sludge in Puget Sound. A schematic flow diagram of the existing plant is presented in Fig. 6-37.

In the project now under construction, effluent from the treatment plant will be discharged through a 42-inch submarine outfall sewer terminating 1,400 feet offshore at a depth of 85 feet. As shown in Chapter 11, it probably will not be possible with such an outfall to maintain adequate protection of the nearby beaches.

Since sufficient head is not available in the plant as constructed, pumping would be required in order to extend the existing outfall beyond its present depth. It is proposed, therefore, to construct a second outfall which will carry the peak sanitary flow and to use the existing line as a storm water overflow. The new line will terminate directly west of Alki Point and will discharge approximately 1,100 feet offshore at a depth of 210 feet. Although construction of the new outfall is not scheduled until Stage II, earlier construction may be required if bacterial contamination of the beaches occurs in the meanwhile.

Piper Creek Subarea, North Puget Sound Sewerage Area. Construction of a sewage treatment plant, trunk sewers, and a submarine outfall are scheduled under Stage I. Also scheduled is acquisition by the central agency of certain existing sewers of the city of Seattle.

Since initial flows will exceed 60 per cent of the ultimate flow (Table 16-9), the treatment plant will be constructed initially to meet ultimate needs. This plant will be of the primary type and will have a capacity of 3.7 mgd. Facilities will be provided for

Table 16-9. Estimated Average Sewage Flows, Sewage Treatment Plants, North Puget Sound Sewerage Area

Year	Average flow, mgd	
	Piper Creek	Boeing Creek
1965	2.3	1.0
1970	2.7	1.4
1975	3.0	1.9
1980	3.2	2.2
1985	3.3	2.4
1990	3.5	2.5
1995	3.6	2.5
2000	3.6	2.6

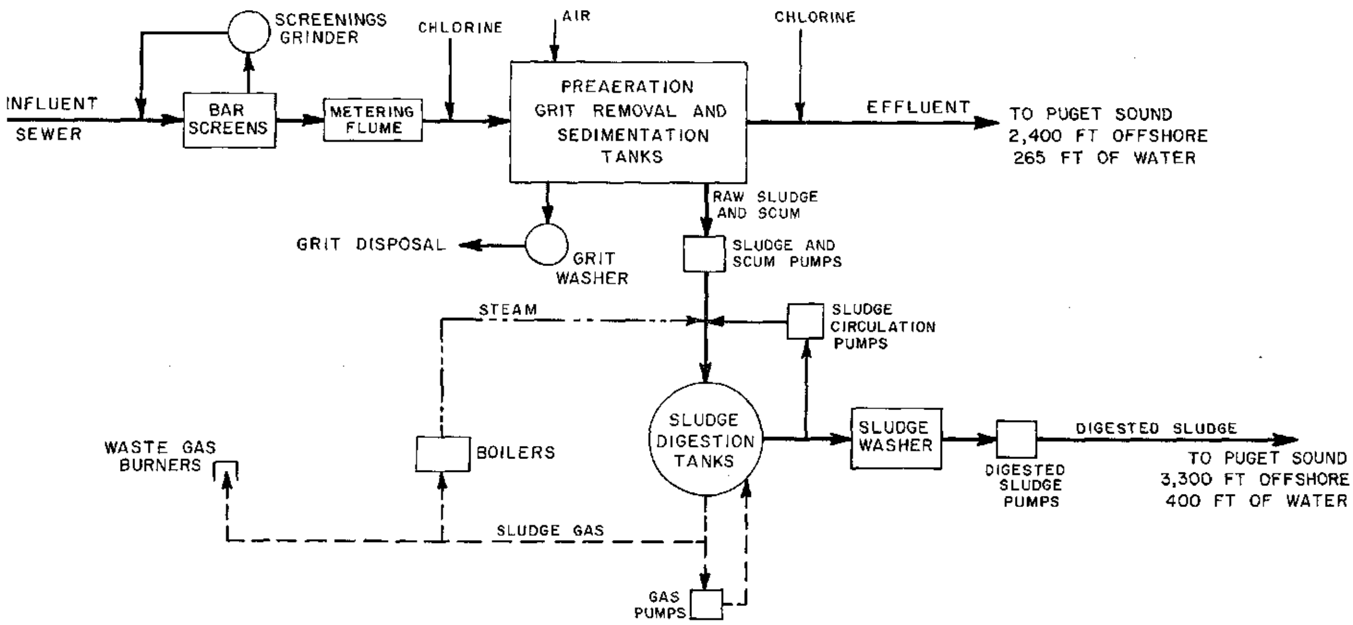

Fig. 16-10. Flow Diagram, Piper Creek Sewage Treatment Plant

pretreatment, preaeration and grit removal, sedimentation, separate sludge digestion, sludge disposal, and effluent chlorination. Approximately three acres of land will be required for the plant site. A schematic flow diagram is shown in Fig. 16-10, and design factors are given in Table 16-10.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 2,400 feet from shore at a depth of 265 feet. This sewer will consist of a land section comprising 2,000 feet of existing 27-inch pipe and a submarine section comprising 2,400 feet of 33-inch reinforced concrete pipe.

Boeing Creek Subarea, North Puget Sound Sewerage Area. Construction of a sewage treatment plant, submarine outfall and trunk sewers to serve the Ronald and Highlands area is scheduled under the Stage I program. Because of financial considerations, construction of the trunk sewer and pumping station to serve Richmond Beach is deferred to Stage II. Due, however, to the fact that raw sewage is presently discharged to Puget Sound at Richmond Beach, construction of facilities required to intercept this discharge should be undertaken at the earliest practicable date.

A primary type treatment plant will be required to serve the Boeing Creek subarea and will have a capacity of 2.6 mgd. An analysis of the predicted average sewage flows (Table 16-9) indicates that initial flows will be 1.0 mgd, or about 40 per cent of ultimate. By 1975, the flow will have increased to 1.9 mgd, or 70 per cent of ultimate. It will be necessary, therefore, to provide initially for ultimate needs.

As shown diagrammatically in Fig. 16-11, the recommended plant will contain facilities for pretreatment, preaeration and grit removal, sedimentation, separate sludge digestion, sludge disposal, and effluent chlorination. Approximately three acres will

Table 16-10. Design Factors, Piper Creek Sewage Treatment Plant

Population in thousands	25	Preaeration and grit removal tanks	
Loading		Number	3
		Detention, minutes	30
Average dry weather flow, mgd	3.7	Hydraulic capacity per tank, mgd	6
Peak wet weather flow, mgd	12	Sedimentation tanks	
Present minimum flow, mgd	0.9	Number	3
		Detention, minutes	90
BOD, 1,000 pounds per day	5	Overflow rate, gal. per sq ft per day	1,070
Suspended solids, 1,000 pounds per day	6	Hydraulic capacity per tank, mgd	6
Bar screens		Sludge digestion tanks	
		Number	2
Number	1	Volume per tank, 1,000 cubic feet	5
Clear spacing, inches	0.75	Loading, pounds per cubic foot per day	0.2

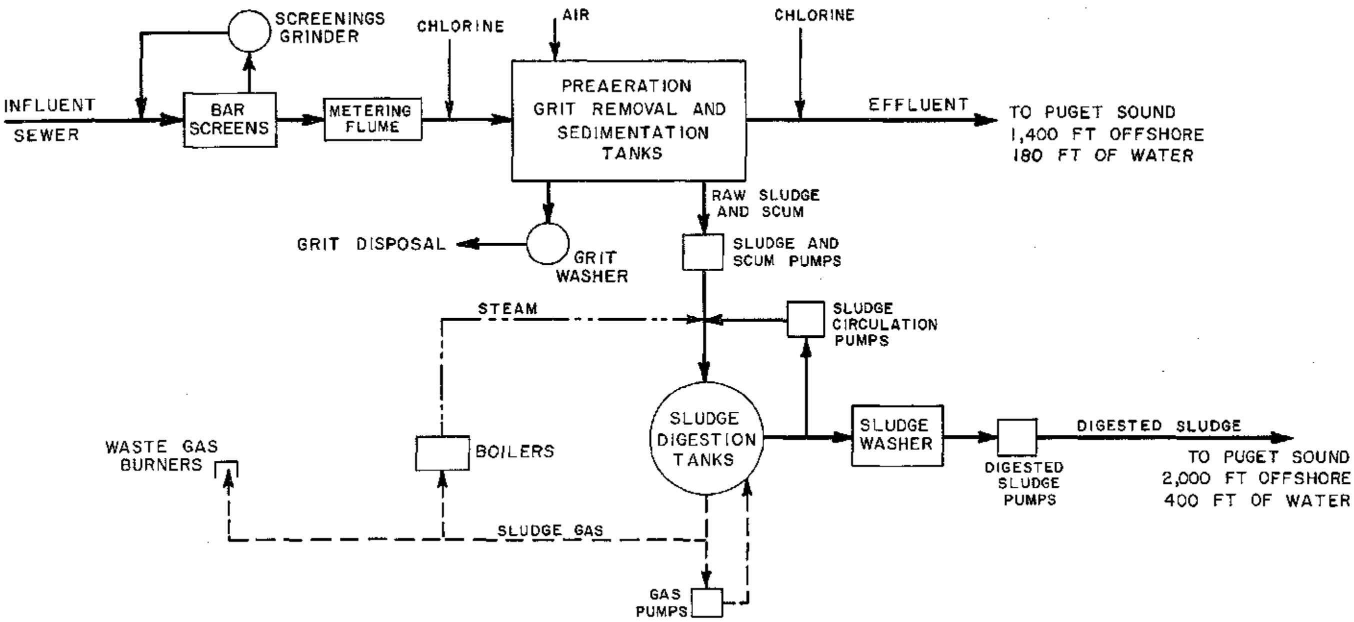

Fig. 16-11. Flow Diagram, Boeing Creek Sewage Treatment Plant

be required for the plant site. Design factors for the plant are presented in Table 6-11.

Treated and chlorinated effluent will be discharged through a gravity outfall sewer terminating 1,400 feet from shore at a depth of 180 feet. This line will consist of a land section comprising 200 feet of 24-inch reinforced concrete pipe and of a submarine section comprising 1,400 feet of 24-inch reinforced concrete pipe.

Temporary Sewage Treatment Plants

Although it is difficult to determine exactly where new temporary treatment plants or other disposal facilities will be required, it is apparent at this time that such facilities will be necessary in the Bothell, Redmond and Mountlake Terrace areas. Similar facilities may be required also to serve isolated developments during the period covered by Stage I construction. Stage I estimates, therefore, include an allowance of $2,000,000 for the construction of temporary treatment plants.

Decisions as to whether temporary plants should be provided during subsequent stages of construction properly should be made as the area develops. It will not be possible until then to determine whether it would be more economical to construct a temporary facility or to obtain service from the central system.

STAGE II CONSTRUCTION, 1970-1980

Stage II construction is scheduled for the years 1970 to 1980. In this period, central sewerage facilities will be extended to the North Lake Washington sewerage area and to the Green River sewerage area. Additionally, sewerage service will be made available to new areas as they develop. For this stage, the total estimated construction costs are $35,417,000 (Table 16-12).

Table 16-11. Design Factors, Boeing Creek Sewage Treatment Plant

Population in thousands	20	Preaeration and grit removal tanks	
Loading		Number	2
Average dry weather flow, mgd	2.6	Detention, minutes	30
Peak wet weather flow, mgd	7.5	Hydraulic capacity per tank, mgd	7.5
Present minimum flow, mgd	0.4	Sedimentation tanks	
BOD, 1,000 pounds per day	4	Number	2
Suspended solids, 1,000 pounds per day	5	Detention, minutes	90
Bar screens		Overflow rate, gallons per square feet per day	1,080
Number	1	Hydraulic capacity per tank, mgd	7.5
Clear spacing, inches	0.75	Sludge digestion tanks	
		Number	2
		Volume per tank, 1,000 cubic feet	4
		Loading, pounds per cubic feet per day	0.20

Table 16-12. Stage II Construction, Recommended Sewerage Facilities

Facility	Construction cost,[a] dollars
Core Plan B[b]	
B-1[c]	1,120,000
STP-B-1[d]	1,638,000
Total, Core Plan B	2,758,000
Core Plan B feeder sewers[e]	
S-2 – S-3	939,000
S-8 – S-10	1,040,000
S-32[f]	3,090,000
S-33[g]	1,280,000
N-1 – N-6	5,245,000
PS-N-1	676,000
PS-N-3 – PS-N-4[h]	1,192,000
Total, feeder sewers	13,462,000
Service sewers	
East Lake Washington[i]	
ELW-1 – ELW-9	735,000
ELW-15 – ELW-16	129,000
ELW-17 – ELW-20	310,000
PS-ELW-1 – PS-ELW-2	304,000
PS-ELW-6	125,000
	1,603,000
North Lake Washington[j]	
NLW-8 – NLW-34	7,406,000
Northwest Lake Washington[j]	
NWW-1 – NWW-5	1,036,000
South Lake Washington[k]	
SLW-3	752,000
SLW-16 – SLW-17	370,000
	1,122,000
Green River[k]	
GR-19	56,000
GR-32 – GR-33	102,000
GR-35 - GR-47	1,802,000
GR-52 – GR-58	918,000
PS-GR-1	139,000
	3,017,000
Southwest Lake Washington[l]	
SWW-1 – SWW-2[m]	174,000
PS-SWW-1[n]	138,000
PS-SWW-3[n]	85,000
	397,000
Total, service sewers	14,581,000
Separate systems	
South Puget Sound[o]	
Redondo Beach subarea	
SPS-2	108,000
SPS-4 – SPS-7	202,000
PS-SPS-1	110,000
STP-SPS-1[p]	709,000
SPS-8	271,000
	1,400,000
Des Moines subarea	
SPS-9	236,000
SPS-12 – SPS-14	185,000
STP-SPS-2[q]	800,000
	1,221,000
Miller Creek subarea	
SPS-18 – SPS-21	424,000
STP-SPS-3[r]	350,000
	774,000
Southwest Suburban subarea	
STP-SPS-4[s]	550,000
SPS-28[t]	56,000
	606,000
West Seattle subarea	
SPS-38[u]	411,000
North Puget Sound[v]	
Boeing Creek subarea	
NPS-8	122,000
PS-NPS-1	82,000
	204,000
Total, separate systems	4,616,000
Total, Stage II construction	35,417,000

See Fig. 16-1 for location of facilities to be constructed under Stage II.

[a] *Includes engineering and contingencies.*

[b] *See Fig. 15-4 for location and Table 15-4 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[c] *Stage II construction: 63 in. at 0.05%, capacity 42 mgd (33% of ultimate). To be paralleled under Stage III construction.*

[d] *Enlargement to capacity of 48 mgd (33.3% of ultimate).*

[e] *See Fig. 15-7 for location and Table 15-11 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[f] *Stage II construction: 60-in. at 0.036 - 0.05%, capacity 31 to 38 mgd (33% of ultimate). To be paralleled under Stage III construction.*

[g] *Stage II construction: 60 in. at 0.063%, capacity 40 mgd (33% of ultimate). To be paralleled under Stage III construction.*

[h] *New stations to replace inadequate existing stations.*

[i] *See Fig. 15-15 for location and Table 15-33 for description of facilities.*

[j] *See Fig. 15-16 for location and Table 15-34 for description of facilities.*

(footnotes continued on next page)

Renton System Sewers

Intercepting sewers for the Renton system to be constructed under the Stage II program include: the Core Plan B interceptor extending south from the treatment plant; the south branch feeder sewer to the city of Kent; and extension of the north branch feeder sewer to serve the Juanita Bay area in the East Lake Washington sewerage area; and the branch feeder sewer to serve the eastern portion of the East Lake Washington sewerage area. Service sewers to be constructed comprise those required to extend sewerage service to developing areas.

Because the rate and extent of population and industrial development in the Green River sewerage area, particularly in the eastern portion, cannot be predicted accurately at the present time, construction of the south Core Plan B interceptor (B1) and the south feeder sewer (S32-33) is planned on an incremental basis. An analysis of predicted flows into these sewers indicates that about one-third the ultimate flow can be expected by 1990. It seems advisable, therefore, to provide initially for one-third the ultimate requirement. When the capacity of the original lines is reached, parallel lines can be constructed. At that time it will be possible to predict more precisely both the rate and extent of future development.

The advisability of incremental construction can be demonstrated by analyzing the estimated costs. If the Core Plan interceptor (B1) were constructed initially for ultimate needs, it would cost $1,504,000 (Table 15-14). On the other hand, if it were constructed for only one-third the ultimate capacity, the total cost would be $1,120,000 (Table 16-12). If that difference in capital outlay, amounting to $384,000, were invested at five per cent interest, it would yield $1,019,000 at the end of 20 years and $1,300,000 at the end of 25 years. The cost of paralleling the interceptor with another sewer having a capacity of two-thirds the ultimate need is estimated at $1,275,000, or about the amount that would accrue at the end of 25 years. Similar values would obtain in the case of the feeder sewers (S32-33).

Renton System Treatment Plant

Enlargement of the Renton sewage treatment plant to a capacity of 48 mgd is scheduled under Stage II construction. New facilities required will include three preaeration and primary sedimentation tanks, three aeration tanks, eight secondary sedimentation tanks, two digesters, and additional sludge lagoon capacity. At the same time, both influent pumping and aeration blower capacity will have to be increased.

West Point System Sewers

Sewers for the West Point system to be constructed under the Stage II program include extension of the north branch feeder sewer to its terminus at the pumping station (PS-N1) in the North Lake Washington sewerage area. With this extension, the entire North Lake Washington sewerage area and the northern portion of the Northwest Lake Washington sewerage area will be tributary to the West Point sewage treatment plant. Service sewers will be constructed as required to extend service to additional developing areas.

West Point System Treatment Plant

Since the West Point sewage treatment plant will be constructed initially with a capacity sufficient for ultimate needs, no additional construction will be required under Stage II.

Independent Systems

With the exception of the Rendondo Beach subarea, construction of facilities to extend service within the various subareas is scheduled under Stage II. In the Redondo Beach subarea, Stage II calls for the construction of a sewerage system, including trunk sewers, treatment works, and a submarine outfall, to serve that area.

STAGE III CONSTRUCTION

Construction of all remaining facilities herein recommended (Fig. 16-1) is provided for under Stage III. This program will be undertaken some time after 1980 as the need develops. Additions and improvements

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Table 16-12 footnotes continued.

[k] *See Fig. 15-17 for location and Table 15-35 for description of facilities.*

[l] *See Fig. 15-18 for location and Table 15-36 for description of facilities.*

[m] *Parallel to existing sewer.*

[n] *Replacement of inadequate existing station.*

[o] *See Fig. 15-12 for location and Table 15-27 for description of facilities; construction for ultimate requirements unless otherwise noted.*

[p] *Initial construction for 1.67 mgd capacity (33% of ultimate).*

[q] *Enlargement to ultimate capacity of 6.5 mgd and provision of secondary treatment facilities.*

[r] *Enlargement to ultimate capacity of 7.5 mgd.*

[s] *Provision of secondary treatment facilities.*

[t] *Extension of submarine outfall.*

[u] *New outfall designed for maximum dry weather sanitary flow (20 mgd).*

[v] *See Fig. 15-13 for location and Table 15-30 for description of facilities.*

Table 16-13. Stage III Construction, Recommended Sewerage Facilities

Facility	Construction costs,[a] dollars
Core Plan B[b]	
B-1[c]	1,275,000
STP-B-1[d]	8,216,000
B-33[e]	318,000
Total, Core Plan B	9,809,000
Core Plan B feeder sewers[f]	
S-1	486,000
S-6 – S-7	305,000
S-16	118,000
S-23 – S-31	2,526,000
S-32[g]	4,099,000
S-33[h]	1,609,000
PS-S-1 – PS-S-4	1,669,000
PS-N-2[i]	100,000
Total, feeder sewers	10,912,000
Service sewers	
North Lake Sammamish[j]	
NLS-1 – NLS-43	8,500,000
PS-NLS-1 – PS-NLS-3	335,000
	8,835,000
South Lake Sammamish[k]	
SLS-1 – SLS-24	3,830,000
PS-SLS-1	364,000
	4,194,000
East Lake Washington[k]	
ELW-25 – ELW-26	231,000
ELW-29 – ELW-31[l]	152,000
ELW-38 – ELW-41	374,000
PS-ELW-8 – PS-ELW-10[m]	296,000
PS-ELW-13	70,000
	1,123,000
North Lake Washington[n]	
NLW-1 – NLW-7	1,102,000
Green River[o]	
GR-1 – GR-18	4,365,000
GR-22 – GR-31	1,858,000
GR-34	41,000
GR-48 – GR-51	372,000
	6,636,000
South Lake Washington[o]	
SLW-1 – SLW-2	402,000
SLW-4 – SLW-15	1,571,000
	1,973,000
Lake Union[p]	
LU-19[q]	169,000
Total, service sewers	24,032,000
Separate systems	
South Puget Sound[r]	
Redondo Beach subarea	
SPS-1	78,000
SPS-3	139,000
STP-SPS-1[s]	364,000
	581,000
Des Moines subarea	
SPS-10 – SPS-11	32,000
Total, separate systems	613,000
Total, Stage III construction	45,366,000

See Fig. 16-1 for location of facilities to be constructed under Stage III.

[a] *Includes engineering and contingencies.*

[b] *See Fig. 15-4 for location and Table 15-4 for description of facilities.*

[c] *84 in. at 0.05% (capacity 90 mgd) to parallel sewer constructed under Stage II.*

[d] *Enlargement in 2 increments to ultimate capacity of 143 mgd.*

[e] *Parallel to outfall constructed under Stage I.*

[f] *See Fig. 15-7 for location and Table 15-11 for description of facilities.*

[g] *78 in. at 0.036 - 0.05% (capacity 63 - 75 mgd) to parallel sewer constructed under Stage II.*

[h] *78 in. at 0.063% (capacity 80 mgd) to parallel sewer constructed under Stage II.*

[i] *Enlargement to ultimate capacity.*

[j] *See Fig. 15-14 for location and Table 15-32 for description of facilities.*

[k] *See Fig. 15-15 for location and Table 15-33 for description of facilities.*

[l] *Parallel to existing sewers.*

[m] *New stations to replace inadequate existing stations.*

[n] *See Fig. 15-16 for location and Table 15-34 for description of facilities.*

[o] *See Fig. 15-17 for location and Table 15-35 for description of facilities.*

[p] *See Fig. 15-18 for location and Table 15-36 for description of facilities.*

[q] *Replacement of existing inverted siphon.*

[r] *See Fig. 15-12 for location and Table 15-27 for description of facilities.*

[s] *Enlargement in 2 increments to ultimate capacity of 5.0 mgd.*

called for under Stage III are estimated to cost a total of $45,366,000 (Table 16-13).

SUMMARY OF STAGE CONSTRUCTION COSTS

As a matter of interest, it is desirable in concluding this chapter to total the construction costs for the three stages and thus to emphasize the magnitude of the expenditures which will be required for sewerage service in the years ahead. For the three stages, the estimated costs are $83,215,000 under Stage I, $35,417,000 under Stage II and $45,366,000 under Stage III. In all, therefore, the metropolitan Seattle area is faced with the prospect over the next 40 or 50 years of having to finance an estimated expenditure of $163,998,000 for facilities which will be required not only to protect the waters of Lake Washington but to bring an end to health and other hazards associated with any failure to provide properly for the sewerage needs of the entire community.

Chapter 17

DEVELOPMENT OF STORM DRAINAGE PLANS

Storm drainage is regarded and undertaken as a municipal function in every modern community throughout the world. The extent of its provision is related to and dependent upon many factors, among which are:

1. Intensity and duration of heavy rains.
2. Topography as related to ground slopes and available natural watercourses.
3. Soil conditions, particularly with respect to perviousness.
4. Proportionate area of paved surfaces and roofs.
5. Value of property to be protected against flooding.
6. Ability and willingness of taxpayers to meet the necessary costs.

Storm drainage serves the dual purpose of preventing damage due to flooding and of providing convenience in the use of city streets. It has little public health or esthetic significance, in which respect it is quite different from sanitary sewerage. Storm drainage facilities, moreover, are relatively expensive because the capacity of adequate conduits to service a given area must be many times that of sewers required to collect and convey sewage from the same area.

Despite the relatively high cost of storm drainage, its provision to the maximum possible extent is becoming regarded as a fundamental necessity. While it is difficult in many cases to justify the required expenditure on purely economic grounds, the insistent demands of personal and public comfort and convenience cannot be ignored. Further, the provision of an adequate storm drainage system will minimize storm water inflow to the sanitary sewerage system.

Unlike the disposal of sewage, the disposal of urban storm drainage does not involve a consideration either of treatment requirements or of dilution capacity available in receiving waters. Changes in land use from rural to urban functions may, however, have a significant effect on the drainage characteristics of a watershed and thus have to be taken into account in drainage planning.

Development of urban improvements in a watershed brings about a decrease in the amount of water entering the ground and thereby in the ground water level. This in turn affects both the outflow from springs and the level of spring-fed lakes. During wet weather, urban improvements serve to increase not only the total volume of surface water runoff but the maximum rate of runoff.

USE OF NATURAL WATERCOURSES

Storm water from most of the metropolitan Seattle area is now disposed of in numerous natural watercourses. In general, continued use of these channels will be essential to any project involving the collection, transportation and disposal of storm water runoff.

Use of natural watercourses as storm drainage channels may affect other beneficial uses. For the most part, however, these other uses are of secondary or incidental importance compared to the storm drainage function. In fact, some channels and stream beds may be utilized exclusively for storm water conveyance, even to the extent of converting them to closed conduits.

While impairment of other beneficial uses is inevitable in some cases, it does not necessarily follow that the storm drainage function is inconsistent with continued use of a watercourse for other purposes. This is particularly true in the case of wooded ravines and similar areas where channel improvements for storm drainage purposes can be kept to a minimum consistent with the protection of adjoining and upstream properties. In other words, many natural streams can be used for the conveyance of storm water without detracting from their value as recreational and esthetic assets.

From the standpoint of preliminary planning, it is obvious that the many lakes of the metropolitan area can be utilized as storage or holding basins for storm water runoff. Proper integration of these lakes in an over-all program of drainage improvements will, of course, make it possible to achieve maximum economy in the provision of facilities for downstream conveyance and disposal. By using a holding basin, it is possible to reduce the maximum rate of flow and thus the size of the downstream facilities.

LEGAL ASPECTS OF STORM DRAINAGE

Based upon a long record of court decisions, common law holds that watercourses and natural drainage channels may not be blocked to the detriment of upstream property. Likewise, surface runoff resulting from precipitation on upstream property can be disposed of to such watercourses, even when the rate of discharge is increased as a result of ordinary improvements on the property. Each owner, therefore, must accept surface waters naturally tributary to his

CITY OF KIRKLAND in the Kirkland-Houghton drainage area lies on the east shore of Lake Washington. Because of good ground slopes and short distances to final points of disposal, the cost of providing storm drainage facilities in areas such as this is reasonably low.

property. In turn, he has the right of unimpaired drainage of his own land, and of the improvements thereon, through properties downstream from him. This concept, by extension, applies also to political entities. On the other hand, if surface runoff is diverted from its natural watershed to another watershed, those responsible for the diversion also become responsible for any damage which may thus be caused.

CENTRAL CONTROL OF STORM DRAINAGE FACILITIES

In most instances, natural watershed boundaries transcend political boundaries. When, as a consequence, two or more political entities lie within a common watershed, cooperative action is required in order both to protect watercourses and to prevent damage by storm water runoff of public and private property. Such cooperation would be assured if the administration of all major drainage facilities were made the responsibility of the central agency proposed herein for the administration of major sewerage facilities.

A central agency would serve not only to provide adequate drainage for all areas but would achieve that objective at a cost which, in general, would be lower than if drainage functions were taken over on a piece-meal basis by individual communities or districts. Furthermore, the provision of adequate storm drainage facilities, as the need for them develops, reduces interruption to travel and communication and thereby benefits not only the area being served but the metropolitan area as a whole.

PRELIMINARY DESIGN OF STORM DRAINAGE FACILITIES

In the layout of drainage facilities, precise information regarding local topography and locations of individual drainage areas is required before even preliminary plans of drainage structures can be evolved. Normally such information cannot be obtained until an area has developed to the extent that drainage patterns as modified by street layouts and other factors can be fully evaluated.

Because much of metropolitan Seattle is presently undeveloped, it is virtually impossible at this time to delineate the over-all drainage areas and subareas required for design purposes. It was not considered feasible, therefore, to attempt a layout of drainage facilities for the entire area. Instead, preliminary designs were developed for four typical areas in which drainage patterns could be properly ascertained. These preliminary layouts are presented in subsequent sections of this chapter and illustrate the methods that should be employed in the design of drainage facilities. A study of storm drainage requirements, as related to separation of the present combined systems serving Seattle, is discussed later in Chapter 18.

When and as the design of drainage projects is undertaken, planning should encompass the entire tributary area. Moreover, each drainage area should be subdivided into appropriate subareas and storm water contributions should be calculated accordingly. This procedure was followed in developing preliminary designs for the four typical areas herein considered.

Design Period

Design of storm drainage facilities to serve a particular area should be based on conditions which are expected to prevail at ultimate or saturation development of that area. This in turn calls for the use of runoff coefficients and times of concentration anticipated in the future. While it is evident that only a portion of a drainage system has to be constructed initially, facilities then provided should have a capacity sufficient for ultimate needs. Subsequent additions may be made on a stage or stepwise basis, timed to keep pace with the growth of the tributary area.

Design Criteria

Criteria to be used in the design of storm drainage facilities were presented in Chapter 13. Briefly, design of urban storm drainage systems is based on the rational formula. This is represented by the formula $Q = ciA$ wherein Q is the runoff rate in cubic feet per second, c is a selected coefficient of runoff expressed as the ratio of runoff to rainfall, i is the mean intensity of rainfall expressed in inches per hour, and A is the tributary area expressed in acres. Values for c were based on projected development of the area and were obtained from Table 13-5. Values of i for calculated times of concentration were based on a storm having a recurrence interval of 10 years (Fig. 13-1 and Tables 13-6 and 13-7).

Storm Water Conduits

Three general categories of storm water conduits were used in developing preliminary layouts for the four typical areas. These were reinforced concrete pipe, open concrete lined channels, and open improved earth channels. Selection of the type of conduit to be used was based on a determination of the expected extent and type of areal development, modified to some degree by economic considerations.

Reinforced concrete pipe was used in areas where a high degree of development, either residential, commercial or industrial, is to be expected. In such areas, closed conduits offer definite advantages with respect to maintenance, safety, community appearance, space requirements, and general convenience.

Concrete lined channels were used in areas where soil conditions are such that scouring would occur at the required velocities. They were used also where the design flow is such that provision of pipe conduits would be economically unsound. Fencing of the channels was provided to the extent deemed necessary for public safety.

Improved earth channels were used in areas where development can be expected to be relatively light, where scouring velocities would not develop, and where the flows are such as to preclude the use of pipe conduits. In general, this type of conduit involved the use of natural watercourses, improved only to the degree necessary to obtain the desired capacity. In all open channels, whether lined or unlined, reinforced concrete box or pipe culverts were provided at all street and railroad crossings.

Storm Drainage for Local Service Areas

Storm drainage systems herein considered were laid out in each case to serve local tributary areas of not less than 160 acres (Chapter 13). As such, they include only those facilities necessary to establish a basic framework in the four areas selected for preliminary study. Laterals and street inlets will have to be constructed to serve smaller areas and connections will have to be made to the basic network.

Design of storm drainage systems for local service areas is beyond the scope of this report. Consideration should be given, nevertheless, to some of the standards applicable to the provision of lateral drains and street inlets.

Use of Street Gutters for Conveying Storm Flows. It is obvious that maximum economy in the design of a storm drainage system can be attained only if street gutters are utilized to the fullest possible extent for the collection and conveyance of the storm runoff. Only thus can the lengths and costs of necessary underground drains be reduced to a minimum.

Street gutters can be utilized to within about one inch of the tops of their curbs, provided pedestrian traffic is not heavy nor the velocity of flow is so great as to be a potential hazard to public safety. In areas where gutters are to be used, roof leaders may discharge either directly onto the ground or through a pipe laid under the sidewalk and curb into the gutter. For design purposes, the extent of gutter use should be determined for each particular street.

Storm Drain Laterals for Foundation Drains. In areas where the local governing body accepts responsibility for the gravity collection of water from foundation drains, storm drain laterals must be provided together with sanitary sewer laterals. Except in areas served by existing combined sewers, design of the sewerage system proposed for the metropolitan area is such that no roof leader or foundation drain connections can be tolerated. Obviously, therefore, storm water from these sources will have to be collected in separate storm drain laterals.

Street Inlets. Street inlets should be provided where necessary to pick up gutter water. In general, good practice requires that flow in gutters be picked up at intervals of about 1,200 feet. In flat areas, the spacing may have to be somewhat less, while that in steep areas may be considerably greater. In commercial and downtown business areas at main street intersections, public convenience requires that very little water be permitted to flow in gutters. Hence, street inlets should be provided at each intersection. Usually four, or sometimes eight, inlets are required at such locations.

Street inlets should be located on the "return", that is, at the beginning of the curve of the corner curb. This eliminates the high curb which results when the inlet is placed at the midpoint of the curve. By using two inlets or a cross curb drain, the entire pedestrian crossing area can be kept free of water. Common practice calls for one inlet in residential areas and two inlets in downtown areas.

Design of street inlets should be such as to facilitate the entrance of water without clogging. A satisfactory design provides for grating bars parallel to the gutter. Provision should be made also for a vertical opening along the curb face both to accommodate increased flow and to permit entrance of water if the grating becomes clogged. To serve these purposes, the horizontal grated section should have a minimum area of six square feet, while the vertical curb face opening should be a minimum of three feet long by four inches high.

Street inlets should be depressed a minimum of two inches below the gutter level. Further, they should be connected to the main drainage system by a pipe at least 10 inches in diameter. The bottom of the inlet should be sloped to the invert of the pipe.

Each inlet must be checked for hydraulic capacity by comparing the design flow which is to be picked up with the known capacity of the inlet. Where the expected flow is greater than the capacity of the inlet, suitable modifications must be made. These include provision of a longer curb face opening, use of two inlets, and provision of a depressed gutter in the vicinity of the inlet.

In storm drainage systems properly designed to prevent deposition in the lines, there is no need for catch basins wherein sand and grit are trapped and accumulated. Catch basin maintenance is usually such that the accumulated material is rarely removed, with the result that the basin soon fails to perform its intended function.

STORM DRAINAGE PLANS FOR SELECTED AREAS

Preliminary designs of storm drainage facilities for four selected drainage areas in metropolitan Seattle were made (1) to demonstrate the method of runoff computation and conduit size selection, and (2) to provide a basis for estimating the cost of a drainage program. Selection of the study areas was based on several factors, among which are:

1. Different problems with respect to drainage design.
2. Similarity to other drainage areas in metropolitan Seattle.
3. Present development to a degree that design factors can be properly ascertained.

Since topography governs the general route, as well as the point of discharge of storm drainage, design alternatives are limited generally to minor deviations in routes and, in some cases, to a choice between an open channel or a closed conduit. In a few instances, the situation with respect to topography is such that it is possible to divert runoff from one drainage area to another. Comparative costs of such alternatives are discussed later in connection with the individual projects to which they apply.

Kirkland - Houghton Area

Occupying a total of 4,510 acres, the Kirkland-Houghton drainage area (Fig. 17-1) lies along the eastern shore of Lake Washington and extends from Juanita Bay on the north to Yarrow Bay on the south. Included within its boundaries are the cities of Kirkland and Houghton. Dense development has taken place in parts of the area and is expected to spread eventually over the entire area.

The Kirkland-Houghton drainage area comprises a number of small subareas, each draining individually to Lake Washington. Surface slopes, which are generally from east to west, and distances to points of disposal are such as to enable effective and low-cost drainage with a minimum of local complications.

All major storm water conduits serving the area follow natural watercourses or grades. Except for possible minor changes during final design, no alternatives are apparent for the suggested routing of storm drains. Because of present and anticipated future developments, pipe drains are mandatory in all but the northernmost subarea. In the latter, some sec-

Table 17-1. Description and Estimated Construction Costs, Drainage Plans KH-A and KH-B, Kirkland – Houghton Area

Number	Design flow, cfs	Description	Construction cost, dollars
Plan KH-A			
KH-1	115	2,200 ft of 30-in. RC at 7.1%, dry, includes repaving and railroad and highway crossings	52,000
KH-2	181	2,100 ft of 42-in. RC at 3.0%, dry, includes railroad and highway crossings	64,000
KH-3	332	3,500 ft of 54-in. RC at 2.7%, dry to moderately wet, includes railroad and highway crossings	132,000
KH-4	228	2,700 ft of 45-in. RC at 3.2%, dry to moderately wet, includes railroad and highway crossing	82,000
KH-5	122	1,800 ft of 33-in. RC at 5.0%, dry	34,000
KH-6	680	1,700 ft of 78-in. RC at 1.6%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	135,000
KH-7	476	2,300 ft of 66-in. RC at 1.9%, dry to wet	110,000
KH-8	219	1,100 ft of 45-in. RC at 3.0%, dry to moderately wet, includes railroad crossing	30,000
KH-9	299	400 ft of 51-in. RC at 3.0%, dry to moderately wet	13,000
KH-10	72	2,600 ft of 27-in. RC at 5.2%, dry	40,000
KH-11	545	2,600 ft of improved open channel at 0.07%, average cross-sectional area 110 sq ft, includes street crossings	58,000
KH-12	465	2,500 ft of improved open channel at 0.07%, average cross-sectional area 93 sq ft, includes street crossing	16,000
KH-13	398	1,800 ft of improved open channel at 0.55%, average cross-sectional area 40 sq ft, includes street crossing	14,000
KH-14	324	1,400 ft of improved open channel at 0.52%, average cross-sectional area 35 sq ft, includes railroad crossing	17,000
KH-15	268	2,200 ft of improved open channel at 0.53%, average cross-sectional area 30 sq ft, includes street crossing	31,000
KH-16	181	1,000 ft of improved open channel at 0.60%, average cross-sectional area 22 sq ft, includes outlet from Forbes Lake	6,000
KH-17	64	2,600 ft of 33-in. RC at 1.6%, dry to wet	50,000
KH-18	126	2,600 ft of 36-in. RC at 4.0%, wet to difficult wet, includes inlet to Forbes Lake	57,000
KH-19	72	3,100 ft of 30-in. RC at 3.1%, dry	53,000
Total contract cost, Plan KH-A			994,000
Engineering and contingencies, 25 per cent			248,000
Total construction cost, Plan KH-A			1,242,000
Plan KH-B			
KH-1 – KH-10	–	Same as KH-1 – KH-10, Plan KH-A	692,000
KH-11	570	2,600 ft of 90-in. RC at 0.54%, difficult wet, includes sheeting, dewatering and highway crossings	299,000
KH-12	485	2,500 ft of 66-in. RC at 2.0%, includes sheeting, dewatering, and highway crossing	149,000
KH-13	414	1,800 ft of 60-in. RC at 2.4%, dry to wet, includes highway crossing	85,000
KH-14	336	1,400 ft of 57-in. RC at 2.1%, dry to wet, includes railroad crossing	66,000
KH-15	268	2,200 ft of 57-in. RC at 1.5%, dry to wet, includes highway crossing	101,000

Continued on next page

Table 17-1. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
KH-16	181	1,000 ft of 45-in. RC at 2.2%, dry to wet	30,000
KH-17 – KH-19	–	Same as KH-17 – KH-19, Plan KH-A	162,000
Total contract cost, Plan KH-B			1,584,000
Engineering and contingencies, 25 per cent			396,000
Total construction cost, Plan KH-B			1,980,000

See Fig. 17-1 for location of facilities.

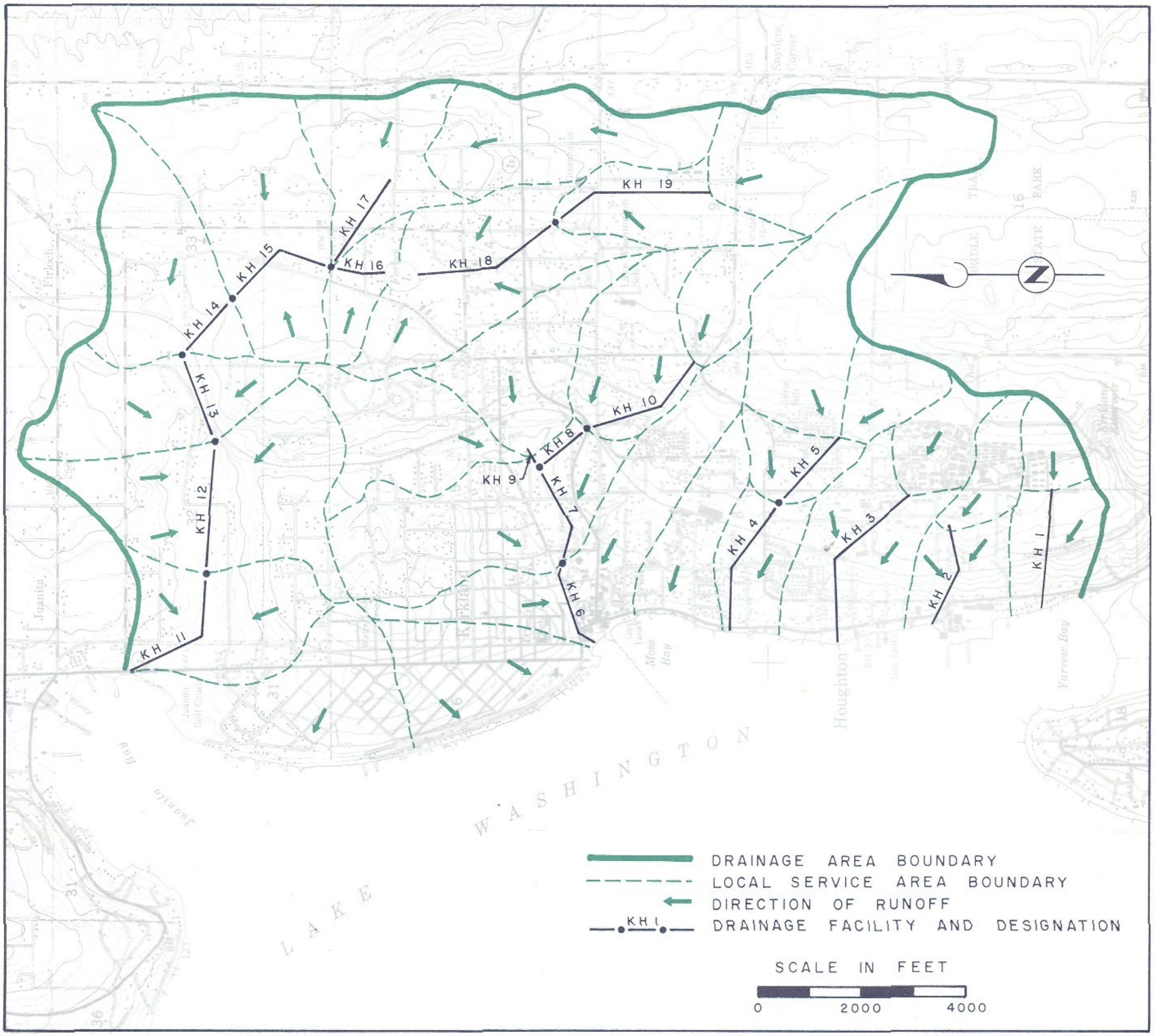


Fig. 17-1. Storm Drainage Plans KH-A and KH-B, Kirkland-Houghton Area

tions could be drained by means of unlined earth channels.

Two plans were laid out to determine the relative costs of closed versus open conduits. The first of these plans, designated as Plan KH-A, provides for using improved earth channels to the fullest possible extent. The second, designated as Plan KH-B, is based on the use of reinforced concrete pipe throughout the entire system.

Locations of major storm drainage facilities for the Kirkland-Houghton drainage area are shown in Fig. 17-1. Descriptions and estimated construction costs of facilities required under each of the two plans are given in Table 17-1. For Plan KH-A, the estimated cost is $1,242,000, while that for Plan KH-B amounts to $1,980,000. Based on a total tributary area of 4,510 acres, the cost of providing trunk storm drainage facilities in areas similar to Kirkland-Houghton will vary between $270 and $440 per acre, depending on the extent to which open channels can be used.

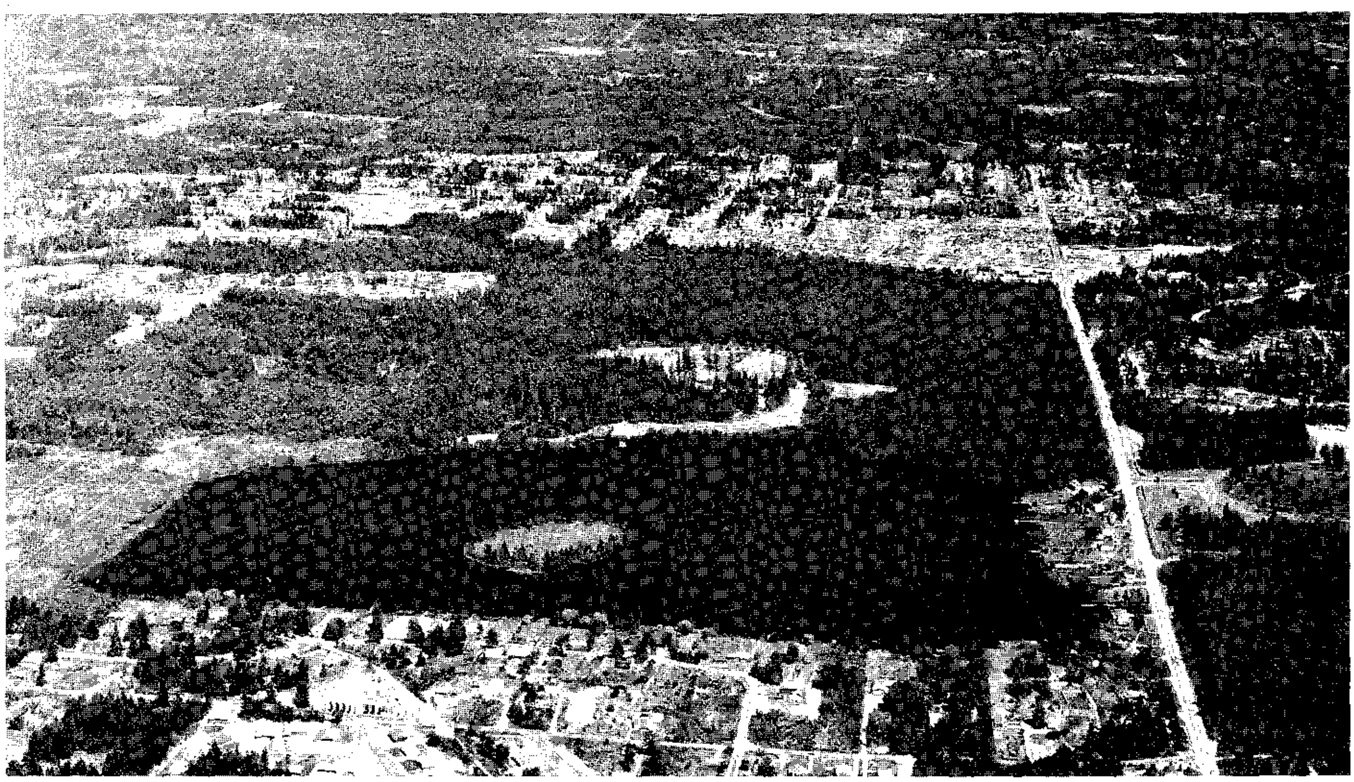
Mountlake Terrace Area

This moderately well developed area contains 7,610 acres, of which 4,550 acres are in Snohomish County. Residential communities are growing rapidly and are expected to continue developing until the area is fully occupied.

Topographically, the Mountlake Terrace area consists of two drainage basins, Lyon Creek and McAleer Creek (Fig. 17-2). Lyon Creek basin, containing a total of 2,510 acres, slopes steeply from north to south and is drained by Lyon Creek which discharges to Lake Washington at the southern end of the basin. McAleer Creek basin is drained by McAleer Creek, which originates at Lake Ballinger and also discharges to Lake Washington. Lake Ballinger occupies an area of approximately 100 acres. Of the total of 5,100 acres in this basin, 3,300 acres are tributary to Lake Ballinger and the remainder directly to McAleer Creek.

Both of the two creeks flow through relatively steep ravines, the sides of which are occupied throughout much of their length by attractive, well landscaped homes. Along their lower reaches, the two creeks traverse a flat, low-lying area potentially valuable for commercial development. For a distance of about one and one-half miles above their points of discharge, the creeks run a few hundred yards apart and parallel to each other.

In the Mountlake Terrace area, storm water runoff originating in Snohomish County must flow into King County. Drainage facilities to be constructed in King County must be designed, therefore, to accommodate both its own runoff plus that from Snohomish County. Such a project should be the joint responsibility of



LAKE BALLINGER in the Mountlake Terrace drainage area acts as a storm water holding basin, thus reducing downstream flows. Similar lakes in the metropolitan area should be utilized as holding basins to achieve economical design of storm drainage facilities.

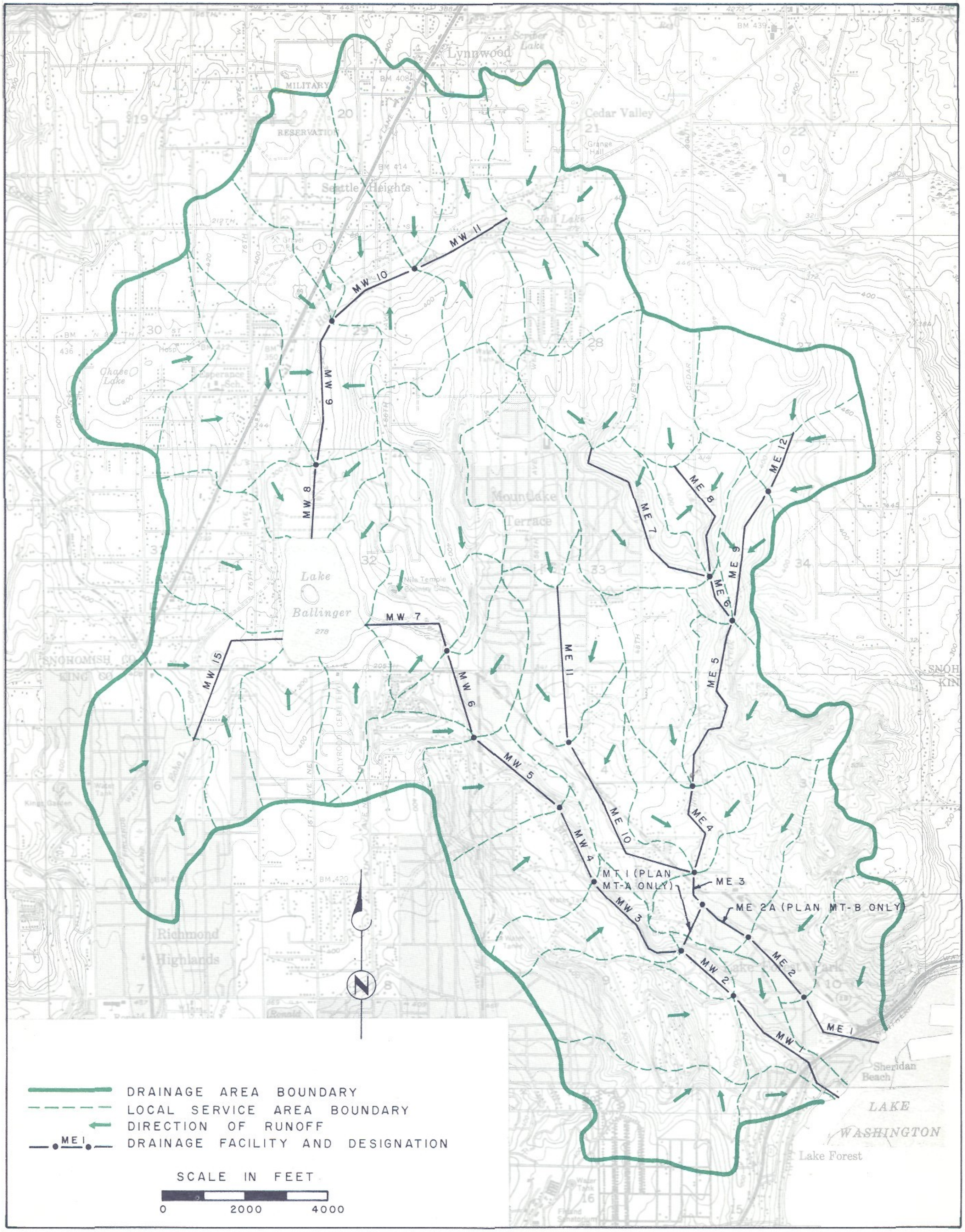


Fig. 17-2. Storm Drainage Plans MT-A and MT-B, Mountain Terrace Area

Table 17-2. Description and Estimated Construction Costs, Drainage Plans MT-A and MT-B, Mountlake Terrace Area

Number	Design flow, cfs	Description	Construction cost, dollars
Plan MT-A			
ME-1	48	2,200 ft of 30-in. RC at 1.8%, difficult wet, includes railroad and highway crossings	65,000
ME-2	13	2,100 ft of 18-in. RC at 1.7%, wet to difficult wet	31,000
ME-3	366	700 ft of improved open channel at 0.09%, average cross-sectional area 75 sq ft, includes street crossings	13,000
ME-4	317	2,100 ft of improved open channel at 0.09%, average cross-sectional area 65 sq ft, includes street crossing	10,000
ME-5	271	4,500 ft of improved open channel at 0.06%, average cross-sectional area 55 sq ft, includes street crossings	28,000
ME-6	127	1,200 ft of improved open channel at 2.85%, average cross-sectional area 9 sq ft, includes street crossing	3,000
ME-7	37	4,600 ft of 27-in. RC at 1.35%, wet, includes street crossing	87,000
ME-8	30	2,800 ft of 24-in. RC at 2.9%, dry to difficult wet, includes street crossings	47,000
ME-9	88	3,200 ft of improved open channel at 1.2%, average cross-sectional area 8 sq ft	5,000
ME-10	64	4,800 ft of 33-in. RC at 1.9%, dry to wet	95,000
ME-11	30	3,600 ft of 21-in. RC at 4.7%, dry, includes repaving and street crossing	45,000
ME-12	48	1,600 ft of 30-in. RC at 1.3%, dry to wet, includes street crossing	29,000
MW-1	570	3,200 ft of improved open channel at 0.06%, average cross-sectional area 115 sq ft, includes street crossings	39,000
MW-2	560	1,700 ft of improved open channel at 0.06%, average cross-sectional area 115 sq ft, includes street crossing	12,000
MT-1	400	1,100 ft of 66-in. RC tunnel at 1.5%, includes allowance of 20% for uncertainties	290,000
MW-3	199	2,900 ft of improved open channel at 0.51%, average cross-sectional area 23 sq ft, includes street crossings	15,000
MW-4	163	2,000 ft of improved open channel at 0.47%, average cross-sectional area 20 sq ft	4,000
MW-5	133	2,800 ft of improved open channel at 0.55%, average cross-sectional area 17 sq ft, includes street crossings	14,000
MW-6	113	1,300 ft of improved open channel at 0.69%, average cross-sectional area 15 sq ft, includes street crossing	7,000
MW-7	105[a]	2,000 ft of 60-in. RC at 0.15%, wet to difficult wet, includes street crossings and outlet structure from Lake Ballinger	93,000
MW-8	326	1,700 ft of 72-in. RC at 0.53%, difficult wet, includes street crossings and inlet structure to Lake Ballinger	122,000
MW-9	206	3,400 ft of 63-in. RC at 0.53%, wet to difficult wet, includes street crossings	154,000
MW-10	128	2,400 ft of 51-in. RC at 0.54%, wet, includes street crossing	85,000
MW-11	64	2,600 ft of 39-in. RC at 0.76%, wet, includes street crossing	65,000
MW-14	73	1,900 ft of 36-in. RC at 1.05%, dry, includes street crossing	36,000
MW-15	41	3,600 ft of 24-in. RC at 3.2%, dry to wet, includes repaving and street crossings	55,000
MW-16	50	3,600 ft of 27-in. RC at 2.6%, dry, includes street crossing	59,000
MW-17	46	2,500 ft of 27-in. RC at 2.2%, wet to difficult wet	45,000
Total contract cost, Plan MT-A			1,553,000
Engineering and contingencies, 25 per cent			388,000
Total construction cost, Plan MT-A			1,941,000

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Table 17-2. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Plan MT-B			
ME-1	455	2,200 ft of 66-in. RC at 1.9%, difficult wet, includes railroad and highway crossings	196,000
ME-2	417	2,100 ft of 66-in. RC at 1.7%, wet to difficult wet	135,000
ME-2A	388	1,300 ft of 66-in. RC at 1.6%, wet	62,000
ME-3 – ME-12	–	Same as ME-3 – ME-12, Plan MT-A	362,000
MW-1	226	3,200 ft of improved open channel at 0.11%, average cross-sectional area 45 sq ft, includes street crossings	38,000
MW-2	205	1,700 ft of improved open channel at 0.11%, average cross-sectional area 45 sq ft, includes street crossing	8,000
MW-3 – MW-17	–	Same as MW-3 – MW-17, Plan MT-A	754,000
Total contract cost, Plan MT-B			1,555,000
Engineering and contingencies, 25 per cent			389,000
Total construction cost, Plan MT-B			1,944,000

See Fig. 17-2 for location of facilities.

[a]Outflow from Lake Ballinger which will act as a holding basin.

both counties, preferably acting through a central agency.

Since the two streams draining the area flow parallel to and close to each other along their lower reaches, diversion of a part of the runoff from the Lyon Creek basin to the McAleer Creek basin would be a feasible alternative to separate disposal in each basin. Two projects were laid out to show their comparative costs. Plan MT-A designates that project wherein part of the flow in the Lyon Creek basin is diverted to the McAleer Creek basin. Under Plan MT-B, all drainage is conveyed and disposed of in each basin separately.

Location of the major storm drainage facilities are shown in Fig. 17-2. Descriptions and estimated construction costs of the facilities required under both plans are given in Table 17-2.

Facilities upstream from the point of diversion of Lyon Creek to McAleer Creek are the same for both plans. Further, both plans propose the use of Lake Ballinger as a storm water holding basin. While no detailed estimates were made of the value of the lake for storm drainage impoundment, this can be readily demonstrated.

As shown in Table 17-2, the flow to be provided for at the lake outlet would be reduced from 326 to 105 cfs by utilizing the lake for storage purposes. This reduction permits the use of a 60-inch line in the first section immediately downstream from the lake (MW 7), whereas a 90-inch line would be required if the total upstream flow were to be accommodated. On that basis, the difference in cost for this section alone would amount to about $100,000. It is apparent, therefore, that continued use of Lake Ballinger for storm drainage impoundment is an economic necessity. Wherever possible, other lakes throughout the metropolitan area should be fully utilized for the same purpose. In the case of Lake Ballinger, storage requirements are such that a water level change of about two feet will obtain under winter storm conditions.

Estimated construction costs of the two plans are almost identical. Plan MT-A is estimated to cost $1,941,000, as compared to $1,944,000 for Plan MT-B. It is obvious, therefore, that the decision as to which of the two plans should be adopted for the Mountlake Terrace area should be based on factors other than cost. These factors would include value of land released for other than storm drainage purposes, magnitude of construction work in congested areas, and reduction of flood potential in business and other districts of high value.

Based on a total tributary area of 7,610 acres, the cost of providing major storm drainage facilities in areas similar to Mountlake Terrace will be about $260 per acre.

Des Moines Area

Situated on the western slope of the metropolitan area, the Des Moines drainage area drains to Puget Sound. This area comprises two drainage basins, both of which empty into the sound within the community of Des Moines (Fig. 17-3). Of the two basins, the northern contains 3,580 acres and drains the greater portion of the Seattle-Tacoma International Airport. Discharge is to the sound through a narrow, steep-sided, heavily wooded natural watercourse. In the southern basin, which contains 1,270 acres, sur-

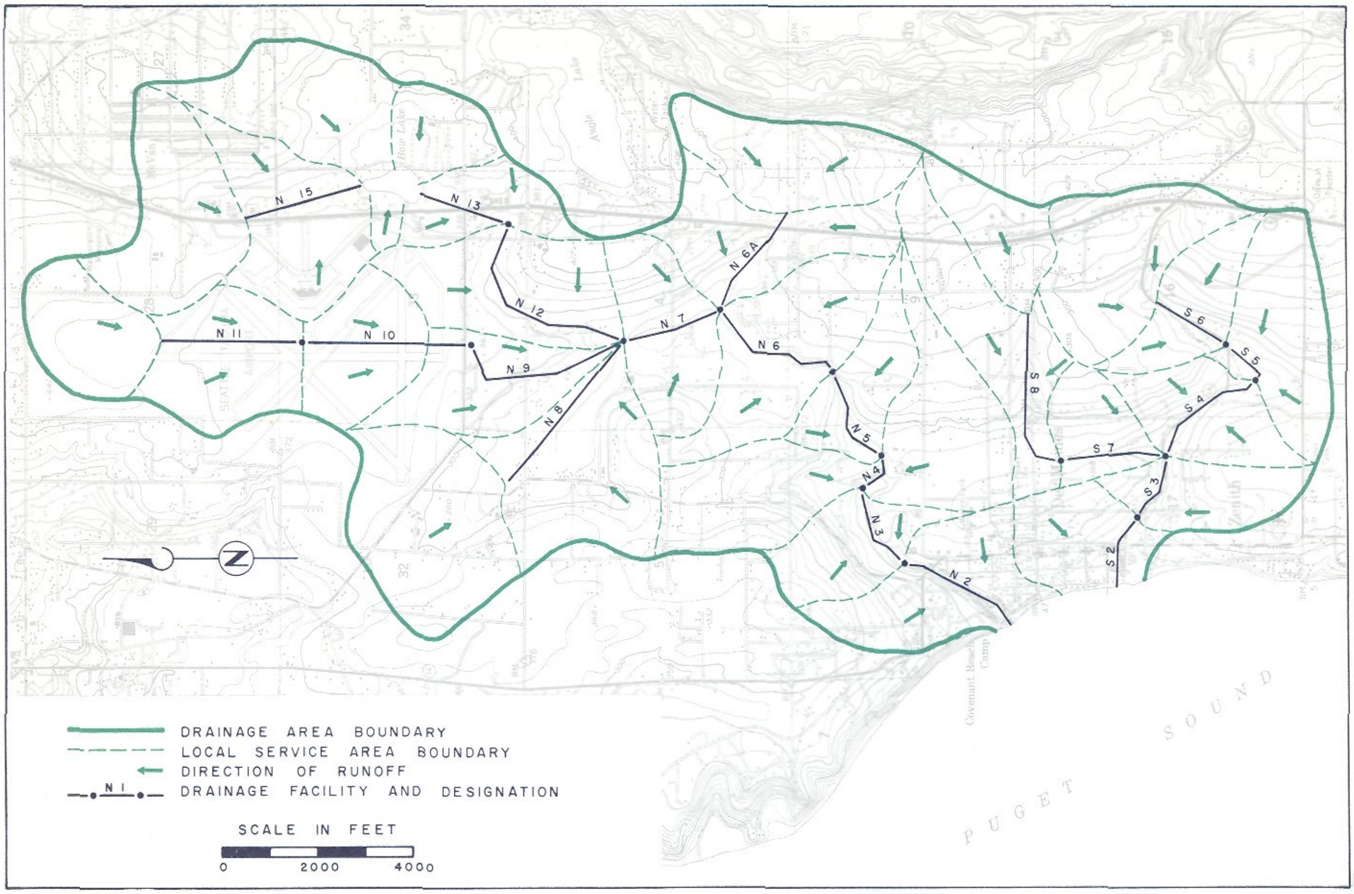

Fig. 17-3. Storm Drainage Plan, Des Moines Area

face slopes and distances to the point of disposal are such that minimum size conduits can be utilized.

Development in the area is presently centered around the community of Des Moines and the airport. Continued residential growth can be expected throughout the area, with some industrial development adjacent to the airport.

The drainage plan proposed for the Des Moines area calls for the use of improved earth channels to the fullest possible extent. Enclosed conduits are to be utilized only where extensive commercial development can be expected, or in areas where elimination of open channels will release land suitable for residential development. No alternatives either as to route or type of facility were considered.

Locations of all major storm drainage facilities required for the Des Moines area are shown in Fig. 17-3. Descriptions and estimated construction costs are given in Table 17-3.

Estimated construction costs amount to a total of $2,655,000. Based on a total tributary area of 4,850 acres, the cost of providing major storm drainage facilities in areas similar to Des Moines will amount to approximately $550 per acre.

Kent Area

Situated in the Green River valley, the Kent drainage area (Fig. 17-4) is bounded on the south and west by Green River, on the east by the ridge separating the Green River and Big Soos Creek drainage basins, and on the north by an arbitrary line along the present north city limit of the city of Kent. Present development is centered around Kent, the only incorporated city in the area. Large scale industrial development can be expected in and near the city. Residential growth undoubtedly will occur in the hilly regions along the eastern boundary of the area. Of the total of 11,480 acres within the drainage area, 6,190 acres lie in the valley and 5,290 acres in the hilly regions to the east.

For a distance of about one and one-half miles to the east, the Kent area is below the flood stage of Green River. For that reason, dikes have been constructed to prevent flooding of adjacent lands. Periodically, however, the river tops the dikes and flooding occurs. While it is expected that flooding may be controlled by the construction of Eagle Gorge dam, the fact still remains that storm water runoff from the floor of the valley cannot be drained to the river by gravity flow. For instance, information obtained from the Seattle office of the U.S. Army Corps of Engineers indicates that the estimated water surface elevation in Green River at Kent was about 40 feet above mean sea level during the flood of 1951. At that time, the measured flow in the river was 19,000 cfs. Even with a controlled maximum flow of 12,000 cfs, the water surface elevation at Kent would be about 35 feet above mean sea level. Ground surface elevations in the valley along the river range from about 40 feet above mean sea level on the south to 22 feet on the north.

Continued development of suburban areas at higher elevations to the east will result not only in increased runoff but in a greatly reduced time of concentration. In consequence, storm water will accumulate rapidly in the low-lying areas. Obviously, therefore, the provision of drainage facilities to prevent flooding these areas by local surface runoff will be a difficult and costly undertaking.

With the valley floor sloping from south to north as well as east to west, several alternatives are pos-

Table 17-3. Description and Estimated Construction Costs, Drainage Plan for Des Moines Area

Number	Design flow, cfs	Description	Construction cost, dollars
S-2	425	1,500 ft of 78-in. RC at 0.8%, difficult wet, includes repaving and street crossings	110,000
S-3	404	1,300 ft of 57-in. RC at 3.1%, wet, includes street crossings	54,000
S-4	176	2,700 ft of 45-in. RC at 2.2%, wet, includes street crossings	87,000
S-5	117	1,000 ft of improved open channel at 0.57%, average cross-sectional area 16 sq ft, includes street crossing	9,000
S-6	74	1,800 ft of improved open channel at 0.70%, average cross-sectional area 11 sq ft	3,000
S-7	118	2,100 ft of 39-in. RC at 2.9%, wet, includes street crossings	58,000
S-8	61	3,600 ft of 27-in. RC at 4.9%, dry to wet, includes repaving and street crossing	64,000
N-2	637	2,300 ft of improved open channel at 0.53%, average cross-sectional area 58 sq ft, includes street crossing	30,000
N-3	589	1,900 ft of improved open channel at 0.55%, average cross-sectional area 54 sq ft	6,000
N-4	502	800 ft of improved open channel at 0.51%, average cross-sectional area 50 sq ft	3,000
N-5	484	2,100 ft of improved open channel at 0.70%, average cross-sectional area 48 sq ft, includes street crossing	14,000
N-6	436	2,800 ft of improved open channel at 0.65%, average cross-sectional area 44 sq ft, includes street crossing	15,000
N-6A	66	2,600 ft of 30-in. RC at 3.3%, dry to moderately wet, includes highway crossing	45,000
N-7	376	1,900 ft of improved open channel at 0.58%, average cross-sectional area 39 sq ft, includes street crossing	16,000
N-8	67	4,100 ft of 42-in. RC at 0.49%, wet	123,000
N-9	156	4,000 ft of 45-in. RC at 1.9%, dry, includes highway crossing	114,000
N-10	80	3,100 ft of 39-in. RC at 2.1%, dry, includes airport runway crossing	83,000
N-11	56	3,100 ft of 39-in. RC at 0.48%, dry, includes airport runway crossing	100,000
N-12	159	4,200 ft of 45-in. RC at 1.8%, dry to wet	114,000
N-13	128	1,900 ft of 51-in RC at 0.53%, wet to difficult wet, includes highway crossing and outlet from Bow Lake	82,000
N-15	59	3,000 ft of 45-in. RC at 0.23%, wet to difficult wet, includes inlet to Bow Lake	124,000
Total contract cost			2,124,000
Engineering and contingencies, 25 per cent			531,000
Total construction cost			2,655,000

See Fig. 17-3 for location of facilities.

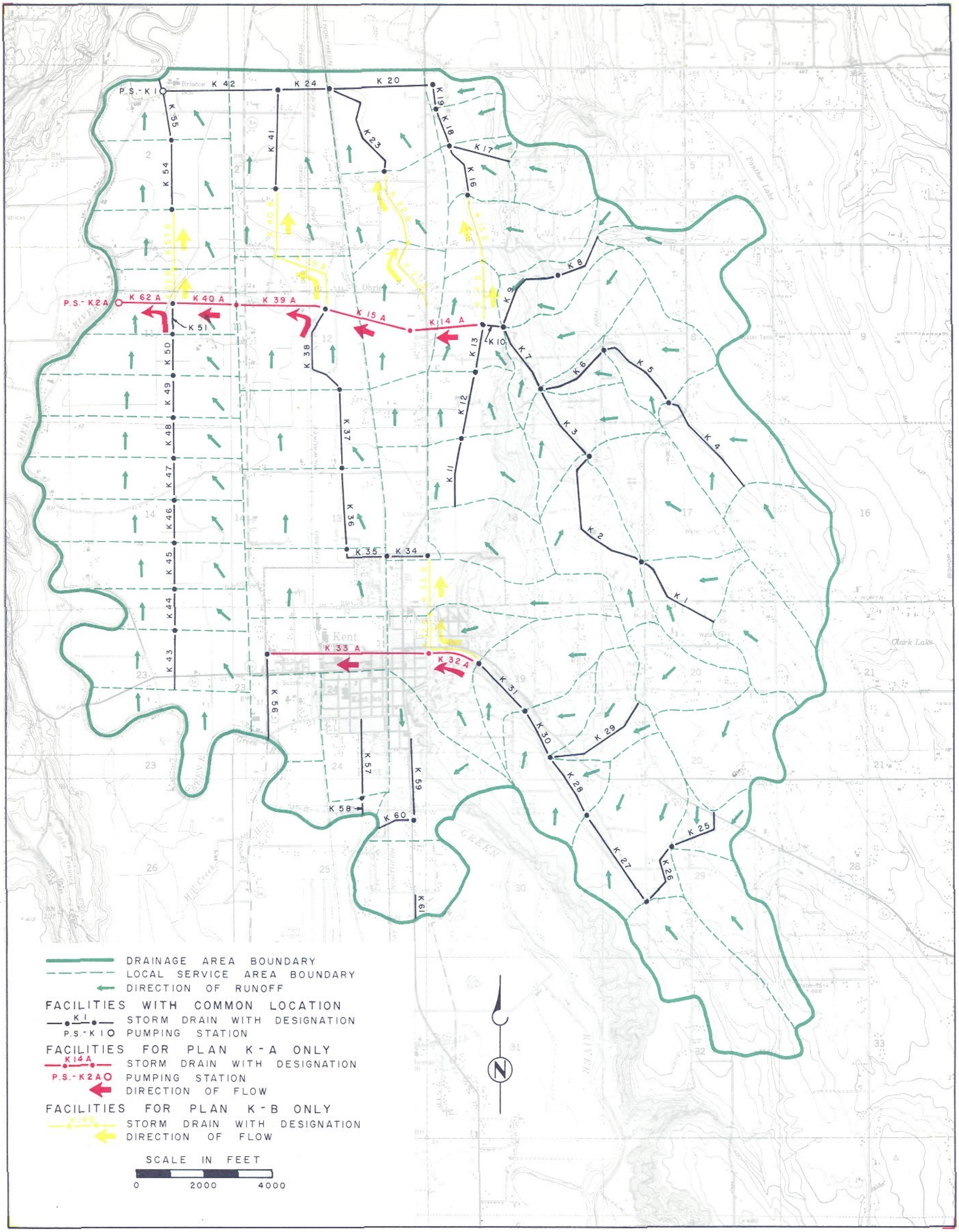

Fig. 17-4. Storm Drainage Plans K-A and K-B, Kent Area

Table 17-4. Description and Estimated Construction Costs, Drainage Plans K-A and K-B, Kent Area

Number	Design flow, cfs	Description	Construction cost, dollars
Plan K-A			
K-1	41	2,800 ft of 30-in. RC at 1.0%, dry, includes street crossings	52,000
K-2	83	4,200 ft of 33-in. RC at 3.7%, dry to wet, includes street crossings	80,000
K-3	168	2,500 ft of 39-in. RC at 5.6%, wet	64,000
K-4	55	3,400 ft of 30-in. RC at 2.1%, dry	58,000
K-5	95	2,500 ft of improved open channel at 0.72%, average cross-sectional area 13 sq ft, includes street crossings	6,000
K-6	123	2,300 ft of improved open channel at 0.72%, average cross-sectional area 16 sq ft	5,000
K-7	306	2,400 ft of improved open channel at 0.54%, average cross-sectional area 34 sq ft, includes street crossings	12,000
K-8	71	1,600 ft of 30-in. RC at 4.4%, dry to wet	27,000
K-9	96	2,300 ft of 30-in. RC at 8.5%, wet	44,000
K-10	402	500 ft of conc lined ditch at 0.4%, average cross-sectional area 42 sq ft	19,000
K-11	59	2,400 ft of 51-in. RC at 0.14%, difficult wet, includes street crossings	89,000
K-12	96	2,000 ft of 60-in. RC at 0.14%, difficult wet, includes street crossings	90,000
K-13	153	1,400 ft of 72-in. RC at 0.14%, difficult wet, includes street crossings	82,000
K-14A	550	2,300 ft of conc lined and fenced ditch at 0.07%, average cross-sectional area 102 sq ft, includes street crossings	166,000
K-15A	587	2,600 ft of conc lined and fenced ditch at 0.08%, average cross-sectional area 102 sq ft, includes street and railroad crossings	188,000
K-34	87	1,100 ft of 57-in. RC at 0.16%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	62,000
K-35	127	1,500 ft of 66-in. RC at 0.16%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	98,000
K-36	189	2,200 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 40 sq ft, includes street crossings	119,000
K-37	244	2,500 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 44 sq ft, includes street crossings	140,000
K-38	286	2,600 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 52 sq ft, includes street crossings	156,000
K-39A	760	2,700 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 150 sq ft, includes street crossings	280,000
K-40A	846	1,600 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 165 sq ft, includes street and railroad crossings	180,000
K-43	55	1,500 ft of 51-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	71,000
K-44	104	1,500 ft of 63-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	86,000
K-45	150	1,400 ft of 72-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	94,000
K-46	191	1,200 ft of 84-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	98,000
K-47	228	1,200 ft of 84-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	98,000
K-48	268	1,200 ft of 90-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	109,000

Continued on next page

Table 17-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
K-49	305	1,200 ft of 96-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	118,000
K-50	340	1,200 ft of 102-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	146,000
K-51	375	900 ft of 102-in. RC at 0.12%, difficult wet, includes sheeting and dewatering	110,000
K-62A	1,160	1,600 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 230 sq ft	194,000
PS-K-2A	1,160	Storm water pumping station	755,000
K-16	47	1,400 ft of 45-in. RC at 0.15%, difficult wet, includes sheeting	98,000
K-17	42	2,000 ft of 24-in. RC at 6.8%, dry to difficult wet	33,000
K-18	98	1,100 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 36 sq ft, includes street crossing	52,000
K-19	116	800 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 43 sq ft, includes street crossing	40,000
K-20	120	3,300 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 43 sq ft, includes street and railroad crossings	156,000
K-23	32	3,200 ft of 45-in. RC at 0.07%, difficult wet	99,000
K-24	159	1,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 55 sq ft, includes street crossings	84,000
K-41	37	3,000 ft of 48-in. RC at 0.067%, difficult wet	100,000
K-42	216	3,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 60 sq ft, includes street crossings	224,000
K-54	70	1,500 ft of 60-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	84,000
K-55	127	1,500 ft of 78-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	110,000
PS-K-1	300	Storm water pumping station	225,000
K-25	40	2,000 ft of 27-in. RC at 2.0%, dry, includes street crossing	34,000
K-26	66	1,700 ft of 30-in. RC at 3.1%, dry	29,000
K-27	177	3,200 ft of 48-in. RC at 1.8%, wet, includes street crossing	123,000
K-28	201	1,800 ft of improved open channel at 0.46%, average cross-sectional area 23 sq ft	4,000
K-29	49	3,000 ft of 24-in. RC at 5.8%, dry to wet, includes street crossing	56,000
K-30	268	1,900 ft of improved open channel at 0.54%, average cross-sectional area 30 sq ft	5,000
K-31	285	2,000 ft of improved open channel at 0.52%, average cross-sectional area 32 sq ft	7,000
K-32A	324	2,800 ft of 78-in. RC at 0.54%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	220,000
K-33A	362	3,600 ft of 96-in. RC at 0.17%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	381,000
K-56	400	2,800 ft of 102-in. RC at 0.11%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	334,000
K-57	37	2,300 ft of 39-in. RC at 0.22%, difficult wet, includes repaving, sheeting and dewatering	79,000

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Table 17-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
K-58	82	800 ft of 42-in. RC at 0.63%, difficult wet, includes sheeting and dewatering	32,000
K-59	70	2,500 ft of 51-in. RC at 0.2%, difficult wet, includes imported backfill, repaving, sheeting, dewatering and railroad crossing	120,000
K-60	164	900 ft of 60-in. RC at 0.45%, difficult wet, includes sheeting and dewatering	52,000
K-61	28	1,000 ft of 36-in. RC at 0.2%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	32,000
Total contract cost, Plan K-A			6,709,000
Engineering and contingencies, 25 per cent			1,677,000
Total construction cost, Plan K-A			8,386,000
Plan K-B			
K-1 – K-13	–	Same as K-1 – K-13, Plan K-A	628,000
K-14B	401	2,100 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft	126,000
K-15B	401	1,700 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossings	126,000
K-16	401	1,400 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossings	134,000
K-17	42	Same as K-17, Plan K-A	33,000
K-18	401	1,100 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossing	87,000
K-19	401	800 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 70 sq ft, includes street crossing	53,000
K-20	410	3,300 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 93 sq ft, includes street and railroad crossings	251,000
K-21B	15	2,900 ft of 30-in. RC at 0.14%, difficult wet, includes street crossing	62,000
K-22-B	35	2,000 ft of 45-in. RC at 0.10%, difficult wet	64,000
K-23	59	3,200 ft of 57-in. RC at 0.07%, difficult wet	127,000
K-24	470	1,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 104 sq ft, includes street crossing	143,000
K-25 – K-31	–	Same as K-25 – K-31, Plan K-A	258,000
K-32B	311	2,600 ft of 78-in. RC at 0.8%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	188,000
K-33-B	352	1,600 ft of 84-in. RC at 0.38%, difficult wet, includes sheeting and dewatering	114,000
K-34	383	1,100 ft of 96-in. RC at 0.18%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	114,000
K-35	408	1,500 ft of 102-in. RC at 0.16%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	177,000
K-36	414	2,200 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 74 sq ft, includes street crossings	163,000
K-37	434	2,500 ft of conc lined and fenced ditch at 0.09%, average cross-sectional area 74 sq ft, includes street crossings	180,000
K-38	460	2,600 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 94 sq ft, includes street crossings	215,000

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Table 17-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
K-39B	468	2,400 ft of conc lined and fenced ditch at 0.05%, average cross-sectional area 94 sq ft, includes street and railroad crossings	206,000
K-40B	483	1,900 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 120 sq ft, includes street crossings	188,000
K-41	510	3,000 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 130 sq ft, includes street crossing	261,000
K-42	972	3,400 ft of conc lined and fenced ditch at 0.03%, average cross-sectional area 177 sq ft, includes street crossings	370,000
K-43	55	1,500 ft of 54-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	75,000
K-44	102	1,500 ft of 72-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	102,000
K-45	147	1,400 ft of 78-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	103,000
K-46	187	1,200 ft of 90-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	109,000
K-47	224	1,200 ft of 90-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	109,000
K-48	262	1,200 ft of 96-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	118,000
K-49	299	1,200 ft of 102-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	134,000
K-50	333	1,200 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	150,000
K-51	366	900 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	113,000
K-52B	375	1,000 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	125,000
K-53B	384	1,800 ft of 108-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	225,000
K-54	412	2,000 ft of 114-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	272,000
K-55	436	1,400 ft of 114-in. RC at 0.08%, difficult wet, includes sheeting and dewatering	190,000
PS-K-1	1,300	Storm water pumping station	845,000
K-56	58	2,800 ft of 51-in. RC at 0.15%, difficult wet, includes imported backfill, repaving, sheeting and dewatering	122,000
K-57 – K-61	–	Same as K-57 – K-61, Plan K-A	315,000
Total contract cost, Plan K-B			7,375,000
Engineering and contingencies, 25 per cent			1,844,000
Total construction cost, Plan K-B			9,219,000

See Fig. 17-4 for location of facilities.

sible with respect to the routing of storm drainage facilities. Preliminary layouts and cost estimates were prepared for two such alternatives. Designated as Plan K-A, the first provides for three major points of discharge to Green River, one at the southern end of the area, the second at the center, and the third at the north end. Under the second alternative, designated as Plan K-B, the greater portion of the drainage would be conveyed to a point in the northern end of the area and discharged to the river. Both plans provide for pumping where necessary and assume a maximum water surface elevation of

20 feet above mean sea level at the pumping station.

Gravity drains serving local drainage areas are also provided under both plans. Design of these drains is based on a controlled water surface elevation in the river of 35 feet above mean sea level.

Locations of all major storm drainage facilities for the Kent area are shown in Fig. 17-4. Descriptions and estimated construction costs are given in Table 17-4. For Plan K-A, the estimated cost is $8,386,000, while that for Plan K-B amounts to $9,219,000. Based on a total tributary area of 11,480 acres, the cost of providing major storm drainage facilities in areas similar to Kent can be expected to vary between $730 and $800 per acre.

SUMMARY OF DRAINAGE COSTS

While it is impossible at this time to estimate accurately the cost of providing major storm drainage facilities throughout the entire metropolitan Seattle area, an approximation based on the unit costs just developed may be of value for planning purposes. To determine the approximate total cost, areas for which drainage facilities are to be provided were divided into five general categories, as follows:

A. An area with favorable surface slopes in which distances to the point of disposal are short and improved earth channels can be used to the maximum extent. Unit cost, $275 per acre.

B. An area with favorable surface slopes in which holding basins and improved earth channels can be utilized. Unit cost, $250 per acre.

C. An area with favorable surface slopes, but which requires a large proportion of enclosed conduits. Unit cost, $450 per acre.

D. An area in which distances to the point of disposal are relatively long. Unit cost, $550 per acre.

E. An area with difficult drainage problems and where pumping may be required. Unit cost, $750 per acre.

In developing the approximate ultimate cost of major storm drainage facilities for the entire metropolitan area, a portion of the city of Seattle was excluded. This portion is presently served by combined sewers. With that exclusion, the area in which storm drainage facilities will ultimately be required amounts to a total of approximately 330,000 acres.

Table 17-5. Approximate Storm Drainage Costs for Ultimate Development in the Metropolitan Seattle Area

Category	Area, acres	Cost per acre,[a] dollars	Total cost,[a] dollars
A	118,000	275	32,450,000
B	33,000	250	8,250,000
C	59,000	450	26,550,000
D	63,000	550	34,650,000
E	57,000	750	42,750,000
Total	330,000	–	144,650,000

[a]Includes engineering and contingencies.

Based upon a study of photographic maps and on information obtained by field inspection, the total area was divided into the five drainage categories outlined above. Estimated construction costs were then developed accordingly and are listed for each category in Table 17-5. As thus determined, the approximate cost of providing major drainage facilities in the metropolitan area for conditions of ultimate or saturation development amounts to a total of $145 million.

It is apparent, of course, that construction of all facilities required at ultimate development need not be undertaken immediately. On the other hand, construction should be undertaken as soon as possible in presently developed areas lacking storm drainage facilities and in other areas where development is limited due to inadequate drainage. This program will involve construction within the next ten years of approximately one-fourth to one-third of the system required for ultimate development. Thereafter, construction can be scheduled as the need for additional facilities develops.

Chapter 18

SEPARATION OF COMBINED SEWERS IN SEATTLE

In common with many other cities throughout the country, the city of Seattle is faced with a diversity of problems brought about by the use of combined sewers for the conveyance of sewage and storm water. These problems are manifested in many ways. For example, heavy rains lead to gross overloading of sewers and thus to sewage overflows in streets, gutters, and basements. Furthermore, contamination of waterways occurs during even moderate rains and is brought about by sewage discharges from overflow structures in combined systems. Correction of these conditions can be achieved only by separation of sanitary sewage and storm water in both trunk and local collection systems. The degree to which separation is required depends, of course, not only on the severity of these problems in local areas but on the extent to which they are brought about by sewage and storm water inputs from upstream locations.

EXTENT OF SEPARATION REQUIREMENTS

As set forth in Chapter 15, the most economic means of preventing bacterial contamination in Lake Washington by storm water overflows is that of constructing holding tanks at overflow locations. These tanks would be designed to store excess water during rainfall periods and to discharge the stored water back to intercepting sewers after the rain has stopped falling.

Beneficial uses of other major bodies of water, namely, Lake Washington Ship Canal, Elliott Bay and Duwamish River, are largely commercial in nature. As such they are not affected adversely by storm water overflows from the combined system. On that basis, and with the provision of holding tanks to protect Lake Washington, the separation program in Seattle need be undertaken only to the extent necessary to relieve overloaded trunk and local sewers.

Design Criteria

Design flows for the facilities required to obtain storm water separation were established on the basis of the criteria presented in Chapter 13. Storm water runoff was calculated by means of the rational formula, using a storm recurrence interval of 10 years. Runoff coefficients and times of concentration were based on ultimate development of each of the selected areas. A proportionate runoff coefficient of two-thirds of the total was used for street and yard drainage, while that used for roof drainage was one-third of the total. These values are based on a detailed analysis by the Seattle engineering department of approximately 20 acres in the Laurelhurst area. Maximum rates of sanitary sewage flow were determined by multiplying the average flow by a factor related to the tributary population and by adding an appropriate allowance for infiltration.

Analysis of Existing System

To determine the extent to which separation is required, it is necessary to analyze the capacity of the combined system. Information thus obtained determines whether the system is adequate (1) to perform its intended function, (2) to carry a part of the storm water flow, as well as all the sanitary sewage flow, (3) to carry only the sanitary sewage flow, or (4) to carry only the storm water runoff.

Previous Studies. In 1951, the Seattle engineering department conducted an overload study of the existing system to determine which sewers were incapable of carrying both storm water and sanitary sewage. This study assumed a uniform storm water runoff rate of 15 cfm per acre, which is equivalent on the average to a storm having a recurrence frequency of once every two years. Even on that basis, which represents only a moderate storm, a substantial number of the combined sewers were found to be deficient in storm flow capacity. Further, this condition was not confined to any particular area or locality but was prevalent throughout the city as a whole.

Present Studies. Detailed studies of separation requirements were made in six areas. In general, it was found that none of the existing systems in their entirety and that very few sewers within these systems have a capacity sufficient for the flow resulting from a 10-year storm. Without exception, all of the systems are capable of carrying the peak flow of sanitary sewage. In most instances, the addition of some relief sewers will enable them also to carry a portion of the storm flow.

Type of Separation

Two complete analyses were made and preliminary layouts and cost estimates were prepared accordingly for each of the six areas. The first analysis assumed partial separation whereby (1) a new storm drainage system to accommodate runoff from streets and yards would be constructed, and (2) runoff from roofs and foundation drains, together with sanitary sewage, would continue to go to the existing system. The second analysis assumed complete separation whereby (1) a new storm drainage system of sufficient capacity to accommodate all storm water runoff, including that from roofs and foundation drains, would be constructed, and (2) the existing system would carry sanitary sewage only. In one area, a study was made also of the possibility of providing a new drainage system to carry runoff from streets and roofs and of using the existing system for sanitary sewage and drainage from foundation drains.

Routes of Storm Drains

In general, the new drainage systems were laid out to follow the routes of the existing combined sewers. For this purpose, use was made of maps prepared by the city engineer's office during the 1951 overload studies.

Since it is possible to discharge storm water to the nearest available body of water, economy in construction could be achieved in some instances by rerouting drains on the basis of using minimum distances to points of disposal. This would make it possible both to shorten the lengths of pipe and to use pipe of smaller diameter. With these advantages in mind, a possible rerouting of storm drains was investigated in one of the six areas.

In the layout of local storm drains for partial separation, it was assumed that, to intercept street inlets, such drains would be required to the next to last intersection along each run. If street gutters are utilized to the fullest possible extent, the lengths of local drains required can, of course, be reduced considerably. In the case of complete separation, it was assumed that local storm drains would be required to the last house on each run to intercept roof leaders and foundation drains.

BASIS OF COST ESTIMATES

Estimates of costs for sewers and storm drains required for the two types of separation are based on the data presented in Chapter 13. As a part of these projects, it will be necessary also to reconnect house sewers and storm water catch basins, to change connections at manholes and, in the case of complete separation, to install new house connections for either storm water drains or sanitary sewers. Unit costs for these purposes are as follows:

House sewer reconnections	$ 40.00
Catch basin reconnections	$ 80.00
Manhole connections	$100.00
New house sewers	$300.00

Drains designed to pick up street drainage only (partial separation) were assumed to be laid at minimum depth, allowing about three feet of cover. Drains designed to pick up all storm runoff, including that from roof leaders and foundation drains (complete separation), were assumed to be laid at a depth of 12 feet, which is about the average required to intercept foundation drains. In one area, a system was laid out which would provide essentially complete separation in that runoff from streets and roofs but not from foundation drains would discharge to the storm drains. With the foundation drains eliminated, the storm drains were assumed to be at minimum depth. Sanitary sewers required for relief of the existing system were assumed to be a minimum of 12 feet deep.

Wherever deemed possible, storm drains were assumed to be laid in parking strips adjacent to the streets. Otherwise, the cost estimates include allowances for repaving. These vary from $1.50 per lineal foot for 8-inch pipe to $4.80 per lineal foot for 60-inch pipe. In paved areas where settlement would be objectionable and where excavated material was considered to be unsuitable for backfill, the costs allow for use of imported granular backfill material.

Construction conditions were assumed to range from dry to moderately wet. In sections where conditions are known to be unfavorable, the costs allow for such items as water control and use of sheet piling.

SEPARATION REQUIREMENTS IN SELECTED AREAS

Development of an actual program of storm water separation in the city of Seattle will require a detailed analysis of all component parts of the existing system. In a preliminary survey of the nature here reported, it is not necessary, nor is it possible, due to lack of topographic and other essential information, to make such an analysis. It is necessary, however, to analyze a sufficient number of typical areas to demonstrate the methods to be employed and to provide a basis for preliminary planning and cost estimating. With this information, it is possible also to reach certain conclusions as to the best procedure to follow in undertaking the separation program.

Selection of Areas for Study

As a first step in the separation study, a number of areas throughout the city were selected as having typical separation problems. These areas were selected in consultation with the engineering department of the city and are among those in which serious overloading of combined sewers is presently occurring. As a second step, six of these areas were selected for detailed study and analysis.

Sewer systems in each of the six areas are shown in detail in maps prepared for the overload studies made in 1951. These maps give sewer sizes and, in most cases, sewer lengths and invert elevations. Additional information was obtained from individual sewer cards on file in the city engineer's office. District numbers used herein in discussing the separation problem coincide with the drawing numbers of the maps prepared for the 1951 study.

District 6 - Briarcliff

Situated on the west side of Magnolia immediately south of Fort Lawton, the Briarcliff district encompasses an area of 150 acres and has a predicted ultimate population of 1,600. Drainage is to the west to Puget Sound and ground slopes are generally steep. This district lies in a major slide area, with slides occurring practically every winter. Subsurface drains have been installed to control and prevent slides and are generally connected to the combined system.

Existing trunk sewers consist of one along Perkins Lane and one along West Ray Street, both of which discharge through an outfall to Puget Sound at the foot of West Ray Street. Under the recommended sewerage plan (Chapter 15), a local intercepting sewer will be required to convey the sewage from the present point of discharge to the central sewerage system at Thirty-Second Avenue West. Lateral sewers range in size from 8-inch to 15-inch.

With a few minor exceptions, principally at the extremity of each lateral, the system does not have a capacity sufficient for combined storm water and sanitary sewage flows. It does, however, have ample capacity for peak sanitary flow. With the addition of a relatively small number of relief sewers,

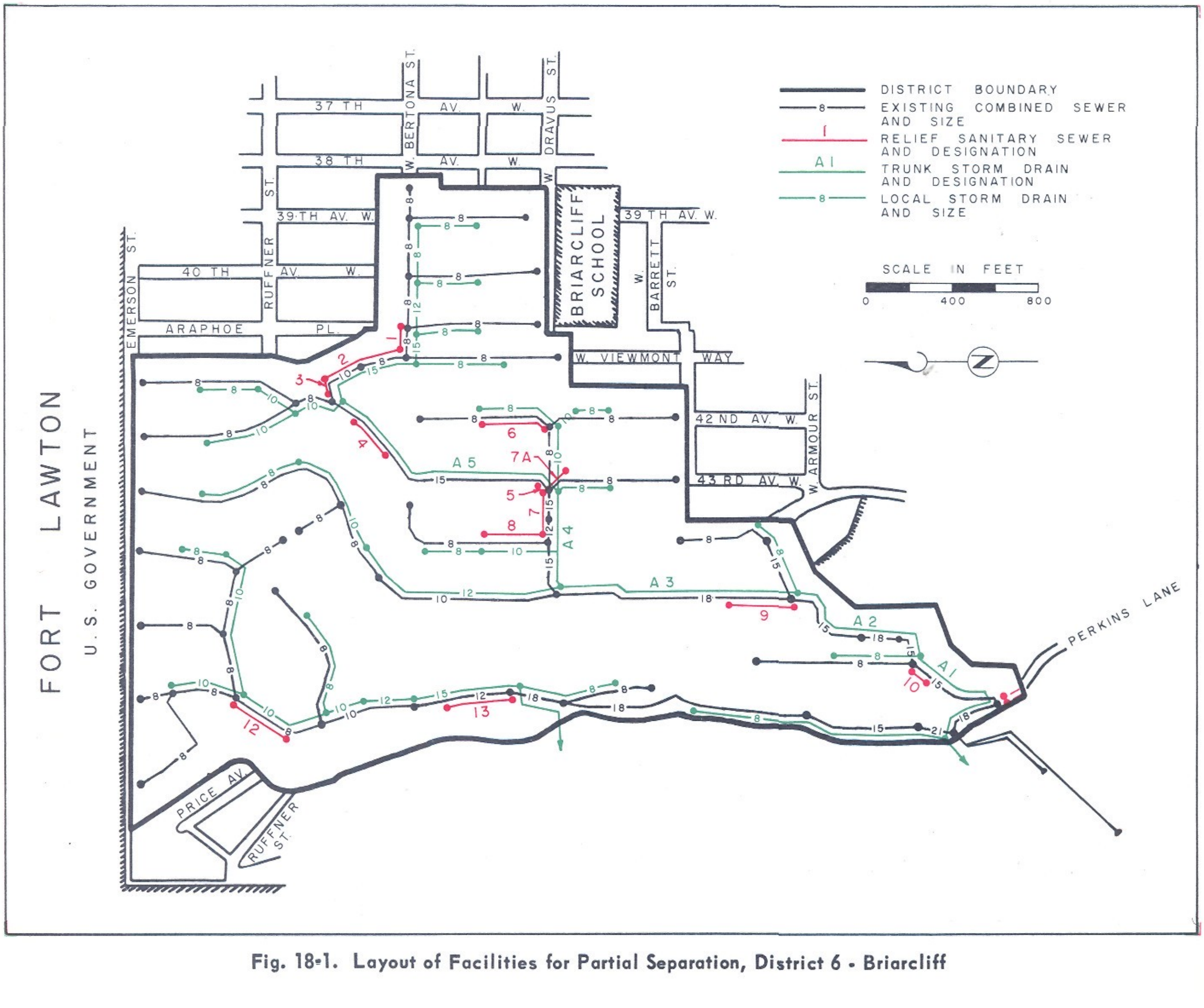

Fig. 18-1. Layout of Facilities for Partial Separation, District 6 - Briarcliff

Table 18-1. Description and Estimated Construction Costs for Separation of District 6 – Briarcliff

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	0.9, 2.7[b]	140 ft of 8-in. conc at 2.1%	1,600
2	1.2, 4.1[b]	410 ft of 8-in. conc at 3.3%	4,700
3	3.2, 4.1[b]	80 ft of 15-in. RC at 0.5%	1,400
4	3.2, 6.0[b]	210 ft of 18-in. RC at 0.2%	4,200
5	2.8, 6.2[b]	70 ft of 15-in. RC at 0.3%	1,300
6	0.6, 1.3[b]	380 ft of 8-in. conc at 0.6%	4,400
7	8.2, 8.2[b]	300 ft of 15-in. RC at 1.5%, required to permit necessary upstream modifications	5,600
7A	0.8	120 ft of 8-in. conc, required to prevent excessive surcharging	1,300
8	0.3, 1.2[b]	310 ft of 8-in. conc at 0.5%	3,600
9	2.0, 13[b]	300 ft of 10-in. conc at 1.1%	3,700
10	5.5, 14[b]	70 ft of 12-in. RC at 2.0%	1,200
11	3.4, 14[b]	30 ft of 10-in. conc at 3.0%	400
12	1.0, 4.5[b]	320 ft of 8-in. conc at 8.4%	3,700
13	1.6, 6.1[b]	260 ft of 8-in. conc at 1.6%	2,900
–	–	Reconnect 30 house connections	1,200
–	–	Reconnect 20 manholes	2,000
Subtotal, relief sanitary sewers			43,200
Trunk storm drains			
A-1	29	640 ft of 24-in. RC at 1.0% minimum slope	10,700
A-2	28	790 ft of 24-in. RC at 2.3% minimum slope	10,600
A-3	25	1,130 ft of 24-in. RC at 1.1% minimum slope	16,300
A-4	18	550 ft of 24-in. RC at 1.8% minimum slope	7,900
A-5	12	1,150 ft of 21-in. RC at 0.6% minimum slope	14,600
Subtotal, trunk storm drains			60,100
Local storm drains			
–	–	7,430 ft of 8-in.	34,400
–	–	4,460 ft of 10-in.	24,600
–	–	2,000 ft of 12-in.	16,000
–	–	600 ft of 15-in.	5,800
–	–	50 intersection crossings, includes catch basin reconnections	18,800
Subtotal, local storm drains			99,600
Total contract cost, partial separation			202,900
Engineering and contingencies, 25 per cent			50,700
Total construction cost, partial separation			253,600
Complete separation[c]			
Trunk storm drains			
A-1	42	640 ft of 30-in. RC at 1.0% minimum slope	13,900

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Table 18-1. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
A-2	41	790 ft of 30-in. RC at 2.3% minimum slope	14,800
A-3	37	1,130 ft of 30-in. RC at 1.1% minimum slope	23,400
A-4	24	550 ft of 30-in. RC at 1.8% minimum slope	11,400
A-5	18	1,150 ft of 30-in. RC at 0.6% minimum slope	23,900
Subtotal, trunk storm drains			87,400
Local storm drains			
–	–	7,430 ft of 8-in.	51,500
–	–	3,460 ft of 10-in.	27,400
–	–	3,620 ft of 12-in.	38,600
–	–	1,800 ft of 15-in.	22,300
–	–	460 ft of 18-in.	6,600
–	–	430 new house connections	129,000
–	–	50 intersection crossings, includes catch basin reconnections	18,800
Subtotal, local storm drains			294,200
Total contract cost, complete separation			381,600
Engineering and contingencies, 25 per cent			95,400
Total construction cost, complete separation			477,000

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-1 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. See Fig. 18-1 for location of trunk storm drains.*

this system could carry the peak sanitary sewage flow plus the storm flow contributed by roof leaders and foundation drains.

Present facilities in the Briarcliff district are shown in Fig. 18-1, together with the relief sewers and storm drains required to effect partial separation. Descriptions and estimated construction costs are given in Table 18-1. The total cost of partial separation amounts to $253,600, or about $1,690 per acre. Of this total, approximately 21 per cent is for relief sanitary sewers, 30 per cent for trunk storm drains, and 49 per cent for local storm drains.

No layout is shown of the facilities required for complete separation. For that purpose, storm drain locations would be substantially the same as those in Fig. 18-1, but local drains would be extended to the end of each existing sewer. No relief sewers would be required because the present system has adequate capacity for the peak flow of sanitary sewage. For complete separation, the estimated cost is $477,000 (Table 18-1), of which about 23 per cent is for trunk storm drains and 77 per cent is for local storm drains, including new house connections.

District 17 - Wedgewood

Occupying a total area of 480 acres, the Wedgewood district lies west of the Sand Point Naval Air Station and north of the Laurelhurst district. Its ultimate population is expected to reach 9,000. Surface slopes are moderate to steep and drain generally to the south.

At present, sewage from this area is discharged through a 36-inch trunk to the North Trunk. Under storm flow conditions, excess flow is diverted by an overflow structure at East 55th Street and 40th Avenue Northeast and is discharged through an outfall line to Union Bay at the foot of 38th Avenue Northeast. It is reported that overflows occur during even relatively light rainfall.

In general, the existing system lacks sufficient capacity to accommodate the flows resulting from a 10-year storm. With a minor amount of relief sewer construction, however, it would have capacity for the peak sanitary flow plus storm water runoff from roof leaders.

District 17 includes the site of the old Cedar Vale

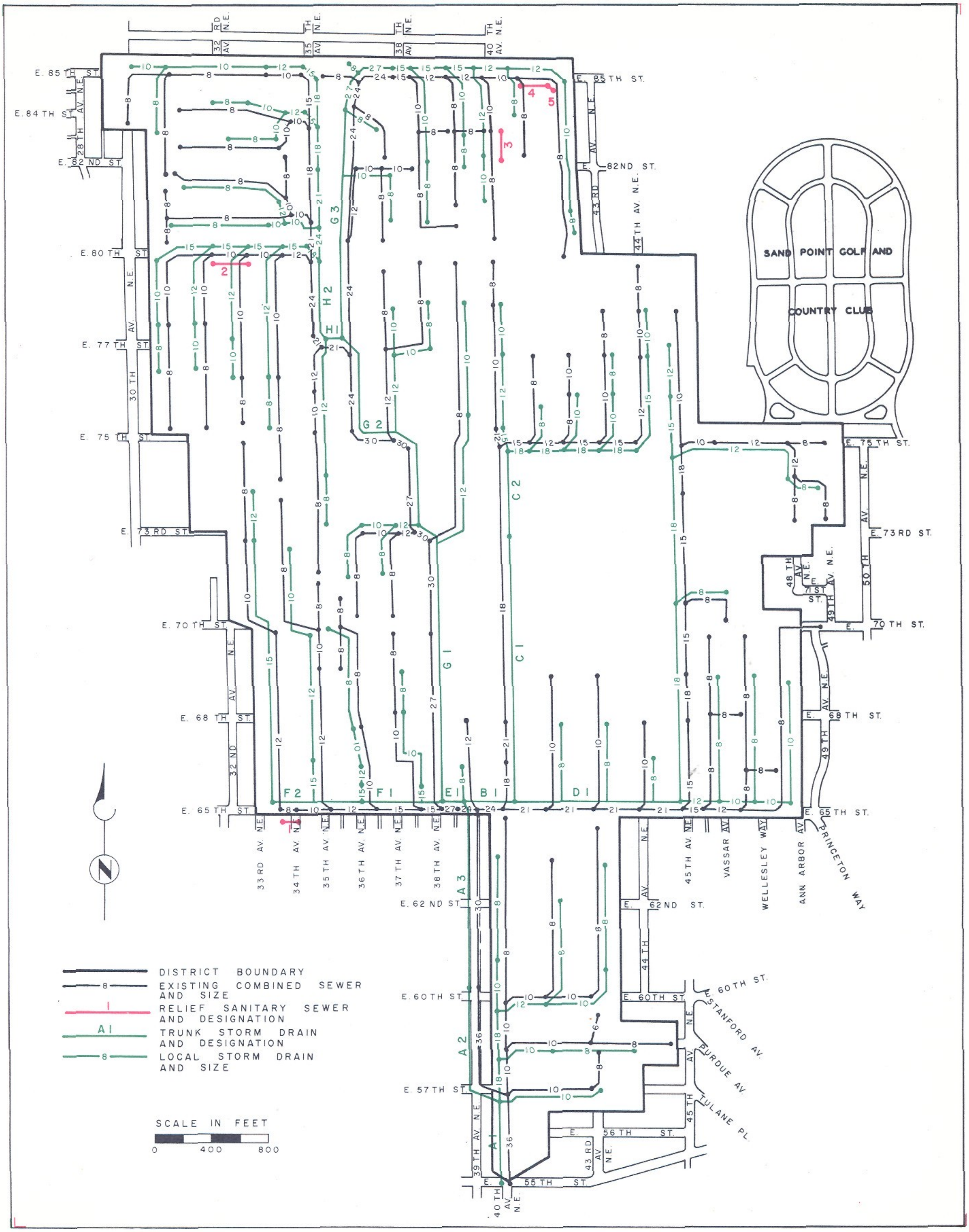

Fig. 18-2. Layout of Facilities for Partial Separation, District 17 - Wedgewood

Table 18-2. Description and Estimated Construction Costs for Separation of District 17 – Wedgewood

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	0.7, 3.6[b]	120 ft of 8-in. conc at 5.8%	800
2	1.4, 4.6[b]	250 ft of 8-in. conc at 2.2%	1,800
3	0.6, 1.8[b]	210 ft of 8-in. conc at 1.0%	1,400
4	0.4, 1.3[b]	240 ft of 8-in. conc at 1.0%	1,500
5	0.2, 1.3[b]	60 ft of 8-in. conc at 2.8%	400
–	–	Reconnect 8 house connections	300
–	–	Reconnect 10 manholes	1,000
Subtotal, relief sanitary sewers			7,200
Trunk storm drains			
A-1	100	660 ft of 39-in. RC at 2.3%	21,000
A-2	98	880 ft of 39-in. RC at 1.9 – 2.8%	28,000
A-3	98	1,220 ft of 33-in. RC at 3.3 – 7.3%	34,300
B-1	49	200 ft of 24-in. RC at 4.9 – 5.4%	3,100
C-1	23 – 26	1,960 ft of 21-in. RC at 3.0 – 5.2%	25,000
C-2	10 – 12	560 ft of 18-in. RC at 0.7 – 5.0%	6,200
D-1	21 – 27	1,300 ft of 21-in. RC at 1.3 – 3.9%	20,000
E-1	55	250 ft of 30-in. RC at 5.0 – 7.7%	5,200
F-1	15 – 18	460 ft of 15-in. RC at 7.7 – 12.1%	5,500
F-2	7 – 12	680 ft of 15-in. RC at 5.8 – 8.8%	8,000
G-1	41 – 44	1,910 ft of 30-in. RC at 1.1 – 4.5%	48,100
G-2	35 – 40	1,550 ft of 30-in. RC at 0.73 – 4.0%	32,400
G-3	13 – 14	1,780 ft of 30-in. RC at 0.1 – 0.4%	31,900
H-1	25	220 ft of 24-in. RC at 3.5%	4,000
H-2	22	640 ft of 24-in. RC at 0.4%	10,800
Subtotal, trunk storm drains			283,500
Local storm drains			
–	–	14,200 ft of 8-in.	69,800
–	–	14,050 ft of 10-in.	83,700
–	–	10,660 ft of 12-in.	89,600
–	–	5,010 ft of 15-in.	49,200
–	–	4,980 ft of 18-in.	56,100
–	–	360 ft of 21-in.	5,200
–	–	280 ft of 24-in.	4,500
–	–	480 ft of 27-in.	8,600
–	–	149 intersection crossings, includes catch basin reconnections	41,000
Subtotal, local storm drains			407,700
Total contract cost, partial separation			698,400
Engineering and contingencies, 25 per cent			174,600
Total construction cost, partial separation			873,000

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Table 18-2. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Complete separation[c]			
Trunk storm drains			
A-1, A-2	155	1,540 ft of 45-in. RC at 1.9 – 2.8%	84,700
A-3	150	1,220 ft of 39-in. RC at 3.3 – 7.3%	60,500
B-1	75	200 ft of 33-in. RC at 4.9 – 5.4%	6,300
C-1	33 – 37	1,960 ft of 24-in. RC at 4.9 – 5.4%	33,800
C-2	16 – 19	560 ft of 21-in. RC at 0.7 – 5.0%	8,700
D-1	42 – 51	1,300 ft of 30-in. RC at 1.3 – 3.9%	36,700
E-1	86	250 ft of 36-in. RC at 5.0 – 7.7%	8,900
F-1, F-2	12 – 25	1,140 ft of 18-in. RC at 5.8 – 12.1%	21,400
G-1, G-2	54 – 67	3,280 ft of 36-in. RC at 1.1 – 4.5%	125,900
G-2, G-3	19 – 54	1,960 ft of 33-in. RC at 0.1 – 2.9%	49,400
H-1, H-2	34 – 40	860 ft of 33-in. RC at 0.4 – 3.5%	30,200
Subtotal, trunk storm drains			466,500
Local storm drains			
–	–	19,100 ft of 8-in.	109,100
–	–	13,000 ft of 10-in.	89,000
–	–	14,400 ft of 12-in.	136,600
–	–	11,300 ft of 15-in.	130,700
–	–	4,600 ft of 18-in.	64,000
–	–	2,600 ft of 21-in.	37,600
–	–	400 ft of 24-in.	6,900
–	–	300 ft of 27-in.	5,800
–	–	440 ft of 33-in.	9,600
–	–	2,200 new house connections	660,000
–	–	149 intersection crossings, includes catch basin reconnections	41,000
Subtotal, local storm drains			1,290,300
Total contract cost, complete separation			1,756,800
Engineering and contingencies, 25 per cent			439,200
Total construction cost, complete separation			2,196,000

[a]*Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-2 for location of facilities.*

[b]*First flow is required relief capacity, second is total design flow.*

[c]*Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. See Fig. 18-2 for location of trunk storm drains.*

housing project east of 40th Avenue Northeast and south of East 75th Street. This site of 45 acres, which formerly was utilized for housing service personnel, is now being subdivided and is more than 50 per cent occupied by new homes. Since detailed layouts of the new sewers being constructed in this area were not available, the entire 45 acres were taken as tributary to the same point in the sewerage system as the old housing project (Fig. 18-2).

Locations of facilities required for partial separation of the Wedgewood district are shown in Fig. 18-2, as are the location and sizes of existing sewers. Descriptions and estimated construction costs are given in Table 18-2. As there listed, the total cost of partial separation amounts to $873,000, or about $1,820 per acre. Of this total, approximately 1 per cent is for relief sanitary sewers, 41 per cent for trunk storm drains, and 58 per cent for local storm drains.

Also given in Table 18-2 are descriptions and estimated construction costs of facilities required for complete separation in the Wedgewood district. For this, the total cost amounts to $2,196,000, or about $4,570 per acre, of which approximately 27 per cent is for trunk storm drains and 73 per cent is for local storm drains, including new house connections. Location of facilities required for complete separation are generally as shown in Fig. 18-2.

District 23 - Southwest Seattle

Lying along the shores of Puget Sound northeast of Lowman Beach Park, the Southwest Seattle district contains 960 acres and is expected to have an ultimate population of 14,500. Drainage in the district is generally to the south and west to Puget Sound through a number of ravines, the most prominent of which runs through the district in a northeasterly direction from Lowman Beach Park.

In this district, the existing combined system consists of one principal trunk in the ravine from Lowman Beach Park and of local sewers connected to that trunk. At present, raw sewage from the area is discharged without treatment to Puget Sound at the foot of Murray Avenue. A pumping station is now under construction, however, which will lift this flow into the waterfront interceptor for conveyance to the Alki Point treatment plant.

Although the principal trunk has sufficient capacity in most sections to accommodate the flow resulting from a 10-year storm, this capacity is not available generally in the local sewers connected to it. Ample capacity is available, however, for the peak sanitary flow. By the addition of some relief sewers, the system would be able also to accommodate the storm flow contributed by roof leaders and foundation drains.

Locations of relief sanitary sewers and of the storm drain additions required to effect partial separation are shown in Fig. 18-3. This figure also shows the locations of the facilities presently in use. Descriptions and estimated costs of the additions required for partial separation are given in Table 18-3.

Under the proposed layout, the principal trunk in the present system would be utilized as a storm drain to the fullest possible extent. There are two reasons for this. First, the trunk is capable of accommodating storm water runoff from streets and yards and, second, the size of the sewer required for the sanitary flow plus the storm flow from roofs is smaller than that required for street drainage.

Estimated construction costs for partial separation in the Southwest Seattle district amount to $1,735,000, or about $1,810 per acre. Of this total, approximately 12 per cent is for relief sanitary sewers, 19 per cent for trunk storm drains, and 69 per cent for local storm drains.

Locations of facilities required for complete separation are substantially the same as those shown in Fig. 18-3. Descriptions and estimated construction costs of these facilities are also given in Table 18-3. Again, the existing principal trunk would be utilized to the fullest possible extent as a storm drain. In this case, construction of relief storm drains would be required in certain sections and some surcharging could be expected of the existing line along Fairmount Avenue. It is estimated that about $85,000 would be saved by using the existing sewer as a storm drain and constructing a new sanitary sewer.

Estimated construction costs for complete separation in the Southwest Seattle district amount to $3,495,800, or about $3,640 per acre (Table 18-3). Of the total cost, approximately 3 per cent is for sanitary sewers, 10 per cent for trunk storm drains, and 87 per cent for local storm drains, including new house connections. In both the partial and complete separation projects, the relatively low percentages of the total cost of trunk drains results from the use of existing sewers for that purpose.

District 29 - South Magnolia

Situated on Puget Sound at the south end of Magnolia, the South Magnolia district occupies an area of 530 acres and has a predicted ultimate population of 5,700. Drainage is variable, with the western portion draining directly to Puget Sound and the remainder draining to the east and west to a flat area culminating in a ravine along 32nd Avenue West. Surface slopes, except in the flat area along 32nd Avenue West, are steep.

The existing combined system is tributary to a principal trunk which is laid along 32nd Avenue West and discharges raw sewage to the sound at the foot of that avenue. Under the sewerage plan recommended in Chapter 15, this discharge will be intercepted and the sewage conveyed to the West Point treatment plant. With a few minor exceptions, the existing system lacks capacity for the combined storm and sanitary flows. It does, however, have capacity for the peak sanitary flow.

Studies were made of two plans for storm water separation. The first, designated as Plan I, provides for the conveyance of storm water to one point of disposal, while the second, Plan II, provides for conveyance to three points of disposal. Under Plan I, storm drain routes would follow the routes of existing sewers. Under Plan II, the routes would be such that storm water would be conveyed the shortest possible distance to the points of disposal.

Under Plan II, a study was made also of an alternative method of separation which would provide for the collection of all runoff from streets and roofs but not for the interception of foundation drains. The latter

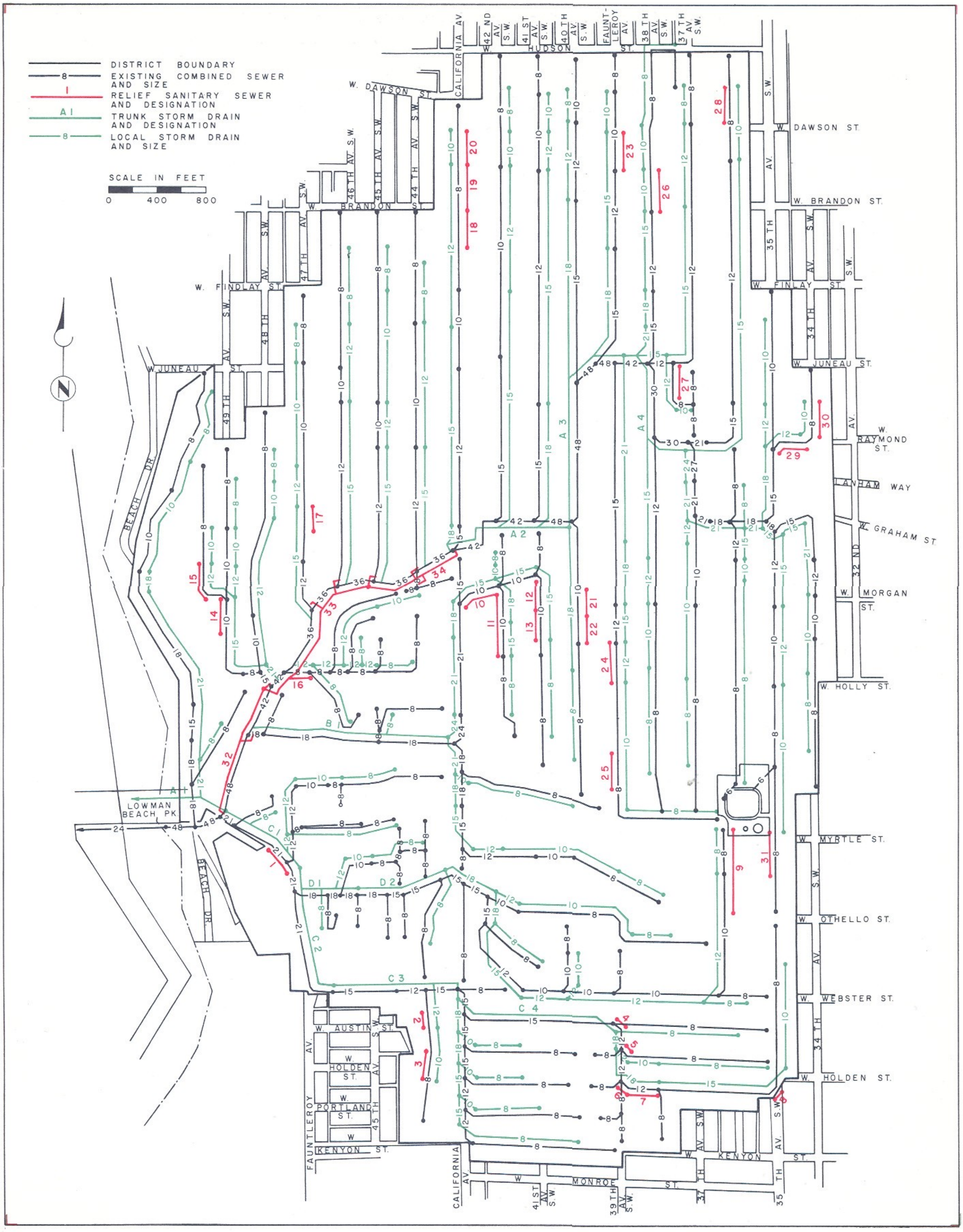

Fig. 18-3. Layout of Facilities for Partial Separation, District 23 - Southwest Seattle

Table 18-3. Description and Estimated Construction Costs for Separation of District 23 – Southwest Seattle

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	15, 30[b]	300 ft of 21-in. RC at 1.0%	6,600
2	0.7, 2.8[b]	100 ft of 8-in. conc at 1.5%	800
3	0.5, 1.7[b]	230 ft of 8-in. conc at 1.0%	1,600
4	1.5, 6.5[b]	80 ft of 8-in. conc at 2.2%	500
5	0.5, 1.7[b]	70 ft of 8-in. conc at 1.0%	500
6	3.9	80 ft of 15-in. RC at 0.4% to replace existing 12-in.	1,100
7	1.5, 3.9[b]	260 ft of 10-in. conc at 0.5%	2,200
8	2.0	90 ft of 15-in. RC at 0.2% to replace existing 8-in.	1,600
9	0.3, 1.1 – 1.8[b]	690 ft of 8-in. conc at 0.4 – 0.6%	4,800
10	1.2, 5.2[b]	280 ft of 8-in. conc at 3.1%	2,600
11	0.4, 1.7[b]	610 ft of 8-in. conc at 0.3 – 0.9%	4,200
12	2.8, 4.1[b]	240 ft of 15-in. RC at 0.3%	3,300
13	0.9, 2.1[b]	240 ft of 10-in. conc at 0.3%	2,000
14	1.0, 3.1[b]	290 ft of 8-in. conc at 0.9%	2,000
15	0.5, 1.5[b]	340 ft of 8-in. conc at 0.7 – 1.0%	2,400
16	2.0, 6.2[b]	220 ft of 8-in. conc at 12.9%	1,200
17	1.4, 3.5[b]	370 ft of 10-in. conc at 0.9%	3,100
18	0.7 – 1.0, 2.7 – 3.0[b]	660 ft of 8-in. conc at 2.4 – 2.8%	7,200
19	1.3, 2.1[b]	380 ft of 10-in. conc at 0.4%	4,900
20	0.5, 1.4[b]	280 ft of 8-in. conc at 0.6%	3,000
21	0.8, 2.9[b]	240 ft of 8-in. conc at 0.9%	1,700
22	1.3, 2.7[b]	240 ft of 10-in. conc at 0.4%	2,000
23	0.3, 1.7[b]	300 ft of 8-in. conc at 0.4%	3,200
24	0.5, 4.3[b]	330 ft of 8-in. conc at 9.7%	2,300
25	0.4, 3.1[b]	290 ft of 8-in. conc at 4.8%	2,000
26	1.0, 2.1[b]	320 ft of 8-in. conc at 0.9%	1,800
27	0.2, 1.3[b]	340 ft of 8-in. conc at 0.8%	2,400
28	0.4, 1.0[b]	280 ft of 8-in. conc at 0.4%	1,900
29	0.4 – 0.8,[b] 1.6	300 ft of 8-in. conc at 0.4 – 0.9%	2,000
30	0.2, 1.0[b]	290 ft of 8-in. conc at 0.4%	2,000
31	0.9, 1.6[b]	360 ft of 10-in. conc at 0.4%	4,600
32	58 – 66	1,100 ft of 30-in. RC at 3.2 – 6.0% to replace existing 42 – 48-in. which is to be used as storm drain	26,000
33	50 – 55	1,200 ft of 27-in. RC at 3.0 – 15% to replace existing 36 – 42-in. which is to be used as storm drain	29,700
34	46 – 47	640 ft of 24-in. RC at 5.6 – 33% to replace existing 36-in. which is to be used as storm drain	13,300
–	–	Reconnect 80 house connections	3,200

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Table 18-3. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
–	–	Reconnect 64 manholes	6,400
–	–	Reconnect 9 lateral sewers	1,800
Subtotal, relief sanitary sewers			161,900
Trunk storm drains			
A-1	220 – 225	600 ft of 54-in. RC at 1.8%	27,600
A-2	86 – 100	1,140 ft of 30-in. RC at 4.5 – 7.0%	21,300
A-3	80 – 83	1,560 ft of 48-in. RC at 0.3 – 0.6%	55,200
A-4	44 – 66	1,200 ft of 33-in. RC at 0.7 – 3.9%	27,300
B-1	32 – 36	1,680 ft of 21-in. RC at 5.6 – 10.5%	22,800
C-1	60	860 ft of 36-in. RC at 1.0 – 20%	21,800
C-2	32	1,170 ft of 27-in. RC at 1.2 – 18%	20,100
C-3	27 – 32	1,280 ft of 18-in. RC at 7.5 – 17%	14,100
C-4	15 – 17	1,260 ft of 18-in. RC at 2.6 – 5.1%	15,300
D-1	25 – 30	520 ft of 24-in. RC at 1.6 – 5.0%	8,300
D-2	23 – 25	710 ft of 18-in. RC at 6.2 – 15%	8,700
–	–	Connections to existing trunk to be utilized as storm drain	2,000
Subtotal, trunk storm drains			244,500
Local storm drains			
–	–	25,200 ft of 8-in.	125,800
–	–	17,600 ft of 10-in.	103,900
–	–	17,700 ft of 12-in.	148,200
–	–	24,200 ft of 15-in.	238,500
–	–	9,800 ft of 18-in.	113,500
–	–	5,700 ft of 21-in.	81,200
–	–	600 ft of 24-in.	9,600
–	–	227 intersection crossings, includes catch basin reconnections	80,900
Subtotal, local storm drains			901,600
Total contract cost, partial separation			1,308,000
Engineering and contingencies, 25 per cent			427,000
Total construction cost, partial separation			1,735,000
Complete separation[c]			
Relief sanitary sewers[d]			
–	3.0 – 4.5	2,940 ft of 15-in. RC at 3.0 – 33% to replace existing 36 – 48-in. which is to be utilized as storm drain	42,600
–	1.9 – 2.4	1,200 ft of 12-in. RC at 3.8 – 7.0% to replace existing 42 – 48-in. which is to be utilized as storm drain	13,100
–	1.5 – 1.9	1,480 ft of 12-in. RC at 0.3 – 0.6% to replace existing 48-in. which is to be utilized as storm drain	16,900
–	–	15 new house connections	4,500
–	–	Reconnect 18 lateral sewers	3,600
Subtotal, relief sanitary sewers			80,700

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Table 18-3. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Trunk storm drains			
A-1	365 – 375	600 ft of 63-in. RC at 1.8%	32,400
–	250 – 260	710 ft of existing 42 – 48-in.	–
–	60, 240[b]	390 ft of 30-in. RC at 3.2% to parallel existing 42-in.	8,300
–	215 – 225	670 ft of existing 42-in.	–
–	190 – 200	540 ft of 45-in. RC at 3.0% to replace existing 36-in.	16,800
–	170 – 190	630 ft of 39-in. RC at 5.6 – 9.2% to replace existing 36-in.	16,700
A-2	135 – 165	1,220 ft of existing 42-in.	–
A-3	100 – 135	1,750 ft of existing 42 – 48-in.	–
A-4	68 – 80	930 ft of 39-in. RC at 0.7 – 3.9%	25,300
B-1	45 – 50	1,680 ft of 27-in. RC at 5.6 – 10.5%	32,500
C-1	100	860 ft of 42-in. RC at 1.0 – 20%	45,300
C-2	55 – 100	1,170 ft of 33-in. RC at 1.2 – 18%	33,400
C-3	50 – 55	1,280 ft of 24-in. RC at 7.5 – 17%	20,300
C-4	28 – 30	1,260 ft of 24-in. RC at 2.6 – 5.1%	21,700
D-1	40 – 45	520 ft of 27-in. RC at 1.6 – 5.0%	10,600
D-2	35	710 ft of 21-in. RC at 6.2 – 15%	11,000
–	–	Connections to existing trunk to be utilized as storm drain	6,000
Subtotal, trunk storm drains			280,300
Local storm drains			
–	–	37,600 ft of 8-in.	226,700
–	–	11,800 ft of 10-in.	84,800
–	–	17,300 ft of 12-in.	170,100
–	–	19,300 ft of 15-in.	224,100
–	–	19,700 ft of 18-in.	264,600
–	–	6,000 ft of 21-in.	98,900
–	–	6,900 ft of 24-in.	125,500
–	–	240 ft of 27-in.	4,600
–	–	340 ft of 30-in.	7,900
–	–	227 intersection crossings, includes catch basin reconnections	80,900
–	–	3,825 new house connections	1,147,500
Subtotal, local storm drains			2,435,600
Total contract cost, complete separation			2,796,600
Engineering and contingencies, 25 per cent			699,200
Total construction cost, complete separation			3,495,800

[a] *Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-3 for location of facilities.*

[b] *First flow is required relief capacity, second is total design flow.*

[c] *Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Routes of trunk storm drains are generally as shown in Fig. 18-3.*

[d] *Routes approximately same as relief sanitary sewers 32-34 and trunk storm drains A2and A3, partial separation.*

would continue to discharge to the existing system, which has ample capacity to take this flow plus the peak sanitary flow. An obvious advantage of such an alternative is that the storm drains could be laid at minimum depth.

Locations of facilities required for partial separation under Plan I are shown in Fig. 18-4, as are the locations and sizes of existing sewers. Under this plan, all storm drains would follow the routes of existing sewers and storm water would be conveyed to a single point of disposal at the foot of 32nd Avenue West. Descriptions and estimated construction costs are given in Table 18-4. As there noted, the total cost amounts to $958,200, or about $1,810 per acre. Of that total, approximately 19 per cent is for relief sanitary sewers, 48 per cent for trunk storm drains and 33 per cent for local storm drains.

Descriptions and estimated costs of facilities required for complete separation under Plan I are given in Table 18-4. It will be seen that the total cost amounts to $1,940,600, of which 32 per cent is for trunk storm drains, and 68 per cent is for local storm drains, including new house connections. Locations of the required facilities would, in general, be as shown in Fig. 18-4.

With three points of storm water disposal, as called for under Plan II, the locations of trunk drains required for separation would be as shown in Fig. 18-5. This figure also shows the locations and sizes of local storm drains required for the separation alternative which calls for the collection of runoff from streets and roofs but not from foundation drains.

Descriptions and estimated construction costs of the facilities required for each of the three alternatives available under Plan II are given in Table 18-4. One alternative involves partial separation, while the other two involve complete separation. For partial separation, the estimated cost amounts to a total of $893,500, or about $1,690 per acre. Of that total, approximately 20 per cent is for relief sanitary sewers, 45 per cent for trunk storm drains, and 35 per cent for local storm drains.

For complete separation under Alternative 1 of Plan II, the estimated cost totals $972,200, or about $1,830 per acre. Approximately 49 per cent of this total is for trunk storm drains and 51 per cent is for local storm drains.

For complete separation under Alternative 2 of Plan II, the estimated cost totals $1,890,000, or about $3,570 per acre. Trunk storm drains account for approximately 29 per cent of the total, while local storm drains, including new house connections, account for 71 per cent.

A summary of the estimated costs for the two plans of separation is presented in Table 18-5. As far as partial separation is concerned, the saving achieved by using three points of disposal rather than a single point is indicated by the lower costs of Plan II. Of more interest, however, is the fact that complete separation could be obtained at a slight additional cost by providing a drainage system designed to take all flow from street gutters and roof leaders. In districts such as South Magnolia, where extensive construction of relief sanitary sewers is required for partial separation, the advantage of such an alternative is that local drains can be extended to critical areas and roof leaders can be connected as required to alleviate overloading of the sanitary system. This advantage is obtained because removal of roof drainage from the sanitary system eliminates the need for relief sanitary sewers.

In areas where the existing system is capable of carrying both sanitary sewage and storm flow runoff from roof leaders, the drainage system may be extended only as required to pick up street inlets. In that manner, construction can be tailored to meet the requirements of any particular area and can be spread over a number of years. A further saving can be achieved by utilizing street gutters to the fullest possible extent for the conveyance of storm water runoff. By so doing, the length of local storm drains can be reduced in areas where only street drainage has to be removed from the existing combined system.

District 33 - Madison Park

Containing a total of 100 acres, the Madison Park district is situated on the shores of Lake Washington and Union Bay and is expected to have an ultimate population of 1,500. Topographically, it is fairly flat, with the surface sloping to Lake Washington on the east and Union Bay on the north.

The existing sewerage system is partly combined and partly separated. In the separated portion, a storm drain on McGilvra Boulevard picks up street drainage only and discharges to Union Bay. This drain is laid about five feet shallower than the sanitary sewer and thus cannot be utilized to intercept foundation drains.

Sewage from both the combined sewers and the sanitary sewers in the separated portion is discharged to a pumping station at 43rd Avenue North and East Lynn Street. At this station, dry weather flows are pumped to a station on East Lee Street, which in turn pumps to the North Trunk sewer. Wet weather flow in excess of the pumping capacity overflows at East Lynn Street.

Although the combined system has ample capacity for the peak flow of sanitary sewage, it is not capable of accommodating the flow resulting from a 10-year storm. Construction of a substantial number of relief sanitary sewers will be required in order to handle the peak sanitary flow plus the

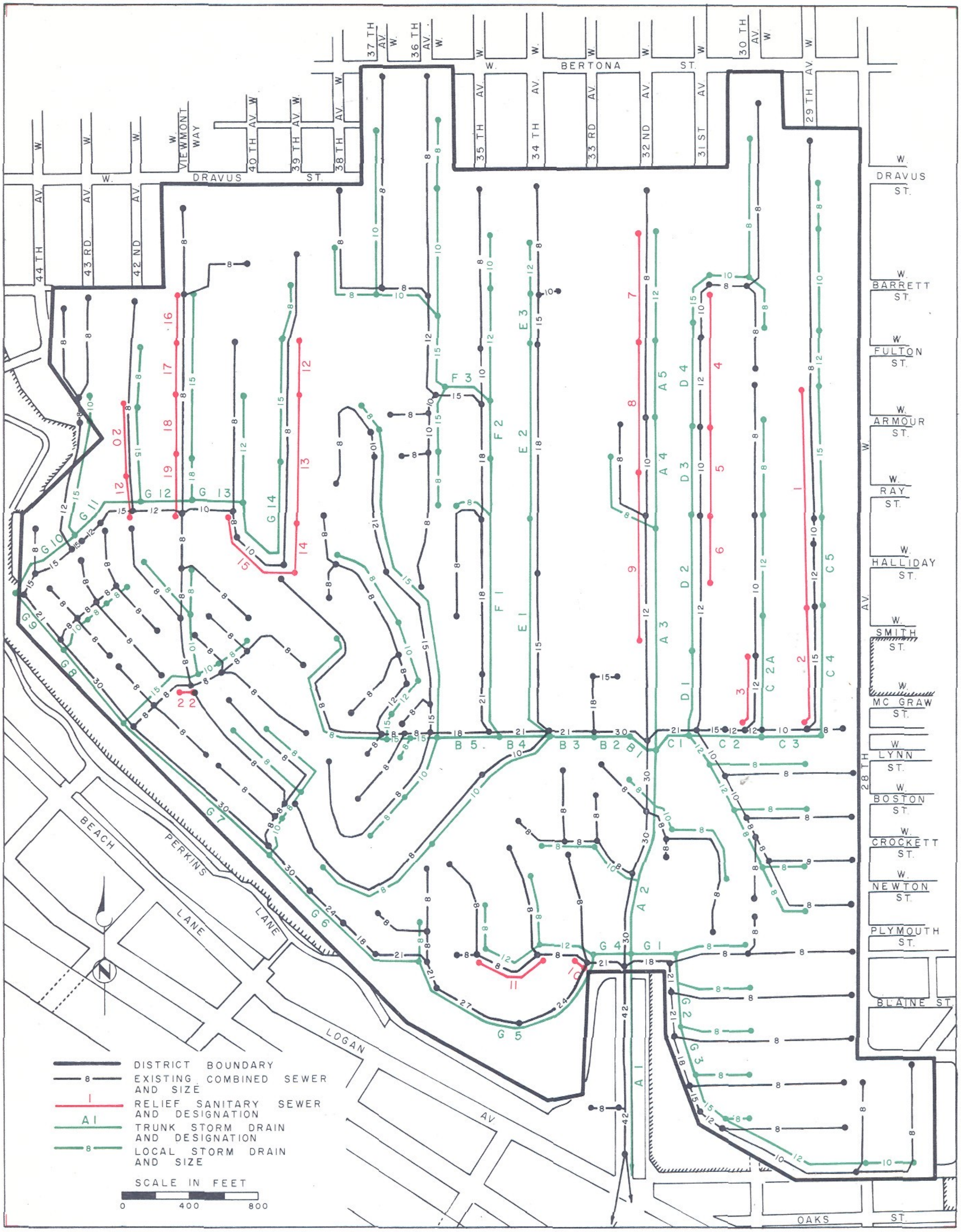

Fig. 18-4. Layout of Facilities for Plan I, Partial Separation, District 29 - South Magnolia

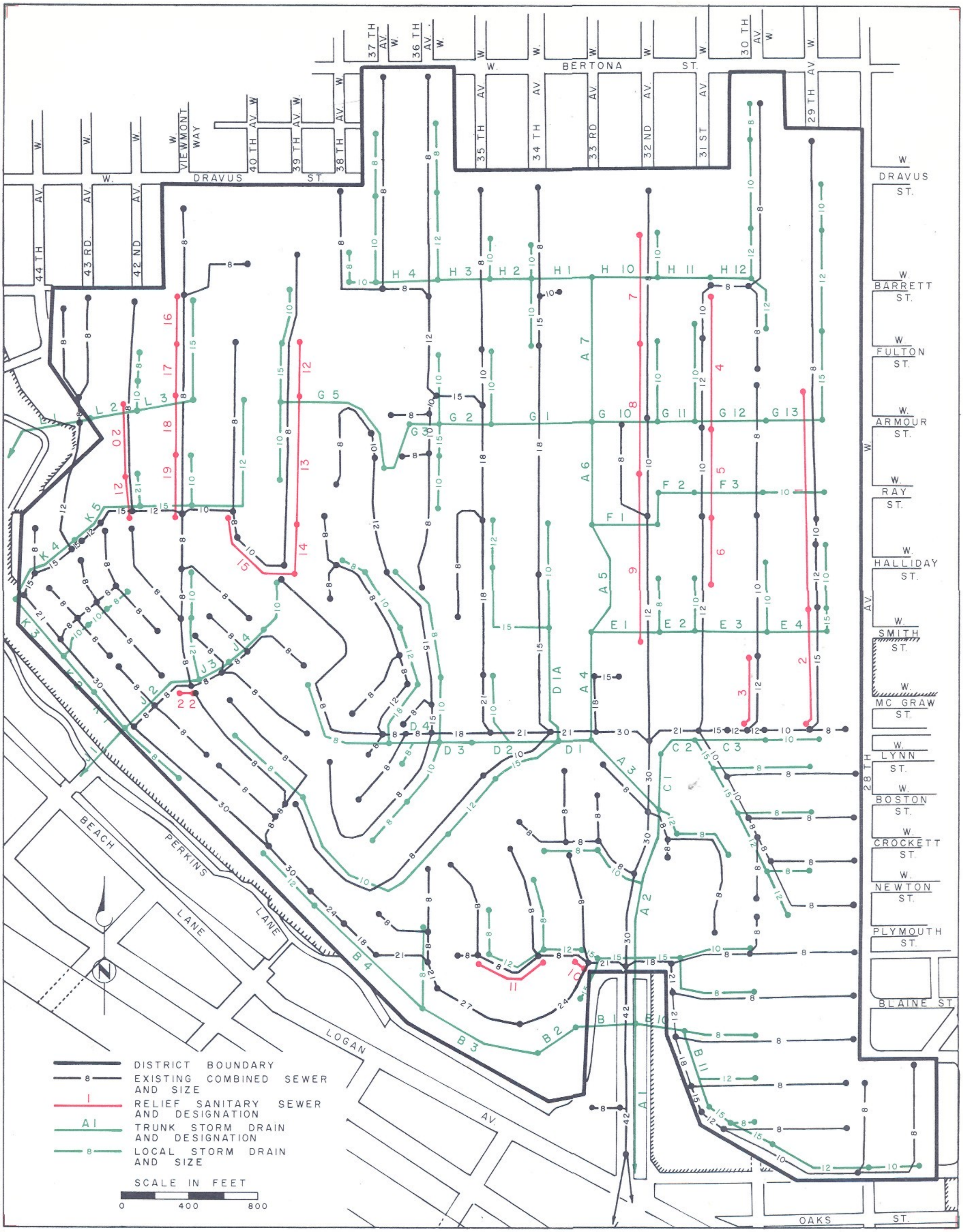

Fig. 18-5. Layout of Facilities Plan II, Complete Separation, Alternative 1, District 29 - South Magnolia

Table 18-4. Description and Estimated Construction Costs for Separation of District 29 – South Magnolia

Number	Design flow, cfs	Description	Construction cost, dollars
Plan I[a]			
Partial separation[b,c]			
Relief sanitary sewers			
1	2.2, 5.3[d]	1,300 ft of 8-in. conc at 3.0%	15,000
2	1.2, 6.2[d]	750 ft of 10-in. conc at 0.6%	9,200
3	1.6, 6.5[d]	500 ft of 10-in. conc at 0.5%	6,200
4	1.2, 4.9[d]	800 ft of 8-in. conc at 1.0%	9,200
5	2.0, 5.1[d]	530 ft of 8-in. conc at 3.0%	6,100
6	1.5, 5.1[d]	370 ft of 8-in. conc at 1.5%	4,300
7	1.2, 2.0[d]	650 ft of 8-in. conc at 2.0%	7,500
8	2.2, 3.5[d]	780 ft of 10-in. conc at 1.0%	9,600
9	1.5, 4.9[d]	1,000 ft of 8-in. conc at 1.5%	11,500
10	1.5, 2.1[d]	100 ft of 8-in. conc at 1.5%	1,200
11	0.8, 1.1[d]	450 ft of 8-in. conc at 0.4%	5,200
12	0.7, 1.6[d]	310 ft of 8-in. conc at 0.5%	3,600
13	1.6, 2.5[d]	750 ft of 10-in. conc at 0.5%	9,200
14	1.9, 2.9[d]	260 ft of 12-in. RC at 0.6%	4,400
15	1.0, 2.9[d]	500 ft of 8-in. conc at 0.8%	5,800
16	1.6, 2.5[d]	340 ft of 8-in. conc at 1.8%	3,800
17	2.1, 3.7[d]	330 ft of 10-in. conc at 1.8%	4,100
18	1.9, 3.9[d]	320 ft of 8-in. conc at 2.8%	3,700
19	3.2, 4.4[d]	320 ft of 12-in. RC at 1.0%	5,400
20	0.7, 1.2[d]	400 ft of 10-in. conc at 0.2%	4,900
21	0.9, 1.7[d]	200 ft of 8-in. conc at 0.5%	2,300
22	1.7, 4.1[d]	100 ft of 8-in. conc at 4.0%	1,200
–	–	Reconnect 106 house connections	4,200
–	–	Reconnect 45 manholes	4,500
Subtotal, relief sanitary sewers			142,100
Trunk storm drains			
A-1	120	1,100 ft of 36-in. RC at 5.0%	20,800
A-2	94	1,400 ft of 36-in. RC at 5.0%	29,300
A-3	12	1,350 ft of 18-in. RC at 2.0%	16,500
A-4	8	570 ft of 18-in. RC at 0.8%	5,800
B-1	46	100 ft of 36-in. RC at 0.6%	2,300
B-2	46	240 ft of 30-in. RC at 1.2%	4,600
B-3	45	280 ft of 27-in. RC at 4.0%	5,200
B-4	33	360 ft of 24-in. RC at 6.0%	6,000
B-5	16	340 ft of 18-in. RC at 5.5%	4,200
C-1	40	350 ft of 24-in. RC at 3.3%	5,100
C-2	22	270 ft of 24-in. RC at 5.0%	3,900
C-3	15	370 ft of 24-in. RC at 3.0%	5,300

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
C-4	14	700 ft of 24-in. RC at 0.6%	10,000
C-5	11	560 ft of 18-in. RC at 1.1%	6,100
C-2A	8	670 ft of 21-in. RC at 0.4%	8,500
D-1	13	460 ft of 18-in. RC at 3.0%	5,000
D-2	12	800 ft of 18-in. RC at 1.5%	8,800
D-3	11	470 ft of 18-in. RC at 2.5%	5,200
D-4	10	620 ft of 18-in. RC at 0.9%	6,800
E-1	12	1,310 ft of 24-in. RC at 2.0%	19,000
E-2	8	1,050 ft of 24-in. RC at 0.2%	15,200
E-3	4	370 ft of 21-in. RC at 0.14%	4,700
F-1	19	1,600 ft of 24-in. RC at 1.0%	23,100
F-2	17	300 ft of 21-in. RC at 1.3%	3,800
F-3	13	320 ft of 18-in. RC at 3.0%	3,500
G-1	16	300 ft of 24-in. CMP at 20%	8,200
G-2	13	450 ft of 24-in. RC at 0.9%	7,500
G-3	8	310 ft of 21-in. RC at 0.3%	4,000
G-4	35	220 ft of 33-in. RC at 5.0%	4,200
G-5	32	1,350 ft of 33-in. RC at 0.7%	26,800
G-6	30	1,200 ft of 33-in. RC at 0.4%	23,600
G-7	28	1,080 ft of 33-in. RC at 0.3%	21,200
G-8	22	700 ft of 33-in. RC at 0.2%	13,800
G-9	22	340 ft of 24-in. RC at 0.9%	4,900
G-10	22	450 ft of 18-in. RC at 9.0%	5,000
G-11	18	460 ft of 18-in. RC at 4.8%	5,100
G-12	16	320 ft of 18-in. RC at 9.7%	3,500
G-13	9	300 ft of 18-in. RC at 4.0%	3,300
G-14	6	1,100 ft of 18-in. RC at 0.8%	12,100
Subtotal, trunk storm drains			371,900
Local storm drains			
–	–	11,970 ft of 8-in.	55,500
–	–	7,210 ft of 10-in.	39,700
–	–	4,570 ft of 12-in.	36,600
–	–	8,040 ft of 15-in.	76,400
–	–	118 intersection crossings, includes catch basin reconnections	44,400
Subtotal, local storm drains			252,600
Total contract cost, partial separation, Plan I			766,600
Engineering and contingencies, 25 per cent			191,600
Total construction cost, partial separation, Plan I			958,200
Complete separation[e,f]			
Trunk storm drains			
A-1	180	1,100 ft of 42-in. RC at 5.0%	27,400
A-2	140	1,400 ft of 42-in. RC at 5.0%	37,800

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
A-3	18	1,350 ft of 21-in. RC at 2.0%	21,900
A-4	11	570 ft of 21-in. RC at 0.8%	8,000
B-1	69	100 ft of 42-in. RC at 0.6%	3,100
B-2	69	240 ft of 36-in. RC at 1.2%	6,000
B-3	68	280 ft of 30-in. RC at 4.0%	6,200
B-4	50	360 ft of 27-in. RC at 6.0%	7,200
B-5	24	340 ft of 18-in. RC at 5.5%	4,800
C-1	60	350 ft of 30-in. RC at 3.3%	7,300
C-2	33	270 ft of 27-in. RC at 5.0%	5,100
C-3	23	370 ft of 27-in. RC at 3.0%	7,000
C-4	21	700 ft of 27-in. RC at 0.6%	13,200
C-5	16	560 ft of 21-in. RC at 1.1%	8,500
C-2A	12	670 ft of 24-in. RC at 0.4%	11,400
D-1	20	460 ft of 24-in. RC at 3.0%	7,800
D-2	18	800 ft of 24-in. RC at 1.5%	13,600
D-3	17	470 ft of 24-in. RC at 2.5%	8,000
D-4	16	620 ft of 24-in. RC at 0.9%	10,500
E-1	17	1,310 ft of 27-in. RC at 2.0%	24,800
E-2	13	1,050 ft of 27-in. RC at 0.2%	19,800
E-3	6	370 ft of 24-in. RC at 0.14%	6,300
F-1	29	1,600 ft of 27-in. RC at 1.0%	30,200
F-2	26	300 ft of 27-in. RC at 1.3%	5,700
F-3	20	320 ft of 21-in. RC at 3.0%	4,900
G-1	24	320 ft of 27-in. CMP at 20%	12,000
G-2	20	450 ft of 27-in. RC at 0.9%	9,000
G-3	13	310 ft of 27-in. RC at 0.3%	5,800
G-4	53	220 ft of 39-in. RC at 5.0%	5,500
G-5	48	1,350 ft of 39-in. RC at 0.7%	36,200
G-6	45	1,200 ft of 39-in. RC at 0.4%	32,200
G-7	42	1,080 ft of 39-in. RC at 0.3%	29,000
G-8	33	700 ft of 39-in. RC at 0.2%	18,800
G-9	33	340 ft of 30-in. RC at 0.9%	7,100
G-10	33	450 ft of 24-in. RC at 9.0%	7,600
G-11	28	460 ft of 21-in. RC at 4.8%	7,000
G-12	24	320 ft of 21-in. RC at 9.0%	4,900
G-13	14	300 ft of 21-in. RC at 4.0%	4,600
G-14	9	1,100 ft of 21-in. RC at 0.8%	16,800
Subtotal, trunk storm drains			503,000
Local storm drains			
–	–	33,050 ft of 8-in.	228,600
–	–	7,930 ft of 10-in.	62,600
–	–	4,730 ft of 12-in.	50,400

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
–	–	6,020 ft of 15-in.	75,000
–	–	7,630 ft of 18-in.	108,500
–	–	1,600 new house connections	480,000
–	–	118 intersection crossings, includes catch basin reconnections	44,400
Subtotal, local storm drains			1,049,500
Total contract cost, complete separation, Plan I			1,552,500
Engineering and contingencies, 25 per cent			388,100
Total construction cost, complete separation, Plan I			1,940,600
Plan II[g]			
Partial separation[b,h]			
–	–	Relief sanitary sewers[i]	142,100
Trunk storm drains			
A-1	147	820 ft of 36-in. RC at 5.0%	15,500
A-2	131	1,200 ft of 36-in. RC at 5.0%	22,800
A-3	101	580 ft of 36-in. RC at 3.5%	13,300
A-4	76	600 ft of 36-in. RC at 1.2%	13,800
A-5	67	750 ft of 36-in. RC at 1.2%	15,400
A-6	61	570 ft of 36-in. RC at 1.0%	10,800
A-7	31	750 ft of 30-in. RC at 0.7%	12,900
B-1	10	400 ft of 15-in. CMP at 10%, anchored to slopes	7,600
B-2	9	250 ft of 18-in. RC at 1.8%	3,100
B-3	7	760 ft of 15-in. RC at 3.3%	7,200
B-4	4	900 ft of 10-in. conc at 4.0%	5,000
B-10	12	350 ft of 18-in. CMP at 10%, anchored to slopes	7,300
B-11	10	480 ft of 18-in. RC at 2.2%	5,300
C-1	20	430 ft of 18-in. RC at 4.0%	4,700
C-2	17	340 ft of 18-in. RC at 4.0%	3,700
C-3	7	300 ft of 12-in. RC at 13%	2,400
D-1	17	380 ft of 18-in. RC at 4.0%	4,700
D-1A	7	650 ft of 15-in. RC at 2.0%	6,200
D-2	17	520 ft of 18-in. RC at 6.0%	6,400
D-3	14	170 ft of 15-in. RC at 5.5%	1,900
D-4	7	300 ft of 12-in. RC at 10%	2,900
E-1	12	300 ft of 18-in. RC at 2.0%	3,300
E-2	11	320 ft of 15-in. RC at 5.0%	3,000
E-3	8	350 ft of 12-in. RC at 11%	2,800
E-4	4	350 ft of 10-in. conc at 15%	1,900
F-1	7	500 ft of 18-in. RC at 0.5%	5,500
F-2	5	320 ft of 10-in. conc at 8.0%	2,600
F-3	4	320 ft of 10-in. conc at 11%	1,800
G-1	19	660 ft of 18-in. RC at 4.5%	7,200

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
G-2	16	320 ft of 18-in. RC at 4.0%	3,500
G-3	11	150 ft of 15-in. RC at 17%	1,400
G-4	11	350 ft of 15-in. RC at 6.0%	3,300
G-5	7	750 ft of 12-in. RC at 10%	6,000
G-10	13	300 ft of 18-in. RC at 1.3%	3,300
G-11	11	320 ft of 15-in. RC at 10%	3,000
G-12	9	320 ft of 15-in. RC at 13%	3,000
G-13	8	320 ft of 15-in. RC at 8.7%	3,000
H-1	17	320 ft of 21-in. RC at 1.6%	4,100
H-2	14	330 ft of 18-in. RC at 8.0%	3,600
H-3	11	320 ft of 15-in. RC at 10%	3,000
H-4	7	250 ft of 12-in. RC at 16%	2,000
H-10	11	320 ft of 18-in. RC at 1.6%	3,500
H-11	9	320 ft of 15-in. CMP at 29%	6,000
H-12	7	320 ft of 12-in. RC at 12%	2,600
J-1	27	600 ft of 24-in. CMP at 20%, anchored to slopes	15,300
J-2	13	500 ft of 15-in. RC at 11%	4,900
J-3	9	200 ft of 15-in. RC at 10%	1,900
J-4	4	400 ft of 10-in. conc at 7.5%	2,200
K-1	15	340 ft of 30-in. RC at 0.2%	6,200
K-2	15	340 ft of 30-in. RC at 0.2%	6,200
K-3	10	340 ft of 18-in. RC at 1.0%	3,700
K-4	10	410 ft of 15-in. RC at 10%	3,900
K-5	8	460 ft of 15-in. RC at 4.8%	4,400
L-1	13	700 ft of 18-in. CMP at 20%, anchored to slopes	14,700
L-2	11	300 ft of 15-in. CMP at 20%	5,800
L-3	8	350 ft of 12-in. RC at 10%	2,800
Subtotal, trunk storm drains			324,300
Local storm drains			
–	–	17,190 ft of 8-in.	80,000
–	–	11,000 ft of 10-in.	60,500
–	–	4,370 ft of 12-in.	35,000
–	–	3,000 ft of 15-in.	28,500
–	–	118 intersection crossings, includes catch basin reconnections	44,400
Subtotal, local storm drains			248,400
Total contract cost, partial separation, Plan II			714,800
Engineering and contingencies, 25 per cent			178,700
Total construction cost, partial separation, Plan II			893,500
Complete separation, Alternative 1[j]			
Trunk storm drains			
A-1	220	820 ft of 42-in. RC at 5.0%	20,400

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Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
A-2	197	1,200 ft of 42-in. RC at 5.0%	29,800
A-3	152	580 ft of 42-in. RC at 3.5%	17,000
A-4	114	600 ft of 42-in. RC at 1.2%	17,300
A-5	101	750 ft of 39-in. RC at 1.2%	18,200
A-6	92	570 ft of 39-in. RC at 1.0%	12,900
A-7	46	750 ft of 33-in. RC at 0.7%	14,200
B-1	15	400 ft of 18-in. CMP at 10%, anchored to slopes	8,000
B-2	14	250 ft of 18-in. RC at 1.8%	3,100
B-3	11	760 ft of 15-in. RC at 3.3%	7,200
B-4	6	900 ft of 12-in. RC at 4.0%	7,200
B-10	18	350 ft of 21-in. CMP at 10%, anchored to slopes	8,400
B-11	15	480 ft of 18-in. RC at 2.2%	5,200
C-1	30	430 ft of 24-in. RC at 4.0%	6,200
C-2	25	340 ft of 21-in. RC at 4.0%	3,700
C-3	10	300 ft of 12-in. RC at 13%	2,400
D-1	26	380 ft of 21-in. RC at 4.0%	5,800
D-1A	11	650 ft of 18-in. RC at 2.0%	7,100
D-2	26	520 ft of 21-in. RC at 6.0%	7,800
D-3	21	170 ft of 18-in. RC at 5.5%	2,100
D-4	11	300 ft of 15-in. RC at 10%	3,300
E-1	18	300 ft of 21-in. RC at 2.0%	3,800
E-2	16	320 ft of 18-in. RC at 5.0%	3,500
E-3	12	350 ft of 15-in. RC at 11%	3,300
E-4	6	350 ft of 12-in. RC at 15%	2,800
F-1	10	500 ft of 21-in. RC at 0.5%	6,400
F-2	7	320 ft of 12-in. RC at 8.0%	2,600
F-3	6	320 ft of 12-in. RC at 11%	2,600
G-1	28	660 ft of 24-in. RC at 4.5%	9,500
G-2	24	320 ft of 21-in. RC at 4.0%	4,100
G-3	16	150 ft of 18-in. RC at 17%	1,600
G-4	16	350 ft of 18-in. RC at 6.0%	3,800
G-5	11	750 ft of 15-in. RC at 10%	7,100
G-10	20	300 ft of 21-in. RC at 1.3%	3,800
G-11	17	320 ft of 18-in. RC at 10%	3,500
G-12	13	320 ft of 18-in. RC at 13%	3,500
G-13	12	320 ft of 15-in. RC at 8.7%	3,000
H-1	26	320 ft of 24-in. RC at 1.6%	4,500
H-2	21	330 ft of 21-in. RC at 8.0%	4,200
H-3	17	320 ft of 18-in. RC at 10%	3,500
H-4	11	250 ft of 15-in. RC at 16%	2,400
H-10	16	320 ft of 21-in. RC at 1.6%	4,100
H-11	14	320 ft of 18-in. CMP at 29%	7,000
H-12	11	320 ft of 15-in. RC at 12%	3,000

Continued on next page

Table 18-4. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
J-1	40	600 ft of 27-in. CMP at 20%, anchored to slopes	16,200
J-2	20	500 ft of 18-in. RC at 11%	5,600
J-3	13	200 ft of 15-in. RC at 10%	1,900
J-4	6	400 ft of 12-in. RC at 7.5%	3,200
K-1	22	340 ft of 33-in. RC at 0.2%	6,800
K-2	22	340 ft of 33-in. RC at 0.2%	6,800
K-3	15	340 ft of 21-in. RC at 1.0%	4,300
K-4	15	410 ft of 18-in. RC at 10%	4,500
K-5	12	460 ft of 15-in. RC at 4.8%	4,400
L-1	20	700 ft of 21-in. CMP at 20%, anchored to slopes	16,800
L-2	16	300 ft of 18-in. CMP at 20%	6,600
L-3	12	350 ft of 15-in. RC at 10%	3,300
Subtotal, trunk storm drains			381,300
Local storm drains			
–	–	8,810 ft of 8-in.	41,000
–	–	13,530 ft of 10-in.	74,500
–	–	7,110 ft of 12-in.	57,000
–	–	4,860 ft of 15-in.	46,000
–	–	1,240 ft of 18-in.	13,600
–	–	118 intersection crossings, includes catch basin reconnections	44,400
–	–	400 new house connections[k]	120,000
Subtotal, local storm drains			396,500
Total contract cost, complete separation, Plan II, Alternative 1			777,800
Engineering and contingencies, 25 per cent			194,400
Total construction cost, complete separation, Plan II, Alternative 1			972,200
Complete separation, Alternative 2[e,h]			
–	–	Trunk storm drains[l]	443,800
–	–	Local storm drains[l]	366,200
–	–	32,250 ft of additional 8-in. local drains to extend to last house on each block	222,000
–	–	1,600 new house connections	480,000
Total contract cost, complete separation, Plan II, Alternative 2			1,512,000
Engineering and contingencies, 25 per cent			378,000
Total construction cost, complete separation, Plan II, Alternative 2			1,890,000

[a] *Plan I provides for installation of storm drains along routes of existing sewers with one point of disposal.*

[b] *Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers.*

[c] *See Fig. 18-4 for location of facilities.*

[d] *First flow is required relief capacity, second is total design flow.*

[e] *Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Assumes that storm drains will be laid at sufficient depth to intercept all foundation drains.*

[f] *See Fig. 18-4 for location of trunk storm drains.*

(Footnotes continued on next page.)

Table 18-5. Summary of Separation Costs, District 29 – South Magnolia

Facility	Construction cost,[a] dollars				
	Plan I[b]		Plan II[c]		
	Partial separation[d]	Complete separation[e]	Partial separation[d]	Complete separation	
				Alternative 1[f]	Alternative 2[e]
Relief sanitary sewers	177,600	–	177,600	–	–
Trunk storm drains	464,900	628,700	405,400	476,600	554,800
Local storm drains[g]	315,700	711,900	310,500	345,600	735,200
New house connections	–	600,000	–	150,000[h]	600,000
Total	958,200	1,940,600	893,500	972,200	1,890,000

[a] *Includes engineering and contingencies.*

[b] *Plan I provides for installation of storm drains along routes of existing combined system with one point of disposal.*

[c] *Plan II provides for rerouting of storm drains with 3 points of disposal.*

[d] *Provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers.*

[e] *Provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers.*

[f] *Provides for removal of all street drainage and roof leaders from sanitary sewers, but permits continued discharge of foundation drains to sanitary sewers.*

[g] *Exclusive of new house connections.*

[h] *Assumes that 25 per cent of all roof leaders will be reconnected to storm drainage system.*

storm flow from roof leaders.

The existing storm drain on McGilvra Boulevard has only enough capacity for street drainage. Some relief will be required in order to accommodate the total flow of storm water.

Locations of relief sanitary sewers and of storm drainage facilities required to effect partial separation are shown in Fig. 18-6 together with a layout of the existing system. Descriptions and estimated construction costs are given in Table 18-6.

Locations of storm drainage facilities required for complete separation would be substantially the same as those shown in Fig. 18-6. Complete separation would necessitate the addition of relief lines for the existing storm drains and the extension of local storm drains to the extremity of each existing sewer line. Descriptions and estimated construction costs of the required facilities are also given in Table 18-6.

For partial separation of the Madison Park district, the estimated cost of construction is $173,900, or about $1,740 per acre. Of this total, approximately 49 per cent is for relief sanitary sewers, 24 per cent for trunk drains and 27 per cent for local storm drains. For complete separation, the estimated cost is $369,100, of which about 27 per cent is for trunk storm drains and 73 per cent is for local storm drains, including new house connections.

District 58 - East Madison

The East Madison district is situated on the east slopes of Capitol Hill south of the University of Washington Arboretum. It comprises an area of 640 acres and has a predicted ultimate population of 20,000. Drainage in this district is northward to Union Bay, with steep surface slopes perpendicular to the main drainage axis.

The existing combined system, which is a part of the Lake Washington district on the North Trunk sys-

Table 18-4 Footnotes (continued).

[g] *Plan II provides for rerouting of storm drains with 3 points of disposal.*

[h] *See Fig. 18-5 for location of trunk storm drains.*

[i] *Relief sanitary sewers same as for Plan I.*

[j] *Complete separation under Alternative 1 provides for removal of all street drainage and roof leaders from sanitary sewers, but permits continued discharge of foundation drains to sanitary sewers. Assumes that storm drains will be laid at minimum depth. See Fig. 18-5 for location of facilities.*

[k] *Assumes that 25 per cent of all roof leaders will be reconnected to storm drainage system.*

[l] *Design flow, length, size and slope of all storm drains same as for Alternative 1. Increased cost due to added depth required for interception of foundation drains.*

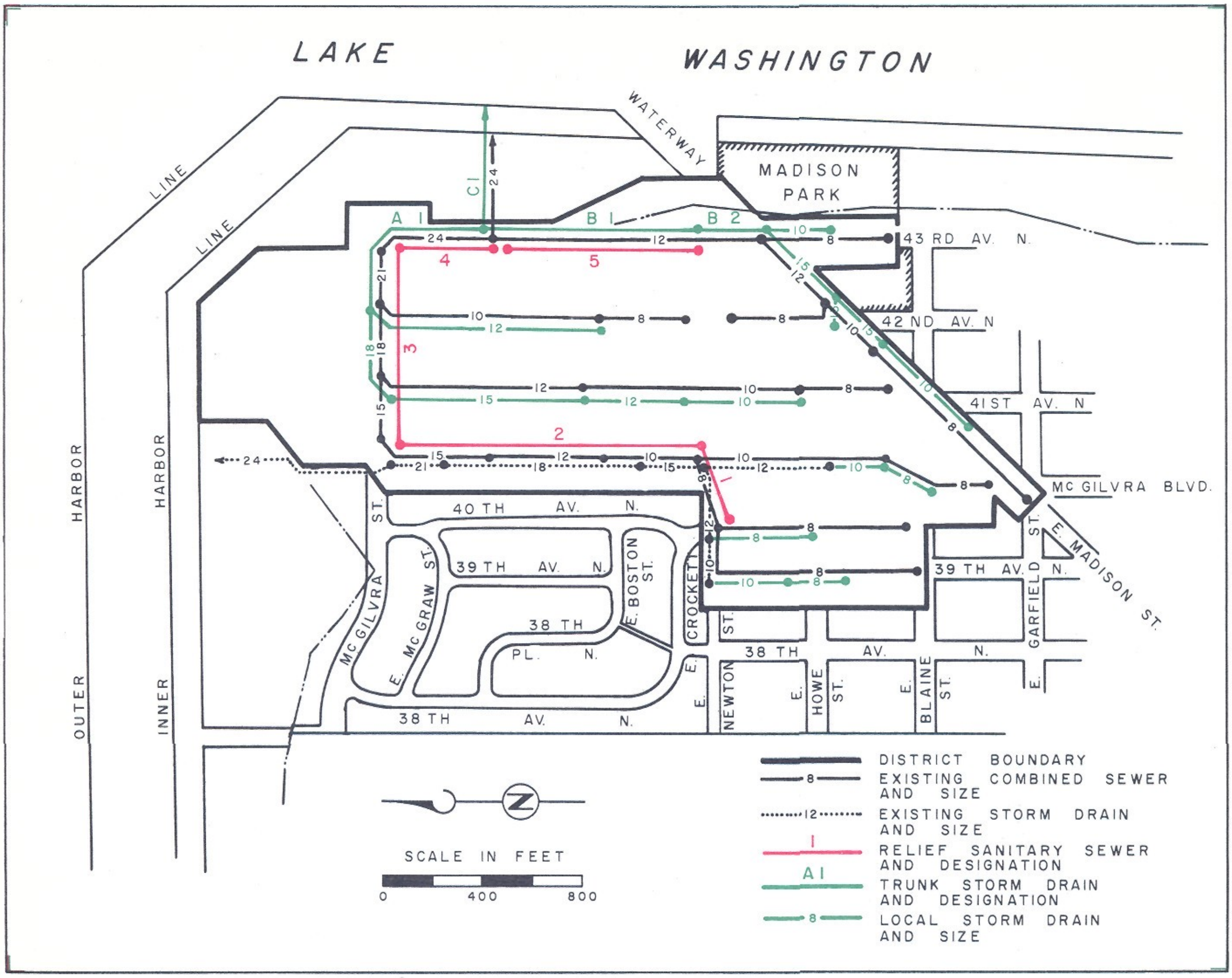

Fig. 18-6. Layout of Facilities for Partial Separation, District 33 - Madison Park

Table 18-6. Description and Estimated Construction Costs for Separation of District 33 – Madison Park

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	0.2, 1.8[b]	290 ft of 8-in. conc at 1.7%	2,000
2	0 – 2.6, 3.3 – 3.7[b]	1,230 ft of 10-in. conc at 0.7 – 1.2%	10,400
3	4.8 – 7.0, 8.9 – 13[b]	920 ft of 21-in. RC at 0.12 – 0.3%	20,000
4	5.5, 13[b]	400 ft of 24-in. RC at 0.11%	15,000
5	0.2, 2.2 – 2.8[b]	830 ft of 8-in. conc at 0.3 – 0.6%	17,400
–	–	Reconnect 36 house connections	1,400
–	–	Reconnect 15 manholes	1,500
Subtotal, relief sanitary sewers			67,700

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Table 18-6. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Trunk storm drains			
A-1	7.3 – 7.5	750 ft of 24-in. RC at 0.11 – 0.19%	12,000
B-1	4.8 – 6.3	830 ft of 18-in. RC at 0.33 – 0.6%	10,000
B-2	3.6	270 ft of 15-in. RC at 0.33%	2,900
C-1	12.5	500 ft of 24-in. RC outfall	8,700
Subtotal, trunk storm drains			33,600
Local storm drains			
–	–	870 ft of 8-in.	4,900
–	–	1,750 ft of 10-in.	11,700
–	–	1,360 ft of 12-in.	12,500
–	–	1,480 ft of 15-in.	17,200
–	–	350 ft of 18-in.	4,300
–	–	26 intersection crossings, includes catch basin reconnections	7,200
Subtotal, local storm drains			37,800
Total contract cost, partial separation			139,100
Engineering and contingencies, 25 per cent			34,800
Total construction cost, partial separation			173,900
Complete separation[c]			
Trunk storm drains			
A-1	15 – 17	400 ft of 33-in. RC at 0.11%	17,300
A-1	15	350 ft of 30-in. RC at 0.12 – 0.19%	13,500
B-1, B-2	6 – 10	1,100 ft of 27-in. RC at 0.33 – 0.6%	29,500
C-1	25	500 ft of 36-in. RC outfall	12,100
–	3.0, 12[b]	350 ft of 12-in. at 0.75% to parallel existing 18-in.	3,200
–	4.2, 11[b]	270 ft of 12-in. at 1.0% to parallel existing 15-in.	2,500
–	–	Reconnect 4 manholes	400
Subtotal, trunk storm drains			78,500
Local storm drains			
–	–	1,520 ft of 8-in.	9,800
–	–	2,180 ft of 10-in.	17,700
–	–	2,150 ft of 12-in.	22,100
–	–	1,010 ft of 15-in.	13,500
–	–	780 ft of 18-in.	11,000
–	–	440 ft of 21-in.	9,100
–	–	300 ft of 24-in.	6,400
–	–	26 intersection crossings, includes catch basin reconnections	7,200
–	–	400 new house connections	120,000
Subtotal, local storm drains			216,800
Total contract cost, complete separation			295,300
Engineering and contingencies, 25 per cent			73,800
Total construction cost, complete separation			369,100

(See page 515 for footnotes.)

tem, presently discharges through a 60-inch trunk to an inverted siphon at the Montlake Bridge. Storm flows in excess of the capacity of the siphon overflow to Montlake Canal.

Originally, the portion of East Madison district west of 23rd Avenue North and north of East Madison Street drained south to the Hanford-Rainier Valley system. Sewage from this area, however, is now diverted eastward along East Denny Way to the principal trunk serving the district.

Although the existing system has ample capacity for the peak sanitary flow, it is unable generally to accommodate the flow from a 10-year storm. In fact, a considerable amount of relief sanitary sewer construction will be required to handle the sanitary peak plus storm runoff from roof leaders. Capacity for a 10-year storm is available, however, along the downstream sections of the principal trunk.

Locations of facilities required for partial separation of the East Madison district are shown in Fig. 18-7 together with a layout of the existing sewers. Descriptions and estimated construction costs are given in Table 18-7. As shown in both the figure and the tabulated data, the existing trunk leaving the district would be utilized as a storm water line to the fullest possible extent. This, of course, would require construction of a new sanitary sewer along the section so utilized.

Locations of facilities required for complete separation are substantially the same as those shown in Fig. 18-7. Descriptions and estimated construction costs are also given in Table 18-7. As in partial separation, the existing main trunk would be utilized to the fullest possible extent as a storm drain. Under design flow conditions, the existing trunk would be surcharged to 30th Avenue North and East John Street.

For partial separation of the East Madison district, the estimated construction cost amounts to a total of $1,390,200, or about $2,170 per acre. Of this total, approximately 24 per cent is for relief sanitary sewers, 12 per cent for trunk storm drains, and 64 per cent for local storm drains. For complete separation, the estimated cost is $2,690,800, or about $4,200 per acre. Of this total, approximately 3 per cent is for new sanitary sewers, 12 per cent for trunk storm drains, and 85 per cent for local storm drains, including new house connections.

Under both partial and complete separation, part of the proposed storm drain (A 9) would be laid along 28th Avenue North. Since this street will shortly become an arterial highway under the Empire Way extension program, it is believed that the 28th Avenue section of the storm drain should be constructed immediately.

Although the study here reported was concerned only with the collection of storm water within the district and not with the problem of disposal, it is evident that disposal could be achieved satisfactorily by discharge to an open channel following an old creek bed through Washington Park. This channel, which would be about 4,200 feet in length and would terminate at Union Bay, would be considerably cheaper than closed conduit of the size here required.

RECOMMENDED SEPARATION PROGRAM

It is evident from the information here presented, as well as that developed in past studies, that the city of Seattle is faced with a continuing program of storm water separation. This program is necessary to prevent the periodic overloading of trunk and local collection systems. In addition, and as set forth in Chapter 15, many of the existing major trunk sewers which are to be incorporated in the central sewerage project lack the capacity to accommodate the flows resulting from a storm with a 10-year recurrence interval. As a consequence, separation will be required in areas tributary to those trunks.

With all of the various factors taken into account, it appears that separation of storm water from sanitary sewage will be required to some extent in all areas presently served by combined sewers. Analysis will have to be made, of course, to determine local requirements. It appears likely, however, that separation in all areas will be required at least to the extent of removing street drainage from existing sewers.

Summary of Separation Costs

A summary of the estimated construction costs for storm water separation in the six areas herein considered is presented in Table 18-8. For partial separation, the estimated cost varies from $1,690 to $2,170 per acre and averages $1,860 per acre. For complete separation, the cost varies from $3,180 to $4,570 per acre and averages $3,890 per acre. Since the deviation from the average is less than 20 per cent, use of the average figures appears justifiable in attempting to assess the total cost to the city of storm water separation.

Table 18-6 Footnotes.

[a] *Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-6 for location of facilities.*

[b] *First flow is required relief capacity, second is total design flow.*

[c] *Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Routes of trunk storm drains are generally as shown in Fig. 18-6.*

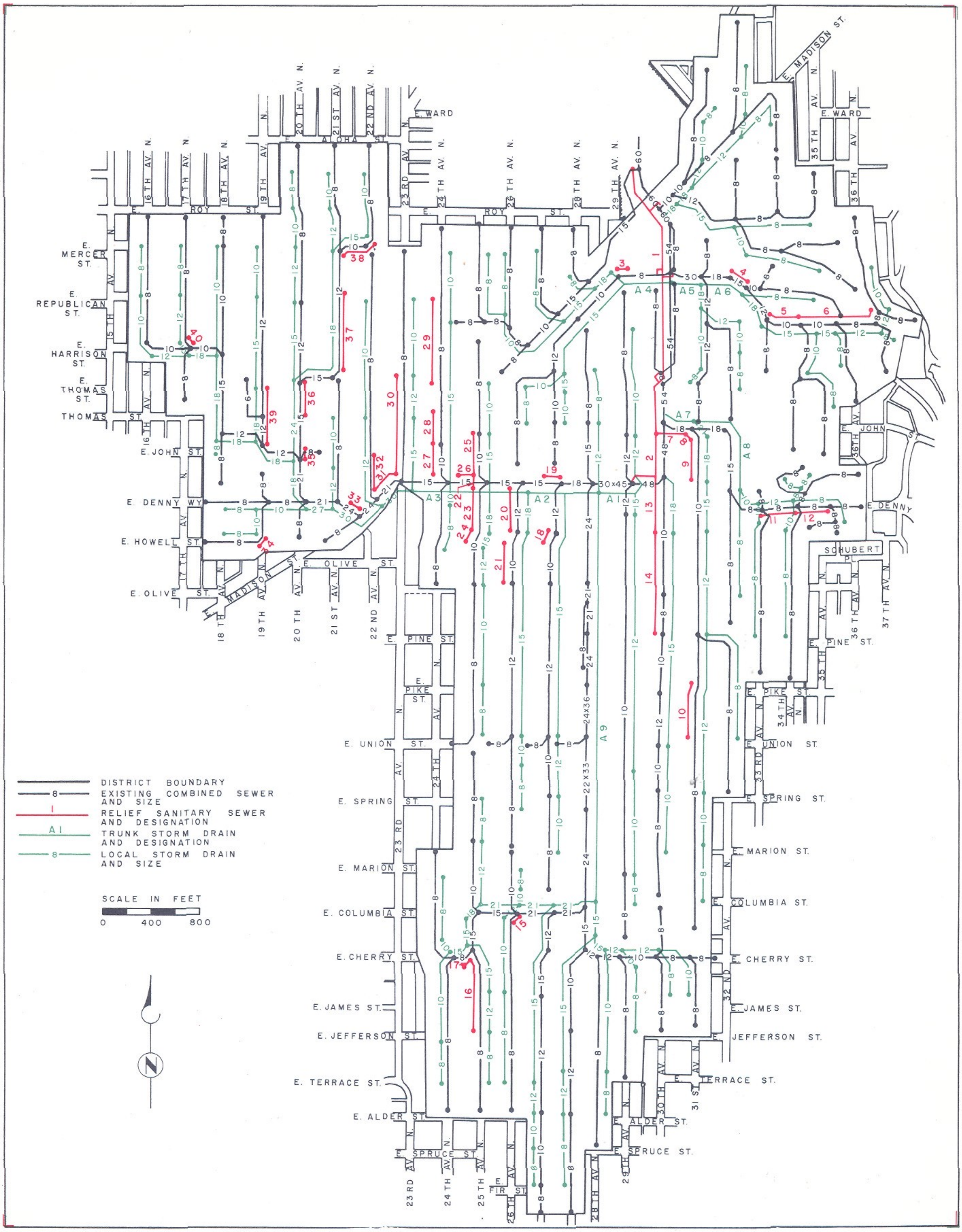

Fig. 18-7. Layout of Facilities for Partial Separation, District 58 - East Madison

Table 18-7. Description and Estimated Construction Costs for Separation of District 58 – East Madison

Number	Design flow, cfs	Description	Construction cost, dollars
Partial separation[a]			
Relief sanitary sewers			
1	64 – 65	2,120 ft of 39-in. RC at 0.50 – 0.70%, to replace existing 54 – 60-in. which is to be used as storm drain	119,000
2	44 – 49	880 ft of 30-in. RC at 1.4 – 2.3%, to replace existing 48-in. which is to be used as storm drain, includes 880 ft of 8-in. lateral parallel to trunk to avoid making house connections to trunk	31,300
3	1.2, 2.7[b]	330 ft of 8-in. conc at 1.4%	2,000
4	1.7, 9.0[b]	210 ft of 10-in. conc at 1.3%	1,500
5	2.4, 7.0[b]	310 ft of 8-in. conc at 4.4 – 8.1%	1,900
6	0.4 – 1.0, 2.0 – 4.0[b]	720 ft of 8-in. conc at 1.0 – 4.0%	4,300
7	3.0, 13[b]	210 ft of 12-in. RC at 0.8%	3,900
8	5.3, 7.4[b]	60 ft of 15-in. RC at 0.9%	1,300
9	2.7, 6.0[b]	470 ft of 10-in. conc at 2.4%	6,200
10	0.7, 2.5[b]	320 ft of 8-in. conc at 2.1 – 3.3%	1,900
11	1.1, 4.0 – 5.3[b]	510 ft of 8-in. conc at 5.4 – 11.8%	3,100
12	2.8, 4.0[b]	270 ft of 12-in. RC at 1.0%	2,900
13	1.3, 3.4[b]	470 ft of 10-in. conc at 0.9%	6,200
14	0.7 – 1.3, 2.7 – 2.9[b]	850 ft of 8-in. conc at 1.0 – 1.4%	5,100
15	0.4, 2.0[b]	60 ft of 8-in. conc at 1.6%	400
16	0.3 – 0.6, 1.0 – 1.5[b]	690 ft of 8-in. conc at 0.4 – 1.1%	4,100
17	2.4, 3.7[b]	60 ft of 10-in. conc at 1.1%	400
18	2.4, 3.9[b]	80 ft of 12-in. conc at 0.4%	900
19	3.6, 28[b]	80 ft of 8-in. conc at 11.6%	500
20	1.1, 3.1[b]	390 ft of 10-in. conc at 0.30%	2,800
21	0.8, 2.6[b]	350 ft of 8-in. conc at 0.70%	2,100
22	0.4, 2.4[b]	60 ft of 8-in. conc at 0.80%	400
23	1.0, 2.2[b]	390 ft of 10-in. conc at 0.30%	2,800
24	0.4, 2.1[b]	90 ft of 8-in. conc at 0.60%	500
25	0.5 – 0.7, 1.7 – 2.0[b]	450 ft of 8-in. conc at 0.60 – 1.45%	2,700
26	3.5, 22[b]	100 ft of 8-in. conc at 7.7%	600
27	0.6, 2.2[b]	330 ft of 8-in. conc at 0.5%	2,300
28	1.2, 2.1[b]	230 ft of 10-in. conc at 0.6%	1,900
29	0.3 – 0.8, 1.0 – 1.6[b]	640 ft of 8-in. conc at 0.3 – 0.4%	4,500
30	0.6 – 0.8, 1.5 – 2.3[b]	890 ft of 8-in. conc at 0.8 – 1.5%	6,200
31	2.7, 18[b]	410 ft of 12-in. RC at 0.9%	5,000
32	0.4, 2.0[b]	230 ft of 8-in. conc at 1.9%	1,400

Continued on next page

Table 18-7. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
33	11, 17[b]	170 ft of 30-in. RC at 0.06%, includes 170 ft of 8-in. lateral parallel to trunk to avoid making house connections to trunk	5,100
34	0.7, 1.1[b]	50 ft of 10-in. conc at 0.11%	400
35	6.9, 8.6[b]	60 ft of 27-in. RC at 0.07%	1,300
36	4.2, 8.6[b]	330 ft of 15-in. RC at 0.45%	4,600
37	1.2 – 1.6, 4.7 – 5.2[b]	660 ft of 8-in. conc at 1.2 – 1.6%	4,000
38	0.8, 2.0[b]	310 ft of 8-in. conc at 0.5 – 0.7%	1,900
39	1.7, 2.1[b]	600 ft of 12-in. RC at 0.25%	7,300
40	0.7, 1.7[b]	50 ft of 8-in. conc at 0.7%	300
–	–	Reconnect 132 house connections	6,500
–	–	Reconnect 89 manholes	8,900
Subtotal, relief sanitary sewers			270,400
Trunk storm drains			
A-1	100	320 ft of 36-in. RC at 2.2%	6,900
A-2	59 – 66	600 ft of 30-in. RC at 8.8 – 16.7%	10,500
A-3	42 – 53	1,020 ft of 30-in. RC at 7.7 – 16.7%	17,800
A-4	11 – 16	560 ft of 15-in. RC at 2.9 – 6.7%	5,700
A-5	22	250 ft of 24-in. RC at 0.80%	4,100
A-6	17	420 ft of 21-in. RC at 1.3 – 1.4%	5,700
A-7	26	270 ft of 27-in. RC at 0.8 – 5.8%	4,900
A-8	15	890 ft of 24-in. RC at 0.6 – 15.3%	13,000
A-9	34 – 37	3,600 ft of 24-in. RC at 2.3 – 10.0%	59,400
–	–	Connections to existing trunk to be utilized as storm drain	1,500
Subtotal, trunk storm drains			129,500
Local storm drains			
–	–	22,000 ft of 8-in.	102,100
–	–	19,000 ft of 10-in.	112,700
–	–	13,000 ft of 12-in.	106,300
–	–	17,000 ft of 15-in.	169,900
–	–	8,000 ft of 18-in.	91,300
–	–	1,000 ft of 21-in.	12,800
–	–	800 ft of 24-in.	13,100
–	–	700 ft of 27-in.	12,000
–	–	900 ft of 30-in.	17,400
–	–	212 intersection crossings, includes catch basin reconnections	74,700
Subtotal, local storm drains			712,300
Total contract cost, partial separation			1,112,200
Engineering and contingencies, 25 per cent			278,000
Total construction cost, partial separation			1,390,200

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Table 18-7. Continued

Number	Design flow, cfs	Description	Construction cost, dollars
Complete separation[c]			
Relief sanitary sewers			
–	3.5 – 4.5	1,940 ft of 15-in. at 0.48 – 0.70%, to replace existing 54 – 60-in. which is to be used as storm drain	62,300
Trunk storm drains			
–	180 – 210	490 ft of 54-in. RC at 0.8 – 1.45%	35,500
–	179	350 ft of 51-in. RC at 0.8%	23,400
A-1	163	320 ft of 42-in. RC at 2.2%	13,000
A-2, A-3	62 – 100	1,620 ft of 30-in. RC at 7.7 – 16.7%	32,200
A-4	19 – 28	560 ft of 18-in. RC at 2.9 – 6.7%	8,300
A-5, A-6	32	460 ft of 30-in. RC at 0.8 – 1.3%	10,500
A-6	26	210 ft of 24-in. RC at 1.4%	3,300
A-7	44	270 ft of 33-in. RC at 0.8 – 5.8%	6,700
A-8	22	890 ft of 24-in. RC at 0.6 – 15.3%	14,100
A-9	54 – 58	3,600 ft of 30-in. RC at 2.3 – 10.0%	100,300
–	–	Connections to existing trunk which is to be utilized as storm drain	1,500
Subtotal, trunk storm drains			248,800
Local storm drains			
–	–	34,100 ft of 8-in.	182,400
–	–	15,600 ft of 10-in.	97,300
–	–	15,600 ft of 12-in.	139,600
–	–	19,600 ft of 15-in.	210,400
–	–	10,100 ft of 18-in.	126,700
–	–	4,200 ft of 21-in.	57,000
–	–	1,600 ft of 24-in.	24,600
–	–	620 ft of 27-in.	11,000
–	–	1,000 ft of 30-in.	19,500
–	–	600 ft of 33-in.	13,300
–	–	170 ft of 48-in.	6,000
–	–	212 intersection crossings, includes catch basin reconnections	74,700
–	–	2,930 new house connections	879,000
Subtotal, local storm drains			1,841,500
Total contract cost, complete separation			2,152,600
Engineering and contingencies, 25 per cent			538,200
Total construction cost, complete separation			2,690,800

[a] *Partial separation provides for removal of all street drainage from sanitary sewers and for continued discharge of roof leaders and foundation drains to sanitary sewers. See Fig. 18-7 for location of facilities.*

[b] *First flow is required relief capacity, second is total design flow.*

[c] *Complete separation provides for removal of all storm drainage, including roof leaders and foundation drains, from sanitary sewers. Routes of trunk drains are generally as shown in Fig. 18-7.*

At present, the sewered area within the city of Seattle comprises about 67.5 square miles, of which approximately 58.5 square miles, or 37,000 acres, are served by combined sewers. Using this area and an average cost for partial separation of $1,860 per acre, the total cost for the entire city amounts to about $69 million. While it is true that this method of calculation results in no more than an approxima-

Table 18-8. Summary of Separation Costs

District	Area, acres	Partial separation		Complete separation	
		Total cost,[a] dollars	Unit cost,[a] dollars per acre	Total cost,[a] dollars	Unit cost,[a] dollars per acre
6 – Briarcliff	150	253,600	1,690	477,000	3,180
17 – Wedgewood	480	873,000	1,820	2,196,000	4,570
23 – Southwest Seattle	960	1,735,000	1,810	3,495,800	3,640
29 – South Magnolia	530	893,500[b]	1,690	1,890,000[c]	3,570
33 – Madison Park	100	173,900	1,740	369,100	3,690
58 – East Madison	640	1,390,200	2,170	2,690,800	4,200
Total or average	2,860	5,319,200	1,860	11,118,700	3,890

[a]*Includes engineering and contingencies.*

[b]*Plan II, see Table 18-6.*

[c]*Plan II, Alternative 2, see Table 18-6.*

tion of the total cost which would thus be incurred, the resulting figure is nevertheless considered to be realistic and should be of value in planning for any separation program that might be undertaken in the future.

Because of the magnitude of the work involved in such a project, it is evident that separation will have to be undertaken as a long-range program. To that end, a program should be planned and initiated under which the first step would involve correction of the most serious flooding conditions and elimination of overflows to Lake Washington. While the latter will be alleviated by construction of storm water holding tanks as recommended in Chapter 15, the size of these tanks could be reduced materially by an effective program of storm water separation. Obviously, therefore, separation in the Lake Washington drainage basin is of paramount importance not only because of the troubles presently being experienced throughout most of the basin with sewage backups (Chapter 8) but because of the saving which could be realized through the construction of smaller holding tanks.

Based on an area of 9,200 acres in the Lake Washington drainage basin within the city of Seattle and an area of approximately 5,000 acres in other locations where serious conditions have developed, the total area to be considered in first-stage construction of separation facilities amounts to approximately 14,000 acres. On that basis, the total cost of first-stage construction would amount to about $26 million. By constructing only those facilities which are required to obtain immediately relief, this cost probably could be reduced to about $18 million. Following the first-stage program, separation should be undertaken throughout the city on a planned yearly basis until all deficiencies associated with the operation of a combined system have been adequately corrected.

Construction of New Sanitary Sewers

Although only minor problems have been encountered in the operation of combined sewers in districts which are not presently fully developed and therefore not completely sewered, it is evident from the information presented herein that the trunks serving many of these districts lack the capacity required to accommodate the flow from a 10-year storm. Further, it is no doubt true that most of the other sewers within these districts are similarly lacking in capacity. With continued development and with further installation of combined sewers, the problems attendant thereto will increase proportionately. Such a situation is bound to result eventually in relatively large expenditures for storm water separation. To reduce future costs to the greatest possible extent, it is essential that all new sewer construction in the city of Seattle be undertaken on the basis of providing separate sewers for sanitary sewage and storm drainage.

In addition to the construction of separate systems, all new buildings, even when situated in areas served by combined sewers, should be required to install separate lines for sanitary sewage and storm water. Under such an arrangement, both lines would be connected initially to the combined system. Eventually, however, following construction of a storm drain, the storm water line would be reconnected. This procedure will eliminate the need for new roof leader connections and thus will serve to reduce the construction cost of new storm drainage facilities.

Construction of New Storm Drains

Only one study was made of a program under which storm drains would be designed to pick up all runoff from streets and roofs, and foundation drains would discharge to the existing system. Cost analyses for this one area (District 29 - South Magnolia) indicate that such a program is both feasible and logical. This type of separation would be particularly economical in similar districts where extensive construction of relief sanitary sewers would otherwise be required.

In the six areas selected for study, the cost of relief sanitary sewers runs as high as 50 per cent of the total separation costs. Elimination of these relief sewers by the procedure noted above would offset to a large extent the cost of providing capacity for the total storm runoff plus the cost of installing new roof leader connections. It appears, therefore, that the most desirable program of separation would involve the provision of a storm drainage system having a capacity sufficient to handle the total storm runoff, including that contributed by roof leaders. Under such a program, storm drains could be constructed initially to intercept street drainage only. Connection of roof leaders could be undertaken subsequently to the extent necessary to relieve the existing combined sewers of storm flows in excess of their capacities.

New storm drains should be designed and constructed in accordance with standard practices. These practices were discussed briefly in Chapters 13 and 17 and cover such items as use of gutters for conveyance of storm water, street inlet design and spacing, and methods of laying pipe.

Chapter 19

FINANCING OF RECOMMENDED FACILITIES

One of the problems common to all major public works projects, including sewerage and storm drainage, is that of developing adequate and economical procedures for financing both their construction and their subsequent operation and maintenance. A closely associated problem, particularly in a metropolitan area, is that of obtaining appropriate enabling legislation under which a project can be administered on a sound and equitable basis. To avoid potential difficulties in financing and administration, it becomes necessary in some cases to enact new laws or to modify existing laws.

In general, financial and legal problems pertaining to public works projects should be assigned to experts in those fields. An engineering report can be of assistance, however, by providing essential preliminary information. In the case of a metropolitan undertaking, this can be achieved by reviewing:

1. The financial resources of the area and the powers and limitations of a metropolitan agency with respect to the use of those resources.
2. The bases for development of financing programs, including both fundamental concepts and specific costs.
3. The approximate magnitude of total annual costs, including those of the construction program and of administration, operation and maintenance of the required facilities.

This chapter contains basic information relating to the above listed items and outlines each of several alternative plans for financing the recommended sewerage and drainage projects. In submitting these plans, it should be emphasized that they are for exploratory and illustrative purposes only and are not to be regarded as specific recommendations.

In line with the balance of this report, this chapter presumes participation of the entire study area in the metropolitan sewerage and drainage programs. In so doing, it is recognized that a metropolitan agency comprising a smaller area may be formed initially. It is recognized also that the city of Seattle is financially capable of undertaking construction of the metropolitan facilities lying within its boundaries. But with problems to solve and conditions to correct which are area-wide in scope, it is obvious that remedial action will require not only a cooperative approach but a coordination of effort through some form of metropolitan sewerage agency. The extent of the area which thus might be served initially is a matter of political rather than engineering decision and obviously is beyond the province of the study here reported.

FINANCIAL RESOURCES OF THE METROPOLITAN AREA

Resources of the metropolitan Seattle area which can be made available for financing the recommended sewerage and drainage projects comprise those funds which could be raised by (1) ad valorem property taxes, (2) service charges and connection fees, and (3) special assessments against benefited properties. The amount of revenue which can be obtained in the future from these sources depends on (1) the assessed valuation of the property within the metropolitan area, (2) the number of residential, commercial and industrial sewer services, and (3) the powers and limitations of a metropolitan agency as authorized under present legislation, or as it may be amended. In turn, assessed valuations and the number of sewer services are related to future increases in population.

Application of Population Forecasts

In general, the planning of sewerage and drainage works should be based on a forecast of the greatest population growth which might reasonably occur (Chapter 5). For financing purposes, however, the forecast of the least growth likely to occur must be considered. In the present study, a low projection was derived for the state as a whole and its relation to the high projection was expressed as a percentage of the high projection for each tenth year (Table 5-8). These percentage values are applicable also to the metropolitan Seattle area. When so applied, the low population projections are found to be 900,000 for 1960, 1,015,000 for 1970, and 1,100,000 for 1980.

Projected Growth of Assessed Valuation

The extent to which revenue can be obtained through the medium of an ad valorem tax will depend on the assessed valuation of property within the metropolitan area. It is necessary, therefore, to determine what increases can be expected during the period allowed for construction of the principal facilities. This requires a determination of present values and an application thereto of ratios based on the predicted lowest rate of population growth.

No information is available concerning the present assessed valuation of real and personal property within the study area. Nevertheless, a reasonably accurate figure can be developed from county assessors' records. In King County, the assessed valuation for 1956, upon which 1957 taxes are levied, was $909,637,000 and amounted to $1,080 per capita. Within the city of Seattle, as would be expected, the per capita value was somewhat higher and amounted to $1,138 in 1956.

In 1956, the assessed valuation of all King County school districts lying either entirely within or largely within the study area was 96.5 per cent of that of the county valuation. At the same time, 95.2 per cent of the county population resided within the study area. On that basis, the per capita assessed valuation within the King County portion of the study area was close to $1,100 per capita.

In Snohomish County, the total assessed valuation in 1956 was $108,209,000 and amounted to $812 per capita. Although the city of Everett, which is outside the study area, and the Everett School District, which is partly in the area, both had per capita valuations of $1,100, values in other portions of the area within Snohomish County apparently were much lower. In the Edmonds School District, which is in the western part of the study area, the assessed value was approximately $640 per capita. In the city of Mountlake Terrace, it was only $360 per capita. At present, however, the Snohomish County portion contains less than 4 per cent of the study area population.

A threefold increase in assessed valuation in King County since 1940 (Fig. 19-1) represents not only the increase in real wealth but the effect of inflation. A similar situation is reflected by the rise in per capita values from $600 in 1940-42 to $1,080 in 1956.

A constant per capita value of $1,100 was assumed in projecting the future assessed valuation. Based on the low population projection, it is estimated that assessed valuations of the study area will be $990 million in 1960, $1,116.5 million in 1970, and $1,210 million in 1980. Values for intermediate years are given in Table 19-1.

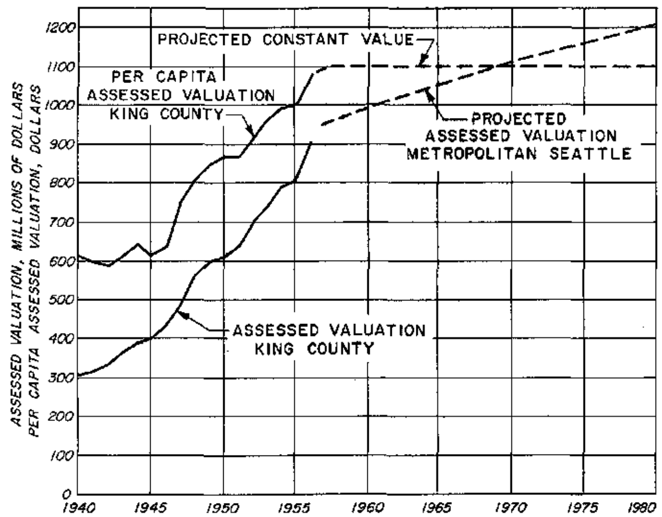

Fig. 19-1. Past and Projected Assessed Valuations, 1940-1980

Past increases in assessed valuation result from increase in real wealth and the effects of inflation. The projected valuation for the metropolitan area is based on a constant per capita value of $1,100 and on the low population projection.

Projected Increases in Service Connections

Based on data obtained from the various sewerage agencies (Chapter 6), the total number of service connections in the study area outside Seattle amounts to about 16,000. Specific figures for Seattle are not available. This is because every water service connection, including all those to premises utilizing septic tanks, pays a sewerage service charge. It is estimated, however, that approximately 13,700 private disposal systems are presently in use within the city.

The situation in Seattle is further complicated by the large number both of commercial and industrial establishments and of multiple unit dwellings. Seattle Water Department records for 1956 show 141,000 accounts within the city. Based on the estimated population of 561,000 for that year, the number of persons per account is 4.0 as compared to a range of 3.0 to 3.5 in suburban areas outside the city.

Due to the large proportion of multiple housing, commercial and industrial accounts within the city, the average revenue per account is much higher than that indicated by the present residential service charge of $12.00 per year. In 1956, for example, the total revenue was $2.75 million, or an average of $19.50 per account. Expressed in another way, the total revenue is equivalent to that which would be obtained from 239,000 single family services. On a population basis, the latter number is equal to one such service for each 2.35 persons in the city.

To estimate the future number of service connections, expressed as equivalent single-family services, a figure of 2.8 persons per sewer service was assumed for the area within the present city limit. This value is somewhat higher than that developed above but was used in order to avoid possible over-estimation upon expansion of the system in residential areas. For all other parts of the study area, a value of 3.5 persons per service was assumed and is the highest of the range reported by the suburban sewerage agencies.

To utilize the foregoing values, separate low projections of population and estimates of the number of persons served by public sewerage agencies were developed both for Seattle and for the balance of the study area (Table 19-1 and Fig. 19-2). In estimating the population served within Seattle, it was assumed that the present program of sewer system extension will continue beyond 1960 and that virtually the entire population will be served by 1970.

In the remainder of the study area, the number of persons served can be expected to increase rapidly after 1960 as trunk facilities become available. On the other hand, as private disposal systems are abandoned in the more densely settled areas, new residential construction in fringe areas can be expected to continue and the number of persons dependent on private systems will, by comparison, decline quite slowly. Outside Seattle, the number served is expected to increase from 70,000 in 1960 to 385,000 by 1980. In the same period, the number of persons not served is expected to decline only from 236,000 to 115,000. This outlook could be improved by more effective control of the location of residential development than has heretofore been practiced. It could be improved also by strengthening state laws to provide for abatement of health hazards through the installation of sewers under local improvement district proceedings.

For the study area as a whole, the number of equivalent single-family services is expected to increase from 210,000 in 1960 to 324,000 in 1980. Estimated numbers for intermediate years are given in Table 19-1.

Financing Powers and Limitations of a Metropolitan Agency

General powers and limitations of a metropolitan agency in the State of Washington are defined by the Metropolitan Municipal Corporation Act of 1957 and were described briefly in Chapter 9. With respect to financing sewerage and drainage functions, such a corporation has power:

1. To levy a one-mill one-year tax. Upon appropriate approval of the electorate at the formation election, a general tax levy of one mill upon all taxable property may be authorized for one year only.
2. To assess component cities and counties for supplemental income.

Table 19-1. Projection of Assessed Valuation and Number of Sewerage Services

Year	Study area		Within Seattle[c]			Balance of study area			Study area	
	Low population estimate,[a] 1,000	Assessed valuation,[b] million dollars	Low population estimate, 1,000	Population served,[d] 1,000	Number of services,[e] 1,000	Low population estimate, 1,000	Population served,[d] 1,000	Number of services,[e] 1,000	Population served,[d] 1,000	Number of services,[c] 1,000
1957	864	950	572	522	186	292	56	16	578	202
1960	900	990	575	535	190	325	70	20	605	210
1961	912	1,003	577	540	192	335	90	26	630	218
1962	924	1,016	578	545	194	346	110	31	655	225
1963	936	1,029	579	550	196	357	130	37	680	233
1964	948	1,042	581	555	198	367	150	43	705	241
1965	960	1,055	583	560	200	377	170	49	730	249
1966	971	1,068	584	565	202	387	193	55	758	257
1967	982	1,080	586	570	204	396	216	62	786	266
1968	993	1,092	587	575	205	406	239	68	814	273
1969	1,004	1,104	589	580	207	415	262	75	842	282
1970	1,015	1,116	590	585	209	425	285	81	870	290
1971	1,024	1,126	591	587	210	433	295	84	882	294
1972	1,033	1,136	592	588	210	441	305	87	893	297
1973	1,045	1,146	593	590	211	452	315	90	905	301
1974	1,051	1,156	594	591	212	457	325	93	916	305
1975	1,060	1,165	595	593	212	465	335	96	928	308
1976	1,068	1,174	596	594	212	472	345	99	939	311
1977	1,076	1,183	597	596	213	479	355	102	951	315
1978	1,084	1,192	598	597	213	486	365	104	962	317
1979	1,092	1,201	599	599	214	493	375	107	974	321
1980	1,100	1,210	600	600	214	500	385	110	985	324

[a] *High projection for study area multiplied by percentage ratio of high to low projections for State of Washington, from Table 5-8.*

[b] *At $1,100 per capita.*

[c] *As defined by 1957 limit.*

[d] *By any public sewerage agency.*

[e] *Equivalent single family swelling services, at 2.8 persons per service within Seattle and 3.5 persons per service in balance of study area.*

3. To fix rates and to charge for the use of metropolitan sewerage and drainage facilities.

4. To issue general obligation bonds, upon appropriate vote of the electorate, and to levy an ad valorem property tax to cover principal and interest payments thereon.

5. To issue revenue bonds, without vote of the electorate, payable solely from revenue and special assessments of the utility which they finance.

6. To levy special assessments against property specially benefited, to form local improvement districts therefor, and to issue warrants or bonds payable entirely from such special assessments.

7. To apply for and receive grants-in-aid from federal or other sources.

The foregoing powers are limited by state constitutional amendments and statutes applicable to all municipal corporations. Limitations thus imposed include:

1. Approval of the electorate for the one-mill one-year tax levy and for authorization of general obligation bonds requires a three-fifths majority vote.

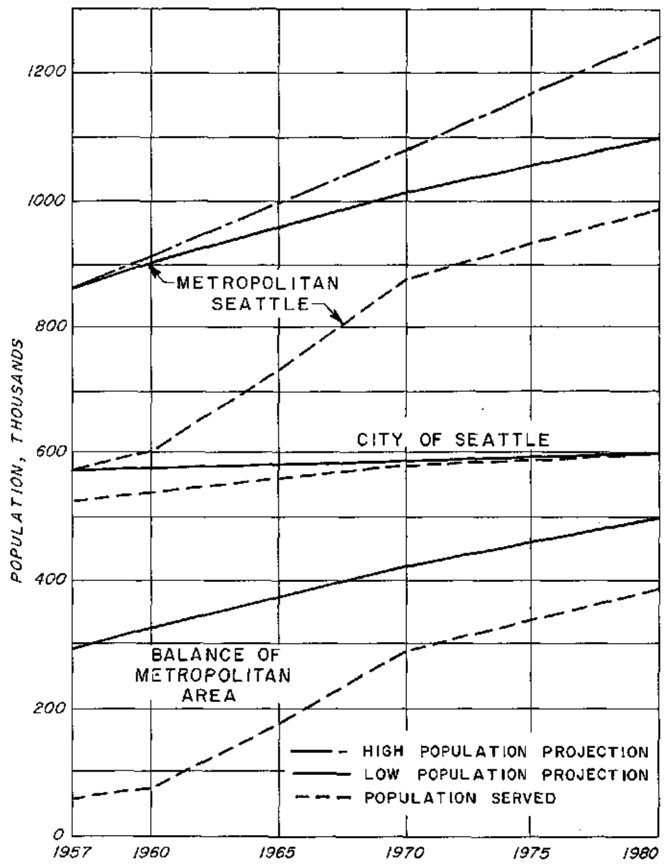

Fig. 19-2. Low Population Projections and Estimate of Population Served by Sewerage System, 1957-1980

Differences in growth rate and percentage of population served by public sewers requires separate projections for Seattle (within present city limit) and for balance of study area.

Further, the number voting must be not less than 40 per cent of the number of votes cast in the last preceding general election.

2. General obligation debt cannot exceed 5 per cent of assessed valuation. Although the Metropolitan Municipal Corporation Act permits deferment of principal repayment on a general obligation bond issue for five years, it requires, in common with other municipal statutes, that the sum of annual principal and interest payments be such that it will be met by equal annual tax levies. While the act does not limit the maximum term of bonds, the 30-year limit for other municipal corporations may be presumed to apply also to a metropolitan agency.

3. The cost of any project to be constructed under special assessment proceedings cannot exceed an amount equal to the true value of the land plus one-fourth the value of improvements on the land. By law, the true value is defined as twice the assessed value, even though the latter in recent years has been only about 20 per cent of actual value. This limit can be exceeded if the property owners involved deposit cash in the amount the project exceeds the limit. In the case of a sanitary sewerage project, the limit can be exceeded if the governing board, by unanimous vote, finds the project necessary to the public health. In any event, assessment proceedings can be nullified by written protest of property owners representing 60 per cent of the value of the assessments. Although such bonds are ordinarily limited to a maximum term of 12 years, the maximum may, under certain conditions, be extended to 22 years.

4. No ad valorem property tax, other than the one-year one-mill initial levy and that required for general obligation bond debt service, is permitted.

Under the above limitations, costs of operation, maintenance and administration of any facilities or services can be met only from service revenue or from supplemental assessments against the component cities and counties. To provide income from a property tax would require either (1) the repeal of a 1934 amendment to the state constitution, which limits ordinary property tax levies to a total of 40 mills, or (2) the raising of assessed valuations to 50 per cent of actual value (as stipulated by the state constitution) and reallocation of allowable levies within the 40-mill limit.

Supplemental Income. Resources available as supplemental income through assessments upon constituent cities and unincorporated portions of county areas are not restricted by the act. Such assessments, nevertheless, are under the control of the governing council, whose members represent the several local agencies. In general, assessments would be limited normally to amounts required for (1) administrative costs not chargeable to specific functions, (2) minor budgetary deficits, and (3) services which supplant a municipal service and for which other means of reimbursement are not available.

Property Taxes and General Obligation Bonds. An initial one-mill one-year levy would, in 1960, raise $990,000 if applied over the entire study area. Maximum general obligation bonding capacity would be $49.5 million in 1960 and would increase to $60.5 million by 1980.

Service Charges and Revenue Bonds. Resources with respect to revenue bonds are limited only by the amount of service charge which the public will accept as necessary to correct present deficiencies and to provide adequately for the future. This in turn depends first upon public recognition of the sewerage and drainage problems, and second upon public assumption of the responsibility for corrective action. While a determination of the upper limit of the resources which may thus be made available is beyond the scope of an engineering report, the general magnitude can be indicated for illustrative purposes.

In considering the use of service charges, it is important to bear in mind the fact that the revenue therefrom must also support operation, maintenance and administrative costs of the central sewerage facilities. After allowing for those costs, each additional dollar per month per equivalent single-family residential service would, in total, support capital improvements of about $2.5 million per year by 1960, and of $3.9 million per year by 1980. For 30-year revenue bonds at 5 per cent interest and equal annual debt service payments, the revenue derived from one additional dollar per month per service would support bond issues in the amount of $39 million in 1960 and of $60 million by 1980.

Special Assessments. In general, resources available through special assessments under local improvement district proceedings are not applicable to the financing of metropolitan sewerage works or major drainage facilities.

Federal Aid. Grants may be available for construction of interceptor sewers, treatment works, and outfalls under Public Law 660 of the 84th Congress, entitled "Water Pollution Control Act Amendments of 1956". These grants are limited to 30 per cent of the cost of a project, or to $250,000, whichever is the lesser, and are to be obtained from appropriations of $50 million per year over a 10-year period. Within each state, not more than one-half the annual allocation to the state can be approved for cities having a population in excess of 125,000.

Since the annual allocation to the State of Washington has been approximately $750,000, only one of the three cities or metropolitan areas having a population in excess of 125,000 is eligible for a maximum grant each year. It is not likely, therefore, than any one of the three would receive such a grant in two successive years.

Actually, the federal aid program is geared primarily to the needs of small cities undertaking necessary construction of a single project. How it will be administered in the case of a metropolitan project with construction undertaken as a continuing program has not yet been revealed. In any event, since the awarding of such grants depends upon annual appropriations by Congress, their availability cannot be assumed in developing a financing program for the central sewerage project.

BASIS FOR DEVELOPMENT OF FINANCING PROGRAMS

Before undertaking the development of alternative programs for financing the sewerage and drainage projects, it is desirable to set forth the basic considerations and data which govern such programs. These include: (1) fundamental concepts of a metropolitan financing program; (2) characteristics of various types of bond issues and income sources as related to these concepts; (3) procedures with respect to reimbursement of existing sewerage agencies for facilities to be abandoned or incorporated in the metropolitan projects; (4) procedures with respect to other administrative operations which may affect financing programs; and (5) annual requirements for capital and for operation and maintenance revenue as affected by the timing of the construction program.

Concepts of Metropolitan Financing

In undertaking any major public works project, the financing program must not only provide adequate funds, but should accomplish this in a manner which, insofar as practicable, is equitable to those bearing the burden. This is particularly important in the case of metropolitan sewerage and drainage projects where the costs are borne by a number of different cities and local government agencies. Three objectives must be considered. These are:

1. Costs of the projects should, in general, be borne by those benefited.
2. Total costs of financing should be kept as low as possible.
3. Interference with other tax-supported needs should be held to a minimum.

Unfortunately, these objectives are not all compatible. For example, the least total outlay for interest charges would obtain with a short-term bond issue. Under such a program, the costs would be borne largely by present users of the system, whereas the facilities to be constructed would serve for many years in the future. Further, high amortization costs of short-term general obligation bonds would interfere with other critical community needs supported by taxation. For these reasons, none of the three objectives can be met fully and compromises have to be made.

Benefits of a Metropolitan Sewerage and Drainage Program

If the cost of a central sewerage system is to be borne by those benefited, it is well to consider what benefits are derived and who gains thereby.

As an approach to the question of benefits, it is desirable to list the three principal functions of a public sewerage system. These functions are (1) to remove sewage and wastes from individual premises, (2) to transport it to a suitable location for treatment and disposal, and (3) to treat it for final disposal. In a metropolitan system, the first function is a local responsibility while the second and third are the responsibility of the metropolitan agency. Insofar as the individual householder is concerned, his direct benefit is the same regardless of whether the transmission, treatment, and disposal facilities are adequate or non-existent. In that respect, a sewerage system is unlike a water, electric or gas utility wherein the adequacy of supply and transmission facilities directly affect the quality of the service.

The benefits of sewerage facilities to be provided under a metropolitan agency, as distinct from those provided by local facilities, pertain to the protection both of public health and of the natural resources of the entire area. They accrue equally to all residents, whether or not served by public sewers. Additionally, metropolitan facilities are designed not only to serve the present population but to provide necessary capacity for future generations. It is obvious, therefore, that it is entirely equitable and proper to spread the cost of a metropolitan system equally over the widest possible base in terms of area and time.

Equal areal distribution of the cost implies that the property tax or service charge be identical throughout the metropolitan area regardless of the cost of serving the individual sewerage areas. For example, the principal trunk sewer, treatment, and disposal facilities of several of the small independent systems discharging to Puget Sound cost more on a per capita or acreage-served basis than those of the central system. Nevertheless, such facilities would be provided without a differential in cost to residents of the independent areas. Since the benefits so derived, namely protection of public waters and beaches, accrue to the public at large, this procedure is amply justified.

Trunk drainage facilities, while not necessary to protect public health or the esthetic and economic value of surface waters, are nevertheless essential to urban development of their tributary areas. Such facilities not only prevent property damage and inconvenience in the lower parts of a watershed but provide in the upland parts for installation of local drainage on a scale necessary to obtain complete development. In those upland areas where the topography, as a result of glaciation, hampers natural drainage, urban development either may be extremely limited or may be halted until trunk drains are made available.

Obviously, direct benefits of trunk drainage accrue primarily to residents and property owners, both present and future, in an entire watershed. But indirect benefits of considerable significance accrue to the metropolitan area as a whole. Of these, the protection of main thoroughfares against flooding and damage is the most obvious. Less tangible but of greater significance is the fact that provision of trunk drainage facilities allows orderly development of areas which otherwise would remain dormant or become substandard or slum neighborhoods. Orderly urban development results in new residential, commercial, and industrial areas which become an asset rather than a liability to the rest of the metropolitan community. It is entirely proper, therefore, to recognize the metropolitan interest in establishing a financing pattern for trunk drainage facilities.

Metropolitan Financing by Bond Issues

Financing by means of three types of bond issues, namely general obligation, revenue, and special assessment, is authorized under the Metropolitan Municipal Corporation Act. Each of these differs with respect to its suitability for metropolitan use.

General Obligation Bonds. Because they are backed by the total assets of a community and, in common with other local government bonds, are exempt from federal income tax, general obligation bonds bear the lowest interest rate of any type of long-term security. When used to finance projects of general public benefit, they have the further advantage of spreading the cost over the entire community on a generally equitable basis through an ad valorem property tax. In addition, debt service costs can be met in part or in whole from other sources of revenue, including service charges. General obligation bonds, therefore, meet two principal objectives, namely, equitable cost distribution and minimum cost financing.

Disadvantages of general obligation bonds include the three-fifths majority vote required for authorization and the indebtedness limitation of 5 per cent of assessed valuation. In the latter case, use of such bonds for sewerage purposes may seriously deplete borrowing capacity for projects, such as metropolitan storm drainage and parks, which cannot be financed by other means.

During the past year or two, interest rates on general obligation issues exhibited a general increase but began to taper off somewhat in the spring and early summer of 1958. Although a rate of 4 per cent per year was assumed for the purpose of the present study, the current trend indicates that lower rates may again prevail in the near future.

Revenue Bonds. Revenue bonds offer the advantages of ease of authorization, unlimited bonding capacity, and noninterference with tax-supported governmental functions. In addition, since bond terms are governed only by their effect on the marketability of an issue, considerable flexibility with respect to repayment terms can be arranged.

Interest rates on revenue bond issues vary over a considerable range and depend upon many factors. These include the financial stability and reserves of the issuing agency, its past and anticipated revenues and expenditures, and, of course, market conditions at the time of sale. In general, the interest rate is three-fourths to one percentage point higher than that for general obligation bonds of the same agency. For the purpose of this report, a rate of 5 per cent was assumed, although present conditions indicate a probable return to somewhat cheaper money.

In addition to the higher interest rate, revenue bonds have an inherent disadvantage in the limited scope of the income which can be applied to the financing of central sewerage and other projects of general public benefit. Under present law, except as noted below, revenue can be derived only from those directly served. Hence, those who are served initially must bear the cost of the capacity which must be provided both to accommodate new areas and to allow adequately for future growth in population and industry.

A second factor leading to inequitable cost distribution in revenue bond financing is the necessity, in order to market such bonds, of maintaining revenues high enough to produce surplus income in an amount equal to fifty per cent of the annual debt service. If not siphoned off to support other governmental activities, the surplus can be used directly for construction, for retirement of callable bonds, or for deposit in a sinking fund established for bond redemption. In all of these, the end result is the same, that is, the capital cost is paid off in a much shorter time than called for by the bond terms and those initially served assume a disproportionate share of the total financial burden.

Under existing state law, the scope of sewerage revenue bond support can be broadened if water supply and sewerage functions are merged in a single utility. In that case, those served by the water system also can be charged for sewerage service regardless of whether they are actually connected to a sewerage system. No consideration is given herein to such a possibility.

As provided for under the enabling legislation, a metropolitan municipal corporation may establish rates and charges for the use of metropolitan drainage facilities. Such a corporation, therefore, could charge cities and other local agencies for drainage service. But there is no practicable way whereby this cost can be passed along to individual property owners on a service charge or tax basis. Moreover, the boundaries of political entities do not coincide with topographic features which define watersheds for drainage purposes. Fundamentally, then, revenue bonds are not appropriate for financing construction of trunk storm drainage facilities. Although the financing of facilities required for storm water separation is a possible exception, separation is primarily a local rather than a metropolitan problem.

Special Assessment Bonds. Local sewerage and drainage collection facilities are financed normally by special assessment bonds of a local improvement district. This procedure, in conjunction with funds from other sources, has been generally employed in Seattle to finance construction of combined trunk sewers.

For separate trunk sewers or drains of the magnitude proposed for the metropolitan system, the special assessment procedure has distinct disadvantages. Experience in general indicates extreme difficulty in obtaining sufficient unanimity among the property owners in a large service area or watershed to permit such a procedure.

Administrative, legal and engineering costs involved in special assessment proceedings are high and interest rates on the bonds are higher than those on general obligation or revenue bonds. And finally, although the proceedings provide for a presumably fair distribution of costs among those benefited, the short term allowed for the life of the bonds prevents an equitable spread of the costs to future residents. For these reasons, no consideration is given to the use of special assessment bonds for financing any of the metropolitan sewerage or drainage facilities.

Reimbursement for Existing Sewerage Facilities

Under the enabling act, a metropolitan municipal corporation is authorized to acquire or use existing facilities and properties of component cities and districts. Acquisition, lease, or contracts for joint use are to be made on such terms as may be fixed by agreement between the legislative body of the local agency and the metropolitan council.

In developing the metropolitan sewerage system recommended in Chapter 16, certain existing facilities will be incorporated therein while others, including some recently constructed, will be abandoned. It is necessary, therefore, to consider (1) what general policy of reimbursement to the local agencies would be equitable, and (2) what effect this would have on the financing program.

Policy with Respect to Reimbursement. The value of a public sewerage facility lies in its capacity both to provide service to those tributary to it and to protect public health and natural resources. It cannot be moved to another location to serve others, nor is it a source of monetary profit. Transfer of ownership from one public body to another, therefore, is simply a paper transaction and, as in the case of local sewers and streets in an annexation proceeding, normally is effected without payment. Problems of reimbursement arise only where the burden of financing prior obligations, such as debt service on outstanding bonds, would result in inequities. For example, if the metropolitan sewerage project included a trunk sewer through each of two communities and one of these had recently constructed a trunk suitable for incorporation in the metropolitan system, it would be inequitable to require that community to carry both the burden of debt service costs for that trunk plus its full share of the metropolitan costs.

A similar situation prevails in the case of the treatment plants which now discharge to Lake Washington and are scheduled to be abandoned upon construction of central facilities. Policies with respect to reimbursement, therefore, should be based upon financial considerations rather than on an appraised value of the existing facility. In other words, the metropolitan agency, when assuming responsibility for operation and maintenance of a facility, should also assume responsibility for its capital costs.

Reimbursement of applicable capital costs may be made in a variety of ways, including:

1. A cash payment in an amount which, with accumulated interest, will pay annual debt service on outstanding bonds.
2. Issuance of refunding bonds to replace original bonds.
3. Assumption of responsibility for outstanding bonds.
4. A lease-purchase agreement under which annual payments are made in an amount equal to debt service requirements.

Choice of the method by which reimbursement is made will depend upon current interest rates, bond terms, approval of bond holders and similar factors which vary with time and place. For the purpose of this report, it is assumed that new capital will be required by the metropolitan agency in an amount equaling the approximate outstanding indebtedness for the principal sewerage works either to be incorporated in the metropolitan system or to be abandoned because of it. Bonds which have been issued to finance such works may vary as to term, but the maximum is thirty years. A 30-year life is therefore assumed equitable for the purpose of determining reimbursements value.

Some facilities, such as Seattle's Alki Point sewage treatment plant and the additions to the Lake City plant, have been financed on a cash basis, rather than through bonds. These cases, as well as others involving significant capital improvements financed from income, are treated the same as those financed by bonds.

Effect of Reimbursement. Reimbursement of local agencies for sewerage works to be incorporated or abandoned will tend to equalize sewerage costs throughout the metropolitan area. While reimbursement will not add to the total indebtedness of the metropolitan area, funds required for that purpose must be available to the metropolitan agency.

To estimate the amount required for reimbursement, information with respect to construction costs and dates was obtained from the participating cities and sewer districts (Table 19-2). Since most of the figures so obtained were actual contract costs, an arbitrary allowance of 10 per cent has been added for engineering.

Reimbursement values given in the table are equivalent to a depreciated value based on a 30-year life and on the straight-line method applied as of 1960. For that reason, facilities constructed prior to 1931, such as Seattle's North Trunk, would be transferred without reimbursement and are not listed.

The initial cost of all sewerage works listed amounts to $12,402,000. Their total reimbursement value is $9,419,000, of which $6,276,000 is for facilities to be incorporated and $3,143,000 is for facilities, mostly treatment plants, to be abandoned. It is of interest to note that 73 per cent of the reimbursement value, or $6,867,000, is for Seattle sewerage works, including Lake City Sewer District. On a per capita basis, reimbursement values are roughly the same both for Seattle and for sewered portions of the suburban area.

Financial Requirements

Before any programs of financing sewerage and drainage facilities can be considered, it is necessary to ascertain all costs which will be incurred and when they will accrue. In addition to capital requirements, these costs include those of operation, maintenance and administration. When they will accrue will depend both on the rate of progress in stage construction and on the growth of population and industry.

Two possible rates of stage construction of sewer-

Table 19-2. Reimbursement Value of Principal Sewerage Works to be Abandoned or Incorporated in Metropolitan System

Sewerage agency	Type of works[a]	Dates of construction	Initial cost,[b] $1,000	Reimbursement value, 1960,[c] $1,000		
				Incorporate[d]	Abandon[d]	Total
Bellevue Sewer District	STP	1954, 1957	469		391	391
Bryn Mawr-Lake Ridge Sewer District	STP	1952	218		160	160
East Mercer Sewer District		1954	73		58	58
Mercer Island Sewer District	PS(2), FM(3)	1957	195	157	18	175
Lake Hills Sewer District	STP	1955	33		28[e]	28
Val-Vue Sewer District	STP	1955	88		73	73
Sewerage and Drainage District No. 4	STP	1943	88		38	38
Southwest Suburban Sewer District	TS, PS(2), STP, O	1955	1,112	927		927
Auburn	STP	1950	74		49[f]	49
Issaquah	STP	1940	22		7	7
Kent	STP	1954	297		234	234
Kirkland	STP	1943-51	375		232	232
Renton	STP	1943-53	335		180	180
Seattle[g]						
Lake City Sewer District	TS, STP, O	1949-57	2,621	594	1,489	2,083
Southwest Lake Washington	TS, PS, STP	1932-52	1,755	703	106	809
Lake Union	TS	1935-58	524	371		371
Laurelhurst	PS(2), TS	1935	159	27		27
Southwest (Alki Point)	TS, PS(3), FM, STP, O	1953-59	3,709[h]	3,416		3,416
Greenwood	TS, STP, O	1949	255	81	80	161
Total			12,402	6,276	3,143	9,419

[a] *TS—trunk sewers; PS—pumping stations; FM—force main; STP—sewage treatment plant; O—outfall.*

[b] *Contract cost plus 10 per cent for engineering.*

[c] *Based on straight-line depreciation, 30-year life.*

[d] *During Stage I, except as noted.*

[e] *Stage II.*

[f] *Stage III.*

[g] *North Beach and Roxbury Heights sewage treatment plants also to be abandoned. Construction costs not available.*

[h] *Including estimated cost of works to be completed before 1960.*

age works are considered, both beginning in 1960. The first assumes that Stage I construction can be completed in five years, or by 1965, and that Stage II construction would begin immediately thereafter and would be completed by 1980. This program is hereinafter termed the 5-year Stage I construction program. The second program assumes that Stage I construction would require ten years for completion and that Stage II construction would be completed by 1980. This second program is hereinafter termed the 10-year Stage I construction program.

If the higher rate of population growth utilized for design purposes is assumed to prevail, certain facilities scheduled for early Stage II construction will be required before 1970 and possibly before 1965. It is obvious, however, that a financing program based on a low rate of population growth will, if a higher rate prevails, yield greater revenue than anticipated. Additional revenue thus obtained would be available for the construction of facilities needed earlier than herein programmed.

For reasons stated previously in Chapter 17, a stage construction program has not been outlined for trunk drainage facilities. As a consequence, discussions involving the financing of these works are limited to general principles.

Capital Funds. Capital funds will be required for the stage construction program, for reimbursement of local agencies for existing facilities, and, as discussed in Chapter 16, for construction of temporary treatment facilities. These requirements are summarized in Tables 19-3 and 19-4 for the two programs of stage construction.

It is assumed that the sum of $9,419,000 for reimbursement purposes will be required in 1960 upon acquisition of all existing facilities. It is assumed also that, if reimbursement is made in annual rather than lump sum payments, these payments would be approximately equal in total to the annual debt service on an equivalent issue of bonds. It is further assumed that the sum of $2 million allowed for construction of

Table 19-3. Summary of Capital Fund Requirements and Operating Costs for Five-Year Stage I Sewerage Program

	Stage I	Stage II		
	1960-64	1965-69	1970-74	1975-79
Capital funds required, total for period, $1,000				
Construction, permanent[a]				
Sewers	44,433	9,240	9,271	9,270
Pumping stations	4,609	445	1,135	1,271
Sewage treatment plants	26,993	550	2,697	800
Outfalls	5,180	467	271	
	81,215	10,702	13,374	11,341
Construction of temporary plants[a]	2,000			
Acquisition of existing facilities[b]	9,419			
Total	92,634	10,702	13,374	11,341
Average annual cost of operation and maintenance, $1,000				
Permanent facilities	464	1,010	1,186	1,324
Temporary facilities[c]	355	161	151	119
Administration[d]	110	130	150	150
Total	929	1,301	1,487	1,593

[a]*From Tables 16-1 and 16-12.*

[b]*From Table 19-2.*

[c]*Including existing works to be abandoned.*

[d]*Administrative, legal, and engineering costs not chargeable to specific operations.*

Table 19-4. Summary of Capital Fund Requirements and Operating Costs for Ten-Year Stage I Sewerage Program

	Stage I, thousand dollars					Stage II, thousand dollars				
	1960-61	1962-63	1964-65	1966-67	1968-69	1970-71	1972-73	1974-75	1976-77	1978-79
Capital funds required, total for period										
Construction, permanent[a]										
Sewers	9,126	8,716	8,716	8,760	8,714	5,915	5,301	5,504	5,761	5,300
Pumping stations	–	304	1,063	2,336	1,307	758	304	333	1,192	264
Sewage treatment plants	2,901	7,899	3,949	8,403	3,841	550	350	709		2,438
Outfalls	781		318	235	3,846	467		271		
	12,808	16,919	14,046	19,734	17,708	7,690	5,955	6,817	6,953	8,002
Construction of temporary plants[a]	2,000									
Acquisition of existing facilities[b]	9,419									
Total	24,227	16,919	14,046	19,734	17,708	7,690	5,955	6,817	6,953	8,002
Average annual cost of operation, maintenance and administration										
Permanent facilities	326	350	485	570	1,022	1,101	1,164	1,238	1,318	1,369
Temporary facilities[c]	406	417	312	315	178	204	119	118	115	126
Administration[d]	100	110	120	130	140	150	150	150	150	150
Total	832	877	917	1,015	1,340	1,455	1,433	1,506	1,583	1,645

[a]*From Tables 16-1 and 16-12.*

[b]*From Table 19-2.*

[c]*Including existing works to be abandoned.*

[d]*Administrative, legal, and engineering costs not chargeable to specific operations.*

temporary treatment facilities will be needed in 1960.

Funds for the 5-year Stage I construction program (Table 19-3) are assumed to be needed in more or less equal amounts per year within each 5-year increment. For the 10-year Stage I program, however, the variation in annual requirements, particularly during the first stage, necessitates the more detailed breakdown in 2-year increments (Table 19-4).

Operation and Maintenance. Annual costs of operation and maintenance include those of (1) new facilities as constructed and placed in operation, (2) existing facilities incorporated in the metropolitan system, and (3) treatment works, both existing and temporary, which must be continued in operation until replaced by permanent facilities. These costs (Tables 19-5 and 19-6) are based on the pertinent cost curves and factors given in Chapter 13 and take into account the average loading on pumping stations and treatment facilities during each incremental period. They do not include operation and maintenance of local collection facilities, either existing or to be constructed hereafter.

Fiscal and Administrative Costs. Regardless of whether capital requirements are obtained from revenue or general obligation bonds, service charges will be necessary to meet operation and maintenance costs of the metropolitan system. Practically all the local sewerage agencies now collect service charges and would continue to do so for maintenance of the local systems. To avoid duplication in billing expense, it is assumed that the metropolitan costs would be charged to the local agencies which, in turn, would add that cost to their local billings. No allowance is made, therefore, for service charge collection expense to the metropolitan agency.

Sewerage service may, in the future, be extended in unincorporated areas by local improvement district proceedings under sponsorship of the metropolitan

Table 19-5. Revenue Bond Financing of Five-Year Stage I Sewerage Program

	Stage I	Stage II		
	1960-64	1965-69	1970-74	1975-79
For period:				
Capital funds required,[a] $1,000	92,634	10,702	13,374	11,341
Revenue bonds issued, $1,000	80,000[b]			
Average number of services,[c] 1,000	225	265	297	315
Monthly service charge, dollars	3.00	2.90	2.70	2.50
Average annual amounts, $1,000				
Operating income	8,100	9,222	9,623	9,450
Debt service	3,643	5,204	5,204	5,204
Operation and maintenance[a]	929	1,301	1,487	1,552
State tax[d]	65	74	77	76
Total costs	4,637	6,579	6,768	6,832
Debt service coverage[e]	1,822	2,602	2,602	2,602
Net income	1,641	41	253	16
Total surplus	3,463	2,643	2,855	2,618
Construction from surplus	2,523	2,140	2,675	2,268
Net to reserve fund	940	503	180	350
Cumulative amounts, $1,000				
Capital fund requirements	92,634	103,336	116,710	128,051
Capital from income	12,634	23,336	36,710	48,051
Capital from revenue bonds	80,000	80,000	80,000	80,000
Bonds outstanding[f]	76,840	69,320	61,780	50,120
Reserve fund[f, g]	4,700	7,215	8,115	9,865

[a] *From Table 19-3.*

[b] *Four $20,000,000 issues of 30-year bonds at 5 per cent, issued in 1960, 1961, 1962, and 1963, with uniform annual interest and redemption payments.*

[c] *All classes, expressed as equivalent single-family residences, from Table 19-1.*

[d] *At 0.8 per cent of gross income.*

[e] *Fifty per cent of bond interest and redemption.*

[f] *At end of period.*

[g] *Exclusive of interest earned.*

Table 19-6. Revenue Bond Financing of Ten-Year Stage I Sewerage Program

	Stage I				
	1960-61	1962-63	1964-65	1966-67	1968-69
For period:					
Capital funds required,[a] $1,000	24,227	16,919	14,046	19,734	17,708
Revenue bonds issued,[b] $1,000	18,000	10,000	7,000	15,000	14,000
Average number of services,[c] 1,000	214	229	245	260	277
Monthly service charge, dollars	2.50	2.50	2.50	2.50	2.40
Average annual amount, $1,000					
Operating income	6,420	6,870	7,350	7,800	7,868
Debt service	1,171	1,821	2,277	3,253	4,163
Operation and maintenance[a]	832	877	917	1,015	1,340
State tax[d]	52	55	59	63	63
Total cost	2,055	2,753	3,253	4,331	5,566
Debt service coverage[e]	586	911	1,139	1,627	2,082
Net income	3,779	3,206	2,958	1,842	220
Total surplus	4,365	4,117	4,097	3,465	2,302
Construction from surplus	3,114	3,460	3,523	2,367	1,854
Net to reserve fund	1,251	657	574	1,098	448
Cumulative amounts, $1,000					
Capital fund requirements	24,227	41,146	55,192	74,926	92,634
Capital from income	6,227	13,146	20,192	24,926	28,634
Capital from revenue bonds	18,000	28,000	35,000	50,000	64,000
Bonds outstanding	17,700	27,000	32,930	46,420	58,370
Reserve fund[f, g]	2,502	3,816	4,964	7,160	8,056
	Stage II				
	1970-71	1972-73	1974-75	1976-77	1978-79
For period:					
Capital funds required,[a] $1,000	7,690	5,955	6,817	6,953	8,002
Revenue bonds issued,[b] $1,000	4,000	1,000	2,000	2,000	3,000
Average number of services,[c] 1,000	292	299	306	313	319
Monthly services charge, dollars	2.40	2.40	2.40	2.40	2.40
Average annual amount, $1,000					
Operating income	8,410	8,611	8,813	9,014	9,187
Debt service	4,423	4,488	4,619	4,749	4,944
Operation and maintenance[a]	1,455	1,433	1,506	1,583	1,645
State tax[d]	67	69	70	72	74
Total cost	5,945	5,990	6,195	6,404	6,563
Debt service coverage[e]	2,212	2,244	2,310	2,375	2,472
Net income	243	377	308	235	152
Total surplus	2,455	2,621	2,618	2,610	2,624
Construction from surplus	1,845	2,478	2,409	2,477	2,501
Net to reserve fund	610	143	209	133	123
Cumulative amounts, $1,000					
Capital fund requirements	100,324	106,279	113,096	120,049	128,051
Capital from income	32,324	37,279	42,096	47,049	52,051
Capital from revenue bonds	68,000	69,000	71,000	73,000	76,000
Bonds outstanding[f]	59,700	57,570	56,380	54,700	52,750
Reserve fund[f, g]	9,276	9,562	9,980	10,246	10,492

[a] *From Table 19-4.*

[b] *Serial bonds, 30-year term at 5.0 per cent interest, with uniform annual interest and redemption payments.*

[c] *All classes, expressed as equivalent single-family residences, from Table 19-1.*

[d] *At 0.8 per cent of gross income.*

[e] *Fifty per cent of bond interest and redemption.*

[f] *At end of period.*

[g] *Exclusive of interest earned.*

agency and without formation of sewer districts. For such areas, direct billing by the metropolitan agency would be necessary and the cost thereof, along with that of local sewer maintenance, would have to be added to the metropolitan charge.

Another expense in connection with service charges is a utility gross earnings tax of 0.8 per cent imposed by the state. The amount thereof will vary with the type of financing and is included later in each alternative program.

Parenthetically, the imposition in Washington of state and city taxes on the gross earnings of a sewerage utility is an application of the principle that a publicly owned utility should be taxed an amount equivalent to what it would pay if privately owned. Because privately owned sewerage utilities are practically nonexistent, there is no real basis for a tax on this type of utility. Further, few sewerage systems are actually administered as true revenue-financed utilities. Subsidies in the form of property tax support and state and federal grants have been necessary in many instances to bring about the construction of treatment facilities which are required both to protect public waters and to maintain environmental sanitation at a safe level. In view of the expressed intention of the state to encourage construction of sewerage facilities as a matter of general welfare, the imposition of a gross earnings tax on sewerage utilities is, to say the least, contradictory. Amendment of pertinent state legislation is indicated. Furthermore, if present legislation would permit component cities or counties to tax the sewerage service revenue of a metropolitan agency, the metropolitan municipal corporation act should be amended to prohibit such taxation.

Finally, allowance must be made for fiscal and administrative costs not included in the foregoing costs of construction, operation, and maintenance. While the latter include engineering, supervision, and overhead costs directly attributable to these functions, provision must be made for:

1. Expenses of the governing board of the metropolitan agency.
2. General expenses, such as for audits, annual reports, and fiscal planning.
3. Legal expenses with respect to amendments of legislation affecting the metropolitan corporation as a sewerage agency.
4. Expenses of bond elections and sales of bond issues.
5. Preliminary engineering planning, including such revisions of the plans herein recommended as are made necessary by changing conditions.
6. Monitoring studies to determine the effectiveness of the sewage treatment and disposal program.
7. Other miscellaneous expenses.

To meet the expenses listed above, an allowance of $120,000 is provided for the first two years. This allowance is increased to $150,000 per year by the seventh year.

FINANCING THE SEWERAGE PROGRAM

Two general plans are presented for financing the sewerage program, one utilizing revenue bonds and the other general obligation bonds. For each of them, two rates of progress are considered for stage construction of the metropolitan system. In presenting these plans, the purpose is to indicate the general magnitude of debt service and total annual costs which are thus incurred and the approximate service charges and tax levies needed to meet these costs.

No attempt has been made to refine bond terms and the reserve fund arrangements which will make it possible to arrive at the lowest possible costs. This is the function of financial consultants who are experienced in the municipal field and are properly familiar with bond market conditions. Costs given for the two plans, therefore, represent the upper limits of an undetermined range within which the sewerage program can be financed. These costs are expressed in terms of 1958 price levels and are subject to adjustment on the basis of further changes in the value of the dollar.

Revenue Bond Financing

The procedures involved in revenue bond financing of the recommended sewerage program can be illustrated by means of two specific examples. Of these, the first provides for construction of Stage I facilities over a period of five years beginning in 1960. This is considered to be the minimum period in which the necessary engineering and construction can be undertaken. The second example calls for construction of Stage I facilities over a period of ten years from 1960 to 1969 inclusive. In both examples, Stage II construction would follow Stage I immediately and would be completed by 1979.

Five-Year Example. In the 5-year Stage I example (Table 19-5), initial capital requirements would be met by four issues of serial revenue bonds, each for $20 million and each having a term of thirty years. Under such a program, the balance of the first-stage capital requirements, along with those of Stage II, would come from the surplus revenue which must be collected to meet bond requirements. In addition, a reserve fund would be accumulated from the surplus. This fund would amount to about 5 per cent of the total capital expenditure by the end of the first 5 years and to almost 8 per cent by the end of Stage II construction in 20 years.

Although a total reserve of almost $10 million might appear to be excessive, it actually would amount to only 5.5 per cent of the total revenue in the 20-year period. In any case, an adequate reserve would be required to protect the revenue bonds and thus to assure a minimum rate of interest. A reserve would be useful also in that it would provide working capital for the construction program as well as funds for replacements, minor improvements, and contingencies.

To support the construction program and to provide necessary funds for operation and maintenance of the sewerage system, the average annual revenue requirement will range from $8.1 million in the first five years to $9.45 million in the 16th to 20th years. Unless state laws are amended, the gross earnings tax will average $73,000 per year and will total $1.47 million for the 20-year period. In other words, the total revenue requirement for the 20 years will amount to $181,975,000. Monthly service charges indicated as necessary to produce that income range from $3.00 for the first period to $2.50 for the last period.

Fig. 19-3 illustrates the status of capital funds throughout the 20-year period. A total of $128 million will be expended for construction, of which $48 million will be derived from service charges. Of the $80 million in revenue bonds, about $30 million will be retired by 1980 and another $10 million will be offset by the reserve fund. On that basis, the net debt will be approximately $40 million. Due to the large proportion of construction financed from income, almost $88 million of the total construction cost will be paid off in the 20-year period.

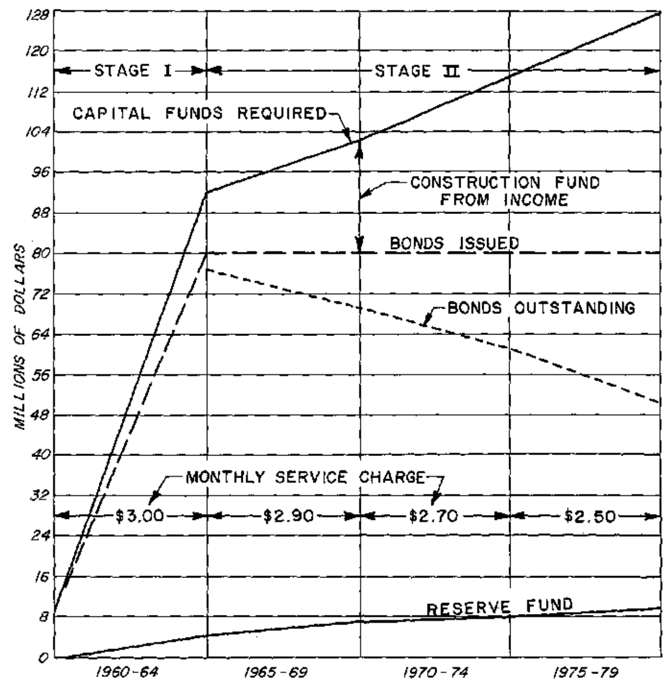

Fig. 19-3. Cumulative Funds, Revenue Bond Financing of Five-Year Stage I Sewerage Program

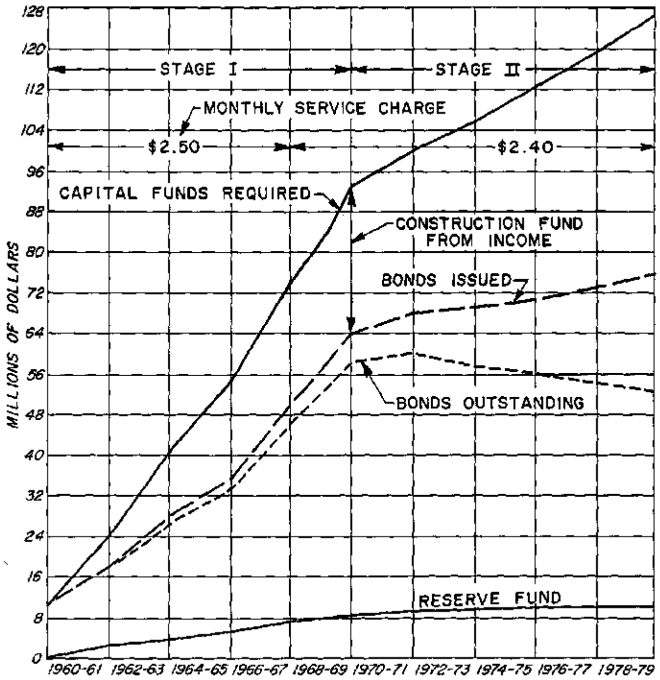

Fig. 19-4. Cumulative Funds, Revenue Bond Financing of Ten-Year Stage I Sewerage Program

Total interest charges to maturity of the four bond issues will amount to about $76 million. That amount, if applied to a 30-year bond issue of $128 million, would represent an interest rate of 3.4 per cent. In all, therefore, the 5-year program would enable relatively low cost financing but would entail a disadvantage in the spread of costs with respect to time.

Ten-Year Example. Under a 10-year program of revenue bond financing (Table 19-6), surplus income again would be used for construction purposes and would thus reduce the total bond issue requirement. During the first stage construction program, five issues totaling $64 million would be sold. During the second stage, annual requirements for construction would be greater than under the 5-year program and would have to be met by the issuance of an additional $12 million in revenue bonds. As in the first example, a reserve fund of approximately $10 million would be accumulated during the 20-year period.

Annual revenue required both to support the construction program and to operate and maintain the system would range from about $6.4 million in 1960 to about $9.2 million in 1979. Reduced costs in the first ten years, as compared to the first example, would result from the deferred construction schedule, which affects not only debt service but operation and maintenance costs as well.

As illustrated in Fig. 19-4, the value of outstanding bonds would reach a peak of slightly less than $60 million at the end of the twelfth year and thereafter would decline to $52.75 million by the end of the 20-

year period. With the accumulated reserve taken into account, the net debt at the end of the second stage of construction would amount to $42 million, or only $2 million more than that estimated under the first example.

Although it would delay construction of facilities needed now to protect Lake Washington and the Puget Sound beaches, the 10-year Stage I program would require a significantly lower revenue during the first ten years than the 5-year program. In other respects, this second example is similar to the first and results in a poor spread of costs with respect to time.

Sinking Fund Alternative. As an alternative to financing part of the construction program from surplus income, revenue bonds could be issued in amounts sufficient to cover all construction costs. Under this alternative, the surplus income could be placed in a sinking fund to retire callable bonds in advance of maturity. Provided the interest earned on the sinking fund is equivalent, or nearly so, to that paid on the bond issues, the net result with respect to financing cost would be practically identical to the examples involving construction from surplus funds. The rate of debt retirement, however, would be more rapid and, as a consequence, annual costs would be higher.

Sewer Service Charge. Present costs of local sewerage agencies for debt service, as well as those for construction, operation and maintenance of sewage works and certain trunk sewers would be assumed by the metropolitan agency and are included in the total annual costs estimated for that agency. In such an event, neither the assumed costs nor the metropolitan service charges would necessarily be added to the costs already being borne by and the service charges being paid to the local agencies. In cities and districts outside Seattle, local sewerage costs would be reduced essentially to those involved in operation, maintenance, and debt service of the collection system. While a detailed analysis of the effect on costs within these individual agencies is premature at this time, a cursory examination indicates that the sum of metropolitan and local charges would range from a nominal increase in present costs to an actual reduction in a few cases.

Within the city of Seattle, financing of metropolitan facilities is complicated by the need for extensive and correspondingly costly separation of combined sewers. In fact, all presently available revenue could be expended on separation and relief sewer construction for many years to come. If that were the case, the cost of metropolitan sewerage facilities would then have to be financed entirely from newly acquired revenue. However, by confining the separation program to the areas most seriously affected by storm flow conditions, a portion of the present revenue from sewer service charges could be utilized for participation in the metropolitan program.

Pending formation of a metropolitan agency, Seattle could finance a substantial portion of the metropolitan facilities lying within its boundaries, plus some of the most urgent separation projects. Although the first phases of these projects could be financed from present revenue, a substantial increase in sewer service charges will be required in Seattle within a few years regardless of whether a metropolitan agency is established.

General Obligation Bond Financing

At present, financing of the entire sewerage program by general obligation bonds is not legally possible. This is because bonded indebtedness cannot exceed 5 per cent of assessment valuation and also because assessed values are considerably below the legal limit of 50 per cent of true value. Nevertheless, examples of such financing are given below to illustrate the differences with respect to revenue financing.

An example of general obligation bond financing of the 5-year Stage I sewerage program is given in Table 19-7. A series of bonds, all having terms of 30 years and assumed to bear a 4.0 per cent interest rate, would be issued as needed to meet all construction requirements. Since no surplus income would be required, no construction from income would be undertaken.

Total annual costs would range from an average of $4,325,000 during the first five years to $8,777,000 during the 16th to 20th years. Total costs for the 20-year period would amount to $140.13 million, or about 77 per cent of that required for revenue bond financing.

Debt service costs could be met by a tax levy averaging 3.34 mills per dollar of assessed valuation during the first five years and 6.06 mills during the last five years of the 20-year period. Because no provision is made in the metropolitan corporation act for a property tax to cover operation and maintenance, a service charge ranging from an average of $0.35 to $0.42 per month would be necessary.

As an alternative to the tax levy and service charge shown in Table 19-7, part or all of the debt service could be included in the service charge. Total annual costs would be increased to the extent of the additional state tax thus incurred. If all costs were raised by service charges, the charge would range from an average of $1.62 to $2.33 per month respectively for the first and last 5-year periods of the twenty years.

Outstanding bonds (Table 19-7 and Fig. 19-5) during the first fifteen years would remain relatively constant near $87 million and would drop to $79 million by the end of the 20-year period. Expressed as a percentage

Table 19-7. General Obligation Bond Financing of Five-Year Stage I Sewerage Program

	Stage I	Stage II		
	1960-64	1965-69	1970-74	1975-79
For period:				
Capital funds required,[a] $1,000	92,634	10,702	13,374	11,341
General Obligation bond issues,[b] $1,000	93,000	10,000	14,000	11,000
Average assessed valuation,[c] million dollars	1,016	1,080	1,136	1,183
Average number of services,[c] 1,000	225	265	297	315
Average annual amount, $1,000				
Debt service	3,389	5,725	6,388	7,171
Operation and maintenance[a]	929	1,301	1,487	1,593
State tax[d]	7	11	12	13
Total cost	4,325	7,037	7,887	8,777
Average annual levy for debt service, mills	3.34	4.80	5.62	6.06
Average monthly service charge for operation and maintenance, dollars	0.35	0.41	0.42	0.42
Cumulative amounts, $1,000				
Capital fund requirements	92,634	103,336	116,710	128,051
Bonds issued	93,000	103,000	117,000	128,000
Bonds outstanding[e]	87,600	86,800	86,700	79,000
Estimated debt limit,[e] $1,000	52,100	55,200	57,800	60,000
Outstanding bonds, per cent of assessed valuation[e]	8.40	7.87	7.50	6.58

[a]*From Table 19-3.*

[b]*General obligation 30-year serial bonds at 4.0 per cent interest, with uniform annual interest and redemption payments.*

[c]*From Table 19-1.*

[d]*At 0.8 per cent of gross revenue from service charges.*

[e]*At end of period.*

of assessed valuation, outstanding bonds would equal 8.4 per cent at the end of the first five years and would drop to 6.6 per cent after twenty years. Total interest costs throughout the life of the bonds would amount to $95 million.

Stretching the first stage construction program to ten years (Table 19-8) would afford no significant advantage under general obligation bond financing. Annual costs during the first six years would be lower than those for the 5-year Stage I program. In both cases, the costs for the six years would be well below the average for the 20 year period.

Outstanding bonds under the 10-year first stage program would reach a maximum of $87 million by the end of the 18th year and thereafter would decline. Expressed as a percentage of assessed valuation, outstanding bonds would exceed the 5 per cent debt limit after the sixth year and would reach a maximum of 7.45 per cent in the 16th through 18th years.

Comparison of Revenue Bond and General Obligation Bond Financing

Use of revenue bonds for financing the metropolitan sewerage program would permit the following advantages:

1. Noninterference with other governmental functions which must be supported by property taxes.
2. Bonds may be authorized without a vote of the electorate. This means that the long-range construction program could be planned and undertaken in an orderly manner without the delays inherent in recurrent bond elections.
3. Lower cost of financing despite a higher interest rate. This is because less indebtedness would be incurred.

As stated earlier, revenue bond financing is subject to a disadvantage in that it would fail to provide an equitable distribution of the costs to all those who would be benefited by the sewerage program. There are two reasons for this disadvantage. First, only those connected to the sewerage system would bear the cost. Second, the surplus income requirement would result in a rapid write-off of capital costs. This in turn means that those served initially would pay also for the capacities then provided to meet future needs. As a consequence, service charges would be higher than otherwise necessary.

General obligation bonds would provide for a more equitable distribution of costs in relation to scope and time. Resulting tax levies and service charges would

Table 19-8. General Obligation Bond Financing of Ten-Year Stage I Sewerage Program

	Stage I				
	1960-61	1962-63	1964-65	1966-67	1968-69
For period:					
Capital funds required,[a] $1,000	24,227	16,919	14,046	19,734	17,708
General obligation bond issues,[b] $1,000	25,000	17,000	14,000	19,000	18,000
Average assessed valuation,[c] million dollars	996	1,022	1,048	1,074	1,098
Average number of services,[c] 1,000	214	229	245	260	277
Average annual amount, $1,000					
Debt service	1,445	2,425	3,240	4,450	5,380
Operation and maintenance[a]	832	877	917	1,015	1,340
State tax[d]	7	7	7	8	11
Total cost	2,284	3,309	4,164	5,473	6,731
Annual levy, mills	1.45	2.37	3.10	4.15	4.90
Monthly service charge, dollars	0.33	0.32	0.31	0.33	0.41
Cumulative amounts, $1,000					
Capital fund requirements	24,227	41,146	55,192	74,926	92,634
Bonds issued	25,000	42,000	56,000	75,000	93,000
Bonds outstanding[e]	24,200	39,600	51,300	67,200	81,300
Estimated debt limit,[e,f] $1,000	50,015	51,400	52,700	54,000	55,300
Outstanding bonds, per cent of assessed valuation[e]	2.42	3.86	4.87	6.22	7.37
	Stage II				
	1970-71	1972-73	1974-75	1976-77	1978-79
For period:					
Capital funds required,[a] $1,000	7,690	5,955	6,817	6,953	8,002
General obligation bond issues,[b] $1,000	7,000	7,000	7,000	7,000	7,000
Average assessed valuation,[c] million dollars	1,121	1,141	1,160	1,178	1,196
Average number of services,[c] 1,000	292	299	306	313	319
Average annual amount, $1,000					
Debt service	5,850	6,200	6,600	7,000	7,402
Operation and maintenance[a]	1,455	1,433	1,506	1,583	1,645
State tax[d]	12	12	12	13	13
Total cost	7,317	7,645	8,118	8,596	9,060
Annual levy, mills	5.20	5.43	5.68	5.95	6.18
Monthly service charge, dollars	0.42	0.40	0.41	0.42	0.43
Cumulative amounts, $1,000					
Capital fund requirements	100,324	106,279	113,096	120,049	128,051
Bonds issued	100,000	107,000	114,000	121,000	128,000
Bonds outstanding[e]	83,800	85,400	86,700	87,000	86,100
Estimated debt limit,[e, f] $1,000	56,300	57,300	58,300	59,200	60,000
Outstanding bonds, per cent of assessed valuation[e]	7.44	7.45	7.45	7.35	7.17

[a]*From Table 19-4.*

[b]*Serial bonds, 30-year term at 4.0 per cent interest, with uniform annual interest and redemption payments.*

[c]*From Table 19-1.*

[d]*At 0.8 per cent of gross revenue from service charges.*

[e]*At end of period.*

[f]*At 5.0 per cent of projected assessed valuation.*

be relatively moderate considering the magnitude of the project.

In order to use general obligation bonds for financing the entire project, the debt limit would have to be raised either by increasing the allowable percentage of assessed valuation or by obtaining a substantial increase in assessed valuation. In the absence of such changes, partial financing by general obligation bonds would be possible but would interfere with the financing of drainage improvements. For the latter, as pointed out previously, general obligation bonds offer the only feasible means of obtaining the necessary funds. Under present conditions, therefore, it is obvious that revenue bonds would provide the most practicable solution to the problem of financing a metropolitan sewerage project.

Independent Financing of Separate Sewerage Systems

While the necessity for financing and administering the central sewerage system through a metropolitan agency is apparent, there may be some question as to the advisability of such an arrangement in the case of several of the independent systems discharging to Puget Sound. Since separate financing and administration of these systems would be practicable, two questions may be raised. These are:

1. What will metropolitan Seattle gain by inclusion of the independent systems in a metropolitan plan of financing and administration?

2. What will the residents of the topographically separate sewerage areas gain by such inclusion?

An answer to the first question may be found in the discussion earlier in this chapter relating to the function or purpose of metropolitan sewerage facilities and the benefits thereby derived. Provision of sewerage service by a metropolitan agency will assure a prompt recognition of local problems, as well as prompt and effective action in providing necessary facilities for sewage collection, treatment and disposal. Under such a program, sewerage construction would be undertaken on a systematic long-range basis and future additions would be made as needed regardless of any apathy on the part of local residents or officials. Technical supervision would be made available to the independent systems, as would a monitoring program to determine the effectiveness of the treatment and disposal operations.

It is not intended to imply that it is either overly difficult or impossible for a small sewerage agency to build suitable facilities and achieve satisfactory results. With efficient management, adequate financing, and a minimum of political interference, sewerage service can be furnished just about as well by a small agency as it can be a large agency. Experience in general, however, indicates that the probability of equal performance is relatively remote.

As an answer to the second question, it is necessary to consider the problems of financing sewerage works in a partially developed but growing area. In such areas, initial financing limitations, rather than sound engineering and long-range economy, tend to govern the design of sewerage facilities. Faced with excessive costs in relation to the initial contributory population, an engineer is compelled to limit his planning to facilities which later must be duplicated or abandoned.

For the purpose of illustration, it can be assumed that facilities equal to those obtainable through a metropolitan agency would be financed locally and that equal performance could be attained at no difference in operating costs. On that basis, an analysis can be made of total annual costs and of resulting service charges for a typical independent sewerage area. Using the Des Moines sewerage area as a typical example, an analysis was made of the total annual costs which would accrue under both metropolitan financing and completely independent financing.

In 1957, the population of the area topographically tributary to the proposed Des Moines treatment plant was estimated to be 13,000. Application of the low population projection results in estimates of 14,000 by 1960, 19,000 by 1970, and 22,000 by 1980. In contrast, application of the high projection used for design purposes gives a 1980 population of 30,000.

For the purpose of revenue calculations, the number of persons served was assumed to be 7,000 by 1960, 14,000 by 1970, and 19,250 by 1980. It was assumed also that first stage construction would be undertaken in 1960 at a cost of $1,618,000 (Chapter 16). Under that program, the initial facilities would consist of trunk sewers, a primary type treatment plant, and a submarine outfall. Additional trunk sewers would be constructed in 1970-71 and 1976-77 at estimated costs of $168,000 and $253,000 respectively. In 1978, the treatment plant would be enlarged at an estimated cost of $800,000.

Estimated costs and revenue requirements for separate financing of the Des Moines sewerage program are given in Table 19-9. Debt service costs are based on 30-year revenue bonds at 5 per cent interest. After the second year, it is assumed that part of the surplus income of each preceding year would be utilized for operation and maintenance, and that the balance would be placed in a reserve fund. Construction of the trunk sewers scheduled for 1970-71 would be financed from the reserve, but additional bonds would have to be issued to finance the sewer and plant additions scheduled to start in 1976.

Operation and maintenance costs are assumed to be identical to those under the metropolitan agency. In this case, however, Des Moines' share of undistributed overhead cost of the metropolitan agency is excluded.

Annual income necessary for separate financing would amount to $185,000 during the first two years and to $160,000 in the third through sixteenth years. It would increase thereafter and would reach $263,000 in the nineteenth and twentieth years. During the 20-year period, the estimated number of equivalent single-family service connections would increase from 2,200 to 5,350.

Service charges would be excessive initially and would amount to $7.00 per month. They would drop to a minimum of $2.80 by 1974-75 and thereafter would increase again to more than $4.00 per month. Under suitable bond terms, it would be possible to obtain an arrangement under which the annual debt service would be the same each year and the sewer service charge would be held at a constant level. In such an event, the latter would still be in excess of $4.00 per month, not including the costs of either local sewer maintenance or revenue collection.

In contrast, the costs to local residents of financing sewerage facilities through a metropolitan agency would be about 60 per cent of those incurred through separate financing. This comparison does not take into account the lower interest rates which metropolitan revenue bonds would command, nor does it include the legal, fiscal and engineering services which would be provided by such an agency.

Part of the difference between metropolitan and local financing lies in the scheduling of construction. At Des Moines, all Stage I facilities would be constructed during the first year or two, as opposed to a five to ten year spread for the metropolitan area as a whole. This means that, by the end of 20 years, a larger share of the indebtedness for first stage facilities at Des Moines would be repaid than would be the case for first stage facilities of the metropolitan area as a whole. In other words, local financing would be more costly for the period under consideration.

A second factor is that facilities at Des Moines require provision for a greater rate of growth than the average for the entire metropolitan area. For that reason, the initial costs per service connection would be higher under local financing.

Table 19-9. Independent Financing of Separate Sewerage System at Des Moines

	Stage I					Stage II				
	1960 -61	1962 -63	1964 -65	1966 -67	1968 -69	1970 -71	1972 -73	1974 -75	1976 -77	1978 -79
For period:										
Construction funds required[a]	1,618					168			253	800
Revenue bonds issued,[b] $1,000	1,618								253	800
Average number of services[c]	2,200	2,600	3,000	3,400	3,800	4,150	4,450	4,750	5,050	5,350
Monthly service charge, dollars	7.00	5.12	4.45	3.92	3.51	3.22	3.00	2.80	3.05	4.10
Average annual amount, $1,000										
Operating income	185	160	160	160	160	160	160	160	185	263
Debt service	106	106	106	106	106	106	106	106	122	174
Operation and maintenance	24	24	27	29	32	36	41	44	47	59
State tax	2	1	1	1	1	1	1	1	2	2
Total cost	132	131	134	136	139	143	148	151	171	235
Debt service coverage[d]	53	53	53	53	53	53	53	53	61	87
Net income[e]	0	(24)	(27)	(29)	(32)	(36)	(41)	(44)	(47)	(59)
Total surplus	53	29	26	24	21	17	12	9	14	28
Construction from surplus	0	0	0	0	0	168	0	0	0	0
Net to reserve fund	53	29	26	24	21	(151)	12	9	14	28
Cumulative amount, $1,000										
Construction fund requirements	1,618	1,618	1,618	1,618	1,618	1,786	1,786	1,786	2,039	2,839
Construction from income	0	0	0	0	0	168	168	168	168	168
Construction from bonds	1,618	1,618	1,618	1,618	1,618	1,618	1,618	1,618	1,871	2,671
Bonds outstanding[f]	1,590	1,530	1,480	1,420	1,350	1,270	1,190	1,080	1,240	1,900
Reserve fund[f, g]	106	164	216	264	306	155	179	197	225	281

[a]*From Table 16-1 and 16-12.*

[b]*Thirty-year serial revenue bonds at 5 per cent interest with uniform annual interest and redemption payments.*

[c]*Based on low population estimate of 14,000 by 1960 and 22,000 by 1980.*

[d]*Fifty per cent of debt service cost.*

[e]*Operation and maintenance costs, in effect, supplied from surplus of preceding year.*

[f]*At end of period.*

[g]*Exclusive of interest earned.*

Granting that it is to the advantage of the independent sewerage areas to participate in the metropolitan plan, a question arises as to whether their inclusion is a disadvantage to areas served by the central system. In that connection, an analysis of Stage I costs shows that the capital outlay per service connection would amount, by 1970, to $332 for the independent systems as compared to $326 for the central system. A similar situation would prevail in 1980. In the long run, therefore, the independent areas as a group would not be subsidized by the central area.

FINANCING THE STORM DRAINAGE AND SEPARATION PROGRAMS

It is not possible at this time to develop a program for stage construction of the proposed drainage improvements. Consideration can be given nevertheless to the rate of progress which could be achieved within financial limitations. Consideration can be given also to the program of separating storm water and sanitary sewage which is to be undertaken by the city of Seattle.

Storm Drainage

In general, two procedures have been found practicable in other metropolitan areas for financing major drainage projects by means of general obligation bonds. Under the first procedure, a schedule of construction is established for the metropolitan area as a whole, with priority based on immediate need and on the value of property to be protected. General obligation bonds in an amount sufficient to carry the program for a considerable number of years are then authorized by a vote of the electorate of the entire area and bonds are issued as required to maintain the construction schedule.

Where taxing powers permit, construction of drainage facilities may be financed entirely on an area-wide basis by means of a fixed annual levy. As an example, this method of financing has been utilized successfully by the Los Angeles County Flood Control District.

Under the second procedure, the metropolitan area is divided into zones or special districts, each of which comprises an entire drainage area. Construction projects therein are financed by bond issues which are independently voted and supported in each zone. This procedure has been used successfully in California by the Alameda County Flood Control District.

Of the two procedures, the first is particularly applicable where drainage problems are wide-spread and general public support can be obtained. The second, while relatively easy to initiate in developed zones subject to flooding, may be impracticable in others because of limited development and low assessed valuation. In the latter case, land development of value to the metropolitan area, such as that for industrial purposes, may be unduly delayed.

Because of the wide-spread need for major drainage improvements throughout the metropolitan area, including those involving separation of combined sewers in Seattle, it is assumed herein that construction will be financed on an area-wide basis in accordance with the first procedure. Further consideration of the second procedure would require at least a preliminary determination of the drainage zones and their assessed valuations. In any case, use of this procedure would necessitate an amendment of the Metropolitan Municipal Corporation Act.

Exclusive of that portion of Seattle served by combined sewers, the estimated eventual cost of constructing major storm drainage facilities in the metropolitan area amounts to a total of $145 million (Chapter 17). Of this total, roughly 25 to 30 per cent, or $35 to $45 million, represents facilities which are needed now or will be needed in the near future.

Major drainage works required to relieve overloaded combined sewers in Seattle are estimated to cost approximately $19 million, which amount is about 27 per cent of the total cost of partial separation. Most of these facilities are needed now and all of them should be constructed as rapidly as the necessary funds can be made available. In all, therefore, the total cost of storm drainage facilities requiring early construction is not less than $54 million and may be as high as $64 million.

Based on the low population projection for the metropolitan area, the general obligation debt limit is estimated to be $49.5 million by 1960 and $55.8 million by 1970 (Table 19-8 and Fig. 19-5). At those levels, a storm drainage construction program involving an expenditure of $6 million per year for 10 years would be feasible.

As an example, assume an issue of 30-year serial bonds at 4 per cent interest, with combined interest and debt retirement charges paid in equal annual installments. Under such a program, outstanding bonds at the end of the tenth year would amount to $53.5 million. Annual debt service would increase from $347,000 the first year to $3.47 million in the tenth year. Tax levies would range from 0.35 mills to 3.15 mills over the 10-year period. Beyond that time, drainage construction would depend largely on population growth and increased assessed valuation. As a minimum, an expenditure of $1.5 million to $2.0 million per year could be supported thereafter based on increased bonding capacity estimated from the low rate of population growth.

No estimates were prepared of operation and maintenance costs for the completed drainage facilities. Normally derived from general fund sources, including tax levies, the amount required for this purpose is small and generally can be raised by a tax levy of

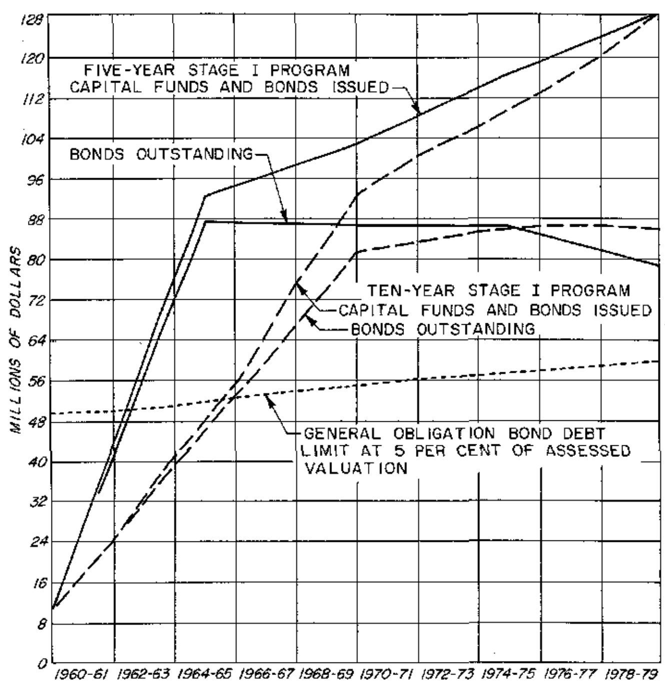

Fig. 19-5. Cumulative Funds, General Obligation Bond Financing of Sewerage Program

about 0.1 mill. Under present limitations, however, a metropolitan agency is not empowered to levy a tax for operation and maintenance purposes. Funds thus required would have to be collected from constituent cities and counties as supplemental income.

Separation of Combined Sewers

Financing the separation of combined sewerage systems in Seattle is primarily the responsibility of that city. Partial separation is estimated to cost a total of $69 million (Chapter 18), of which $19 million is for major storm drains and could be financed as part of a general drainage program undertaken by a metropolitan agency. This would leave a balance of $50 million to be financed by the city.

Particular urgency is attached to the separation of systems within the Lake Washington watershed. For these systems, the estimated initial cost of partial separation is $18 million, or $13 million exclusive of the trunk drains which could be financed by a metropolitan agency.

Financial resources which are now or may become available in the future for the separation program include:

1. Approximately $5.5 million as reimbursement for existing facilities by the metropolitan agency. This amount (Table 19-2) excludes $1,333,000 due Lake City Sewer District for facilities still being financed by that district.
2. Up to $2 million per year from the city sewer service fund. Upon inauguration of the metropolitan sewerage program, normal expenses to be met from this fund will be reduced largely to those of collection system maintenance. The foregoing income will become available each year in the event that the local charge is not reduced.
3. Existing general fund revenues.
4. General obligation bonding capacity of the city available only for water, sewerage, and electric utilities.

If income available under items 1 and 2 were utilized directly for construction, the Lake Washington watershed separation program could be achieved in four to seven years, depending on whether the major storm drainage works were financed by the metropolitan agency. Continuing on the same basis, the balance of the separation work could be completed in an additional 18 to 25 years.