A REPORT

ON AN

ENGINEERING INVESTIGATION

OF

SEWAGE COLLECTION

AND

TREATMENT FACILITIES

CITY OF PENDLETON

OREGON

NOVEMBER 1962

CORNELL, HOWLAND, HAYES & MERRYFIELD

Consulting Engineers

SEATTLE CORVALLIS BOISE

CITY OF PENDLETON, OREGON

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December, 1962

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

APPENDIXES

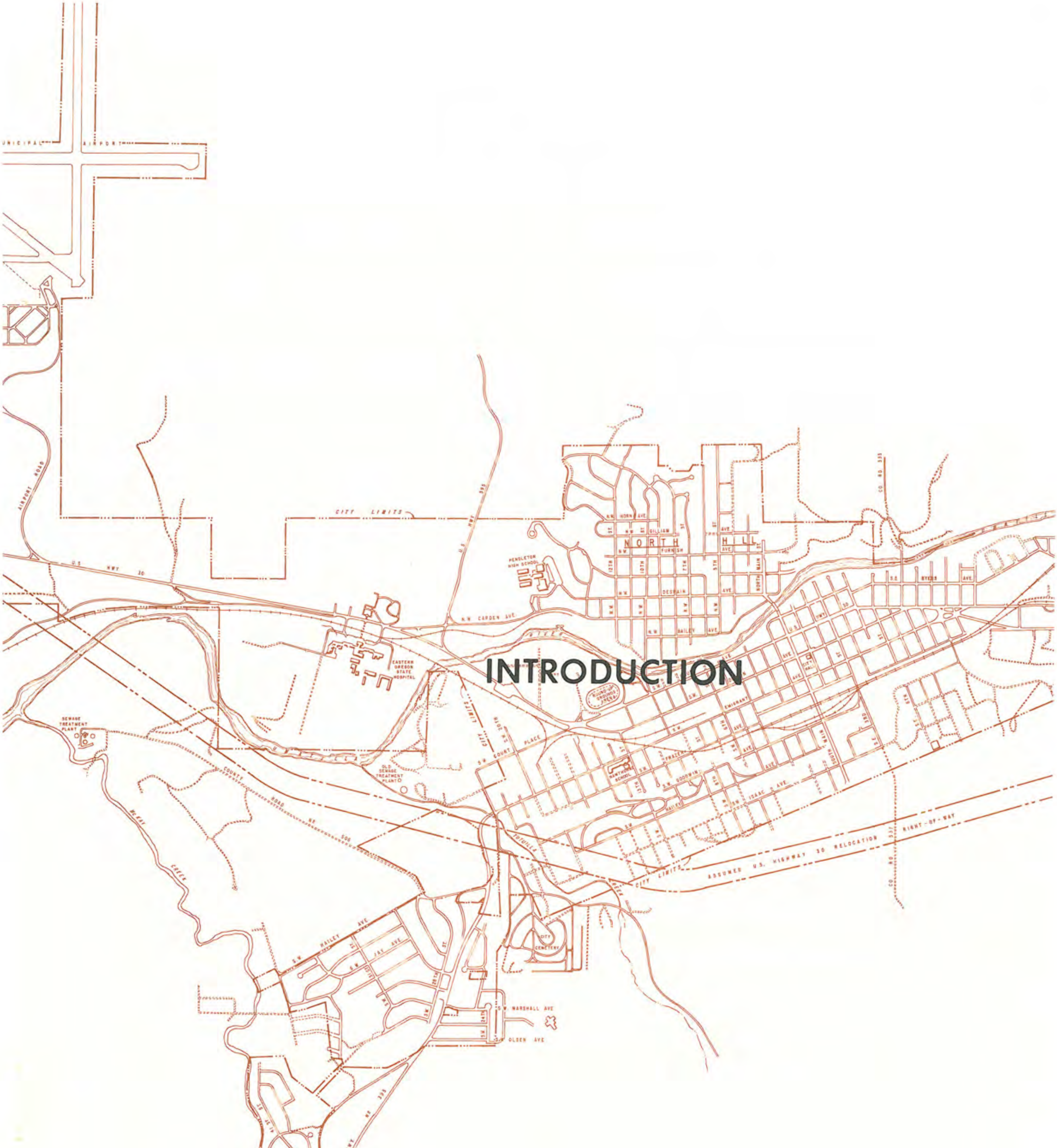
TABLE OF CONTENTS (Continued)

TABLES

FIGURES

PLATES

INTRODUCTION



CHAPTER I

INTRODUCTION

NEED FOR SEWER STUDY

When planning studies were undertaken to develop a suitable route for relocating U. S. Highway 30 through Pendleton and for providing utility service to subdivisions developing at the edge of the City limits, it was recognized that the utility systems did not have an overall master plan which would provide a guide for orderly expansion. It was also recognized that many of the existing sewers in the business district of the City, as well as in other areas, were not shown on sewer maps; and their location, size, and condition were unknown. Also, even though the sewage treatment plant was relatively new, an orderly expansion over a long-range program had not been specifically established so that budgeting for its expansion along with the collection system could be accomplished.

Upon recognizing these facts and knowing that sewer service must be provided for unsewered areas near the City limits, it was concluded that an overall master plan should be developed for the sewer system and an analysis should be made for programming maintenance, rehabilitation, and new construction.

AUTHORIZATION

After the need for an overall sewer study was established, this investigation was initially authorized in a letter by Andrew J. Browning, City Manager, dated October 14, 1959. Before the study could actually commence, however, an accurate mapping program was considered essential. Upon completion of the detailed topographic maps, notice to proceed on the overall sewer study was given on October 26, 1961, in a letter by Mr. M. O. Gardner, City Manager.

SCOPE

The scope of the sewerage investigation included the following items:

1. Prepare detailed maps of existing sewerage facilities.

2. Develop a forecast of the future population for the City of Pendleton.

3. Establish existing and future sewage flow conditions.

4. Determine location, size, and condition of existing sewers and establish a program for repair and replacement of sewers considered to be of little value in their present condition.

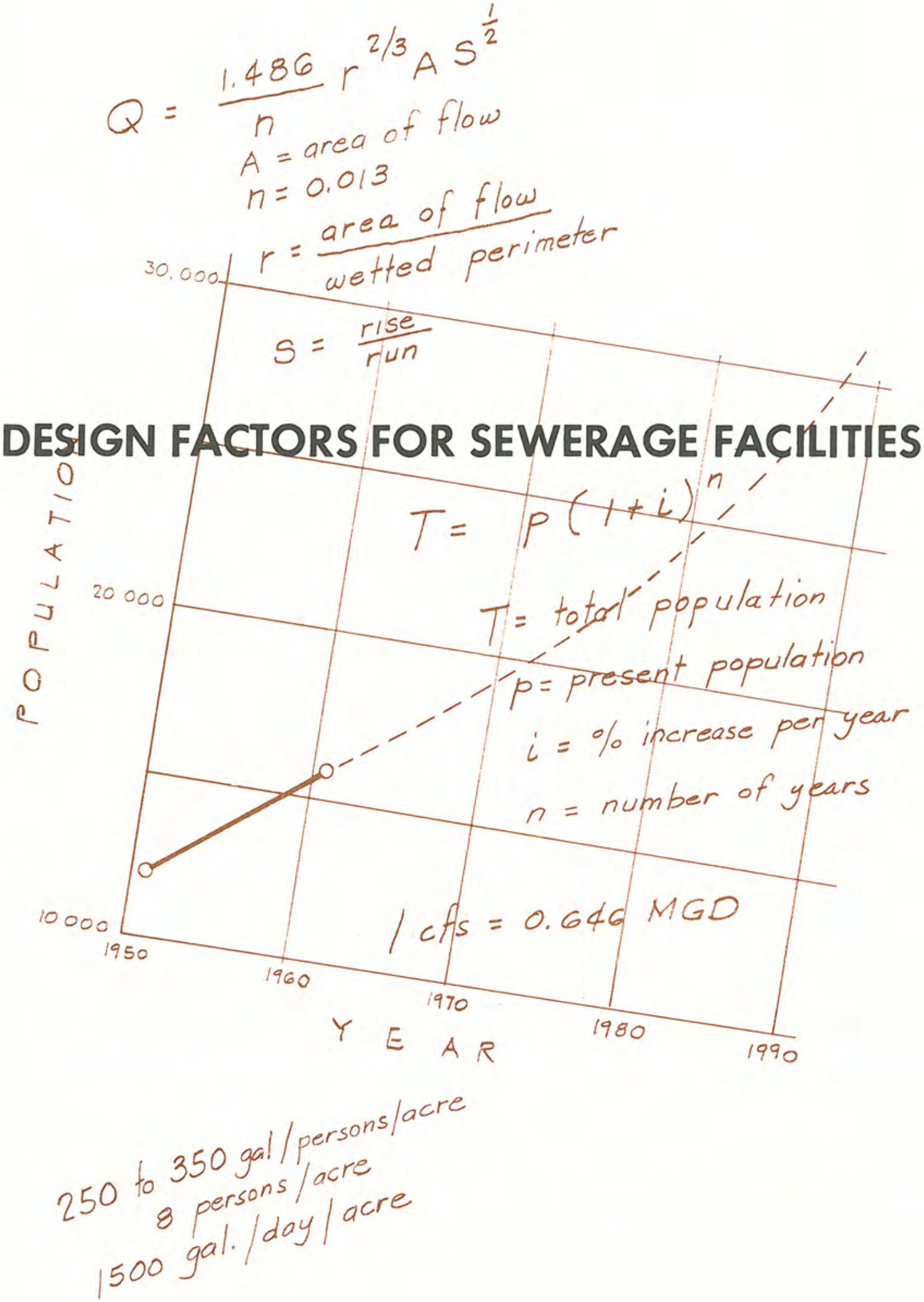
5. Prepare a master plan of the ultimate sewage collection system.

6. Review the condition and capacity of the present treatment facilities and recommend a program for the orderly expansion of these facilities.

ACKNOWLEDGMENTS

We are indebted to the Engineering and Utility Departments of the City of Pendleton for the assistance provided in developing the data and many historical features of the sewer system. The City Engineering Department under the direction of Mr. Anton Groh, the Sewer Department under the direction of Mr. Ray Corwin, and the Water Department and the Construction and Repair Department--both under the direction of Mr. Raymond Struthers--provided valuable assistance throughout the investigation.

DESIGN FACTORS FOR SEWERAGE FACILITIES



CHAPTER II

DESIGN FACTORS FOR SEWERAGE FACILITIES

DEFINITION OF TERMS

Some of the various terms frequently used in conjunction with sewerage systems and referred to throughout this report are defined as follows:

Domestic Sewage. Flow of polluted waste from residential dwellings, apartment houses, and other living accommodations (expressed in gallons per day).

Commercial Sewage. Flow of polluted waste from commercial establishments containing domestic sewage along with possible other waste waters such as those originating from laundries, bottling plants, ice plants, and dairies (expressed in gallons per day).

Industrial Wastes. Flow of waste liquids from industries using large volumes of water for processing industrial products, such as food processing plants, paper mills, plywood and lumber mills, and in Pendleton, the Pendleton Woolen Mills (expressed in millions of gallons per day, mgd).

Infiltration. Flow of groundwater and surface water into sanitary sewers intentionally or unintentionally through pipe joints, pipe walls, manholes, and other direct or indirect connections (expressed in gallons per acre per day).

Average Daily Flow. The flow from a complete sewer system or a defined portion thereof measured in total gallons throughout a 24-hour period (expressed in millions of gallons per day, mgd).

Peak Flow. The highest average daily flow occurring throughout a 12-month period (expressed in millions of gallons per day, mgd).

Per Capita Flow. The average daily flow of domestic sewage divided by the number of persons contributing to the specific quantity involved (expressed in gallons per capita per day).

Trunk Sewer Lines. Sewers normally over 18 inches in diameter intercepting sewage from mains and laterals and carrying it to the sewage disposal facility.

Main Sewer Lines. Sewers from 10 inches to 18 inches in diameter (sometimes including 8 inches) carrying sewage from laterals to trunk sewers.

Lateral Sewer Lines. Sewers 8 inches and smaller which intercept sewage from individual residences, etc., through side sewer and service connections.

Side Sewer Connections. Sewers normally 4 inches and 6 inches in diameter which run from the lateral sewer to the private property line.

Service Lateral. An extension of the side sewer connections from the property line to the plumbing facilities leading from a residence or other building.

BOD. Biochemical oxygen demand, which is a measure of the oxygen necessary to satisfy the requirements for the aerobic decomposition of the sewage. This provides an accurate indication of the organic content of the sewage.

Settleable Solids. Suspended solids which will settle in sedimentation tanks in normal detention periods.

Suspended Solids. Solids which can be filtered from the sewage.

Total Solids. Both suspended and dissolved solids.

SEWAGE FLOW

The Pendleton sanitary sewer system collects domestic sewage, commercial sewage, industrial wastes, and infiltration. Since these wastes have varying effects upon the design of a sewage collection system and treatment plant, they are discussed separately.

Domestic Sewage. Inasmuch as the daily routine of most inhabitants of a residential community are similar, the peak sewage flow from a given area may be several times greater than the average flow. Therefore the flow used in the design of a sewer system must provide capacity for the peak flow expected as well as the average flow. The domestic sewage flow is, of course directly related to the water use in a residence. During recent years, residential water use has increased due to the use of modern plumbing fixtures, automatic washers, dishwashers, and multiple bathroom facilities. The average domestic sewage flow ranges from 60 to 100 gallons per person per day.

Experience throughout the Northwest has resulted in a design criteria for the per capita flow from residential areas, and it has been generally recognized that,although peak flows from small areas may reach as high as 350 gallons per person per day, measurements indicate that as the contributing area is increase, the peak flow tends to drop and seldom attains 250 gallons per person per day in areas of 250 acres or more.

Commercial Sewage. Sewage flow from most commercial establishments normally does not create a significant demand on sewerage facilities. Most commercial buildings have restrooms only and do not have unusual amounts of sewage entering the system. However, a few commercial operations, such as laundries, ice plants, dairies, and bottling works will produce fairly large sewage flows which will enter the sanitary sewer system. In cities similar to Pendleton, it has been established that the areas having primarily commercial buildings will produce sewage flows comparable to those from residential areas. Sewage flow measurements in the Pendleton sewer system (see Appendix C) indicate that there is no reason to assume this criteria should not apply in Pendleton; therefore, it has been assumed in this investigation that the flow developed in the business district is comparable to an area having a density of six persons per acre.

Industrial Sewage. Industries such as the Pendleton Woolen Mills and especially the food processing plants in Pendleton produce large volumes of wastes which enter the sanitary sewage collection system and are processed through the sewage treatment plant. Although these industries do not have industrial waste meters, other than the Smith Canning Company, a measure of the volume from each industry has been established through the water use record. Since these industries operate during the spring and summer when high infiltration occurs from irrigation, a peak flow develops in the sewage collection system and the treatment facilities (see Plates 8 and 9 at the back of this report), which therefore requires special consideration of the wastes produced from the industries to insure adequate hydraulic design of the sewers and treatment facilities. Also, since the food processing industries contribute a high organic content to the sanitary sewage, the treatment plant operation must allow for increased organic loads during the canning season.

Infiltration. The problem of excessive flow in the sewer system caused by excess infiltration is not a new one and is a problem faced by many communities with older systems throughout the Northwest. Cities in western Oregon, such as Salem, Eugene, Sweet Home, Silverton, and in Washington, such as Yakima, Richland, and many others, have significant infiltration problems. As the existing sewers become more heavily loaded with domestic sewage, the problem of eliminating excessive infiltration becomes more severe.

Although some sanitary sewer systems in western Oregon have been known to have infiltration rates which exceed 8,000 gallons per acre per day during storm periods, the sewer system in Pendleton does not appear to have large areas with infiltration rates of this magnitude.

However, some areas in Pendleton have extremely high infiltration rates which are now limited by the carrying capacity of the sewer pipes. These areas have been given special attention in this report and are discussed in more detail in Chapter III. In general, however, an average system constructed within the past ten to twenty years could have infiltration rates averaging 1, 500 gallons per acre per day. As the construction standards have increased and the pipe manufacturers have developed rubber ring joints and more dense pipes with essentially no leakage through the walls, watertight sewers can now be constructed. However, even though future public sewers can be constructed watertight, allowances for infiltration must be made because it is difficult to prevent direct storm drain connections from around private homes; and it is often difficult to control the construction standards of all service laterals, even though individual leakage tests are required. Therefore, even in new sewer systems, the allowance of 1, 500 gallons per acre per day is provided to insure a proper safety factor in the hydraulic design of the sewer system.

Summary of Sewage Flow. The above considerations of the various types of sewage flow have resulted in the following design criteria.

TABLE 1

FLOW CRITERIA

Capacity of existing sewers (See Chapter III)

Design of New Sewers

Minimum Velocity:	2 feet per second
Average Domestic Flow:	100 gallons per capita per day
Peak Domestic Flow: (Design Flow)	250 gallons per capita per day
Industrial Flow: (Average day for maximum month)	
Smith Canning Company	0. 82 mgd
Pendleton Frozen Foods	1. 51 mgd
Pendleton Woolen Mills	0. 12 mgd
Infiltration:	1, 500 gallons per acre per day

POPULATION

Since the volume of domestic sewage flowing in a sanitary sewer system is a function of the population served, a study was necessary of the existing and future population densities in the present City and in areas which may develop and require sewer service.

Previous Studies. Previous studies which were reviewed in preparing the population forecast for Pendleton include the following:

1. Population Trends, dated 1959, conducted by the Bureau of Municipal Research and Service, University of Oregon.

2. Planning for Land Use, dated 1961, conducted by the Bureau of Municipal Research and Service, University of Oregon, and Lutes & Amundson, Springfield, Oregon.

3. Population of McKay Creek area by Community Planning Office, Pendleton.

4. Population of Oregon Cities, Counties, and Metropolitan Areas, 1950 to 1958, by the Bureau of Municipal Research and Service, University of Oregon.

5. United States Census of Population of Oregon, dated 1960, by the U. S. Department of Commerce, Bureau of the Census.

Population History. After reviewing the above studies, the population history of Pendleton was compared with the history of similar cities in eastern Washington and Oregon, the State of Oregon and Umatilla County. As shown on Figure 1, the population trends for the Pacific Northwest, the State of Oregon, Umatilla County, and Pendleton have increased at rates of 0.6 to 2.6 percent per year over the past 10 to 50 years. In 1960, the total population within the City limits of Pendleton reached 14,434 persons.

The design of City utilities such as sewer and water systems is not only affected by the total population but also by the population densities throughout the service areas. These densities have been studied in detail and are shown on Plate 1 at the back of this report.

House counts were made in various typical areas to establish the existing population densities. These counts were made from photographs taken for mapping purposes in 1960. The results are shown in Table 2 which is a summary of present average densities (also shown on Plate 1) totaling 14,430 persons, the 1960 population of the City.

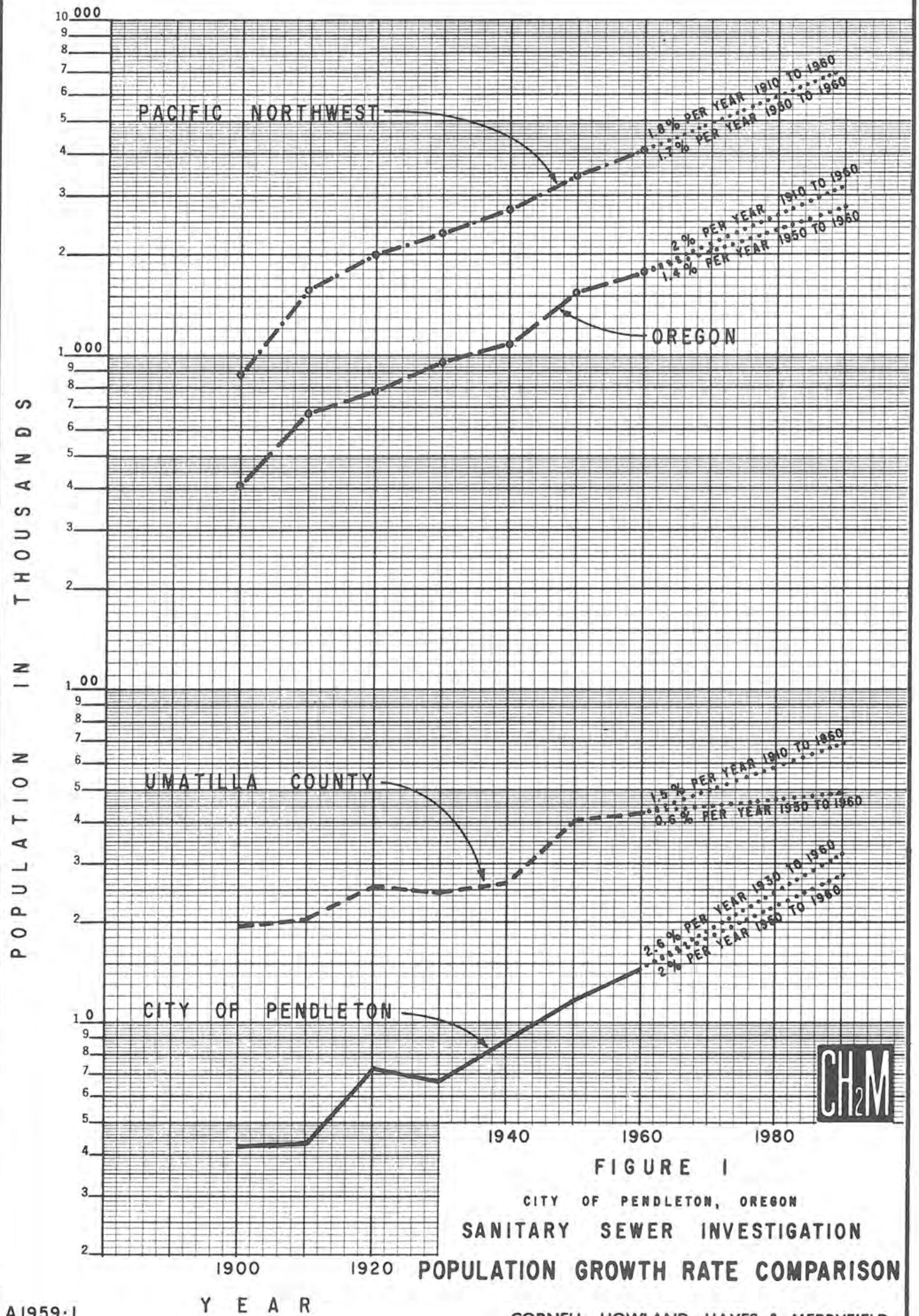


FIGURE 1

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

POPULATION GROWTH RATE COMPARISON

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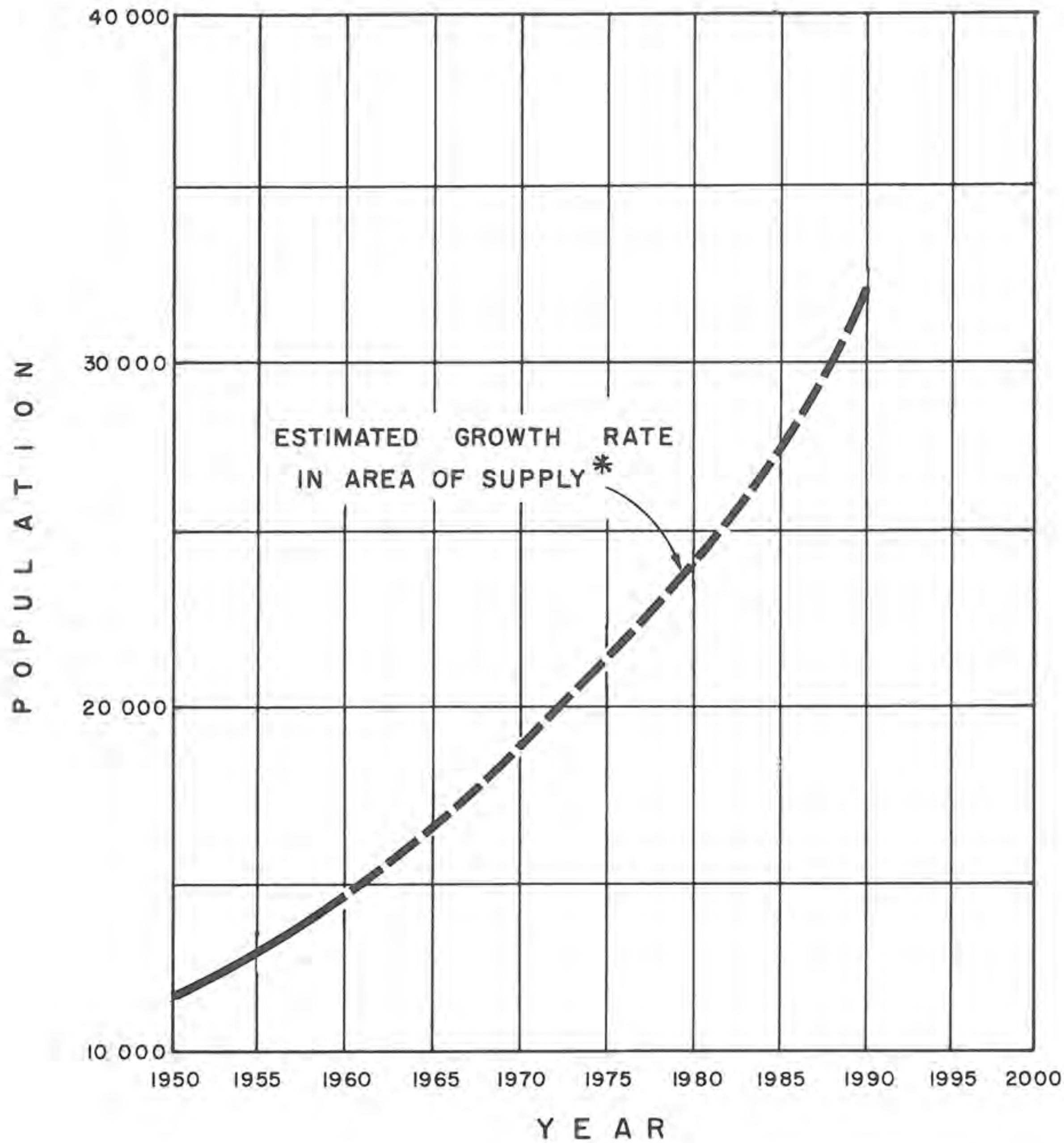
TABLE 2

SUMMARY OF AVERAGE POPULATION DENSITIES FOR 1960

Area	Acres	Persons per Acre	Persons
Pendair	115	2	230
McKay Creek			
	30	8	240
	160	10	1,600
Downtown &			
South Hill	50	2	100
	380	6	2,280
	120	8	960
	550	10	5,500
North Hill			
	290	8	2,320
	120	10	1,200
Undeveloped Areas	385	0	0
TOTAL	2,200		14,430

Population Growth. After studying the population growths in other cities and the population history of Pendleton, a growth study was made which indicated a reasonable population forecast would be one which increased at a rate between 2.0 and 2.6 percent per year. Although the lower rate appears more realistic, the higher rate of 2.6 percent per year was used because in sizing utilities, such as sewer and water systems, it is extremely difficult and costly if added capacity is required beyond the peak design capacity of the systems. The population forecast shown on Figure 2 for the City of Pendleton was developed by compounding the 1960 population of 14,434 at a rate of 2.6 percent, which resulted in a population of 32,000 in 1990. The projected population to receive sewer service includes annexation of new areas to the City and allows for increased densities within portions of the present City.

No method of predicting populations is exact nor can any method eliminate the human errors in judgment which affect population growth estimates. Because of the many factors which affect this growth, no one can predict the precise effect a change in any one of these factors may have on the established growth trend. However, this is not necessarily significant in that the population prediction is used as a



* GROWTH RATE COMPUTED AT 2.6 % PER ANNUM COMPOUNDED ON CITY CENSUS POPULATION OF 14,434 FOR 1960.

FIGURE 2

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

POPULATION FORECAST

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general guide for the City in planning and budgeting for major expenditures. It is, of course, anticipated that the future growth will not follow a smooth curve as shown on Figure 2, but will instead increase in increments resulting from periodic industrial expansion in the area or from annexation.

Areas to Accommodate Future Population. The increase in population through 1990 can be accommodated in the existing City limits plus five additional areas outside the City boundaries. These are referred to as McKay Creek, South Hill, Riverside, North Hill, and Lower Pendair. They are discussed as follows:

Present City Limits: Vacant lots within the present City limits could accommodate approximately 3,550 persons. However, since the trend of residential development is along the edge of the City, it is assumed that only 1,800 persons would actually construct homes on the vacant lots through 1990. The remaining lots will no doubt be utilized for commercial and industrial purposes as the City zoning changes throughout the next 20 to 30 years.

McKay Creek: Recent studies of the McKay Creek area (which excludes the area that is presently within the City limits) by the Community Planning Office at Pendleton indicated an ultimate population of 9,900 could be located in the McKay Creek area. For the purposes of this study, it has been assumed that this area will be approximately 70 percent developed at the end of the study period, or 7,000 people will be located in the McKay Creek area by 1990. This compares with the present development within the City limits of Pendleton.

Riverside District: The Riverside district is located east of the City of Pendleton along the east bank of the Umatilla River as shown on Plate 1. It comprises an area of approximately 520 acres, which can be subdivided into three general areas as follows:

1. The River Bottom area lying south and west of State Highway No. 11 and north of the river. This is presently an agricultural area but is rapidly developing into an urban area which will require sanitary sewers in the near future.

2. Pendleton Canning area which is the river bottom land between the Northern Pacific Railroad and State Highway No. 11. This area is primarily an industrial area today. However, it may also accommodate a residential district in the future.

3. Mt. Hebron area which is the hilly terrain lying to the north of Riverside and Pendleton Canning areas. The central portion of this area has developed into residential tracts today, and the eastern and western portions are primarily agricultural areas.

The development of the Riverside area is progressing steadily and is a key area considered in the sewer study because the sewage from this area must flow through the entire City to reach the treatment plant. Therefore, all sewer planning within the existing City must allow for the ultimate development in the Riverside area. This district has been estimated to ultimately accommodate a population of 4,496; however, as shown in Table 3, the population is estimated to only total 3,670 by 1990.

TABLE 3

RIVERSIDE DISTRICT POPULATION FORECAST

Location	Area (Acres)	Density (Pers./AC) 1990	Density (Pers./AC) Ultimate	Population 1990	Population Ultimate
Riverside Bottom Area	210	8.1	10	1,700	2,100
Pendleton Canning Area	34	4.9	6	170	204
Mt. Hebron Area	274	6.5	8	1,800	2,192
			TOTALS	3,670	4,496

As can be recognized from the above table, the population development is governed by the topography and existing developments. The Riverside Bottom land has favorable topography for residential development in that it is generally flat with good soil conditions for lawns and gardens. The Mt. Hebron area is a sloping area with rock outcroppings and is somewhat difficult to develop into residential lots. The local conditions of industrial environment in the Pendleton Canning area are reflected in Table 3 in that a less dense residential population is expected because of the industrial atmosphere in this area. These factors, of course, will govern other areas throughout Pendleton and have been taken into account in projecting the population densities in the other various areas discussed herein.

South Hill: The South Hill area is comprised of 555 acres and is divided into two separate areas. It is estimated that a population of

5, 724 people can ultimately be located in this area; however, only 3, 020 people are estimated to be located in the South Hill district by 1990, with 2, 400 living in the area having a gentle slope and 620 in the area having steeper slopes. The 191 acres in the steeper terrain would result in a density of 3. 25 persons per acre in 1990.

North Hill: The area north of the Umatilla River and beyond the existing City limits is described as the North Hill area. The terrain is generally sloping and is highly desirable land. This area is now undeveloped, however, because of the view from the high land, it could be characterized by large homes and oversized lots. Therefore, the density has been estimated at 3. 2 persons per acre which results in a population of 1, 600 developing within the 500 acres in the North Hill area by 1990. This area can accommodate an ultimate population of approximately 2, 000.

Lower Pendair: The Lower Pendair area consists of approximately 48 acres below U. S. Highway No. 30 and above the railroad in the vicinity of the McKay Creek confluence with the Umatilla River. This is sloping land, rising to the north from the Umatilla River and has been estimated to develop ultimately to a density of 6. 5 persons per acre. This results in approximately 310 people being accommodated in this area by 1990 with an ultimate population of 400 persons.

Population Summary. As shown in Table 4, below, the 1990 population of 32, 000 can easily be accommodated in the areas discussed above. This does not necessarily indicate that these areas will be developed specifically as indicated; however, sometime in the future these areas will develop into residential units and will require sewerage facilities which must be sized for the population they will ultimately serve.

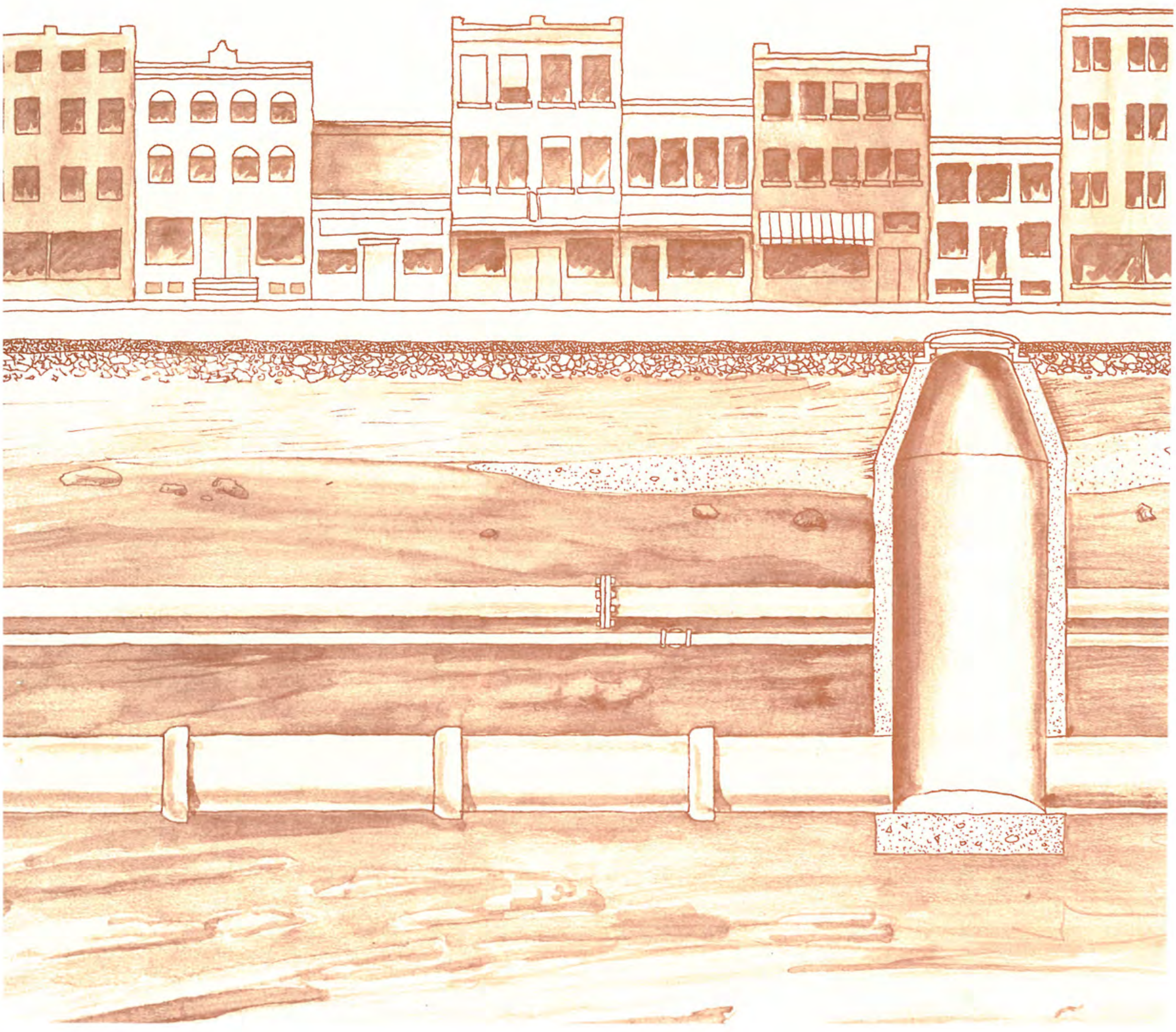
TABLE -

1990 POPULATION ACCOMMODATION SUMMARY

Location	Area Acres	Average Density	Persons
Present City Limits (1)	2,200	6.5	14,400 (4)
Present City Limits (2)			
(Increase through study period)	310	6.5	2,000 (4)
McKay Creek (3)	1,482	4.7	7,000
Riverside:			
Pendleton Canning Area	34	4.9	170
Lower Area	210	8.1	1,700
Mount Hebron	274	6.5	1,800
South Hill:			
Level Area	364	6.5	2,400
Sloping Area	191	3.25	620
North Hill	500	3.2	1,600
Lower Pendair Area	48	6.5	310
	Total Population by 1990 (4)		32,000

(1) 1960 census figure (14,443).
(2) Approximately 50 percent of possible population increase (Ref. Population Trends prepared for the Pendleton City Planning Commission by the Bureau of Municipal Research and Service).
(3) Approximately 70 percent of possible population increase (Ref. Memorandum, Subject - Population of the McKay Creek Area by Community Planning Office, Pendleton.
(4) Exluding area and population of Eastern Oregon State Hospital and area of Pendleton Municipal Airport.

EXISTING SEWER SYSTEM



CHAPTER III

EXISTING SEWER SYSTEM

SYSTEM HISTORY

The first organized effort for a municipal sewage collection system was made in the early 1900's when a sanitary sewer system was constructed in the older portion of the City. This basic system was extended by the City and by private individuals without the benefit of a long-range sewerage plan and with little or no construction supervision. Over the years maintenance of the private lines has been turned over to the City.

One of the first major additions to the original system was in the mid-thirties when the system was extended to the east by the addition of sewers on S. E. Byers, S. E. Court Avenue, and S. E. Court Place. Much of the work was done under the WPA program; and the sewers were constructed with concrete pipe, having mortar joints. The upper or easterly end of this sewer was laid at very shallow depths with some portions of the sewer only inches below ground surface. (See Figure 3.)

The next major addition to the ever-growing system came during World War II when the Federal government provided sewer service to the government housing development on the western slopes of the North Hill. Here again this sewer was constructed of concrete pipe with mortar joints. Also, during this same period an independent sewer system was added at the Pendleton Airport, which included a separate treatment plant because the area was somewhat remote from the existing sewerage system and the airport was not then included within the City limits. The airport (Pendair) was later annexed to the City in 1950.

The most significant development in the City's history of sewage collection and treatment was the construction of a new treatment plant and interceptor sewer which bypassed the old treatment plant at the mouth of Tutuilla Creek and carried the domestic sewage to the new treatment plant constructed at the mouth of McKay Creek. (See Plate 2 at the back of this report.) This made it possible for the sewerage system to expand as necessary to meet the needs of the constantly growing City. An interesting aspect of the sewage treatment program was the use of the old treatment plant as pretreatment for industrial wastes, primarily cannery wastes. Chapter V discusses the treatment facilities in more detail.

VIEW OF MANHOLE ON N.W. 12TH STREET NEAR UMATILLA RIVER. NOTE TWO SEWERS AT DIFFERENT LEVELS USING THE SAME MANHOLE.
(FLOW FROM SEWER ON LEFT IS AT LOWER ELEVATION)

VIEW OF MANHOLE ON S.E. ALEXANDER AVENUE AND S.E. 12TH STREET. NOTE HOW NEAR THE SEWER IS TO THE GROUND SURFACE.

FIGURE 3

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

EXISTING SEWER FACILITIES

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The new interceptor sewer was constructed across the Umatilla River in two locations, using concrete pipe embedded in reinforced concrete as shown in Figure 4. The remaining portion of the interceptor sewer was constructed with concrete pipe having mortar joints.

With the treatment plant located where it could serve the McKay Creek drainage basin, a portion of this basin adjacent to U. S. Highway 395 was annexed to the City in 1949;and sewers were installed shortly thereafter. This system at present is the most rapidly expanding system in Pendleton. The McKay sewers were constructed of concrete pipe with the early portions of the system joined by mortar joints and the more recent portions having rubber ring joints. The latest major addition to the system was a sewer serving the Tutuilla Creek area constructed in 1959. This sewer was constructed of concrete pipe using rubber ring joints.

SEWER MAIN SYSTEMS

For the purpose of this report, the existing sewer mains were divided into separate systems as follows:

1. Pendair System
2. McKay Creek System
3. Tutuilla Creek System
4. S. W. 20th Street System
5. Byers and Court Avenue System
6. Dorion and Court Avenue System
7. Emigrant Avenue System
8. N. W. 10th Street System
9. North Main Street System

These systems are described in Appendix A and are shown on Plate 2 bound at the back of this report.

INVERTED SIPHON RIVER CROSSING DURING CONSTRUCTION IN 1948 LOCATED ON THE UMATILLA RIVER NORTH WEST OF THE OLD SEWAGE TREATMENT PLANT. PIPE DIAMETERS 15 INCH, 12 INCH AND 24 INCH.

INVERTED SIPHON RIVER CROSSING DURING CONSTRUCTION IN 1948 LOCATED ON THE UMATILLA RIVER NORTH WEST OF THE OLD SEWAGE TREATMENT PLANT. NOTE ROCK CONDITIONS AT RIGHT OF PICTURE WHICH ARE TYPICAL THROUGHOUT PENDLETON WHERE OVER-BURDEN IS SHALLOW.

FIGURE 4

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

INVERTED SIPHON RIVER CROSSING

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NOV. '62

CONDITION OF SEWERS

Records. Throughout the years many of the sewer system maps and records have been lost. Such information as pipe size, location of sewers, and location of side sewer connections have been misplaced or were never recorded. As previously described, the scope of this study included the preparation of detailed maps showing the present sewer system. In compiling these large maps (submitted separately from this report), every attempt was made to make them as accurate as possible and be representative of the existing sewer system. The maps, however, must be verified in the older systems by noting pipe materials and diameters during cleaning operations, because it is suspected that numerous sewers which were shown to be 8 inches on the older maps are actually 6 inches in diameter.

Materials. The present sewer system consists of clay pipe sewers in the old portion of the City, which were laid starting some sixty years ago, and concrete pipe sewers in the newer areas. At least one section of the system on S. E. Byers Avenue was constructed with steel "invasion pipe" which is now in extremely poor condition. In some areas where the sewers have been exposed, grave signs of deterioration were observed and portions of the old pipe were collapsed. One such section uncovered during the course of being repaired had eroded a new channel outside the old pipe for a considerable distance, where it re-entered the sewer line through another broken section.

Size. The detailed sewer maps show many 6-inch sewers in existing. These sewers, by present-day standards, are considered too small for sewers over 200 feet in length. The 6-inch pipes were originally placed at the upper ends of laterals; but as the system grew and the laterals were extended, many of the 6-inch pipes were used as mains and some were extended with larger diameter pipes upstream. In general, the extension of 6-inch laterals should be accomplished with 8-inch pipe; and the existing 6-inch sewers should be replaced with 8-inch pipe to provide a consistent sewer throughout its length, which could be properly maintained with modern sewer cleaning equipment.

Cover. Generally, the depth of cover over the sewer is adequate to afford protection from freezing and to allow users to connect by gravity flow. As described in Appendix A, however, there are some areas having extremely shallow sewers located near the ends of the older systems. The correction of these conditions is included in the projects described in Chapter IV and shown on Plate 5.

Slope. Another existing situation in the system is that some lines have less slope than is required to keep the sewage solids in suspension.

To correct this condition, it is often necessary to reconstruct several hundred feet of sewer on either side of the flat section to obtain the necessary slopes. The correction of some of the more critical slope conditions is included in project Nos. 2, 6, and 7 as described in Chapter IV.

TELEVISION INSPECTION

Sewers with known problems, along with other sewers selected at random, were inspected by use of a closed circuit television system. The purpose of the inspection was to determine the general condition of the sewers and to establish the value of inspecting sewers by use of television cameras. As would be expected of any random sampling, some lines were in relatively good condition and other lines were in poor condition. Some lines could not be inspected properly because of construction conditions and because some sewers had not been cleaned. (See Figure 5.) A detailed description of the television inspection is bound herewith as Appendix B-1 and B-2.

It was concluded that television inspection of sewers is quite helpful in determining the general condition of the sewers and in locating maintenance problems. Its greatest values are in determining the extent of repair and reconstruction necessary to correct faulty sewer conditions and inspecting new construction for the purpose of accepting or rejecting the completed sewers.

SMOKE INSPECTION

Appendix B-3 describes the smoke tests conducted to determine the value of this procedure in locating infiltration and exfiltration areas, sewer locations, buildings connected to specific sewers, and storm drain connections.

Infiltration, exfiltration, and broken pipes are difficult to locate with smoke where the soil is saturated or primarily clay, silt, and organic materials. It is further complicated if the sewer is under paved streets.

Sewer connections, however, can be easily located with smoke by observing the sewer vents on buildings. (See Figure 6.)

VIEW OF TELEVISION MONITOR SHOWING SEWER ON S.W. 18TH STREET OFF S.W. BYERS AVE.
NOTE RINGS ON SIDES OF PIPE WHICH INDICATE SEEPAGE THROUGH PIPE WALLS.

VIEW OF TELEVISION MONITOR SHOWING 12-INCH SEWER ALONG S.W. EMIGRANT AVENUE BETWEEN S.W. 6TH & S.W. 7TH STREETS. SEWER WAS FLOWING 1/3 FULL AND HAD NOT BEEN CLEANED.
OBJECT ON LEFT OF PIPE IS SIDE SEWER STUBBED INTO MAIN.

FIGURE 5

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

TELEVISION PICTURES OF SEWERS

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A1959·5
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SMOKE TEST FOR SEWER LEAKAGE ON S.E. 4TH. STREET AND S.E. BYERS AVENUE.
NOTE EQUIPMENT
(1) 8 INCH ELBOW AND 1500 C.F.M. HOMELITE PORTABLE BLOWER.
(2) 8 INCH CANVAS DUCT.
(3) 8 INCH ELBOW, PLYWOOD TOP AND WEIGHTS ON MANHOLE.

HOUSE CONNECTION LOCATED BY SMOKE TEST ON S.E. 4TH. STREET AND S.E. BYERS AVENUE. NOTE SMOKE COMING OUT OF ROOF VENTS.

FIGURE 6

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

SMOKE TEST

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

A1959·6
NOV. '62

INFILTRATION

One of the most critical problems in the Pendleton sewer system is infiltration. Excessive infiltration is not new in sewers and is a common problem faced by many communities with older sewer systems. As the sewers become more heavily loaded, the problem of eliminating infiltration becomes more severe.

Observations of the maintenance crew indicate that during high water table periods portions of the Byers-Court Avenue System (see Plate 2) are continually surcharged. This indicates the sewers are not capable of carrying away the infiltration as fast as it enters the sewers; and if the infiltration could be carried away, it would flow into the sewers at increased rates because the hydraulic head (difference in water levels) would be greater if the sewers and manholes were no longer surcharged. The high water table in the Byers-Court Avenue system occurs when the Umatilla River is high and flowing over 1,000 cubic feet per second. (See Plate 7.)

A review of the sewage treatment plant flow records from 1954 to the present indicated Monday, April 22, 1958, represented the date of maximum sewage flow when the canneries were not operating. (See Plate 8.) On this date a section of the McKay Creek sewer had water flowing over the manholes which resulted in surface water entering the system. The total flow at the treatment plant was 5.5 million gallons per day (mgd). The infiltration rate was estimated to be 3.1 mgd by making the following assumptions:

1. Average Sunday dry weather flow was 1.4 mgd.

2. Average variation from Sunday to Monday was plus 0.3 mgd. (Monday having a greater flow than Sunday.)

3. McKay Creek surface water infiltration estimated at 0.7 mgd.

4. Peak infiltration throughout system (other than McKay Creek area) equals remaining flow on April 22, 1958 (3.1 mgd).

By further assuming that most of the groundwater infiltration enters the sewers in the more level portion of the existing sewer system, which comprises about 520 acres, the peak infiltration rate becomes 6,000 gallons per acre per day which is four times the accepted allowable design rate of 1,500 gallons per acre per day. It is believed that the infiltration is primarily concentrated in the Byers-Court Avenue system, which has been considered for major rehabilitation as described in Chapter IV.

An attempt to verify the infiltration conditions was made by conducting flow measurements at various points throughout the sewer system. The data collected during this program is tabulated in Appendix C, and the flow measuring points are shown on Plate 2. Data were collected pertaining to the depth of flow, velocity of flow, amount of rain and river stage for the duration of the investigation. Readings were taken on Monday, Wednesday, and Friday at approximately the same time of day, which minimized the variation in readings resulting from the normal fluctuation in flow throughout each day. Therefore, the significant changes indicated the presence of infiltration. As can be seen on Plate 7, the flow measured on S. E. Byers Avenue at S. E. 6th Street and at Main Street have marked increases and decreases depending on the river stage. The section of sewer on Byers Avenue between S. E. 4th and S. E. 5th Streets was plugged at the upper end, and the normal flow was diverted on two different occasions. Both times a flow of 0.15 foot deep in a 10-inch sewer was observed. The water was clear and cold and of constant depth for over an hour. It was concluded that the source was not a side sewer and that this represented an infiltration flow of approximately 45 gpm or 0.06 mgd. It is apparent that the Byers-Court Avenue system is one of the major contributors to infiltration.

ULTIMATE COLLECTION SYSTEM

CHAPTER IV

ULTIMATE COLLECTION SYSTEM

AREA SERVED

The first step essential to developing a master plan for the sewer system is the determination of the boundaries of the area for which the sewer service is ultimately to be provided. In most cases, establishment of these boundaries for sewer service consideration is determined by City planning policy and is dependent, to a great extent, upon how far into the future a sewer system master plan is projected. Often the ultimate service area is quite well defined by topography and other natural limits. In other instances, however, no apparent limiting factors exist, and the extent of the service area must be based on estimates of future land use and development, existing zoning laws and regulations, value and nature of the land for future development, and the extent to which sewer service planning is to be extended into the future.

Plate 1 indicates the limits of these ultimate service areas under consideration in this study. These limits were established after studying Planning Commission reports and surrounding topography and after giving due consideration to the potential development area and to the need for sanitary service in the areas surrounding the City of Pendleton. The existing City sewer system has been utilized to the fullest extent in providing service to this area and is considered the core of the ultimate system to serve the areas selected for service.

The ultimate service area considered in this study includes approximately 5,300 acres. (The Airport and Eastern State Hospital are excluded.) Of this total, 2,200 acres are within the present City limits and generally are served or can be served by the existing City sewer system. A population density equal to the present City average of 6.5 persons per acre would result in the ultimate service area accommodating a population of at least 34,450 people.

The design of the sewers proposed herein was based principally on two factors: (1) domestic sewage flow from the various areas; and (2) population density for each area as shown on Plate 1. Allowances have been made for stage development of the trunks and mains to serve the expanding needs of the service area. If certain areas do not develop as rapidly as anticipated, then service can be delayed until a demand exists. The limits of the service area, as shown on Plate 1, are by no means fixed and can be altered to suit subsequent population growth patterns. The overall sewer plan has been developed to provide as much flexibility as possible, and to allow an accelerated construction program if the need arises and financial capabilities permit.

PRELIMINARY PLANS

The location of the proposed sewers can only be tentative, and it is expected that alterations and modifications to the preliminary designs, including occasional resizing of the facilities, will be required as a result of the detailed engineering analysis necessary for the preparation of construction drawings. However, the plans presented herein are sufficiently valid to compare alternate routes and to use as a basis for financial planning.

In those areas in which the development pattern and streets have been established, cost estimates have been prepared. Preliminary locations of sewer mains and laterals are shown on Plates 3 and 5. In those areas which are still undeveloped to a large extent, the location and estimated costs of the principal mains serving the area are shown, but no attempt was made to locate or estimate the costs of the smaller mains and laterals. As development in these areas creates a need for sewer service, the overall plan can be supplemented by more detailed planning for the individual lateral sewers.

BASIS FOR SEWER DESIGN

All proposed main and interceptor sewers have been designed to handle the maximum flow from their respective service areas during the design life of the system. Since it is virtually impossible to enlarge the sewer system without complete replacement, it is important that all factors affecting the ultimate capacity of the sewer be considered and that design criteria be used which will insure satisfactory service to the area intended. Variables in each area under consideration, such as expected population densities, maximum per capita flow, maximum expected groundwater infiltration, and industrial waste flows, all affect design criteria for sewers and may change considerably from area to area.

All designs developed herein have been based on the assumption that the sewers will only carry domestic and industrial sewage with certain allowances for unavoidable infiltration, while normal storm water runoff will be disposed of separately. The practice of constructing separate storm sewers is being universally adopted in the United States and is a requirement of the Oregon State Sanitary Authority.

The design population densities and flow criteria for new sewers were described in Chapter II.

PROPOSED ULTIMATE COLLECTION SYSTEM

The proposed ultimate sewer system consists of three systems served by one treatment plant as shown on Plate 3. The systems are the Pendleton system, the McKay system, and the Pendair system.

The Pendleton System. The Pendleton system will serve the area east and north of the treatment plant. The primary trunk sewer was located on Byers and Court Avenue as shown on Plate 3. This sewer was selected for the primary trunk because the location places it at a common low point for the entire system; and the reconstruction of the existing sewer, which lies near the river, gives partial relief to the excessive infiltration in the existing sewer. Where added capacity was needed for the primary trunk sewer, it was decided to completely replace the old Byers line to S. W. 10th Street instead of constructing a parallel line for the following reasons:

1. The new line would have less infiltration.

2. Most of the existing sewer should be at a lower elevation for proper side sewer and lateral connection.

3. The problem of interference during construction with existing side sewers and other utilities would be minimized.

The ultimate primary trunk sewer diverts from the existing trunk at S. W. 10th Street and S. W. Court Avenue to allow the ultimate sewer to be constructed as low as possible; to reduce the cost and difficulty of construction; to eliminate the pump station now serving the north portion of the S. W. 20th Street system; and because the size and condition of the existing sewer beyond this point are adequate.

As can be seen on Plate 3, the primary trunk rejoins the existing system just upstream from the first Umatilla River crossing near the old treatment plant. The existing trunk sewer beyond this point has adequate capacity to handle the ultimate sewage flow.

McKay System. The McKay system will serve the McKay Creek drainage basin south of the old treatment plant. The trunk sewer will be constructed at the low point in the valley to serve the maximum area and will extend to the vicinity of the U. S. Highway 395 crossing of McKay Creek and will generally follow the creek southward beyond the limits of Plate 3.

Pendair System. The Pendair system serves the area from the airport to the Umatilla River. A river crossing and pumping station are necessary, as shown on Plate 3, to transmit all the sewage from the Pendair system into the treatment plant. With the exception of the collection system at the airport, the entire Pendair system will be new construction.

PROJECTS

These systems were broken down into individual projects which can be constructed as the need develops. An approximate time schedule for the projects has been prepared as shown on Plate 5; and detailed cost estimates for each project are included herein as Appendix D. The estimates are all based on projected construction costs for 1963. Projects constructed at later dates must have the estimates updated as necessary to correspond with the future changes in costs of sewer construction. The projects are as follows:

1. Pendair Main. The Pendair main, as shown on Plate 5, should be constructed in the immediate future because of the extremely poor condition of the Pendair sewage treatment plant and effluent line. The treatment plant was constructed during the war, and many of the deteriorated parts of the plant are virtually irreplaceable. The estimated cost for this project is $96, 300 (see Appendix D for detailed cost estimates), which includes the cost of the pump station and river crossing, using pipe diameters of 8 and 12 inches. As shown on Plate 6, the depth to rock is variable throughout the main and should be determined in more detail before construction to insure that rock excavation costs are reasonably accurate for budgeting purposes.

2. S. E. Byers Avenue Trunk. This primary trunk should be reconstructed before 1965 at an estimated cost of $117, 600, using pipe diameters from 15 to 24 inches. It must be constructed before the Riverside area can be served. The lower termination point for this project was located at S. E. Second Avenue because the existing sewer beyond this point has the capacity to handle the sewage from the area served for the next 5 to 10 years and the sewer line is in relatively good physical condition. Also included in this project is the sewer on S. E. 9th Drive. This section will intercept the existing main on Court Place, which serves the woolen mills and the area of St. Anthony's Hospital. This is considered desirable to improve the flow conditions in the larger trunk until the Riverside sewers are completed, to relieve the existing sewer on S. E. Court Place, and to comply with the proposed ultimate sewerage plan for serving a portion of the south hill through a new 12-inch main on S. E. 10th Street (Project 13).

3. State Hospital - River Main. As shown on Plate 5, this 10-inch main will serve the entire State Hospital system and should be constructed before 1965 at an estimated cost of $21,000. The high school, State Hospital, a potential junior college site, and residential areas along U. S. Highway 395 will ultimately be served by this main.

4. State Hospital - High School Main. This main shown on Plate 5 will serve the high school and the western portion of the existing N. W. 10th Street system, which was constructed during World War II and has limited expansion capabilities and questionable physical conditions such as the manholes shown in Figure 3. This main should be constructed before 1965, possibly in conjunction with the State Hospital-River Main, Project No. 3. The estimated cost for this main is $22,400, using a pipe diameter of 8 inches.

5. Riverside - Initial Project. The initial Riverside project will probably materialize shortly after 1965. The area must first be annexed to the City. However, because of the present concentration of homes along Riverside Avenue, it may not be too long before a health hazard develops if sewers are not provided soon. The estimated cost for this project, as shown on Plates 3 and 5, is $217,300. The project includes a pump station, river crossing, and the various sewers ranging in diameter from 8 to 18 inches.

6. S. E. Court Place Reconstruction. A section of the existing Court Avenue-Court Place system, as described in Appendix A (see Plate 5), has a negative grade which should be replaced in the not too distant future. Funds should be available for this reconstruction project between 1965 and 1970. The work is estimated to cost $14,900 for a new 10-inch sewer.

7. Alexander Avenue Reconstruction. It has also been assumed that funds should be provided for rebuilding the sewer on Alexander Avenue and on S. E. 11th Street to the main on S. E. Byers Avenue sometime between 1965 and 1970. This 8-inch sewer is estimated to cost $13,800 and is considered necessary because the existing sewer is far too shallow, as shown on Figure 3, which makes it difficult to serve the adjoining property by gravity and presents operational and maintenance problems during freezing weather.

8. State Hospital - U. S. 30 Eastern Main. This main is the initial portion of the sewer to serve the State Hospital area north of U. S. Highway 30. The remaining portion of this sewer is Project 22. The existing hospital buildings north of the highway are now using septic tanks. The eastern section is anticipated to be constructed

between the years 1965 and 1970 at an estimated cost of $14,700, using only 8-inch pipe. The laterals required to serve the State buildings are not shown and are assumed to be constructed by the State as necessary to intercept the existing septic tanks.

9. State Hospital - U. S. 395 Main. This main extends northward on U. S. Highway 395, as shown on Plate 5, and is planned for construction in two sections. The first section is shown to extend up the highway 1,660 feet to serve the area which may develop between the years 1965 and 1970. The cost of this 8-inch sewer is estimated to be $19,600. The remaining portion will be constructed sometime in the future as the area is developed.

10. S. E. Byers Avenue - Infiltration. Infiltration correction should be carried out on the laterals and side sewer in addition to the new primary trunk construction on S. E. Byers Avenue (Project No. 2). Infiltration correction can be accomplished by two methods. The first consists of replacing the laterals and side sewers. This can be very expensive and has questionable value when the cost of replacement is compared to the benefits received. The second method consists of treating the existing sewer by utilizing a pressure grout applied from the inside of the line. The grout is applied with a packer, as shown in Figure 7, which consists of a hollow tube with two rubber tires mounted at each end. It is drawn through the line in conjunction with a TV camera; and when a questionable point is seen by the TV camera, the pneumatic tires are inflated and the questionable point is tested by air pressure. If the pressure is not sufficient, the section is then pressure grouted. The side sewers are bypassed after observations on the TV monitor. There are also recent processes for applying pressure grout internally to the side sewers and then flushing them to insure that they are fully usable without obstructions to flow.

Pressure grouting can also be used in external treatment of areas, such as around manholes, which have infiltration. This type of arrangement is shown on Figure 7.

There are an estimated 7,400 feet of sewer laterals and 2,200 feet of side sewers which should be treated for infiltration. The internal pressure grouting would cost an estimated $23,200 and should be scheduled as soon as possible after the S. E. Byers Avenue trunk is completed.

11. Bailey Avenue Main Reconstruction. The reconstruction will replace a portion of the 10-inch sewer between N. W. 8th and 9th Streets at an estimated cost of $8,300. This section needs replacement because of buildings located over the present sewer line and the exfiltration problems that exist in this area. This project probably will be constructed between 1970 and 1975.

PACKER USED FOR REPAIRING SEWER LEAKS BY INTERNAL PRESSURE GROUTING. SEWAGE PASSES THROUGH HOLLOW CENTER WHILE GROUT IS FORCED INTO CHAMBER BETWEEN TIRES AND INSIDE FACE OF PIPE.

HOLDING PACKER IS ED WRIGHT OF SOIL SOLIDIFIERS, INC., SEATTLE, WASH.

SETUP USED FOR REPAIRING SEWER LEAKS BY EXTERNAL GROUTING.

NOTE LINE OF GROUT TUBES AT RIGHT CENTER OF PICTURE.

FIGURE 7

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

INFILTRATION CORRECTION METHODS

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NOV. '62

12. Riverside - Future Project. This final portion of the Riverside project represents the remaining sewers to serve the district as shown on Plates 3 and 5. The project may develop between 1970 and 1975, and is estimated to cost $27,000 for the principal main as shown on Plate 5.

13. S. E. 10th Street. This main will probably not be constructed until after 1970 when the residential area south of this area begins to develop (see Plates 1 and 5). The 8- and 10-inch sewers are estimated to cost $23,900.

14. S. W. Court Avenue Trunk. Depending upon the growth rate of the area served, this portion of the primary trunk, as shown on Plate 5, will be reconstructed between 1970 and 1975 with 24-inch diameter pipe at an estimated cost of $63,400.

15. Riverside - Mount Hebron. This section of sewer consists of a main sewer that will serve the area as shown on Plates 1 and 5, and laterals that will serve the Mount Hebron Drive area. The estimated 1963 construction costs total $96,100. The main will probably not be necessary for at least 10 years. This area is unusually rocky; and even though over $7,000 has been included for rock excavation, additional rock excavation costs may occur depending upon the final location of the sewer lines. The sewers included in this project are shown on Plate 5 and include 8- and 10-inch sewers.

16. North McKay Creek Trunk. The North McKay Creek area is the fastest growing area in Pendleton. At present, areas needing sewers can be connected to the existing system until a trunk is built. The construction of this sewer trunk, however, may not be justified until after 1975. It is estimated to cost $127,800 and will consist of 24-inch pipe with two 12-inch sewers connecting to the existing system.

17. South McKay Creek Trunk. This 21-inch trunk will probably not be constructed until after 1980, and the estimated 1963 construction costs total $84,400. This portion of the McKay Creek trunk will no doubt be extended in sections and at present, it is impossible to determine the length of the sections until the area develops further.

18. N. W. King Avenue Main Reconstruction. The costs for upgrading this system to handle the ultimate service area involves the replacement of the undersized sewers (6 inches in diameter) with 8-inch pipe at an estimated cost of $20,600. This may be required between 1975 and 1980.

19. S. E. Third Street Main. This main, as shown on Plate 5, is planned for construction during the period of 1975 and 1980 at an estimated cost of $31,600, using a pipe diameter of 8 inches. It intercepts (see Plate 3) the portion of the Emigrant Avenue system which lies east of S. E. Third Street and is terminated at Dorion Avenue, as the remaining sewers are in relatively good condition and are adequately sized for serving the western portion of South Hill. The detailed cost estimate includes an allowance for rock excavation. As previously mentioned, the cost of rock excavation is difficult to estimate and, at the time of construction, can result in some variance from the estimated cost of the project.

20. Lower Pendair. This project provides service to the area between the Umatilla River and U. S. Highway 30 and is not planned for construction until after 1975. The 1963 estimated construction costs total $93,000. It is probable that this area may develop prior to 1990 in lieu of one of the other areas pictured on Plate 1.

21. Roundup Trunk. This sewer completes the primary trunk, as shown on Plate 5, and will probably not be justified for construction until sometime after 1975. It is estimated to cost $106,400 using a pipe diameter of 24 inches. Timing of this project depends particularly upon the rate of growth of the service area.

22. State Hospital - U. S. 30 Western. This main will not be constructed until after 1980 at an estimated cost of $24,300, using a pipe diameter of 8 inches.

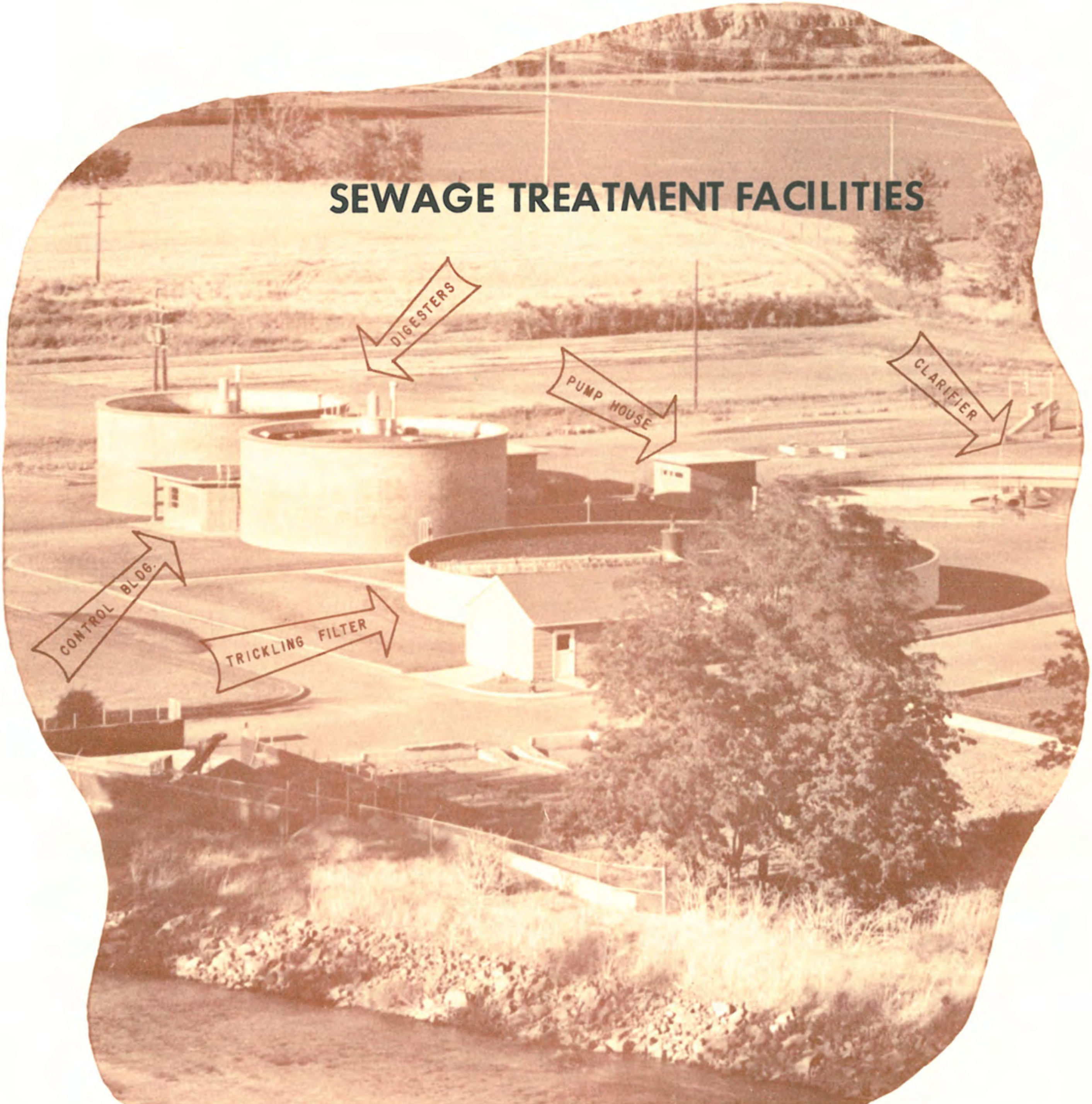
23. East Pendleton Railroad Main. As shown on Plate 5, this 8-inch main will serve the future developments on the eastern side of Pendleton. It is not anticipated as necessary until after 1980 and the 1963 estimated construction costs total $34,400.

A summary of the project costs is presented in Table 5.

TABLE 5

SUMMARY OF SEWER PROJECTS

Construction Period	Proj. No.	Description	Cost	Total Cost of Construction Period
Before 1965	1	Pendair Main	$ 96,300	
	2	S. E. Byers Ave. Trunk	117,600	
	3	State Hospital-River Main	21,000	
	4	State Hospital-High Sch. Main	22,400	$ 257,300
1965-1970	5	Riverside-Initial Project	217,300	
	6	S.E. Court Place Reconst.	14,900	
	7	Alexander Ave. Reconst.	13,800	
	8	State Hosp. -U.S. 30 Eastern Main	14,700	
	9	State Hosp. -U.S. 395 Main	19,600	
	10	S.E. Byers Area-Infiltration	23,200	303,500
1970-1975	11	Bailey Ave. Main Reconst.	8,300	
	12	Riverside-Future Project	27,000	
	13	S.E. 10th Street Main	23,900	
	14	S. W. Court Avenue Trunk	63,400	
	15	Riverside - Mount Hebron	96,100	218,700
1975-1980	16	N. McKay Creek Trunk	127,800	
	18	N. W. King Ave. Main Recon.	20,600	
	19	S. E. 3rd Street Main	31,600	
	20	Lower Pendair	93,000	
	21	Roundup Trunk	106,400	379,400
Beyond 1980	17	South McKay Creek Trunk	84,400	
	22	State Hosp. -U.S. 30 W. Main	24,300	
	23	East Pendleton Railroad Main	34,400	143,100
		TOTAL		$1,302,000


SEWAGE TREATMENT FACILITIES
DIGESTERS
PUMP HOUSE
CLARIFIER
CONTROL BLDG.
TRICKLING FILTER

CHAPTER V

SEWAGE TREATMENT FACILITIES

EXISTING TREATMENT FACILITIES

The existing sewage treatment facilities were completed in 1951 and are shown on Figure 8. The sewage treatment plant was designed to handle a hydraulic load of 5.0 million gallons per day and to treat a dissolved organic waste having a population equivalent of 53,750, of which 20,000 would be from the future connected population, and 33,750 would be waste from the pea processing industry, after it had received pretreatment in facilities located at the City's old treatment plant site.

The sludge digestion facilities were designed to provide three cubic feet of capacity for a connected population of 20,000. Chlorination facilities were provided for disinfection of the plant effluent while it passed through the plant outfall line into the Umatilla River.

Sludge drying facilities were constructed so as to provide 1.36 square feet of drying area per capita for a connected population of 20,000.

PRESENT PLANT LOAD

As a routine operating procedure, the City's plant operating personnel prepare and submit to the Oregon State Sanitary Authority monthly operating reports which indicate the nature of the waste being received at the treatment plant, together with results of the tests conducted to establish the degree of treatment obtained in the various plant units.

A review of these reports indicates that, except for the period of the year that waste is received from the pea processing industry, the plant is capable of treating the waste in a proper manner. The volume of both the hydraulic and the organic load from the industrial waste has increased to such an extent that, for this period of the year, plant effluent is discharged into the Umatilla River without the reduction in organic load that the State Sanitary Authority has required. (See Appendix E.)

The hydraulic load placed on the treatment plant during the pea processing season has increased to a major extent, resulting in reduced efficiency in the plant units. As an example, the hydraulic load

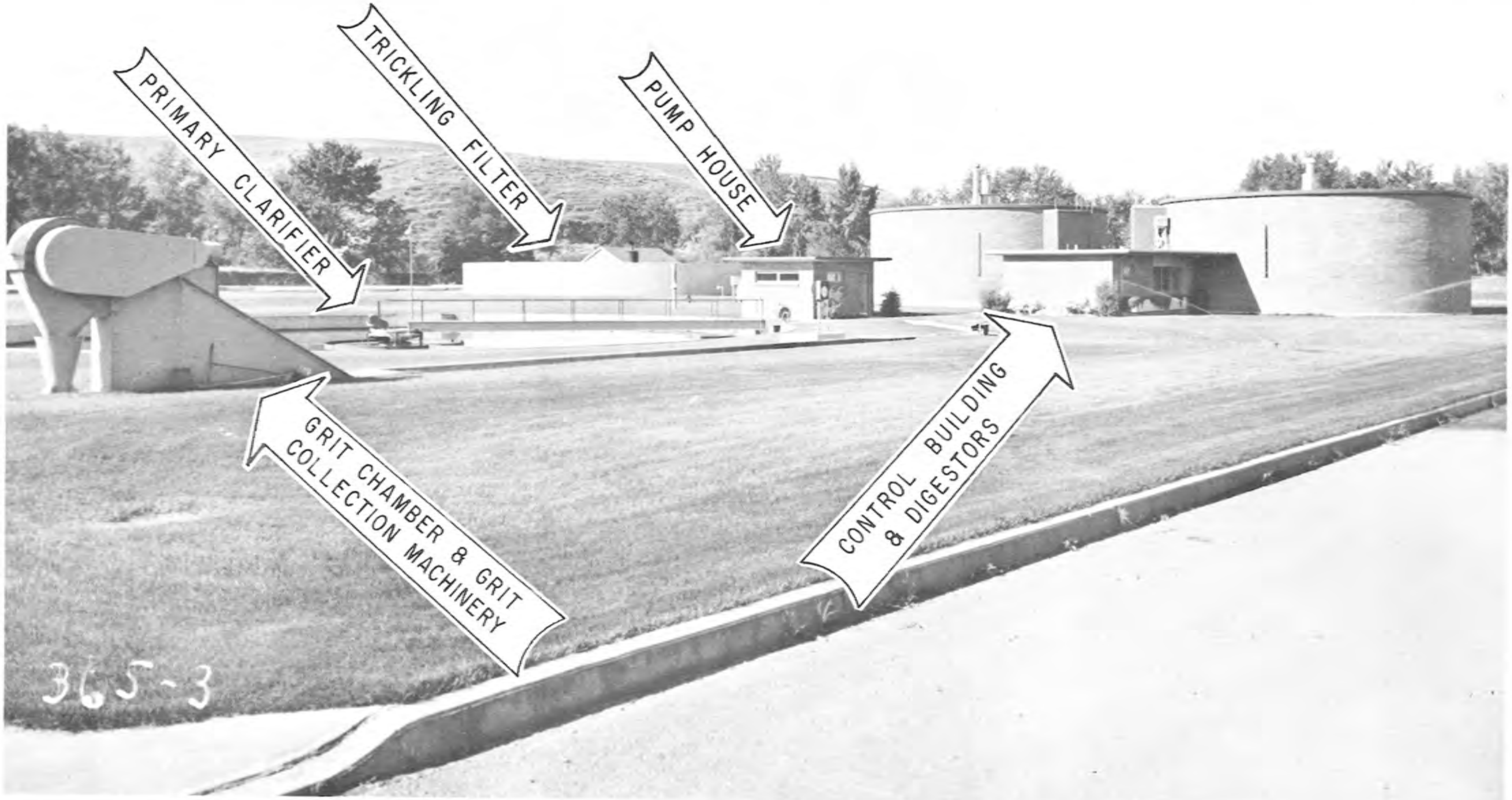


FIGURE 8

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

EXISTING SEWAGE TREATMENT PLANT

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during the last week of July 1962 was 7.2 million gallons per day. This resulted in a detention time in both the primary and final clarifier of one hour and twenty-one minutes, rather than the two hours for which they were designed.

Based upon the test results referred to by the Oregon State Sanitary Authority in their letter dated September 28, 1962, as shown in Appendix E hereof, and in conformance with periodic tests as conducted by the treatment plant personnel, the trickling filter is not capable of providing the required oxidation of the combined industrial and domestic waste as received at the main plant. These tests indicate that the volume of industrial waste, as received in 1962, can only be properly treated by the construction of additional facilities at the main treatment plant.

Plant tests indicate that the sludge digesters have been able to digest most of the solids received in a proper manner. Considerable difficulty has been experienced in the accumulation of floating solids on the surface of the digester. This difficulty can be most economically corrected by the installation of a mechanical sludge mixer in the primary digester. Experience has indicated that the capacity of the digesters can be increased by at least 30 to 50 percent by the addition of the mechanical mixer. This addition would result in adequate capacity for the proposed third-stage construction.

Plant operational reports indicate that the volume of chlorine used each year is such that a sizeable reduction in chlorine costs can be obtained by the installation of chlorine handling facilities, so the chlorine could be obtained in one-ton containers. It is estimated that, with this equipment, the current chlorine costs could be reduced by $750 a year. Plant effluent during the canning season often requires chlorine to be added at a rate in excess of the capacity of the present meter. New chlorine metering equipment should be provided which will be capable of feeding the required dosage at all times.

PRETREATMENT PLANT OPERATION

Experience in the operation of the industries' pretreatment facilities indicates that there are many times when the screening equipment fails to operate properly, resulting in an excessive volume of industrial waste solids remaining in the waste received at the City's pretreatment plant. Where these solids remain in the waste an extended period of time, an increased portion of the solids are dissolved and are thereby retained in the waste for treatment in the main treatment plant.

The remaining solids also require decomposition and this increases materially the load on the plant.

Failure to obtain proper operation of the industries' screening equipment is a common problem and can probably best be corrected by installing nonremovable screens in the sewer line leading to the City's plant, so that a failure in proper operation of the screening equipment will result in flooding of the industries' facilities.

The installation of equipment to continuously sample the waste which is received for treatment and the enforcement of sewer service charge which reflects an increased charge resulting from improper operation of their equipment should also be provided. This procedure has recently been adopted by two major cities in Oregon. It requires, however, a considerable expense in providing the technical help in the plant laboratory to carry out these tests.

ADDITIONAL TREATMENT PLANT FACILITIES

The preliminary plant layout shown on Plate 4 provides for expanding the present treatment facilities to serve a connected population of 32,000, plus an industrial load of two times the present load. The proposed layout provides for the addition of the clarifiers and filters in two stages. While the layout indicates units of equal size, it is possible that final design of either of the stages could develop requirements for varying these sizes to some extent. Since the existing treatment facilities were constructed in two stages, the future construction steps have been designated the third and fourth stages. Stage 3 should be constructed before 1965 and Stage 4 will probably be required between 1975 and 1980.

CLARIFICATION FACILITIES

The primary and final clarifiers will effectively remove the suspended organic matter if the clarifier basins have sufficient capacity to permit these solids to settle out of the water-borne waste before the liquid is permitted to flow on out of the settling basin. Experience has indicated that approximately 120 minutes should be provided for this unit of the treatment facility. The design of adequate clarification capacity is based, therefore, on the volume of waste to be treated when the plant is operating at design capacity.

As set forth in other sections of this report, it is estimated that the 1990 population of Pendleton would be approximately 32,000; and at that time or before, it can be expected that waste from the

Pendleton Pea Processing Plant will need to be treated in the City's plant in the same manner that the waste from the Smith Pea Processing Plant is now being treated.

It has been assumed, therefore, that the expanded plant should be capable of serving a connected population of 32,000 plus an industrial waste of a volume of twice that now being received.

Analysis of the existing plant records indicates that in 1961, the average per capita flow in July without industrial waste was 185 gallon per day. This is based upon a metered flow of 2.80 mgd and an assumed connected population of 15,000. Using the same per capita flow for the estimated 32,000 population would result in a domestic load of 6.0 mgd. Flow records for June and July 1962 also indicate an industrial waste volume of approximately 3.0 mgd.

Estimated (1990) volume during pea processing season:

Source	Volume, mgd
Domestic waste (32,000 pop.)	6.00
Smith Cannery	3.00
Pendleton	3.00
Total Flow	12.00

The capacity of the existing primary and final clarifier is 5.0 mgd each. This would require new clarifiers to handle the 7.0 mgd. This can be provided in one unit having a diameter of 110 feet or two units each having a diameter of 80 feet.

TRICKLING FILTER

The size of high rate trickling filter facilities is based upon the volume of dissolved organic matter to be applied to the filter under design conditions and based upon the degree of removal obtained from such a loading. As indicated above, it appears that any additional facilities should be capable of providing a total plant capacity to serve a domestic population of 32,000; and in addition, an industrial load approximately twice the volume of the existing load. In order to establish the volume of the existing load, it will be necessary to carry out a detailed daily sampling and testing program for a complete canning season. The volume of these wastes varies considerably from day to day and, therefore, any design of treatment facilities based upon results from only a few samples could be considerably in error.

It seems essential, therefore, that plans be made to provide the staff to conduct a detailed sampling and testing program throughout the next canning season. These tests should be conducted on both the influent to the plant as well as on the treated effluent. These tests would establish the extent of the organic load applied to the existing units and the percent removal being provided by the existing facilities. Prior to such a sampling and testing program, it is essential that the food processing industry improve, in every manner possible, their pretreatment facilities so that the design of the trickling filter can be based upon organic loads which cannot be more economically removed by improved operation of the industry's facilities.

Following the analysis of these test results, the proper size of the additional filter units could then be established.

The trickling filter recirculation pumping stations, as with the primary and final clarifiers, are also designed based upon the volume of waste to be pumped. The size of each of these units has, therefore, been established as approximately twelve feet square and equipped with pumping facilities similar to the existing pump.

DIGESTER

In order to eliminate the floating scum problem in the digester facilities and in order to expand the capabilities of the existing digester units to decompose a larger volume of organic solids, it is proposed that internal mixing and heating facilities be added in one of the units. This proposed revision would provide for selecting one of the two units to be converted to a primary type digester. The floating cover would then be supported permanently in a maximum elevated position. A draft-tube type mechanical mixer would be installed in the digester with the mixing equipment supported from the fixed cover and properly positioned to force the sludge downward through the tube. The draft tube would be of a double tube design to provide a watertight jacket surrounding the inner tube. Circulation of hot water through the jacket would provide the required heat to maintain the sludge at the optimum temperature for high-rate digestion.

CHLORINATION EQUIPMENT

During periods of maximum organic load from the canneries, the chlorine demand exceeds the capacity of the existing chlorine meter; and since the chlorine demand varies depending on the strength of the organic waste, it is recommended that chlorine metering equipment be installed which is capable of being automatically controlled by a chlorine residual measuring equipment. This equipment would assure the required amount of chlorine at all times and would prevent the feeding of an excess amount at any time.

One-ton chlorine container handling equipment should also be installed in order to purchase the chlorine in a more economical manner.

SEWAGE GRINDER

At the time the existing sewage grinding basin was constructed, space was provided for the addition of a second grinder at a later date. Standby bar screens were installed in the space for the future grinder. Maintenance and repair on the one grinding unit has increased to the extent that an additional grinding unit should be included in any proposed plant additions.

ESTIMATED CONSTRUCTION COSTS

Preliminary estimates have been prepared for the probable construction cost of the additions considered in this report. These costs have been tabulated and are presented in Table 6 of this report. The estimates are in most instances based upon records of construction cost for similar size units for other communities and corrected to conform with the 1963 construction cost index. Any construction contemplated later than the 1963 construction period should have the estimated cost adjusted to conform to the current construction cost index.

It must be pointed out here that the estimated cost shown should be approximate only, as the size of the trickling filter units will need to be definitely established following the results of the testing program hereinbefore recommended. A supplement to this report will be prepared and presented at such time as the size and cost of this unit is definitely established.

TABLE 6

COST ESTIMATE
OF
SEWAGE TREATMENT PLANT ADDITIONS

Item No.	Item	Estimated Construction Costs Third Stage	Fourth Stage
1.	Chlorine handling facilities	$ 7,500.00	
2.	Sewage grinder	7,500.00	
3.	Sludge mixing and heating facilities	18,000.00	
4.	Primary clarifier (80'dia.)	52,000.00	$ 52,000.00
5.	Filter recirculation pumping station	16,000.00	10,000.00
6.	Trickling filter (110' dia.)	82,000.00	82,000.00
7.	Final calrifier (110' dia.)	52,000.00	52,000.00
8.	Interconnecting piping	20,000.00	4,000.00
	Total Construction Cost (1963)	$255,000.00	$200,000.00
	Add 25% Contingencies & Engineering	63,750.00	50,000.00
	Total Cost	$318,750.00	$250,000.00

SEWER MAINTENANCE

CHAPTER VI

SEWER MAINTENANCE

NECESSITY OF MAINTENANCE

The history of the Pendleton sewers, as previously described, has resulted in sketchy maintenance records throughout the life of most of the Pendleton sewers. It is not known if many of the old clay sewers shown on the City maps are 6 or 8 inches in diameter. Numerous service connections and actual sewer laterals are not accurately located and in some cases not known to exist. Poor horizontal and vertical alignments of sewers are identified only when the sewers become filled with solids and fail to function properly. Deficient sewer construction and pipe materials are located similarly when the sewers no longer function adequately.

As a result of the historical developments in the sewer construction and maintenance, the sewer maintenance crew is operating on a basis of correcting stoppages, when they occur, with only minor flushing and rodding activities as a preventative maintenance program. Often the problem areas are identified too late for normal rodding, bucketing, and flushing to be successful because the sewer has deteriorated and eroded to require complete replacement.

The increase in population and resulting increase in sewage flow will increase the stoppages, and the rate of detecting faulty sewer materials and construction will increase unless a preventative maintenance program is carried out continuously. Even sewers constructed perfectly with the latest pipe materials will require occasional maintenance because objects such as rocks, rags, diapers, sand, and rodents will create stoppages. Also, sewers having little flow will often not develop scouring velocities of at least two feet per second; and the solids will settle out or hang up on the sides of the pipe, creating offensive odors and eventual stoppage of the flow. A sewer that is plugged with solids to the extent that rodding is required to create a flow cannot be expected to have the hydraulic capacity of a clean sewer. Its hydraulic capacity can be redeveloped only by removing all of the solid material which has been deposited in the sewer. These conditions can only be controlled and corrected by a properly organized maintenance program.

PREVENTATIVE MAINTENANCE PROGRAM

Some cities in Oregon and Washington have recently intitiated maintenance programs which result in each sewer being cleaned at least once every three to five years. Eugene, Oregon, is now on a five-year cycle. Yakima, Washington, has a similar program starting. The programs consist of:

1. Rodding the sewer to break open any fully plugged portions and to cut up the roots at faulty joints and at breaks in the pipe.

2. Bucketing the sewer to remove the large volumes of heavy solids into trucks to be hauled away.

3. Balling the sewer to remove the remaining sludge, scum, and smaller solids. (Often simultaneous with flushing.)

4. Flush the sewer to insure everything loose is carried away from the pipe walls.

In a few instances, cities find that bucketing is often not necessary after the program is underway and all solids can be removed by pulling an inflated ball through the sewer.

In addition to a recurring maintenance program, some cities, such as Yakima and Seattle, are developing a photographic log of their entire sewer systems by taking colored pictures every two to five feet throughout the sanitary sewer. This locates faulty side sewer connections, broken pipe, poor joints, roots, faulty alignment, and frequent infiltration. The camera and other equipment to carry out the photographic program cost from $6,000 to $8,000. It takes a three-man crew to efficiently take the pictures,and from 500 to 1,000 feet can be photographed per day after the sewers are thoroughly cleaned.

Although the entire cleaning and photographic program,as described above,is probably not justified in the City of Pendleton, a minimum program as follows should be initiated.

1. Purchase a new power rodder, additional rods, and tools which will cost approximately $5,500 for modern equipment as manufactured by Flexable or O'Brien.

2. Purchase a combination bucketing machine and truckloader at a cost of approximately $2,000.

3. Purchase large-size buckets and balls to supplement the existing City equipment, estimated to cost $500.

4. Conduct a program two days per week with a two-man crew of rodding, bucketing, balling, and flushing an average of 1,000 feet of

sewer per day. This will allow coverage of the entire existing system once every three years.

5. During the above maintenance program, keep records of sewer sizes, pipe materials, conditions, problems and pipe failures.

6. Program construction and repair projects to rectify the defective sewers identified in the maintenance program.

7. When at least five major projects are identified and scheduled for repair, retain the services of a television inspection organization such as Video Scan to establish the actual repair method and the location.

8. Budget for one additional man in the sewer maintenance department to provide the implementation of the above program.

9. Review the program costs annually and modify as necessary to insure its success.

The initial cleaning program will be difficult to accomplish until all sewers have been cleaned once and all the unusual problems have been encountered and the manhole and sewer locations have been established. The cleaning system should start at the upper ends of the laterals and continue methodically to the treatment plant to allow all flushings to be continuously washed through the entire sewer system.

During the course of accomplishing items Nos. 4, 5, 6, and 7 of the preventative maintenance program, the repair projects should be classified as follows:

1. Complete reconstruction between manholes

2. Isolated reconstruction at service connections, broken pipe, or faulty initial construction.

3. Infiltration and exfiltration repair by pressure grouting.

4. Continued surveillance without repair.

The reconstruction projects can be scheduled as soon as they are identified. However, the grouting projects should accumulate until the first circuit of the cleaning program is completed, because the mobilization of a pressure grouting organization such as Soil Solidifiers, Inc., in Seattle is expensive and as much work as possible should be concentrated into one effort. The City of Walla Walla has just completed a program of infiltration correction during which Soil Solidifiers, Inc., was employed. The cement-chemical grout was considered quite successful and appears to be one of the better methods of grouting sewers of the many systems tried in recent years. As shown on Figure 7, Soil Solidifiers, Inc., is capable of grouting from within the sewer or from the outside. Figure 9 shows pictures of the same sewer before and after infiltration correction by pressure grouting. Service connections are often located by use of a television camera used in conjunction with the grouting. The grouting costs vary from \$0.50 to \$4.00 per foot for normal sewers, although the grouting company works on a daily basis of approximately \$300 per day.

MAINTENANCE SUMMARY

It is often difficult to recognize the value of maintaining sewers if they seem to be carrying the sewage away from the homes properly. The object of any preventative maintenance program is to minimize the overall costs by correcting deficiencies before they become critically expensive. This is applicable to sewers as with all utility systems and is becoming widely practiced by all municipalities as a method of reducing overall maintenance costs, maintaining a constant sized maintenance force, keeping detailed records of the sewer system, and instilling a competitive, morale-building spirit in the sewer maintenance force by competing for the best sewer system with the least service calls. This latter feature is accomplished in some cities by dividing the sewers into two areas under separate foremen for maintenance repsonsibilities.

The costs of properly maintaining a sewer system are not cheap and will cost an estimated \$7,000 more than the 1962 maintenance costs, but these costs will be justified by minimizing future reconstruction projects.

VIEW OF TELEVISION MONITOR SHOWING INTERIOR OF SEWER BEFORE INFILTRATION CORRECTION BY GROUTING FROM THE SURFACE. LIGHT AREA ON LEFT OF PICTURE IS WATER ENTERING SEWER.

VIEW OF TELEVISION MONITOR SHOWING INTERIOR OF SEWER AFTER INFILTRATION CORRECTION BY GROUTING FROM THE SURFACE.

NOTE: FLOW OF WATER HAS STOPPED.

FIGURE 9

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

EFFECT OF INFILTRATION CORRECTION BY GROUTING

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CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

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SUMMARY & RECOMMENDATIONS

CHAPTER VII

SUMMARY AND RECOMMENDATIONS

SUMMARY

The Pendleton sewerage facilities are typical of many sewer systems that evolved over the past fifty to seventy-five years, except that many features in Pendleton, such as the treatment facilities and many sewer trunk and lateral installations, are well advanced in planning and construction to fit the ultimate sewerage plan.

The sewage treatment facilities require relatively minor expenses to bring them in line with the program for expanding to meet the population growth.

The sewer construction program throughout the next thirty years is expensive as presented herein; but since the sewer system can be developed in many construction steps, the expense at any given time can be maintained at a reasonable cost, providing advance fiscal planning and budgeting is accomplished. Table 7 summarizes the estimated capital expenditures, in 5-year increments, which should be scheduled for both the collection system and the treatment facilities.

TABLE 7

SUMMARY OF CAPITAL EXPENDITURES
FOR
SEWERAGE FACILITIES

Construction Period	Facilities	Cost	Total Cost For Construction Period
Before 1965	Sewers	$257,300	
	Treatment Facilities	312,500	
			$569,800
1965-1970	Sewers		303,500
1970-1975	Sewers		218,700
1975-1980	Sewers	$379,400	
	Treatment Facilities	250,000	
			629,400
Beyond 1980	Sewers		143,100

The sewer maintenance in Pendleton compares with many cities. However, to stay abreast of the maintenance problems with the expanding system;and to better correct the known deficiencies, additional expenditures are necessary in the sewer maintenance department. The capital expenditures for maintenance equipment total $8,000, and additional annual costs for labor and operations is estimated at $7,000.

RECOMMENDATIONS

As a result of this investigation, it is recommended that:

1. The City of Pendleton adopt the ultimate sewerage system presented herein.

2. At the time a financial analysis is undertaken to establish proper rates for the water department, a similar analysis be made for sewer rates to insure adequate income to construct and maintain proper sewer facilities.

3. An immediate program be implemented to provide systematic maintenance procedures as described herein.

4. That the staff at the treatment plant be increased to provide an added laboratory technician during the 1963 pea processing season to determine the organic load applied and the degree of reduction obtained by the treatment facilities.

APPENDIX A

APPENDIX A

DESCRIPTION OF EXISTING SEWERS

SEWER MAIN SYSTEMS

For the purposes of this report, the existing sewer mains were divided into separate systems as follows:

1. Pendair System
2. McKay Creek System
3. Tutuilla Creek System
4. S. W. 20th Street System
5. Byers and Court Avenue System
6. Dorion Avenue System
7. Emigrant Avenue System
8. N. W. 10th Street System
9. North Main Street System

The outlines of these systems are shown on Plate 2 and their general physical conditions are described below.

Pendair System. The Pendair sewer system and sewage treatment plant was acquired from the government in 1950, along with other utilities, housing, etc., at Pendair. These sewerage facilities were constructed in 1941 by the Federal government to serve the Pendleton Air Base. The system consists of concrete pipe with mortar joints ranging in size from 6 inches to 12 inches in diameter. However, most of the sewers are 8 inches in diameter which collect the sewage from the area and terminate in a 12-inch pipe at the Pendair sewage treatment plant, which is in a state of almost complete disrepair. The effluent from the treatment plant originally flowed to the river in an open ditch, but more recently has been conveyed in a 4-inch steel pipe to an abandoned mill pond near the river. Much of the time this line is overloaded and the partially treated sewage continues to flow in the open ditch. It is highly desirable to eliminate this treatment system and route the sewage from this area to City's sewage treatment plant. It is understood that this will be accomplished in the immediate future.

The slopes of the sewers are not known in this area. However, the sewer locations are shown, along with all other existing sewers, on a detailed topographic map submitted to the City separately from this report.

McKay Creek System. This system was constructed after 1950 with concrete pipe having mortar joints initially. The later portions had rubber-ring joints. Reports by the maintenance personnel indicate there has been very little trouble in this area with the exception of minor infiltration along McKay Creek.

The system consists of pipe sizes ranging from a maximum diameter of 12 inches to a minimum of 8 inches. A portion of the 8-inch pipe in the system was constructed at a grade of 0.0028 feet per foot which is less than the minimum slope desired of 0.004 for this size sewer. A flat grade of this nature could cause some of the solids to settle out and eventually develop a block in the line. This section of the sewer is on S. W. Jay Avenue west of S. W. 31st Street.

The maximum grade is 0.143 feet per foot which is well under the maximum desirable grade of 0.33 feet per foot. The maximum depth of the sewer in this area is 10 feet and the minimum depth is just over 2 feet. The shallow depth of 2 feet occurs at the end of a lateral and is of adequate depth to protect the sewer from traffic damage. However, because of its location, it cannot be extended. The area which lies on the west slopes of the McKay Creek Valley has rock depths which vary from a minimum of one foot to a maximum of about 6 feet, with the deeper rock depths being near the creek.

Tutuilla Creek System. This system consists of two independent sewer mains, one serving the old portion of the City and a new main constructed in 1959 which follows the creek, using concrete pipe and rubber-ring joints. The old system has the predominant characteristic of a 6-inch diameter clay pipe with three sections having slopes less than the desirable minimum of 0.006 feet per foot. These sections are located on S. W. Emigrant Avenue between S. W. 21st and S. W. 23rd Streets, the alley between S. W. 21st and S. W. 22nd Streets, and south of S. W. Dorion Avenue. As mentioned before, slopes of this nature are constant sources of trouble.

The new portion is constructed of concrete pipe having rubber-ring joints and there has been no trouble reported by the maintenance personnel. The size of the pipe in this portion varies from a maximum diameter of 12 inches to a minimum diameter of 8 inches. The slopes vary from a maximum of 0.1254 feet per foot for 8-inch diameter pipe to a minimum of 0.0023 for 12-inch diameter pipe. These slopes meet the recommended design standards. The rock depth in this area varies from outcroppings to a maximum of 10 feet, with the greater depths being in the flatter areas. The sewer depths in this area are generally adequate to afford protection from traffic and freezing. However, the sewers should be deeper in the areas of less than minimum slope.

S. W. 20th Street System. This system is made up of two sections, one flowing from the north side of S. W. Court Place and the other flowing from the south side of S. W. Court Place. The north portion collects the sewage from S. W. Byers in an 8-inch diameter sewer which flows into the only existing pump station in the system. The pump station is located on the south side of U. S. Highway 30 where S. W. 20th Street intersects the highway. There are two pumps rated at 185 gal/minute with a static head of 15 feet, and there is an overflow line which follows U. S. Highway 30 and discharges into the river. The pumps lift the sewage to the gravity line on S. W. 20th Street which has a section of sewer with a slope of 0.003 feet per foot which is less than recommended and could cause occasional stoppages.

The second portion which flows from the south side of S. W. Court Place was partially constructed with the initial City sewers. The ends of the various branches were constructed with 6-inch pipe. As the system was expanded, one of the 6-inch diameter sewers was extended by adding 8-inch diameter pipe. This line is located at S. W. 16th Street between S. W. Emigrant and S. W. Frazer Avenues and serves the Goodwin Avenue, Haley Place, and Haley Lane area between S. W. 13th and S. W. 18th Streets. This condition could create future problems because the flow from the upper portion of the sewer could exceed the capacity of the 6-inch sewer.

Another source of possible future trouble is the 10-inch diameter sewer which is located under the Hawthorne School on S. W. 14th Street. This is probably the result of a sewer being constructed initially in a street which was later vacated. If major difficulties develop in this sewer, such as broken or deteriorated pipe, it may be necessary to construct a new line around the school.

The maximum slope in this portion of the system is 1.2 feet per foot which is greater than desirable to insure adequate flow depth to carry the solids through the sewer. This sewer line is located between S. W. Frazer and Goodwin Avenues on S. W. Ninth Street. Less than desired minimum slopes appear on S. W. Frazer Avenue between the following streets: a) 12th and 13th; b) 13th and 15th; c) 16th and 19th. The latter section of sewer is the only one where trouble has been reported. However, as mentioned before, all sections could eventually cause considerable trouble.

The depth to rock in this portion of the system varies from 8 to 10 feet in the more level area,and from being exposed at the surface 4 feet deep as the system progresses up to the bluff to the more gentle slopes above. The greater depth to rock is generally near the river in the more level area, as shown on Plate 6 at the back of this report. The depth of cover over the sewer is adequate in this portion of the system.

Byers and Court Avenue System. As shown on Plate 2, this system serves as the main sewer for the other systems, as well as serving the adjacent areas.

The system was laid in various increments; the first portion was part of the original system running through the business district and extending to the City shops on S. E. Byers Avenue. When additional sewers were required beyond the City shops, a second portion was installed in Byers Avenue parallel to the existing sewer from S. E. 11th Street eastward. This duplication of sewers (see Plate 2) was necessary because the original sewer was too shallow for service connections in the newer area. This latter sewer was also constructed in the thirties under the W.P.A. program.

Being one of the older systems, much of the information concerning the physical characteristics of this system has been misplaced or was never recorded. However, from field observations, most of this system is constructed of concrete pipe with mortar joints. One section from S. E. 4th Street to S. E. 5th Street on S. E. Byers Avenue was constructed with metal pipe ("invasion pipe") which is in an advanced stage of decomposition.

The maximum sewer diameter is 22 inches and the minimum diameter is 6 inches. Less than minimum slopes appear in the following four areas:

1. The main on S. E. 5th Street between the S. E. Byers Avenue offset.

2. The lateral on S. E. 11th Street from S. E. Byers Avenue northward and on S. E. Alexander Avenue. In addition to having a flat slope, this line is very near the surface and, in places, at the street surface.

3. The lateral on S. E. 15th Street north of S. E. Byers Avenue and on S. E. Alexander Place.

4. The lateral that joins the system at Byers Avenue between S. E. 18th and S. E. 19th Streets and which collects the sewage between S. E. 18th and 19th Streets and on Byers Place.

One lateral that is giving the maintenance personnel some trouble joins the main at S. E. Byers Avenue and S. E. 17th Street, collecting sewage from this street. The maintenance personnel reported a break in the sewer somewhere between S. E. Byers Avenue and S. E. Byers Place, but when flushing water is added to the upper manhole, no water appears at the manhole on Byers Avenue. It is also reported that it is impossible to surcharge the upper manhole.

The depth of cover for the sewers in this system varies from a maximum of 15 feet to a minimum of zero feet. The depth to rock in the area varies from approximately 6 to 10 feet with the shallow rock cover appearing at the east end of the system.

Dorion and Court Avenue System. This system also has a good portion of its length consisting of the original system which is lacking in construction information. The original part of this sewer extends between S. E. 12th Street and S. E. 12th Place Drive on S. E. Court Place. This system was extended during the 1930's under the W.P.A. program by constructing a shallower sewer parallel to the original sewer beginning at S. E. 6th Street and extending beyond the older system to serve the St. Anthony's Hospital area. The pipe slopes in the system vary from a maximum of 0.173 feet per foot for an 8-inch diameter sewer to a negative slope in a 10-inch diameter sewer. The negative slope is located between S. E. 12th Street and S. E. 12th Drive on the newer portion of the line, and maintenance problems have been continuous in this portion of the sewer. The Pendleton Woolen Mill periodically discharges wastes from dye vats just upstream from the point of this negative slope which flushes this section and helps prevent more serious maintenance problems.

Another portion of the system having less than minimum slope is that portion of the system serving S. E. 16th Street north of S. E. Court Place which, on occasion, has resulted in maintenance problems. Also, in the older portion of the system, there are numerous changes in slope between manholes which makes the sewers more difficult to maintain. One such sewer was discovered when relaying a lateral on S. E. 6th Street south of S. E. Emigrant Avenue.

The maximum size pipe diameter is 18 inches and the minimum size pipe diameter is six inches. The depth of the sewer varies from a maximum of approximately 12 feet to a minimum depth of about 2 feet, the shallower sewers being at the east end of the system. The depth to rock in the area of this system varies from a maximum overburden of 10 feet to a minimum of 6 feet. The greater depths are found in the western portion of the system.

Emigrant Avenue System. This system is part of the original sewers in Pendleton and much of the construction information has been misplaced. However, some of the "lost" sewers have been rediscovered. For instance, a sewer line on S. W. Frazer Avenue was relocated a few years ago by a maintenance crew and two sewer lines have been laid because on S. W. 8th Street the second line was constructed without knowledge of the first line's existence.

The maximum sewer size in the system is 12 inches in diameter and the minimum size is 6 inches in diameter. The upper portion of the system (eastern portion) is predominantly 6-inch clay pipe.

The maximum slope is 0.25 feet per foot in a 6-inch sewer located on S. W. 5th Street between S. W. Hailey and Isaac Avenues. This slope approaches the point where the flow is too shallow due to the high velocities and the solids will tend to hang up along the pipe and finally create a stoppage in the line. Often stoppages of this nature may not be apparent for some time because of exfiltration. Slopes flatter than recommended are in an 8-inch sewer located on S. W. Hailey between S. W. 2nd and 3rd Streets.

As in other systems, and as shown on Plate 6, the depths to rock vary from zero to 10 feet with the greater depths being near the river.

N. W. 10th Street System. This system collects sewage from the area known as North Hill. As with the other older systems, much of the information about the construction of this system has been misplaced or never recorded. Field inspections indicated the presence of clay and concrete pipe, but the extent of either pipe material is not known. As formerly was the case in the S. W. 20th Street system, 6-inch diameter pipes were used at the extremes of the old system;and when extensions were necessary, they were often constructed with larger diameter pipes. However, the small pipe has caused relatively little trouble because most of the sewer slopes are steep. The maximum slope is 0.33. However, the run is very short (less than 30 feet) between two adjacent manholes and should cause no serious trouble. There are two sewers in the system having slopes less than recommended for the diameter of the pipe used. One is on N. W. 10th Street between N. W. Ellis and N. W. Furnish Avenues in a 6-inch sewer with a slope of 0.00511. This line requires periodic attention to keep it flowing freely. Another sewer consisting of two consecutive sections of 8-inch pipe having slopes of 0.00233 and 0.00385 feet per foot is located near the river from N. W. 14th Street to the middle of the block between N. W. 12th and N. W. 13th Streets. Although the slopes are flat in these sections, there have been no maintenance problems reported. A second sewer near the river between N. W. 10th and N. W. 13th Streets was constructed parallel to an older sewer. The newer sewer was constructed through the original manholes in the older sewer and at different elevations. (See Figure 3.)

Another reported troublespot is between N. W. 6th and N. W. 7th Streets where the sewer is adjacent to the river. In this section, a 10-inch sewer is located under an apartment house. It is believed that the sewer is exfiltrating into the river because of continual swampy ground conditions between the apartment house and the river.

The depth to rock in this system varies from 1 foot to 8 feet with the greater depth to rock being near the river. The depth of cover over the sewer varies from approximately 10 feet to 3 feet with the shallower cover being in an area where rock is closer to the surface. The shallow sewers should cause no significant trouble in expanding the system due to the sloping terrain in this area.

North Main Street System. This system serves the remaining portion of the North Hill that lies east of the N. W. 10th Street system. Like most of the older systems in Pendleton, very little is known about it. The only troublespot reported is an 8-inch sewer which flows down N. E. 1st Street. This sewer is not straight between manholes and has two abrupt changes in horizontal alignment between manholes which makes it extremely difficult to clean this section of sewer.

The slopes in this system are relatively steep and apparently do not create maintenance problems. The depth to rock in this area varies from 1 to 6 feet with the greater depth being closer to the river.

APPENDIX B-1

APPENDIX B-1

TELEVISION INSPECTION OF SEWERS

INSPECTION PROCEDURES

On April 9, 10, and 11, 1962, an inspection of sewer lines was carried out using closed-circuit television equipment and personnel from Video-Scan, 1929 S. W. Palatine Road, Portland 19, Oregon, in conjunction with City personnel. Prior to moving the equipment to Pendleton, the sewer lines were to be cleaned and flushed. Cleaning of the sewers is necessary to remove the accumulcation of scum and dirt covering the interior walls of the pipe, so the television camera has clean pipe to view.

The equipment consisted of a panel truck, television monitor and camera, portable generator to power the television equipment, portable blower, and the necessary winches, cables, and electric cables to operate the television camera in a section of sewer. With the panel truck located near the upstream manhole, the two hand winches are placed over the two manholes that make up the run to be inpsected. A cable from the upstream winch is floated or blown from the upper manhole to the lower manhole by means of a parachute. This cable is then attached to the cable on the lower winch which is pulled back to the upstream manhole. The television camera is then attached. The downstream cable being attached to the front of the television camera dolly is used to pull the camera and dolly through the sewer. The upstream cable is attached to the back of the dolly as a safety measure in case the camera becomes fouled and has to be pulled back. The electric cables that power the camera, the camera's lights, and carry the picture back to the monitor in the panel truck are attached to the back of the camera.

After the camera is lowered through the manhole and placed in the sewer pipe, the electric cable is passed through a device which measures the footage of cable. This measurement enables the viewer to log any troubles on the interior of the pipe as seen on the monitor and relate it back to the actual trouble spot on the line.

DISCUSSION AND SUMMARY

Television is a very useful tool in the determination of certain physical characteristics existing in sewer lines. Even though it is sometimes difficult to photograph the view on the monitor screen, the television set normally produces a clear picture of the sewer and is quite helpful in establishing the internal condition of sewers. The photographs shown on Figure 9 are representative of the quality of picture normally viewed; however, the pictures of other sewer inspection as shown on Figure 5 indicate the type of picture to be expected if the equipment is operating properly. Such features as bad joints, mortar in the pipe, infiltration, broken pipe, and bad side sewer connections can be seen. In addition adverse grade conditions can be detected by noting the flow conditions of the sewer. However, as any other tool, television inspection has its limitations, such as:

1. Exfiltration cannot be located, only the bad joints and broken pipe can be found.

2. If the sewers are flowing over one-third full, flow in the sewers must be diverted as the camera lens submerges and very little information is gained.

3. Physical size (length of the camera) poses some limitations, particularly where the sewer direction changes abruptly. Usually, there is not enough clearance to get the camera in the sewer line, short of breaking the bottom of the manhole. The minimum diameter of sewer that the camera can fit into is 8 inches. Since the Pendleton television inspection was completed, smaller cameras have been developed for viewing sewers as small as 4 inches in diameter.

As would be expected from a random inspection of various sewers within an old system, some lines were in poor condition and others were in relatively good condition. The sewers inspected are shown on Plate 2. The following list of sewers indicates where television inspection was attempted without success.

1. The sewer located on S. E. Eighth Street between Dorion and Frazer Avenues was plugged.

2. The sewers on N. W. 10th Street between N. W. Ellis and N. W. Furnish was found to be a 6-inch pipe and not an 8-inch pipe as indicated on the sewer maps.

3. The sewer located on S. W. 10th Street between Dorion and Emigrant Avenues had abrupt changes in direction at both ends.

The major difficulty encountered was that the lines to be inspected had not been properly cleaned.

INSPECTION DATA

The following log was recorded in the field at the time the inspection was conducted:

APRIL 9, 1962

Court Place - S. E. 14th to S. E. 16th

The 8-inch sewer on S. E. Court Place between S. E. 14th and S. E. 16th Streets (manhole 58 to 57) was inspected. The TV camera was placed in manhole 58 and pulled downstream through the line to manhole 57. All distances in the following log refer to distances from the center of manhole 58.

233 feet	Rough joint on bottom.
239 feet	Adverse grade - rough joint - flow sluggish.
259 feet	Side service on left side.
309 feet	Encountered roots (dirt from roots obscured view).
312 feet	View obscured .

General Condition: Line needs some maintenance such as bucketing to remove the roots, rocks, etc.

Bailey - N. W. Sixth to N. W. Seventh Streets

This 10-inch line was viewed (manholes 4-7 to 3-7) by putting the camera in manhole 4-7 and pulling it downstream to manhole 3-7. There were two buildings built over this line and some suspicion of exfiltration to the river from this line. The following log was recorded; all distances refer to manhole 4-7.

71 feet	Wye in top of line.
80 feet	Wide joint.
87 feet	Wye in top of line.
136 feet	6-inch wye in top of line.
172 feet	Wye in top.
222 feet	Wye 6 inches on right appears to be inactive.
224 feet	Wye in top.
288 feet	End of run - Manhole 3-7.

General Condition: The flow was moving along quite well through the line and there was no build-up of grease. With the TV camera, exfiltration is impossible to locate, except that wide joints can be found. A better check would be to plug the line, divert the flow, and check with dye. This would take approximately two hours.

Byers - S. E. Fifth to S. E. Fourth Streets

The flow was too high for proper television inspection; and the sewer was plugged on S. E. Fifth at Manhole 11-8. The camera could not be used on S. E. Fifth due to a bend in the sewer limiting the amount of room for the camera to be inserted. Therefore it was decided to plug the next manhole downstream (10-8) and pull the camera upstream from Manhole 9-8. Even though the flow was blocked at S. E. Fifth (Manhole 10-8) there was still 1-1/2 inches of clear cold water flowing at the next manhole (No. 9-8). After inspecting the 8-inch sewer flowing into 9-8, it was found to be a 7-1/2-inch steel pipe, which was paper thin and decomposed. The use of television inspection for this line was abandoned because of the possibility of destroying the existing line.

APRIL 10, 1962

Emigrant - S. W. Seventh to S. W. Eighth Streets

A 12-inch line was inspected, setting the camera in at S. W. Seventh Street and pulling it downstream toward S. W. Eighth, the following log was recorded. All distances refer to the manhole at S. W. Seventh Street.

6.5 feet	Wye top.
13.5 feet	Wye top not in use.
85 feet	Wye left, break along side (possible).
132 feet	Wye left.
135 feet	Wye right.
174 feet	Wye left.
223 feet	Wye left.
253 feet	First manhole on S. W. Eighth Street.
265 feet	End of run, second manhole on S. W. Eighth.

General Condition: The general condition of this sewer was very good except for the possible break along one side at 85 feet. There was no build-up of grease and from the pull of the camera there seemed to be no rocks or debris on the bottom of this sewer.

Emigrant - S. W. Sixth to S. W. Seventh Streets

A 12-inch line was inspected with the TV camera, setting the camera in at S. W. Sixth and pulling toward S. W. Seventh Street. It was reported that some difficulty had been experienced along this run. The following log was recorded. All distances refer back to the manhole at S. W. Sixth Street.

90 feet	Wye left.
100 feet	Grease and scum heavy.
129 feet	Active wye right.
190 feet	Grease build-up very heavy in sewer. Top third of sewer full.
210 feet	Grease has been building up badly, starting to slough off in front of the camera lens. Camera seems to be wedged pretty tight. After pressure was applied to break camera free, the pulling line broke and the camera had to be pulled out with the retrieving line attached to the back of the camera. Camera was 43 feet from the downstream manhole when the pulling line broke.

Byers - S. E. Fourth to S. E. Third Streets

A 10-inch line was inspected along this run inserting the camera at S. E. Fourth and pulling toward S. E. Third Street. A little trouble was encountered when floating the parachute through this run. The following log was recorded; all distances refer back to the manhole at S. E. Fourth Street.

6 feet	Wye right.
65 feet	Wye on the left.
67 feet	Wye right.
78 feet	Wye left, very poor joint, the pipe sticks into the sewer about 3 inches. Camera lodged against it and was retracted.

General Condition: The general condition of this sewer to this point is very poor and the sewage was ponding and had sludge floating on the surface.

APRIL 11, 1962

Byers - S. E. Third to S. E. Second Streets

A 10-inch sewer was inspected starting at S. E. Third and pulling through toward S. E. Second Street with the following log recorded. All distances refer to the manhole at S. E. Third Street and Byers Avenue.

76 feet	Wye top right.
85 feet	Wye left.
130 feet	Wye top left. Camera seems to be rolling up on one side.
343 feet	Wye left. Camera rolling up on one side. It appears that when we pulled up, it caught in this wye that we just passed; however, this was most of the way through the line and the camera was retracted.

General Condition: The general condition is good. It appears to have tight joints and the sewage is moving along very well. There is no build-up of sludge.

Byers - S. E. Third To S. E. Fourth Streets

It was decided to go back on the line that we looked at on the previous day where the camera lodged against an incoming wye 78 feet into the line. The camera was inserted at S. E. Third Street and pulled upstream toward S. E. Fourth Street. The following log was recorded. All distances refer to the manhole at S. E. Fourth Street and Byers Avenue.

35 to 42 feet	Low spots, sewage was ponding and covering the camera lens.
43 feet	Wye top.
49 feet	Wye left.
65 feet	Wye left.
91 feet	Wye left.
97 feet	Change of grade, sewage backing up badly.
141 feet	Wye top.
174 feet	Out of the bad grade, sewage moving along better now.
176 feet	Wye sticking out into the sewer, (same wye encountered on the log of the previous day).

General Condition: The condition of the line is poor. There was sludge and rocks in the line and removing the camera was difficult.

APPENDIX B-2

VIDEO SCAN
01929 S. W. PALATINE ROAD
PORTLAND 19, OREGON

April 15, 1962

Cornell, Holland, Hayes & Merryfield
Logan Bldg.
500 Union Street
Seattle 1, Washington

Inspection Report for Sewer Lines in City of Pendleton, Oregon inspected April ~~8, 9, 10,~~ 1962
9, 10, 11,

4-9-62
Court Place S.E. 14 to S.E. ~~12~~ 16

233'	Rough joint on bottom
239'	Adverse grade - Rough joint
259'	Side Service - Water High- question.
309'	Roots

Roots showed from this point to the end of the line. Due to high water and dirt on the faceplate this section should be rerun but high water makes success improbable.
MH registered 482'.

6th to 7th on N.W. Bailey
(Under apartments and house)

71'	Wye top
80'	Wide joint- apparently OK
87'	Wye top
136'	Wye top (could be manhole)
172'	Wye top
222'	Wye right 6" - not used
224'	Wye Top - used
288'	MH

4-10-62
S.W. Emigrant 7th to 8th 12"

6 1/2'	Wye Left
13 1/2'	Wye top - not used
85'	Wye Left -cracked inside of Wye
132'	Wye Left
135'	WyeRight

J Brenton

4-10-62
S.W. Emigrant

174'	Wye Left
223'	Wye Left
253'	First MH
265'	MH

S.W. Emigrant 6th to 7th 12"

90'	Wye Left
129'	Wye Right

Pipe from 100' on very heavy with grease, roots, etc.

209'	Pipe jammed - Cable broke 43' to MH

4-10-62
Byers 4th to 5th 10"

6'	Wye Right - Stub-in Wye, intruding into pipe managed to get camear by.
65'	Wye Left
67'	Wye Right
78'	Wye left - Stub-in wye, concrete sticking out impossible to get camera by.

N.B. These l nes should be cleaned by bucketing, porcupine etc. before camera goes through again. Water quite high, about 4".

4-11-62
Byers 3rd to 2nd

76'	Wye top Right
85'	Wye Left
132'	Wye top Left
143'	Wye Left Camera stuck on wye. Necessary to pull back

Byers 3rd to 4th. 10"

43'	Wye Left Top
49'	Wye Left
65'	Wye Left top
91'	Wye Left
97'	Adverse Grade Extends to 172'. Pipe 1/2 to 3/4 full.
141'	Wye left top

Byers 3rd to 4th.

172' Out of Adverse
176' Wye Right - Stub-in extends into pipe. Impossible to pass. 78' to end of line.

Byers 4th to 5th. 8"

Steel pipe, badly oxidized. Impossible to put camera through without ruining what is left of pipe.

12 Noon, 4-11-62
Finished job for present.

APPENDIX B-3

APPENDIX B - 3

SMOKE TESTS IN THE PENDLETON SEWER SYSTEM

DESCRIPTION OF TESTS

Smoke tests are used to locate leaks in sewers which could be caused by broken pipe or bad joints and illegal connections, such as roof and foundation drains. The test equipment used in Pendleton consisted of a portable Homelite blower having a capacity of 1500 cfm, a canvas air duct, a plywood manhole cover lined with sponge rubber to make a seal with the manhole ring, and two sheet metal elbows attached to the intake side of the blower and between the plywood manhole cover and the canvas duct. This equipment is pictured in Figure 6.

The equipment was assembled and a 3-minute smoke bomb was placed in the intake elbow on the blower. Smoke was pumped through the blower into the manhole filling the sewer lines. The sewers in the test section should normally be restricted in such a way to allow the flow of sewage under the obstruction and to trap the smoke in the section to be tested. For these tests to be effective in locating sewer breaks, the soil must be somewhat porous and obviously the voids cannot be filled with water. When the smoke appears at the surface, the location of the leak is not always directly under the spot where the smoke appears because the smoke will often travel laterally for some distance if there are zones of previous materials. Also, these tests are difficult to use under pavements, except for locating faulty service connections.

Prior to beginning the smoke tests in Pendleton, newspaper and radio media were used to notify the public of the nature of these tests so the residents in the test area would not become alarmed. The Fire and Police Departments were also notified to forestall emergency calls arising from these tests. The nature of the smoke generated by the smoke bombs used in the test is harmless.

TEST DATA

Three tests were conducted, two on April 9, 1962, and the final test on April 11, 1962. The source of smoke in the first two tests, instead of being a smoke bomb, was an insect fogger which produces a dense smoke type fog. This fog is produced by partially combusting diesel oil, forming an emulsion of oil and air. Prior to these dates, the City had experienced a period of rain as shown on Plate 7, which diminishes the value of the smoke test. No attempt was made to obstruct the sewer lines in these tests.

In the first test, the blower was located at the intersection of S. W. 39th Street and S. W. 41st Street. In the vicinity of the test, the streets are paved, but only house foundations with plumbing connections exist. Smoke was only detected coming out of the plumbing connections.

In the second test, the blower was located at the manhole just north of S. E. Byers Place on S. E. 17th Street, which is paved. The sewer to be tested proceeds from this point and joins the main sewer at S. E. Byers Avenue. Residential type dwellings are built along each side of the line and holes were drilled in the pavement on 50-foot centers to allow the smoke to come through the pavement. Smoke was observed coming out of the roof vents and in the yard of one resident where a cleanout existed.

In the third test, the blower was placed at the manhole which is located at the intersection of S. E. Third and S. E. Byers Avenue. This is an area of paved streets, residential homes, and commercial buildings. In this area, the source of smoke was from 3-minute smoke bombs.

As before, smoke appeared only at roof vents in this area. There tests were of little success because of the following reasons:

1. The soil was wet. In the third test, it was believed that the groundwater was above the sewer line, which would, of course, seal this line.

2. The test sewers were not isolated as recommended.

3. The smoke used in the first two tests was not a true smoke, but emulsion, which might not have passed through the soil if any leaks were present.

As mentioned in Chapte II, the records on many of the connections in the downtown area have been lost or misplaced. The information gained from these and future smoke tests in locating building sewer connections may be invaluable to the City of Pendleton in the future.

APPENDIX C

APPENDIX C

FLOW MEASUREMENTS

Flow measurements were taken at ten manholes throughout the Pendleton sewer system at the various locations shown on Plate 2. These measurements of depth, flow and velocity, along with the precipitation for the City and the Umatilla River flow records, were obtained for the duration of the measurement. The depth of flow was measured and velocity determined at about the same time daily on Mondays, Wednesdays, and Fridays. The weather and river flows were obtained for every day during the measurements.

Velocity of the flow was measured by means of a Gurley-Pygmey flow meter which was set at 0.6 of the depth of the flow off the bottom. The average velocity was obtained by adjusting the velocity recorded in the field.

Data was collected at all ten manholes from February 5, 1962, to April 9, 1962. On the latter date, the collection of data was suspended on all but two of the manholes because there was no significant change in the remaining eight manholes. From April 13, 1962, to April 20, 1962, depth measurements only were taken in the two stations on S. E. Byers Avenue. The depth measurements were stopped on the above date because the river flow was dropping rapidly and sufficient information had been obtained.

The only problem encountered was at some stations there was not adequate depth of flow to allow the flow measure to be used. These are noted in the following tabulation of the data collected.

Sewer: Byers Avenue
Location: S. E. Sixth & Byers
Size: 8-inch

Date	Time	Depth of Flow/ Ft.	Average Velocity Ft/Sec	Flow cfs	mgd
2/ 5/62	12:50	0. 12	1. 57	0. 067	. 043
2/ 7/62	12:55	0. 10	1. 98	0. 065	. 042
2/ 9/62	1:05	0. 15	1. 48	0. 084	. 054
2/12/62	1:10	0. 11	2. 45	0. 088	. 057
2/14/62	12:40	0. 12	1. 34	0. 057	. 037
2/16/62	12:45	0. 10	0. 81	0. 026	. 017
2/19/62	12:50	0. 11	1. 34	0. 048	. 031
2/21/62	12:45	0. 12	1. 77	0. 075	. 049
2/23/62	12:45	0. 11	2. 51	0. 090	. 058
2/26/62	12:45	0. 11	1. 33	0. 048	. 031
2/28/62	12:40	0. 13	1. 52	0. 070	. 045
3/ 2/62	12:40	0. 11	1. 93	0. 070	. 045
3/ 5/62	12:45	0. 11	1. 28	0. 046	. 030
3/ 7/62	too shallow				
3/ 9/62	12:40	0. 12	1. 33	0. 057	. 037
3/12/62	12:40	0. 11	1. 52	0. 054	. 035
3/14/62	12:45	0. 12	1. 35	0. 058	. 038
3/16/62	12:45	0. 12	1. 81	0. 077	. 050
3/19/62	12:40	0. 14	2. 26	0. 120	. 078
3/21/62	12:45	0. 11	1. 43	0. 051	. 033
3/23/62	no record	0. 12	1. 43	0. 061	. 040
3/26/62	12:40	0. 12	1. 25	0. 053	. 034
3/28/62	12:40	0. 18	1. 43	0. 108	. 070
3/30/62	12:40	0. 21	2. 17	0. 20	. 130
4/ 2/62	12:40	0. 24	2. 11	0. 238	. 154
4/ 4/62	12:40	0. 16	1. 31	0. 084	. 054
4/ 6/62	12:40	0. 23	1. 69	0. 177	. 115
4/ 9/62	12:40	0. 24	2. 17	0. 245	. 159
4/13/62	2:25	0. 30	2. 17	0. 32	. 207
4/16/62	2:10	0. 23	1. 69	0. 177	. 115
4/18/62	2:05	0. 17		0. 14	. 091
4/20/62	2:10	0. 14	2. 26	0. 12	. 078

Sewer: Byers Avenue
Location: Byers & Main
Size: 10-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs	mgd
2/ 5/62	1:55-1:56	0.36	0.44	0.098	.064
2/ 7/62	1:10	0.38	0.42	0.103	.067
2/ 9/62	1:15	0.33	0.46	0.098	.064
2/12/62	1:15	0.34	0.40	0.083	.054
2/14/62	12:50	0.34	0.49	0.103	.068
2/16/62	12:55	0.30	0.45	0.079	.051
2/19/62	1:05	0.30	0.53	0.098	.064
2/21/62	12:55	0.35	0.41	0.088	.057
2/23/62	12:55	0.32	0.42	0.078	.051
2/26/62	12:50	0.36	0.50	0.112	.073
2/28/62	12:50	0.33	0.48	0.096	.062
3/ 2/62	12:50	0.31	0.54	0.099	.064
3/ 5/62	12:55	0.32	0.50	0.095	.062
3/ 7/62	12:50	0.32	0.48	0.088	.057
3/ 9/62	12:50	0.32	0.49	0.092	.060
3/12/62	12:50	0.48	0.54	0.177	.115
3/14/62	12:55	0.32	0.45	0.086	.056
3/16/62	12:55	0.36	0.42	0.091	.059
3/19/62	12:50	0.25	0.54	0.074	.048
3/21/62	12:55	0.41	0.42	0.112	.073
3/23/62	12:55	0.35	0.59	0.129	.084
3/26/62	12:50	0.35	0.54	0.117	.076
3/28/62	12:50	0.32	0.81	0.153	.099
3/30/62	12:50	0.41	1.19	0.317	.205
4/ 2/62	12:50	0.60	0.81	0.343	.222
4/ 4/62	12:50	0.47	0.92	0.29	.19
4/ 6/62	12:50	0.38	1.19	0.29	.19
4/ 9/62	12:50	0.40	1.14	0.29	.19
4/13/62	2:30	0.60	0.81	0.343	.222
4/16/62	2:15	0.56		0.32	.207
4/18/62	2:10	0.60	0.81	0.343	.222
4/20/62	2:15	0.44		0.17	.11

Sewer: S. W. Dorion Avenue
Location: Dorion & Second West
Size: 15-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	2:00	0.45	1.23	0.488
2/ 7/62	1:20	0.62	1.71	1.043
2/ 9/62	1:25	0.54	1.60	0.805
2/12/62	1:25	0.43	1.33	0.488
2/14/62	1:00	0.47	1.65	0.700
2/16/62	1:05	0.62	1.85	1.135
2/19/62	1:15	0.45	1.35	0.541
2/21/62	1:05	0.64	1.62	1.016
2/23/62	1:05	0.62	1.72	1.056
2/26/62	1:00	0.45	1.41	0.554
2/28/62	1:00	0.48	1.48	0.634
3/ 2/62	1:00	0.63	1.62	0.99
3/5 /62	1:05	0.64	1.62	1.016
3/ 7/62	1:00	0.56	1.52	0.818
3/ 9/62	1:00	0.48	1.48	0.634
3/12/62	1:00	0.52	1.38	0.673
3/14/62	1:05	0.50	1.36	0.620
3/16/62	1:05	0.50	1.48	0.673
3/19/62	1:00	0.56	1.62	0.858
3/21/62	1:05	0.64	1.81	1.135
3/23/62	1:05	0.49	1.41	0.620
3/26/62	1:00	0.50	1.52	0.7
3/28/62	1:00	0.52	1.76	0.858
3/30/62	1:00	0.48	1.43	0.607
4/ 2/62	1:00	0.65	1.85	1.188
4/ 4/62	1:00	0.66	1.89	1.248
4/ 6/62	1:00	0.53	1.65	0.807
4/ 9/62	1:00	0.58	1.35	0.744

Sewer: N. W. 12th Street
Location: N. W. 12th Street & Gilliam
Size: 8-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	2:10	0.11	2.62	0.094
2/ 7/62	1:30	0.07	1.93	0.036
2/ 9/62	1:40	0.05	1.17	0.026
2/12/62	1:35	0.04	too shallow	
2/14/62	1:15	0.04	too shallow	
2/16/62	1:20	0.05	too shallow	
2/19/62	1:20	0.06	too shallow	
2/21/62	1:15	0.04	too shallow	
2/23/62	1:20	0.04	too shallow	
2/26/62	1:10	0.04	too shallow	
2/28/62	1:15	0.04	too shallow	
3/ 2/62	1:15	0.04	too shallow	
3/ 5/62	1:15	0.04	too shallow	
3/ 7/62	1:10	0.04	too shallow	
3/ 9/62	1:10	0.05	too shallow	
3.12/62	1:10	0.05	too shallow	
3/14/62	1:15	0.04	too shallow	
3/16/62	1:15	0.05	too shallow	
3/19/62	1:10	0.04	too shallow	
3/21/62	1:15	0.04	too shallow	
3/23/62	1:20	0.03	too shallow	
3/26/62	1:10	0.05	too shallow	
3/28/62	1:10	0.03	too shallow	
3/30/62	1:10	0.06	too shallow	
4/ 2/62	1:10	0.04	too shallow	
4/ 4/62	1:10	0.03	too shallow	
4/ 6/62	1:10	0.03	too shallow	
4/ 9/62	1:10	0.03	too shallow	

Sewer: S. W. Ninth Street
Location: S. W. Ninth & Goodwin
Size: 8-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	2:45	0.09	0.55	0.015
2/ 7/62	2:10	0.06	0.32	0.002
2/ 9/62	2:15	0.15	0.44	0.025
2/12/62	2:00	0.10	0.27	0.009
2/14/62	1:45	0.11	0.52	0.018
2/16/62	1:50	0.12	0.42	0.018
2/19/62	1:45	0.09	too shallow	
2/21/62	1:40	0.11	0.38	0.014
2/23/62	1:50	0.07	too shallow	
2/26/62	1:35	0.11	0.37	0.013
2/28/62	1:40	0.09	too shallow	
3/ 2/62	1:50	0.09	too shallow	
3/ 5/62	1:45	0.08	too shallow	
3/ 7/62	1:40	0.09	too shallow	
3/ 9/62	1:40	0.09	too shallow	
3/12/62	1:40	0.11	0.34	0.013
3/14/62	1:50	0.08	too shallow	
3/16/62	1:40	0.12	0.30	0.013
3/19/62	1:35	0.07	too shallow	
3/21/62	1:40	0.09	too shallow	
3/23/62	1:40	0.09	too shallow	
3/26/62	1:30	0.09	too shallow	
3/28/62	1:35	0.09	too shallow	
3/30/62	1:35	0.09	too shallow	
4/ 2/62	1:30	0.08	too shallow	
4/ 4/62	1:40	0.09	too shallow	
4/ 6/62	1:30	0.08	too shallow	
4/ 9/62	1:30	0.08	too shallow	

Sewer: S. W. Goodwin Avenue
Location: S. W. Sixth & Goodwin
Size: 10-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	2:40	0.24	1.00	0.132
2/ 7/62	2:00	0.22	1.65	0.185
2/ 9/62	2:05	0.21	1.35	0.145
2/12/62	1:50	0.17	1.39	0.107
2/14/62	1:35	0.18	0.92	0.081
2/16/62	1:40	0.18	1.43	0.132
2/19/62	1:35	0.18	1.38	0.121
2/21/62	1:30	0.14	1.12	0.069
2/23/62	1:40	0.19	1.45	0.132
2/26/62	1:30	0.20	1.36	0.145
2/28/62	1:35	0.23	1.23	0.158
3/ 2/62	1:40	0.18	1.21	0.112
3/ 5/62	1:35	0.20	1.21	0.12
3/ 7/62	1:30	0.20	1.06	0.11
3/ 9/62	1:30	0.20	1.26	0.13
3/12/62	1:30	0.19	1.29	0.12
3/14/62	1:40	0.22	1.22	0.14
3/16/62	1:30	0.17	1.46	0.11
3/19/62	1:30	0.17	1.12	0.10
3/21/62	1:35	0.20	1.35	0.14
3/23/62	1:35	0.18	1.41	0.13
3/26/62	1:25	0.18	1.33	0.12
3/28/62	1:30	0.16	1.33	0.10
3/30/62	1:30	0.18	1.38	0.12
4/ 2/62	1:25	0.19	1.21	0.11
4/ 4/62	1:30	0.20	1.27	0.13
4/ 6/62	1:25	0.17	1.25	0.10
4/ 9/62	1:25	0.21	1.33	0.14

Sewer: S. W. Fifth Street
Location: S. W. Sixth & Frazer
Size: 10-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	2:30	0.21	3.41	0.356
2/ 7/62	1:50	0.17	2.62	0.198
2/ 9/62	2:00	0.16	2.73	0.198
2/12/62	1:45	0.16	2.73	0.198
2/14/62	1:25	0.16	2.45	0.172
2/16/62	1:30	0.15	2.42	0.158
2/19/62	1:30	0.15	2.36	0.158
2/21/62	1:25	0.14	2.47	0.145
2/23/62	1:30	0.16	2.40	0.172
2/26/62	1:20	0.17	2.26	0.172
2/28/62	1:25	0.16	2.40	0.172
3/ 2/62	1:30	0.14	2.37	0.145
3/ 5/62	1:25	0.16	2.37	0.172
3/ 7/62	1:20	0.15	2.20	0.145
3/ 9/62	1:20	0.19	2.20	0.211
3/12/62	1:20	0.16	2.30	0.172
3/14/62	1:30	0.18	2.23	0.198
3/16/62	1:25	0.13	2.42	0.132
3/19/62	1:20	0.15	2.37	0.114
3/21/62	1:25	0.16	2.47	0.172
3/23/62	1:30	0.15	2.37	0.114
3/26/62	1:20	0.18	2.30	0.198
3/28/62	1:20	0.15	2.30	0.158
3/30/62	1:20	0.14	2.45	0.145
4/ 2/62	1:20	0.15	2.37	0.114
4/ 4/62	1:20	0.14	2.30	0.14
4/ 6/62	1:20	0.14	2.30	0.14
4/ 9/62	1:20	0.14	2.30	0.14

Sewer: N. W. 10th Street
Location: N. W. 10th & Garden
Size: 8-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	2:20	0.11	1.51	0.054
2/ 7/62	1:40	0.05	2.16	0.022
2/ 9/62	1:50	0.03	too shallow	
2/12/62	1:40	0.07	too shallow	
2/14/62	1:20	0.06	too shallow	
2/16/62	1:25	0.08	too shallow	
2/19/62	1:25	0.05	too shallow	
2/21/62	1:20	0.05	too shallow	
2/23/62	1:25	0.06	too shallow	
2/26/62	1:15	0.07	too shallow	
2/28/62	1:20	0.06	too shallow	
3/ 2/62	1:20	0.06	too shallow	
3/ 5/62	1:20	0.08	too shallow	
3/ 7/62	1:15	0.07	too shallow	
3/ 9/62	1:15	0.04	too shallow	
3/12/62	1:10	0.05	too shallow	
3/14/62	1:20	0.04	too shallow	
3/16/62	1:20	0.06	too shallow	
3/19/62	1:15	0.06	too shallow	
3/21/62	1:20	0.06	too shallow	
3/23/62	1:25	0.04	too shallow	
3/26/62	1:15	0.06	too shallow	
3/28/62	1:15	0.05	too shallow	
3/30/62	1:15	0.03	too shallow	
4/ 2/62	1:15	0.06	too shallow	
4/ 4/62	1:15	0.03	too shallow	
4/ 6/62	1:15	0.05	too shallow	
4/ 9/62	1:15	0.07	too shallow	

Sewer: S. W. Hailey Avenue
Location: S. W. 31st & Hailey
Size: 10-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	3:05	0. 12	2. 57	0. 119
2/ 7/62	2:20	0. 08	3. 91	0. 112
2/ 9/62	2:30	0. 11	3. 02	0. 125
2/12/62	2:15	0. 11	2. 92	0. 119
2/14/62	2:00	0. 11	3. 41	0. 139
2/16/62	2:05	0. 11	3. 81	0. 158
2/19/62	1:50	0. 12	4. 22	0. 198
2/21/62	1:55	0. 10	3. 67	0. 158
2/23/62	2:00	0. 12	3. 67	0. 172
2/26/62	1:50	0. 10	4. 12	0. 152
2/28/62	1:50	0. 11	3. 81	0. 158
3/ 2/62	2:00	0. 14	4. 12	0. 251
3/ 5/62	1:55	0. 12	4. 30	0. 205
3/ 7/62	1:50	0. 12	4. 30	0. 205
3/ 9/62	1:50	0. 12	4. 34	0. 205
3/12/62	1:55	0. 11	3. 54	0. 145
3/14/62	2:00	0. 11	4. 30	0. 178
3/16/62	1:55	0. 11	3. 49	0. 145
3/19/62	1:45	0. 13	3. 96	0. 224
3/21/62	1:50	0. 11	4. 30	0. 178
3/23/62	1:50	0. 11	3. 96	0. 165
3/26/62	1:40	0. 11	3. 67	0. 152
3/28/62	1:45	0. 11	4. 30	0. 178
3/30/62	1:45	0. 10	2. 92	0. 107
4/ 2/62	1:40	0. 11	4. 12	0. 172
4/ 4/62	1:50	0. 10	4. 12	0. 152
4/ 6/62	1:40	0. 10	4. 05	0. 152
4/ 9/62	1:40	0. 10	3. 96	0. 145

Sewer: McKay Creek
Location: S. W. 39th Street
Size 12-inch

Date	Time	Depth of Flow/Ft.	Average Velocity Ft/Sec	Flow cfs
2/ 5/62	3:25	0.04	0.36	0.004
2/ 7/62	2:35	0.05	----	-----
2/ 9/62	2:45	0.05	too shallow	
2/12/62	2:25	0.006	too shallow	
2/14/62	2:15	0.07	too shallow	
2/16/62	2:20	0.06	too shallow	
2/19/62	2:05	0.05	too shallow	
2/21/62	2:05	0.09	too shallow	
2/23/62	2:10	0.05	too shallow	
2/26/62	2:00	0.06	too shallow	
2/28/62	2:00	0.06	too shallow	
3/ 2/62	2:10	0.07	too shallow	
3/ 5/62	2:05	0.05	too shallow	
3/ 7/62	2:00	0.05	too shallow	
3/ 9/62	2:00	0.05	too shallow	
3/12/62	2:05	0.05	too shallow	
3/14/62	2:10	0.06	too shallow	
3/16/62	2:05	0.04	too shallow	
3/19/62	1:55	0.08	too shallow	
3/21/62	2:00	0.06	too shallow	
3/23/62	2:00	0.06	too shallow	
3/26/62	1:50	0.06	too shallow	
3/28/62	2:00	0.07	too shallow	
3/30/62	1:55	0.06	too shallow	
4/ 2/62	1:50	0.06	too shallow	
4/ 4/62	2:00	0.03	too shallow	
4/ 6/62	1:50	0.06	too shallow	
4/ 9/62	1:50	0.05	too shallow	

APPENDIX D

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

PENDAIR MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class C				
0 - 6 feet	3,750	l.f.	1.20	$ 4,284.00
6 - 8 feet	500	l.f.	1.55	775.00
8 - 10 feet	1,400	l.f.	2.00	2,800.00
Addition for Rock Excavation	3,570	l.f.	3.35	11,959.50
Sewer Pipe In Place				
8-inch Concrete	3.550	l.f.	2.10	7,455.00
12-inch Concrete	1,920	l.f.	3.05	5,856.00
18-inch Concrete	300	l.f.	5.35	1,605.00
Manholes				
Std. MH, 0 - 6	15	each	190.00	2,850.00
Additional Depth	14	l.f.	18.00	252.00
Gravel Base	5,470	l.f.	0.35	1,914.50
River Crossing	300	l.f.	32.00	9,600.00
Highway Crossing	100	l.f.	43.00	4,300.00
PumpStation		l.s.		18,000.00
Pressure Line	1,500	l.f.	4.50	6,750.00
Trench	1,500	l.f.	1.20	1,800.00
Subtotal				$80,201.00
20% Engineering & Contingencies				16,040.20
TOTAL				$96,241.20

ESTIMATE $96,300

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

S. E. BYERS AVENUE TRUNK

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D (18-inch pipe and under)				
6 - 8 feet	870	l. f.	3. 10	$ 2, 697. 00
8 - 10 feet	400	l. f.	4. 00	1, 600. 00
(21-inch to 30-inch pipe)				
0 - 6 feet	2, 160	l. f.	2. 90	6, 264. 00
6 - 8 feet	1, 590	l. f.	4. 00	6, 360. 00
8 - 10 feet	1, 800	l. f.	5. 15	9, 270. 00
Sewer Pipe in Place				
15-inch Concrete	470	l. f.	4. 40	2, 068. 00
18-inch Concrete	800	l. f.	5. 35	4, 280. 00
21-inch Concrete	2, 960	l. f.	6. 80	20, 128. 00
24-inch Concrete	2, 590	l. f.	7. 80	20, 202. 00
Manholes				
Std. MH, 0 - 6 feet	26	each	190. 00	4, 940. 00
Additional Depth	32	l. f.	18. 00	576. 00
Gravel Base				
18-inch Pipe & Under	1, 270	l. f.	0. 35	444. 50
21-inch to 36-inch Pipe	5, 550	l. f.	0. 70	3, 885. 00
Street Surface Replacement				
Gravel	1, 560	l. f.	0. 80	1, 248. 00
Asphalt	5, 260	l. f.	2. 30	12, 098. 00
Y-Stubouts	130	each	15. 00	1, 950. 00
Subtotal				$ 98, 010. 50
20% Engineering & Contingencies				19, 602. 10
TOTAL				$117, 612. 60

ESTIMATE $117, 600

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

STATE HOSPITAL - RIVER MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class C 0 - 6 feet	2,500	l.f.	1.20	$ 3,000.00
Sewer Pipe in Place 10-inch Concrete	2,500	l.f.	2.60	6,500.00
Manholes Std. MH, 0 - 6 feet	6	each	190.00	1,140.00
Gravel Base	2,420	l.f.	0.35	847.00
Highway Crossing	90	l.f.	43.00	3,870.00
Railroad Crossing	50	l.f.	43.00	2,150.00
Subtotal				$17,507.00
20% Engineering & Contingencies				3,501.40
TOTAL				$21,008.40

ESTIMATE $21,000

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

STATE HOSPITAL HIGH SCHOOL MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
6 - 8 feet	1,750	l.f.	3.10	$ 5,425.00
Class E				
0 - 6 feet	100	l.f.	1.85	185.00
Sewer Pipe in Place				
8-inch Concrete	1,850	l.f.	2.10	3,885.00
Manholes				
Std. MH, 0 - 6 feet	6	each	190.00	1,140.00
Additional Depth	12	l.f.	18.00	216.00
Gravel Base	1,850	l.f.	0.35	647.50
Street Surface Replacement				
Asphalt	1,750	l.f.	2.30	4,025.00
Y-Stubouts	35	l.f.	15.00	525.00
Highway Crossing	60	l.f.	43.00	2,580.00
Subtotal				$18,628.50
20% Engineering & Contingencies				3,725.70
TOTAL				$22,354.20

ESTIMATE $22,400

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

RIVERSIDE INITIAL PROJECT

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
0 - 6 feet deep	2,005	l.f.	2.30	$ 4,611.50
6 - 8 feet deep	6,345	l.f.	3.10	19,669.50
Class E				
0 - 6 feet deep	3,140	l.f.	1.85	5,809.00
6 - 8 feet deep	3,820	l.f.	2.15	8,213.00
8 - 10 feet deep	220	l.f.	2.60	572.00
Sewer Pipe in Place				
8-inch Concrete	11,830	l.f.	2.10	24,843.00
10-inch Concrete	1,600	l.f.	2.60	4,160.00
12-inch Concrete	700	l.f.	3.05	2,135.00
18-inch Concrete	1,400	l.f.	5.35	7,490.00
Manholes				
Std. MH, 0 - 6 feet	46	each	190.00	8,740.00
Additional Depth	29	l.f.	18.00	522.00
Gravel Base	19,760	l.f.	0.35	6,916.00
Street Surface Replacement				
Gravel	4,250	l.f.	0.80	3,400.00
Asphalt	6,590	l.f.	2.30	15,157.00
River Crossing	320	l.f.	32.00	10,240.00
Highway Crossing	60	l.f.	43.00	2,580.00
Pump Station		l.s.		33,000.00
Side Sewers	230	each	100.00	23,000.00
Subtotal				$181,058.00
20% Engineering & Contingencies				36,211.60
TOTAL				$217,269.60

ESTIMATE $217,300

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

S. E. COURT PLACE RECONSTRUCTION

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
0 - 6 feet	570	l. f.	2. 30	$ 1, 311. 00
6 - 8 feet	570	l. f.	3. 10	1, 767. 00
Sewer Pipe in Place				
10-inch Concrete	1, 140	l. f.	2. 60	2, 964. 00
Manholes				
Std. MH, 0 - 6 feet	5	each	190. 00	950. 00
Additional Depth	2	l. f.	18. 00	36. 00
Gravel Base	1, 140	l. f.	0. 35	399. 00
Street Surface Replacement				
Asphalt	1, 140	l. f.	2. 30	2, 622. 00
Y-Stubouts	23	each	15. 00	345. 00
Bridge Crossing		l. s.		2, 000. 00
Subtotal				$ 12, 394. 00
20% Engineering & Contingencies				2, 478. 80
TOTAL				$ 14, 872. 80

ESTIMATE $14, 900

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

ALEXANDER AVENUE RECONSTRUCTION

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D 0 - 6 feet	1,150	l.f.	2.30	$ 2,645.00
Sewer Pipe in Place 8-inch Concrete	1,150	l.f.	2.10	2,415.00
Manholes Std. MH, 0 - 6 feet	3	each	190.00	570.00
Gravel Base	1,150	l.f.	0.35	402.50
Street Surface Replacement Asphalt	1,150	l.f.	2.30	2,645.00
Side Sewers	28	each	100.00	2,800.00
Subtotal				$ 11,477.50
20% Engineering & Contingencies				2,295.50
TOTAL				$ 13,773.00

ESTIMATE $13,800

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

STATE HOSPITAL U. S. 30 EASTERN MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class C				
0 - 6 feet	1,000	l. f.	1.20	$ 1,200.00
Class D				
0 - 6 feet	800	l. f.	2.30	1,840.00
Sewer Pipe in Place				
8-inch Concrete	1,800	l. f.	2.10	3,780.00
Manholes				
Std. MH, 0 - 6 feet	5	each	190.00	950.00
Gravel Base	1,800	l. f.	0.35	630.00
Street Surface Replacement				
Asphalt	800	l. f.	2.30	1,840.00
Y-Stubouts	6	each	15.00	90.00
Highway Crossing	60	l. f.	32.00	1,920.00
Subtotal				$ 12,250.00
20% Engineering & Contingencies				2,450.00
TOTAL				$ 14,700.00

ESTIMATE $14,700

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

STATE HOSPITAL U. S. 395 MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D 0 - 6 feet deep	1,450	l.f.	2.30	$ 3,335.00
Addition for Rock Excavation	1,450	l.f.	3.35	4,857.50
Sewer Pipe in Place 8-inch Concrete	1,450	l.f.	2.10	3,045.00
Manholes Std. MH, 0 - 6 feet	4	each	190.00	760.00
Gravel Base	1,450	l.f.	0.35	507.50
Street Surface Replacement Asphalt	1,450	l.f.	2.30	3,335.00
Y-Stubouts	35	each	15.00	525.00
Subtotal				$16,365.00
20% Engineering & Contingencies				3,273.00
TOTAL				$19,638.00

ESTIMATE $19,600

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

S. E. BYERS AREA - INFILTRATION

Item	Quantity	Unit	Unit Price	Amount
Move In & Out		l. s.		$ 1,000.00
Lateral Pressure Grout	7,400	l. f.	2.00	14,800.00
Side Sewers	2,200	l. f.	1.60	3,520.00
Subtotal				$19,320.00
20% Engineering & Contingencies				3,864.00
TOTAL				$23,184.00

ESTIMATE $23,200

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

BAILEY AVENUE MAIN RECONSTRUCTION

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
6 - 8 feet	174	l. f.	3.10	$ 539.40
8 - 10 feet	142	l. f.	4.00	568.00
10 - 12 feet	142	l. f.	5.25	745.50
12 - 14 feet	142	l. f.	6.90	979.80
Sewer Pipe in Place				
10-inch Concrete	600	l. f.	2.60	1,560.00
Manholes				
Std. MH, 0 - 6 feet	2	each	190.00	380.00
Additional Depth	10	l. f.	18.00	180.00
Single Drop MH, 0 - 6 feet	1	each	260.00	260.00
Additional Depth	3	l. f.	23.00	69.00
Gravel Base	600	l. f.	0.35	210.00
Street Surface Replacement				
Asphalt	600	l. f.	2.30	1,380.00
Subtotal				$ 6,871.70
20% Engineering & Contingencies				1,374.34
TOTAL				$ 8,246.04

ESTIMATE $8,300

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

RIVERSIDE FUTURE PROJECT

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class E 6 - 8 feet deep	4,080	l.f.	2.15	$ 8,772.00
Sewer Pipe in Place 8-inch Concrete	4.080	l.f.	2.10	8,568.00
Manholes				
Std. MH, 0 - 6 feet	11	each	190.00	2,090.00
Extra Depth	22	l.f.	18.00	396.00
Gravel Base	4,080	l.f.	0.35	1,428.00
Y Stubouts	80	each	15.00	1,200.00
Subtotal				$ 22,454.00
20% Engineering & Contingencies				4,490.80
TOTAL				$ 26,944.80

ESTIMATE $27,000

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

S. E. 10TH STREET MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
6 - 8 feet	850	l.f.	3.10	$ 2,635.00
8 - 10 feet	250	l.f.	4.00	1,000.00
Class C				
6 - 8 feet	870	l.f.	1.55	1,348.50
8 - 10 feet	500	l.f.	2.00	1,000.00
Sewer Pipe in Place				
8-inch Concrete	1,370	l.f.	2.10	2,877.00
10-inch Concrete	1,100	l.f.	2.60	2,860.00
Manholes				
Std. MH, 0 6 feet	9	each	190.00	1,710.00
Additional Depth	15	l.f.	18.00	270.00
Gravel Base	2,470	l.f.	0.35	864.50
Street Surface Replacement				
Asphalt	1,100	l.f.	2.30	2,530.00
Y-Stubouts	17	l.f.	15.00	255.00
Railroad Crossing	60	l.f.	43.00	2,580.00
Subtotal				$19,930.00
20% Engineering & Contingencies				3,986.00
TOTAL				$23,916.00

ESTIMATE $23,900

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

S. W. COURT AVENUE TRUNK

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
6 - 8 feet	1,900	l. f.	4.00	$ 7,600.00
8 - 10 feet	910	l. f.	5.15	4,686.50
10 - 12 feet	340	l. f.	6.65	2,261.00
Sewer in Place				
24-inch Concrete	3,150	l. f.	7.80	24,570.00
Manholes				
Std. MH, 0 - 6 feet	14	each	190.00	2,660.00
Additional Depth	36	l. f.	18.00	648.00
Gravel Base	3,150	l. f.	0.70	2,205.00
Street Surface Replacement				
Asphalt	3,150	l. f.	2.30	7,245.00
Y-Stubouts	65	each	15.00	975.00
Subtotal				$52,850.50
20% Engineering & Contingencies				10,570.10
TOTAL				$63,420.60

ESTIMATE $63,400

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

RIVERSIDE MOUNT HEBRON

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
0 - 6 feet	4,350	l.f.	2.30	$ 10,005.00
Class E				
0 - 6 feet	3,410	l.f.	1.85	6,308.50
6 - 8 feet	990	l.f.	2.15	2,128.50
Addition for Rock Excavation	2,200	l.f.	3.35	7,370.00
Sewer Pipe In Place				
8-inch Concrete	4,350	l.f.	2.10	9,135.00
10-inch Concrete	4.400	l.f.	2.60	11,440.00
Manholes				
Std. MH, 0 - 6 feet	35	each	190.00	6,650.00
Additional Depth	3	l.f.	18.00	54.00
Gravel Base	8,750	l.f.	0.35	3,062.50
Street Surface Replacement				
Asphalt	4,350	l.f.	2.30	10,005.00
Y-Stubouts	88	each	100.00	8,800.00
Highway Crossing	120	l.f.	43.00	5,160.00
Subtotal				$80,118.50
20% Engineering & Contingencies				16,023.70
TOTAL				$96,142.20

ESTIMATE $96,100

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

NORTH MCKAY CREEK TRUNK

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation &				
Backfill, Class C				
(18-inch pipe and under)				
6 - 8 feet	1,500	l.f.	1.55	$ 2,325.00
(18 - 30-inch pipe)				
0 - 6 feet	500	l.f.	1.50	750.00
6 - 8 feet	2,340	l.f.	2.00	4,680.00
Class X				
8 - 10 feet	2,600	l.f.	15.15	39,390.00
Sewer Pipe in Place				
12-inch Concrete	1,500	l.f.	3.05	4,575.00
24-inch Concrete	5,440	l.f.	7.80	42,432.00
Manholes				
Std. MH, 0 - 6 feet	15	each	190.00	2,850.00
Additional Depth	20	l.f.	18.00	360.00
Gravel Base				
18-inch Pipe & Under	1,500	l.f.	0.35	525.00
21-to 36-inch Pipe	5,440	l.f.	0.70	3,808.00
River Crossing	150	l.f.	32.00	4,800.00
Subtotal				$106,495.00
20% Engineering & Contingencies				21,299.00
TOTAL				$127,794.00

ESTIMATE $127,800

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

SOUTH MCKAY CREEK TRUNK

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class C (18-inch pipe to 30-inch pipe)				
0 - 6 feet	1,750	l.f.	1.50	$ 2,625.00
6 - 8 feet	2,450	l.f.	2.00	4,900.00
Class X				
8 - 10 feet	1,260	l.f.	15.15	19,089.00
Sewer Pipe in Place				
21-inch Concrete	5,460	l.f.	6.80	37,128.00
Manholes				
Std. MH, 0 - 6 feet	12	each	190.00	2,280.00
Additional Depth	13	l.f.	18.00	234.00
Gravel Base	5,460	l.f.	0.70	3,822.00
Subtotal				$70,078.00
20% Engineering & Contingencies				14,015.60
TOTAL				$84,093.60

ESTIMATE $84,400

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

N. W. KING AVENUE MAIN RECONSTRUCTION

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D				
0 - 6 feet	1,575	l. f.	2.30	$ 3,622.50
6 - 8 feet	400	l. f.	3.10	1,240.00
Sewer Pipe in Place				
8-inch Concrete	1,975	l. f.	2.10	4,147.50
Manholes				
Std. MH, 0 - 6 feet	12	each	190.00	2,280.00
Gravel Base	1,975	l. f.	0.35	691.25
Street Surface Replacement				
Asphalt	1,975	l. f.	2.30	4,542.50
Y-Stubouts	40	each	15.00	600.00
Subtotal				$ 17,123.75
20% Engineering & Contingencies				3,424.75
TOTAL				$ 20,548.50

ESTIMATE $20,600

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

S. E. THIRD STREET MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class C				
0 - 6 feet deep	500	l.f.	1.20	600.00
Trench Excavation & Backfill, Class D				
0 - 6 feet deep	1,080	l.f.	2.30	2,484.00
6 - 8 feet	950	l.f.	3.10	2,945.00
8 - 10 feet	180	l.f.	4.00	720.00
10 - 12 feet	90	l.f.	5.25	472.50
Addition for Rock Excavation	470	l.f.	3.35	1,574.50
Sewer Pipe in Place				
8-inch Concrete	2,800	l.f.	2.10	5,880.00
Manholes				
Std. MH, 0 - 6 feet	6	each	190.00	1,140.00
Additional Depth	10	l.f.	18.00	180.00
Gravel Base	2,800	l.f.	0.35	980.00
Street Surface Replacement				
Asphalt	2,300	l.f.	2.30	5,290.00
Y- Stubouts	42	each	15.00	630,00
Railroad Crossing	80	l.f.	43.00	3,440.00
Subtotal				$26,336.00
20% Engineering & Contingencies				5,267.20
TOTAL				$31,603.20

ESTIMATE $31,600

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

LOWER PENDAIR

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class				
0 - 6 feet	2,280	l.f.	1.20	$ 2,736.00
6 - 8 feet	2,820	l.f.	1.55	4,371.00
8 - 10 feet	570	l.f.	2.00	1,140.00
Class D				
6 - 8 feet	3,000	l.f.	3.10	9,300.00
Addition for Rock Excavation	3,000	l.f.	2.80	8,400.00
Sewer Pipe in Place				
8-inch Concrete	5,280	l.f.	2.10	11,088.00
10-inch Concrete	1,680	l.f.	2.60	4,368.00
12-inch Concrete	1,710	l.f.	3.05	5,215.50
Manholes				
Std. MH, 0 - 6 feet	25	each	190.00	4,750.00
Additional Depth	50	l.f.	18.00	900.00
Gravel Base	8,670	l.f.	0.35	3,034.50
Street Surface Replacement				
Asphalt	3,000	l.f.	2.30	6,900.00
Y-Stubouts	184	each	15.00	2,760.00
Highway Crossing	240	l.f.	43.00	10,320.00
Railroad Crossing	50	l.f.	43.00	2,150.00
Subtotal				$77,433.00
20% Engineering & Contingencies				15,486.60
TOTAL				$92,919.60

ESTIMATE $93,000

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

ROUNDUP TRUNK

Item	Quantity	Unit	Unit Price	Amount
Trunk Excavation & Backfill, Class E				
8 - 10 feet	3,659	l.f.	3.65	$ 13,355.35
10 - 12 feet	799	l.f.	4.50	3,595.50
12 - 14 feet	637	l.f.	5.50	3,503.50
14 - 16 feet	835	l.f.	6.25	5,218.75
Addition for Rock Excavation	1,320	l.f.	2.80	3,696.00
Sewer Pipe in Place				
24-inch Concrete	5,930	l.f.	7.80	46,254.00
Manholes				
Std. MH, 0 - 6 feet	13	each	190.00	2,470.00
Additional Depth	55	l.f.	18.00	990.00
Gravel Base	5,930	l.f.	0.70	4,151.00
Y-Stubouts	15	each	15.00	225.00
Highway Crossing	60	l.f.	43.00	2,580.00
Railroad Crossing	60	l.f.	43.00	2,580.00
Subtotal				$ 88,619.10
20% Engineering & Contingencies				17,723.82
TOTAL				$ 106,342.92

ESTIMATE $106,400

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

STATE HOSPITAL - U. S. 30 WESTERN MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class D 0 to 6 feet deep	2,550	l.f.	2.30	$ 5,865.00
Sewer Pipe in Place 8-inch Concrete	2,550	l.f.	2.10	5,355.00
Manholes Std. MH, 0 - 6 feet	8	each	190.00	1,520.00
Gravel Base	2,550	l.f.	0.35	892.50
Street Surface Replacement Asphalt	2,550	l.f.	2.30	5,865.00
Y-Stubouts	50	each	15.00	750.00
Subtotal				$20,247.50
20% Engineering & Contingencies				4,049.50
TOTAL				$24,297.00

ESTIMATE $24,300

DETAILED COST ESTIMATE
ULTIMATE COLLECTION SYSTEM

EAST PENDLETON RAILROAD MAIN

Item	Quantity	Unit	Unit Price	Amount
Trench Excavation & Backfill, Class E				
0 - 6 feet deep	2,250	l.f.	1.85	$ 4,162.50
6 - 8 feet deep	760	l.f.	2.15	1,634.00
8 - 10 feet deep	460	l.f.	2.60	1,196.00
10 - 12 feet deep	130	l.f.	3.25	422.50
Addition for Rock Excavation	800	l.f.	2.80	2,240.00
Sewer Pipe in Place				
8-inch Concrete	3,600	l.f.	2.10	7,560.00
Manholes				
Std. MH, 0 - 6 feet	10	each	190.00	1,900.00
Additional Depth	5	l.f.	18.00	90.00
Single Drop MH, 0 - 6 feet	1	each	260.00	260.00
Additional Depth	7	l.f.	23.00	161.00
Gravel Base	3,600	l.f.	0.35	1,260.00
Highway Crossing	120	l.f.	43.00	5,160.00
Railroad Crossing	60	l.f.	43.00	2,580.00
Subtotal				$28,626.00
20% Engineering & Contingencies				5,725.00
TOTAL				$34,351.00

ESTIMATE $34,400

APPENDIX E

OREGON STATE BOARD OF HEALTH
STATE OFFICE BUILDING
1400 S. W. 5TH AVENUE
PORTLAND 1, OREGON

TELEPHONE:
DAYS—CAPITOL 6-2161
AFTER HOURS—CAPITOL 2-1800

MAILING ADDRESS:
P. O. BOX 231
PORTLAND 7, OREGON

District Office:
218 Raley Building
29 S. E. Court Street
Pendleton, Oregon
Phone: CRestview 6-0841

September 28, 1962

Mayor and City Council
City of Pendleton
City Hall
Pendleton, Oregon

Attention: M. O. Gardner, City Manager

Gentlemen:

The Pendleton Sewage and Industrial Waste Treatment plants failed to function at efficiencies expected during the 1962 pea processing season.

Hydraulically at the Main Plant flows exceeded the designed average flow of 5 million gallons per day.

On two of the four measured days at the Main Plant efficiencies of removal were 25% and 48% (in terms of organic loading B.O.D.-5) as compared to a 63% minimum removal expected under existing loadings and the 80 to 90% normally obtained during the non-pea processing season.

Removal efficiencies at the Industrial Waste Pretreatment Plant on the two occasions measured were 13 to 19% as compared to an expected reduction in excess of 39%.

Since it is the statutary resposibility of the Oregon State Sanitary Authority to maintain the maximum degree of purity in the waters of the state and since O.A.R. Chapter 334, Section 1205, requires that all sewage and industrial waste treatment and disposal plants shall be operated at their highest practical efficiency, this matter is being called to your attention with the following comments and recommendations:

It is suggested that a thorough review of pretreatment facilities and operation may disclose methods by which industrial waste loadings may be substantially reduced.

COPY

It is recommended that contact be made with your engineering firm to thoroughly evaluate and recommend remedial measures.

It is understood that a sewer system study program is currently underway.

Very truly yours,

H. M. Patterson
District Sanitary Engineer

HMP:mp
cc: Tony Groh, City Engineer
cc: Portland

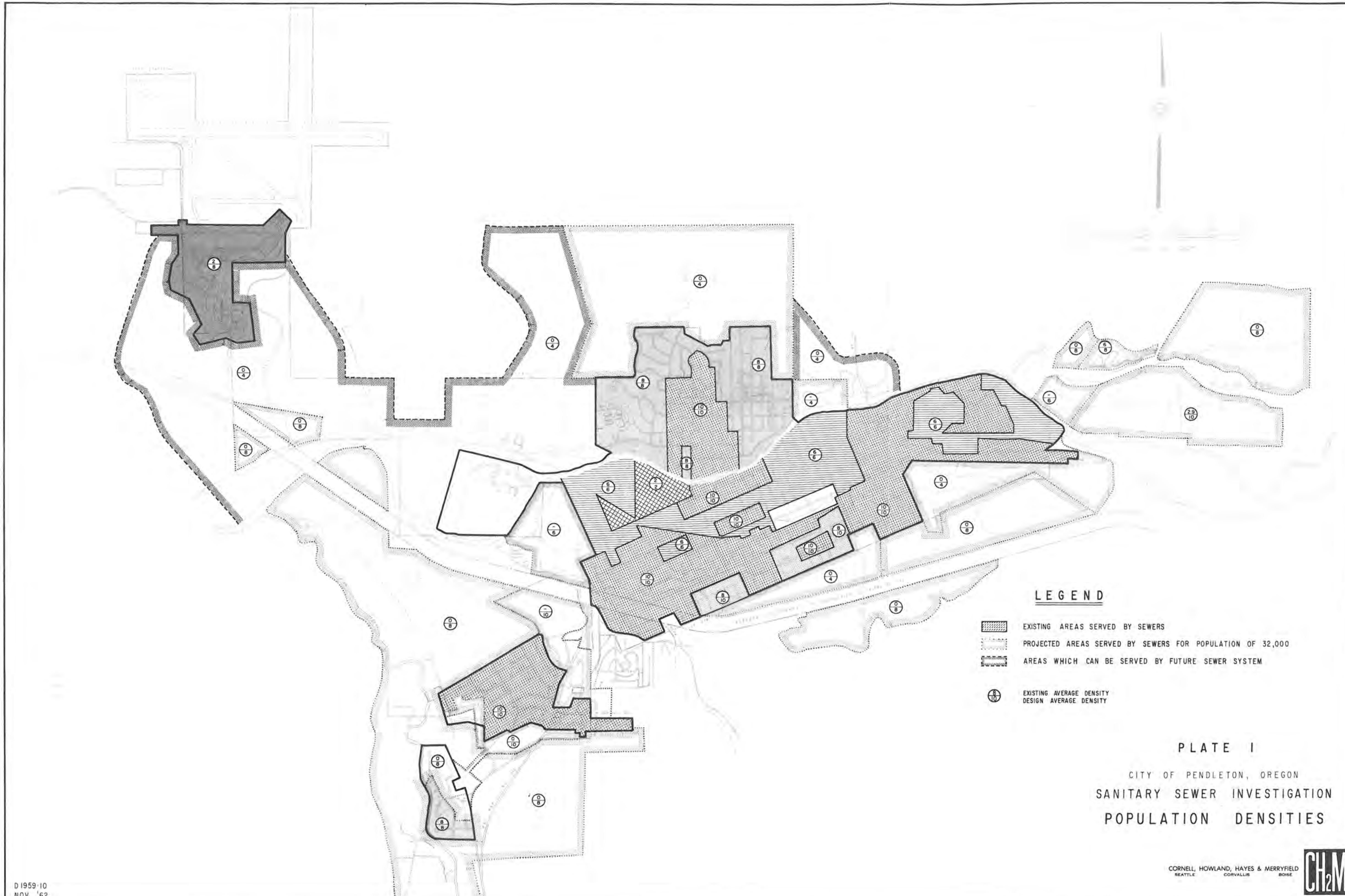

PLATE I

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

POPULATION DENSITIES

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

CH2M

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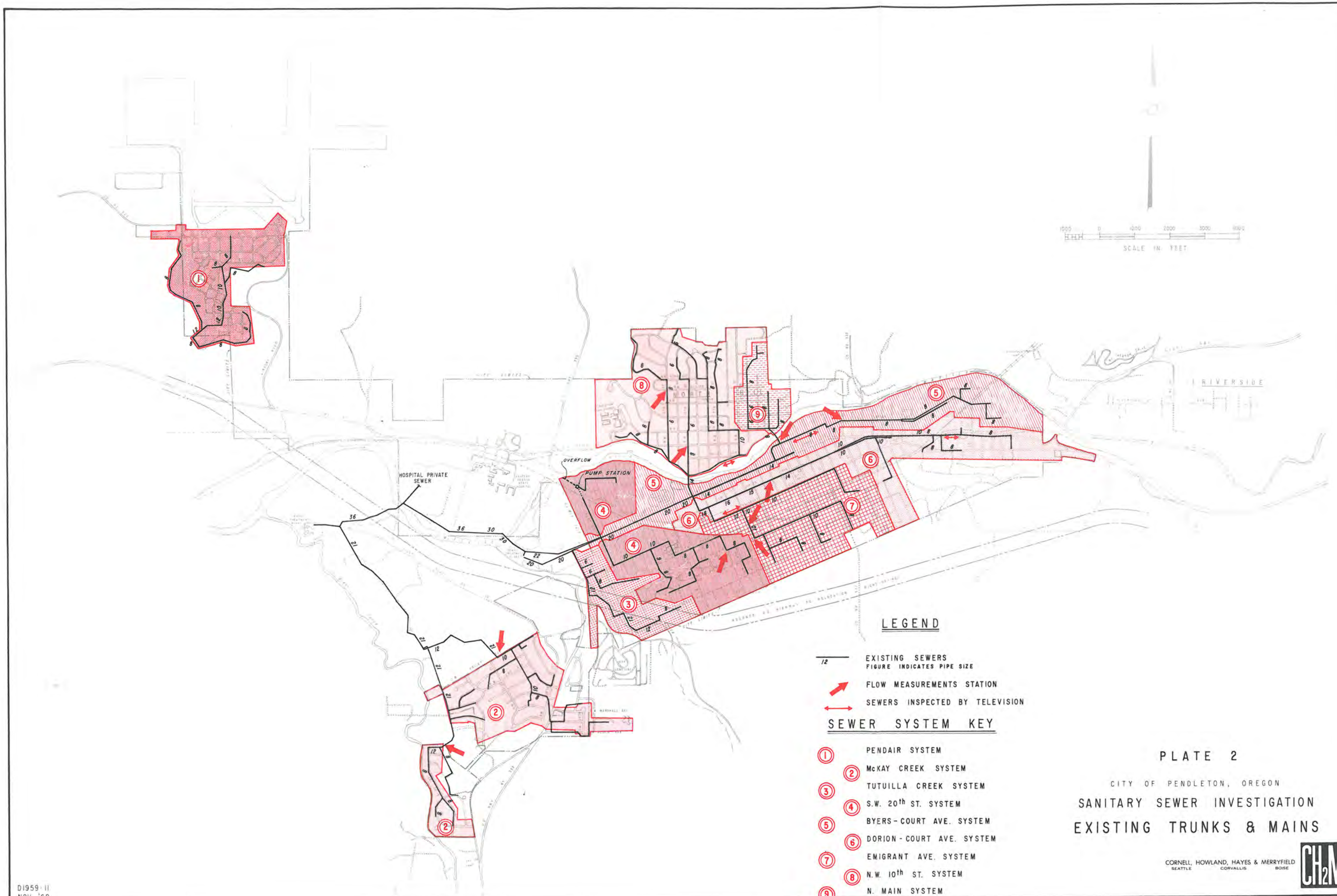
LEGEND
EXISTING SEWERS
FIGURE INDICATES PIPE SIZE
FLOW MEASUREMENTS STATION
SEWERS INSPECTED BY TELEVISION
SEWER SYSTEM KEY
1 PENDAIR SYSTEM
2 McKAY CREEK SYSTEM
3 TUTUILLA CREEK SYSTEM
4 S.W. 20th ST. SYSTEM
5 BYERS-COURT AVE. SYSTEM
6 DORION-COURT AVE. SYSTEM
7 EMIGRANT AVE. SYSTEM
8 N.W. 10th ST. SYSTEM
9 N. MAIN SYSTEM
HOSPITAL PRIVATE SEWER
OVERFLOW
PUMP STATION
RIVERSIDE
SCALE IN FEET
PLATE 2
CITY OF PENDLETON, OREGON
SANITARY SEWER INVESTIGATION
EXISTING TRUNKS & MAINS
CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE
CH2M
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NOV. '62

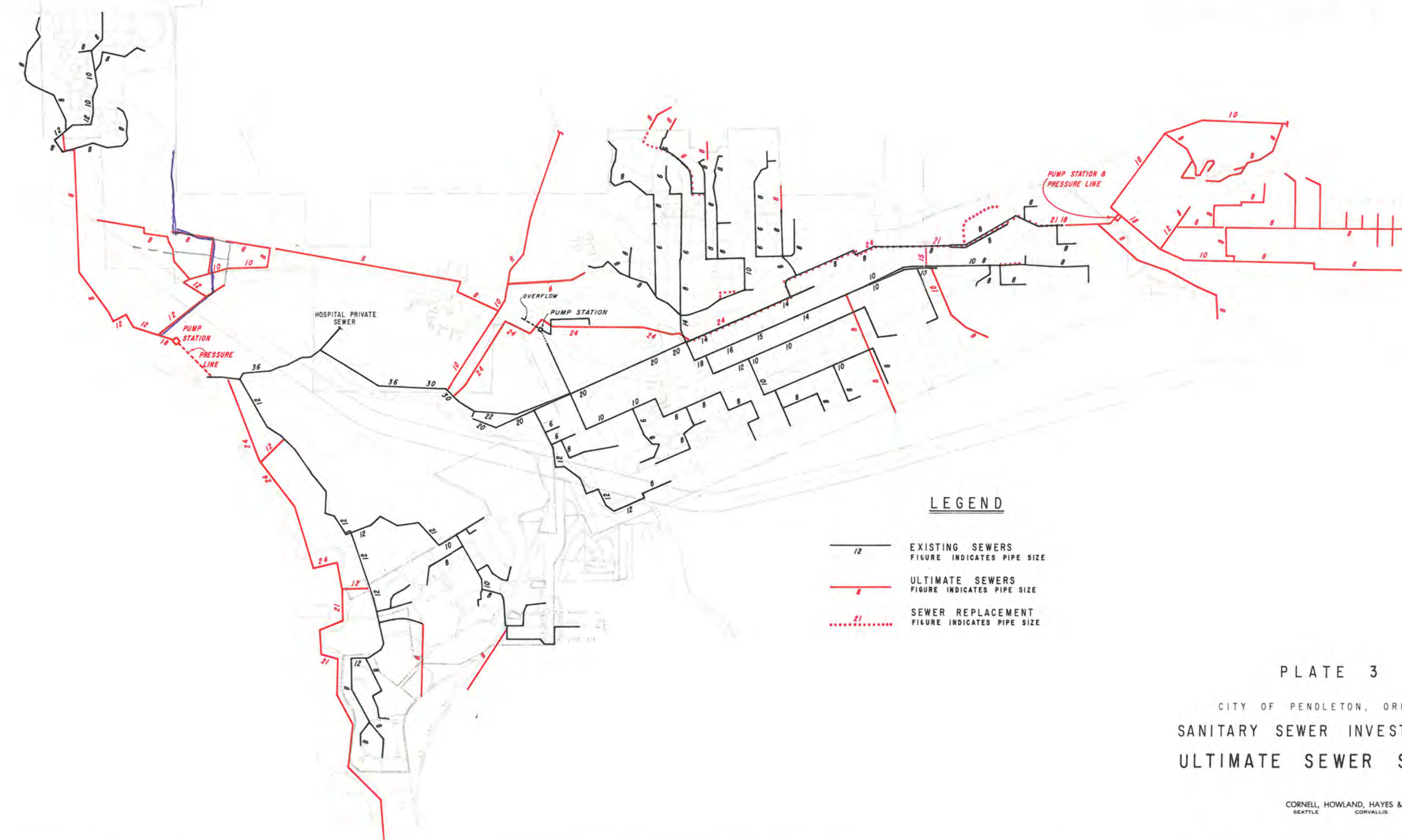

PLATE 3

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

ULTIMATE SEWER SYSTEM

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

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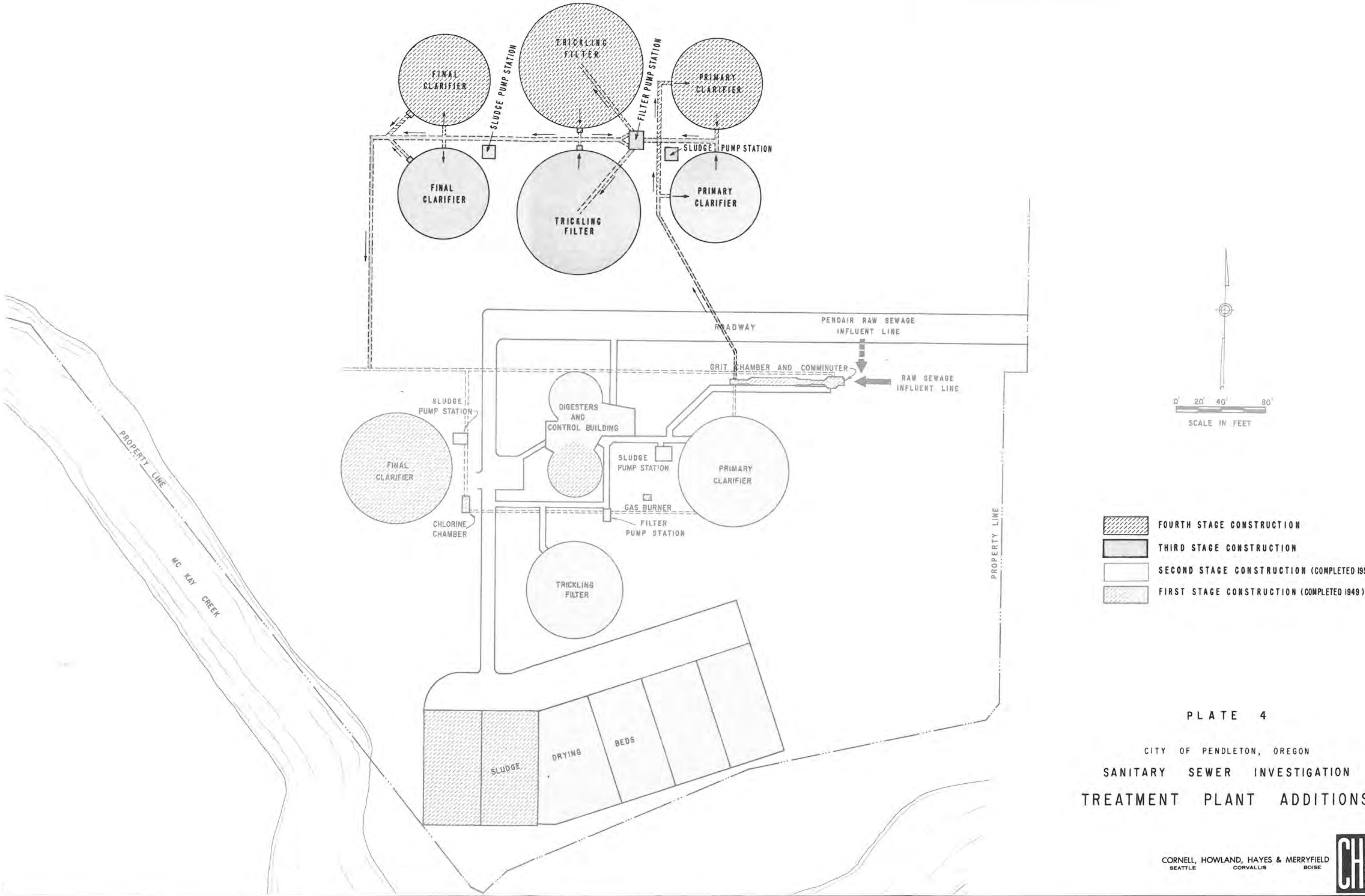

PLATE 4

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

TREATMENT PLANT ADDITIONS

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

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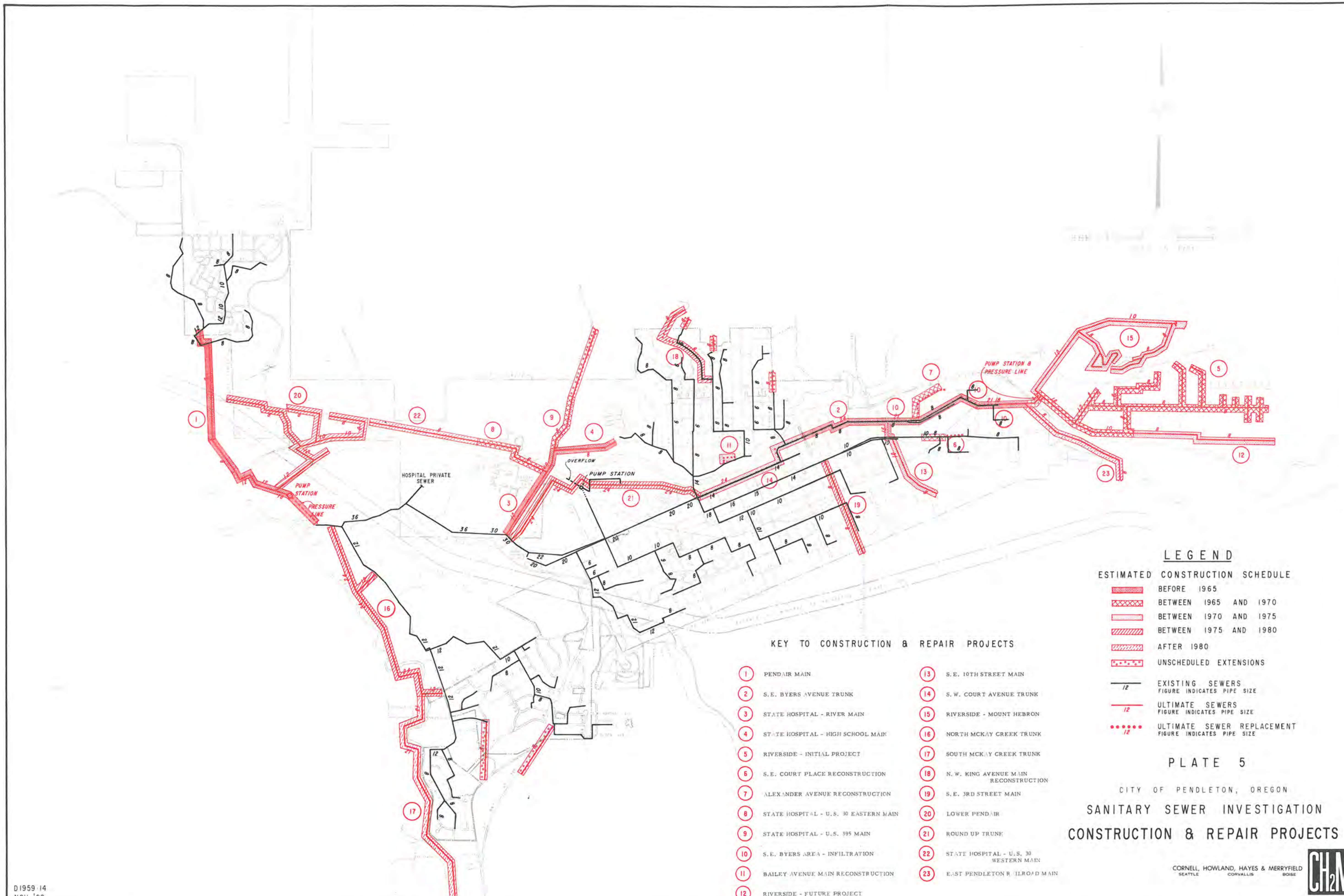

PLATE 5

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

CONSTRUCTION & REPAIR PROJECTS

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

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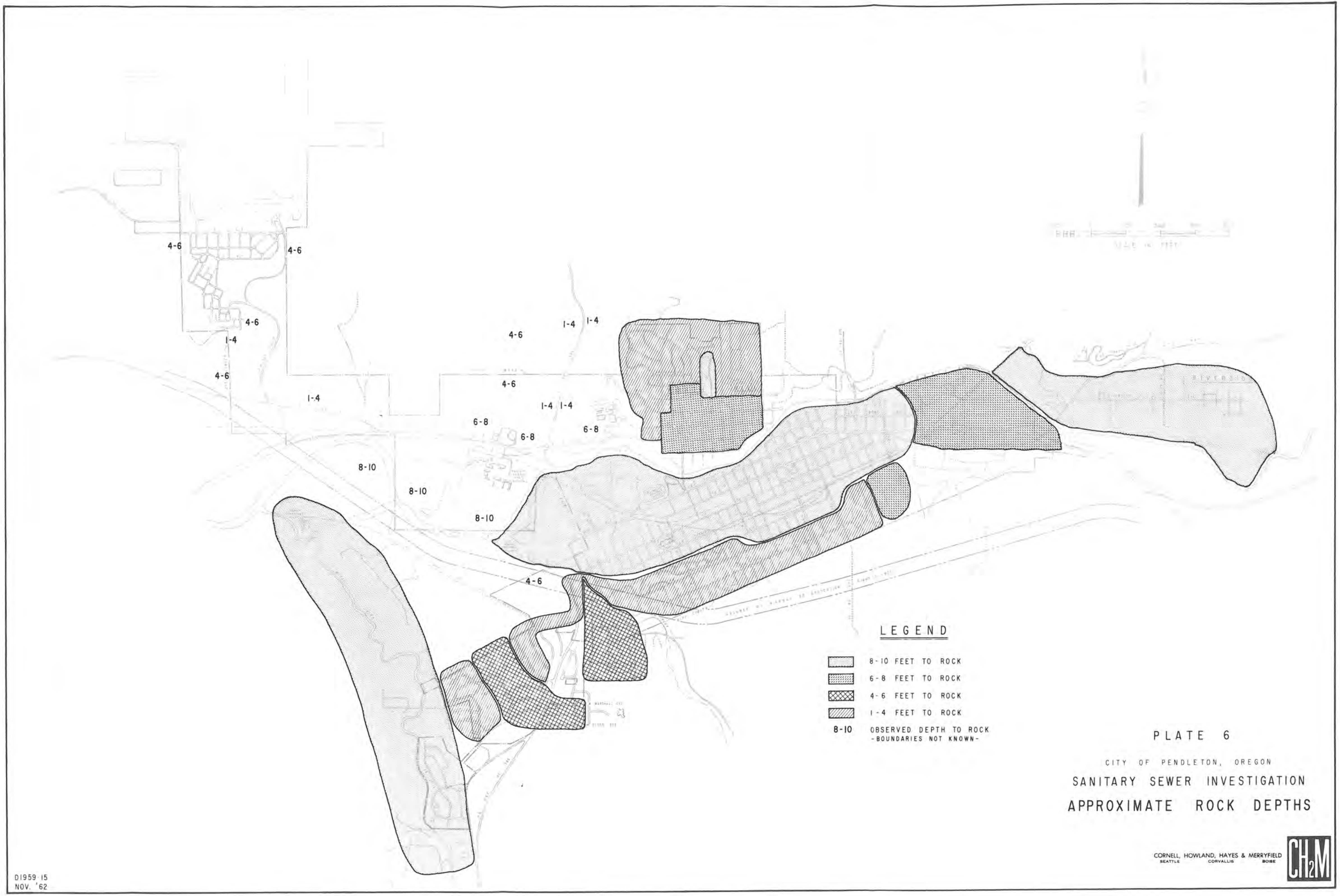

PLATE 6

CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

APPROXIMATE ROCK DEPTHS

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE
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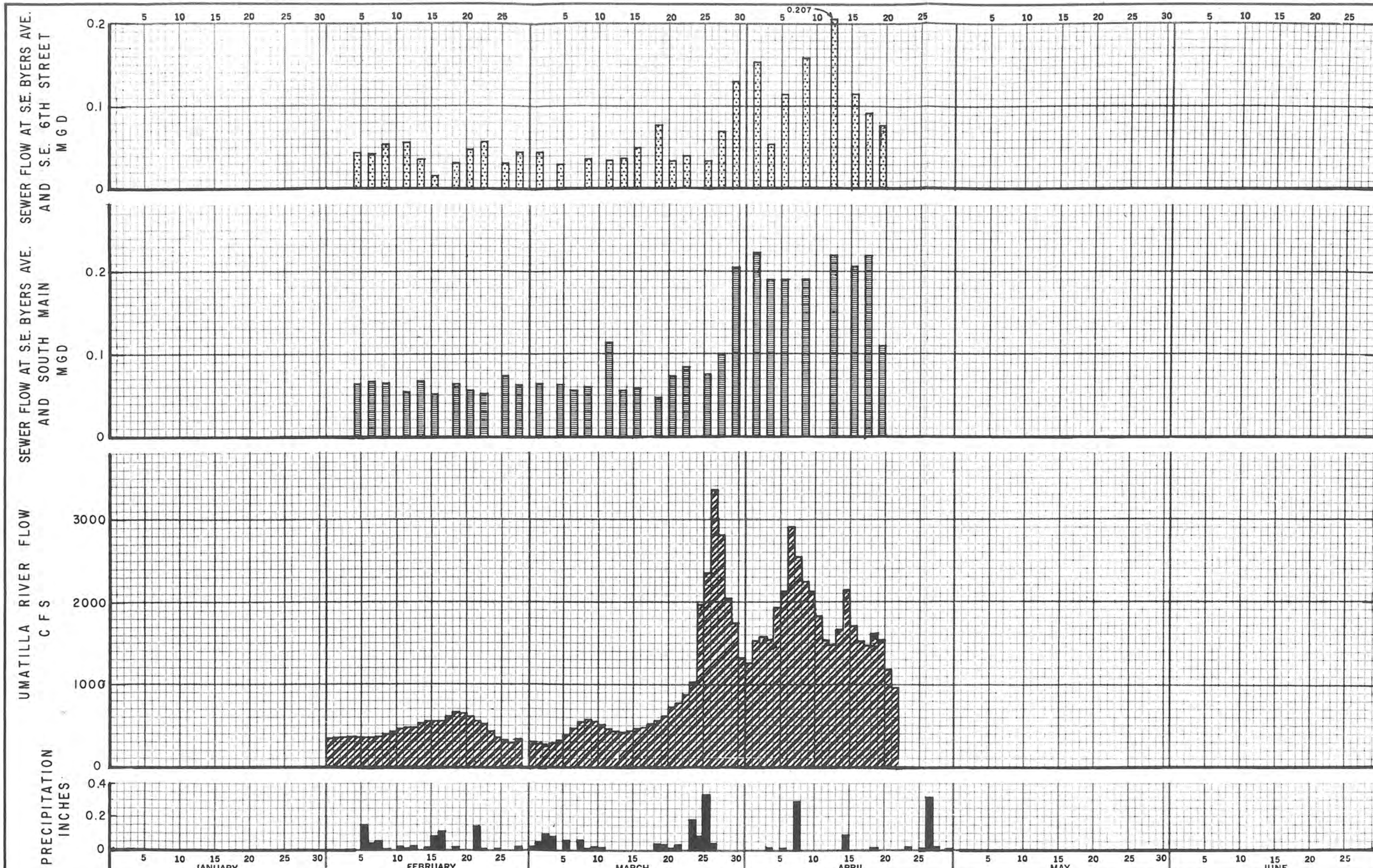

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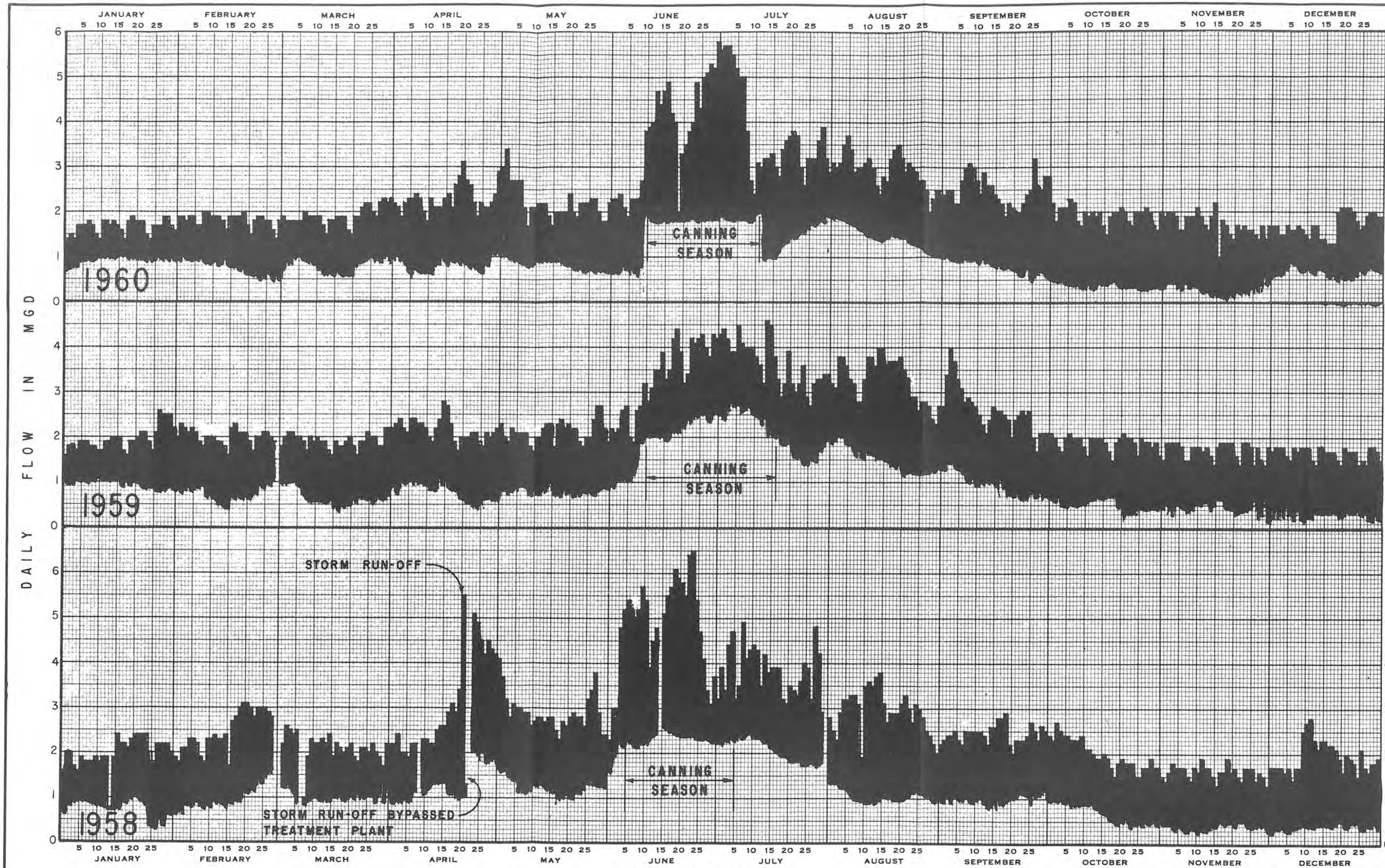
CITY OF PENDLETON, OREGON
SANITARY SEWER INVESTIGATION
INFILTRATION DATA - 1962

PLATE 7
CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE



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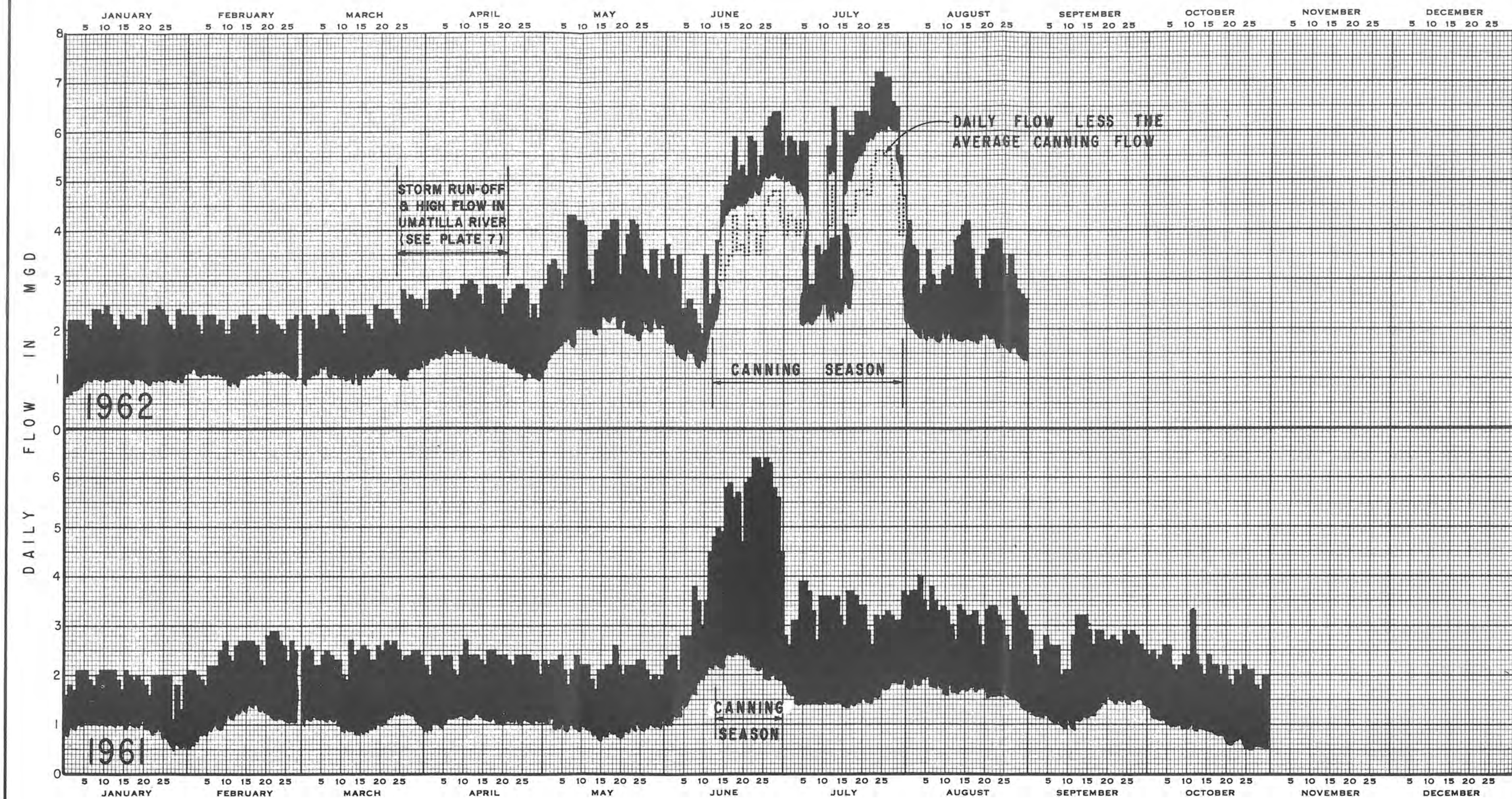
CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

SEWAGE TREATMENT PLANT FLOW – 1958, 1959 & 1960

PLATE 8

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE



CITY OF PENDLETON, OREGON

SANITARY SEWER INVESTIGATION

SEWAGE TREATMENT PLANT FLOW - 1961 & 1962

PLATE 9

CORNELL, HOWLAND, HAYES & MERRYFIELD
SEATTLE CORVALLIS BOISE

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