OREGON CITY

DRAINAGE MASTER PLAN

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Prepared for

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TABLE OF CONTENTS

CHAPTER	TITLE	PAGE
	EXECUTIVE SUMMARY	ES-1
1	INTRODUCTION	
1.1	Authorization	1-1
1.2	Study Purposes and Goals	1-1
1.3	Drainage Master Planning	1-1
2	PHYSICAL ENVIRONMENT	
2.1	General	2-1
2.2	Climate	2-1
2.3	Soils	2-1
2.4	Land Use	2-3
2.5	Flood Plain Information	2-5
3	EXISTING DRAINAGE SYSTEM	
3.1	General	3-1
3.2	Major Drainage System	3-1
3.3	Minor Drainage System	3-25
4	HYDROLOGIC/HYDRAULIC ANALYSIS	
4.1	General	4-1
4.2	Drainage Planning Criteria	4-2
4.3	HEC-1 Description, Input Parameters and Calibration	4-6
4.4	Hydrological Results	4-14
4.5	Drainage System Deficiencies	4-22
5	DRAINAGE PLAN DEVELOPMENT	
5.1	Planning Objectives	5-1
5.2	Alternative Approaches	5-1
5.3	Drainage Plan Development Process	5-3
6	RECOMMENDED MASTER DRAINAGE PLAN	
6.1	General	6-1
6.2	Drainage System Design Criteria	6-1
6.3	Capital Improvements Cost Estimating	6-4
6.4	Recommended Capital Improvement Programs	6-4
6.5	Recommended Drainage Maintenance Program	6-41

TABLE OF CONTENTS (CONTINUED)

CHAPTER	TITLE	PAGE
7	FINANCING PLAN	
7.1	General	7-1
7.2	Cost Apportioning	7-1
7.3	Funding Options	7-3
7.4	Recommendations	7-5
7.5	Drainage Utility Rates	7-8
	REFERENCES	
Appendix A	DRAINAGE DESIGN PROCEDURES AND STANDARDS	A-1
Appendix B	SCS RAINFALL DISTRIBUTIONS	B-1
Appendix C	SUBBASIN PARAMETER ESTIMATION TECHNIQUES	C-1

LIST OF TABLES

TABLE	TITLE	PAGE
2.1	Hydrological Groupings of Soils Found Throughout the Oregon City Study Area	2-4
3.1	Major Hydraulic Structures Inventory	3-23
4.1	Storm Recurrence Intervals for Planning and Design of Drainage Improvements	4-3
4.2	Curve Numbers Established for Each Hydrologic Soil Group	4-7
4.3	Mapped Impervious Area Used in Oregon City by Land Use Category	4-8
4.4	Subbasin Parameters Used in Hydrological Modeling	4-11
4.5	Subbasin Peak Flows for Existing Conditions of Development	4-15
4.6	Subbasin Peak Flows for Ultimately Planned (Future) Development Conditions	4-18
4.7	Simulated Peak Flows Along the Major Drainage Collector Systems	4-20
4.8	Performance of Major Hydraulic Structures Under Existing Conditions of Development	4-23
4.9	Performance of Major Hydraulic Structures Under Future Conditions of Development	4-25
6.1	Proposed Capital Improvements for the Caufield Basin	6-7
6.2	Proposed Capital Improvements for the Central Point Basin	6-10
6.3	Proposed Capital Improvements for the Mud Basin	6-12
6.4	Proposed Capital Improvements for the South End Basin	6-16
6.5	Proposed Capital Improvements for the Coffee Basin	6-20

LIST OF TABLES (CONTINUED)

TABLE	TITLE	PAGE
6.6	Proposed Capital Improvements for the Clinton Basin	6-24
6.7	Proposed Capital Improvements for the Upper Newel Basin	6-26
6.8	Proposed Capital Improvements for the Park Place and Livesay Basins	6-28
6.9	Proposed Capital Improvements for the John Adams Basin	6-32
6.10	Proposed Capital Improvements for the Singer Basin	6-37
6.11	Estimated Major Drainage System Maintenance Costs	6-47

LIST OF FIGURES

FIGURE	TITLE	PAGE
2.1	Oregon City Study Area	2-2
2.2	100-Year Floodplain Throughout the Oregon City Study Area	2-7
3.1	Drainage Basin Boundaries Throughout the Oregon City Study Area	3-3
3.2	Caufield Drainage Basin	3-6
3.3	Central Point and Mud Drainage Basins	3-8
3.4	South End Drainage Basin	3-10
3.5	Coffee, Clinton and Tumwater Drainage Basins	3-12
3.6	Upper Newell Drainage Basin	3-14
3.7	Lower Newell Drainage Basin	3-15
3.8	Park Place and Livesay Drainage Basins	3-17
3.9	John Adams and Singer Drainage Basins	3-18
4.1	Rainfall Intensity Duration and Frequency Curves for Oregon City	4-5
6.1	Capital Improvement Projects for the Caufield Drainage Basin	6-6
6.2	Capital Improvement Projects for the Central Point and Mud Drainage Basins	6-9
6.3	Proposed Improvement of Existing Pipe Inlet to M20.20	6-14
6.4	Capital Improvement Projects for the South End Drainage Basin	6-15
6.5	Capital Improvement Projects for the Coffee, Clinton and Tumwater Drainage Basins	6-19
6.6	Proposed Improvement of Existing Culvert Inlet to CO30.30	6-22
6.7	Capital Improvement Projects for the Upper Newell Drainage Basin	6-25

LIST OF FIGURES (CONTINUED)

FIGURE	TITLE	PAGE
6.8	Capital Improvement Projects for the Park Place and Livesay Drainage Basins	6-27
6.9	Proposed Improvements at the Intersection of S. Cleveland and S. Harley Street P 40.21 to P40.23	6-29
6.10	Capital Improvement Projects for the John Adams and Singer Drainage Basins	6-31
6.11	Typical Detention Outlet Structure	6-35
6.12	Proposed Improvement in John Adams Street from 12th Street to 15th Street	6-36
6.13	Proposed Improvement of Existing Pipe Inlet to S60.45	6-40

Executive Summary

EXECUTIVE SUMMARY

The Oregon City Drainage Master Plan has identified and presented:

- A capital improvements program that will solve existing major drainage problems and provide adequate drainage facilities for the study area under ultimate planned development conditions.

- Drainage design procedures and standards needed to plan, design, construct and maintain drainage collection and detention facilities.

- Funding methods that should be used to offset the cost of maintaining the existing drainage system and the cost of construction and maintenance of the recommended capital improvement programs.

EXISTING DRAINAGE SYSTEM

The primary drainage problem in the Oregon City area is overbank flooding that occurs when the quantity of storm water exceeds channel capacity and/or the capacity of culverts and pipes. Overbank flooding can destroy property and endanger lives. High floodwater velocities can erode channels and banks endangering bridge piers and approaches. Floodwaters can remove valuable topsoil from farmland and decrease normal channel capacity with mud and debris.

The Oregon City study area has been divided into 22 drainage areas. Twelve of these areas have been defined as major drainage basins and have been further divided into subbasins. Hydrological computer models have been developed for these 12 basins and capital improvement projects, described in Chapter 6, address the major drainage collector systems.

Drainage system deficiencies are assumed to exist if the capacity of the various channels, culverts and pipes along the major drainage courses are found to be significantly less than those specified in the drainage planning criteria. This condition was found to occur in approximately 33 of the 59 major hydraulic structures inventoried in the study area. This means that approximately 56 percent of the existing hydraulic structures appear to be deficient in effectively passing peak flows under existing conditions of development. Of these 33 structures, approximately 26 of them cannot pass the estimated 5-year peak flow. This indicates that overbank flooding problems occur frequently at some locations throughout the study area.

In addition, approximately 38 of the 59 hydraulic structures that were inventoried appear to be deficient under future flow conditions. It is interesting to note that the impact of continued urbanization on the culverts and pipes deemed to be adequate under existing conditions was not as great as what is normally

found in drainage planning. Only 5 more structures are classified as deficient under future flow conditions when compared to existing flow conditions. This means that 80 percent of the drainage structures deemed adequate under existing development conditions will continue to function adequately under ultimate development conditions.

CAPITAL IMPROVEMENT PROGRAMS

A total of fifty-five capital improvement projects (CIPs) have been recommended for the major drainage collector systems in the study area. The total estimated capital improvement program costs for completing these construction projects is $1,306,480. This program cost includes construction with contingency; at $1,004,910; engineering and surveying at $170,920; legal and administration at $40,200 and financing at $90,450.

The capital improvement projects are recommended to be implemented over a 20-year period. Each of the improvement projects was assigned a priority ranking from 1 to 4. The priority ranking represents a 5-year time period in which the project should most likely be constructed. These rankings are based on several subjective criteria. First and foremost was the severity of the drainage problem. Another concern was the location of the project. Is the CIP located within the city now or is it anticipated to be located in the city later? In addition, the priority rankings were based on the assumption that CIP expenditures would be maintained at approximately $65,000 per year (1987 dollars). Thus, the total program costs for all the projects with the same priority ranking would be approximately $325,000 in 1987 dollars.

MAINTENANCE PROGRAM

Continued maintenance of the major drainage system is necessary if it is to act as an effective outfall for stormwater runoff. To maximize the use of the major drainageways during runoff and flood periods, and thereby reduce the damage potential, a good drainage maintenance program is an absolute necessity.

Based on field observations during the collection of data of this study, it was concluded that the actual hydraulic capacity of the existing major drainage systems throughout most of the Oregon City study area is functioning at a small fraction of its potential capacity due to its poorly maintained condition. This statement is not intended as a criticism of existing maintenance practices. As a general rule, adequate funding for system maintenance simply is not allocated. Maintenance personnel do the best they can with the budgets they are given. Drainage typically receives a low priority in the competition for limited maintenance dollars.

Numerous studies throughout the country have shown that the most economical expenditure of drainage monies is for the continued maintenance of the drainage system. Given the tremendous investment the City already has in their drainage systems, it makes sense that they should be functioning at close to 100 percent of their available capacity when that infrequent flood event occurs.

The primary goal of the recommended drainage maintenance program presented in Chapter 6 could be stated as:

"Within available funds, the City of Oregon City will provide preventive maintenance and rehabilitate drainageway facilities in a manner that will ensure reasonably adequate functioning of the drainageways and hydraulic structures during periods of stormwater runoff."

FUNDING PROGRAM

A lack of adequate funding is a common frustration of all municipalities in dealing with drainage problems. Drainage system maintenance and capital improvement funding is basic to a system's successful operation as an efficient transporter of stormwater runoff. If funds are not available to address existing and potential drainage problems, the chance for further flood damage must become a recognized fact. The recommendations of the master drainage plan range from the preservation of natural drainageways to structural modifications of channels, and culverts. To prevent damages, funds must be invested.

A drainage utility has been recommended as the most appropriate funding program for the City of Oregon City. The service charges or user fees from the utility can be used to finance:

1. The cost of implementing and continuing the drainageway maintenance program described in Chapter 6.

2. The cost of administration, engineering, design and construction of the major capital improvements presented in Chapter 6.

3. The cost of administration, engineering, design and construction of anticipated capital improvements to the minor drainage systems which have not been identified as part of this drainage study.

4. The cost of obtaining drainage easements for the major drainageways throughout the study area (these costs have not been estimated as part of this drainage study).

The recommended rate structure for the Oregon City drainage utility is a flat rate for all single family residences combined with a variable rate charge for other properties based on gross area and the intensity of development. The flat rate charge for

all single family residences would be based on a typical residential property having an 8,000 to 10,000 square foot lot with approximately 2,000 to 3,000 square feet of impervious coverage.

The variable rate charge for non-single family residential properties would be based on the representative number of equivalent residential units for that property. The equivalent residential units (ERU) of a non-single family residential property would be computed by multiplying the property's area range number (ARN) by its development intensity factory (DIF) as follows:

$$ERU = ARN \times DIF$$

The area range number (ARN) is a simplified way of grouping non-single family residential properties into groups of similar parcel size. It is recommended that all parcels with gross areas of 1 to 2,500 square feet would be assigned an area range number (ARN) of 1. Parcels with gross areas of 2,501 to 5,000 square feet would have an ARN of 2. ARNs equal to 3 represent parcels whose area ranges between 5,001 and 7,500 square feet and so on.

The development intensity factory (DIF) can be viewed as a runoff coefficient that is indicative of the land use or impervious coverage of each property. Chapter 7, Section 7.5.2, represents the recommended DIFs for each of the existing land uses and zoning throughout Oregon City.

It is recommended that a system development charge should be established as part of the drainage utility rate structure. The system development charge could be assessed when a parcel is developed or substantially redeveloped. The charge is structured so that developing properties will pay their equitable share of the cost of drainage improvements that were designed to eventually accommodate their development. Therefore, developing properties must "buy into" the community's previous investment in these drainage systems.

The system development charge is based on the accumulated capital improvement costs the property owner would have paid if his property had been developed since the drainage utility was originally established. The system development charge would reach an accumulated maximum at 20 years.

RECOMMENDATIONS

1. Adopt the Oregon City Drainage Master Plan with specific reference to the adoption of the Drainage Design Procedures and Standards outline in Appendix A of this report.

2. Establish a drainage utility for the City of Oregon City (e.g. ORS 454) with an initial utility rate of $2.00 per month for each equivalent residential unit (ERU). All

single family residential owners are to be charged for one ERU regardless of actual lot size. Monies from the utility are used for maintenance of the drainage system, CIP engineering and construction, drainage easement acquisition and utility administration.

3. Monthly utility rates for non-single family residential properties are based on their estimated ERUs multiplied by $2.00. The properties ERU will be based on its Area Range Number (ARN) multiplied by its Development Intensity Factor (DIF; see Section 7.5.2).

4. A system development charge should be included within the drainage utility's established rate structure (see Section 7.5.5).

5. Construct the recommended 55 major capital improvement projects for drainage over the next 20-year period. Capital improvement program costs are estimated at $1,306,480 for an average expenditure of $65,000 per year (i.e., 1987 dollars). CIP costs are to be funded by approximately 35 percent of the drainage utility's collections.

6. Utilize approximately 15 percent of the drainage utility's annual collections to design and construct needed minor capital improvement projects for drainage. Expenditures are estimated to be approximately $28,000 per year.

7. Implement the recommended maintenance program for the entire Oregon City drainage system with special emphasis on the major drainage system (i.e., Section 6.5). Maintenance and drainageway easement acquisition programs are to be funded by approximately 40 percent of the drainage utility collections. Expenditures are estimated to be approximately $74,000 per year (i.e., 1987 dollars).

8. Utilize approximately 10 percent of the drainage utility's annual collections to administer the utility. Expenditures are estimated to be approximately $18,000 per year.

9. Conduct a detailed hydraulic analysis of the Singer Creek culvert that includes a field survey of the culvert's exact alignment, flowline elevations, dimensions and surface roughness (i.e., see Section 6.4.10). Based on the results of this hydraulic analysis, adjust the recommended CIPs for the Singer Creek basin, if necessary.

Chapter 1
Introduction

CHAPTER 1

INTRODUCTION

1.1 AUTHORIZATION

The preparation of this report was authorized by the City of Oregon City on November 13, 1986 as part of a professional services agreement to prepare a Sanitary Sewer Master Plan and Storm Drainage Master Plan for the City of Oregon City's Urban Growth Boundary (UGB).

This report documents the storm drainage master planning work that was conducted under the agreement.

1.2 STUDY PURPOSES AND GOALS

The purpose of the Drainage Master Plan is to provide the City of Oregon City with specific engineering recommendations for the control of storm drainage in the study area.

The goals of the plan are to:

- Identify the capital improvement projects that will solve current drainage problems and provide adequate drainage for the study area when it reaches ultimate planned development conditions.

- Establish the criteria and standards needed to plan, design, construct, and maintain drainage collection and detention facilities.

- Identify the funding methods that should be used to offset the cost of maintaining the existing drainage system and the costs of construction and maintenance of the recommended capital improvement programs.

1.3 DRAINAGE MASTER PLANNING

Drainage Master Planning involves a broad planning process for urban drainage and flood control that will assist the City of Oregon City to achieve its comprehensive goals and objectives. This drainage master plan should reduce the public cost for storm sewers, channels, culverts, and bridges, and lower the costs for culvert replacement, maintenance, and relief and rehabilitation following floods.

It will identify drainage problems and constraints before future development can impose additional burdens on the existing drainage system. The Drainage Master Plan will also identify various methods by which the plan could be funded while ensuring that future development shares the costs of the plan's implementation.

The drainage master plan will identify a course of action for urban drainage through corrective and preventive measures. Corrective, or structural, measures seek to mitigate flood damage potential that has been created by urban development encroaching on natural floodplains. Preventive, or nonstructural, measures are taken through floodplain management and attempt to reduce the damage potential of minor and major storms before future development can encroach on the floodplain.

Master drainage planning for the urban drainage network is based on the premise that two separate and functionally distinct drainage systems exist. The first, or minor, system accommodates the storm runoff expected once every 2 to 10 years. It consists of property line swales, streets, gutters, storm sewers, and smaller open drainageways. It provides convenience drainage, reduces street maintenance costs, and directly affects the orderliness of an urbanized area.

The second, or major, system handles runoff from the 10- to 100-year storm in order to prevent loss of life and major damage. This storm runoff follows the major drainage system route during a major storm, whether or not the route is planned. The 100-year storm is studied to ensure compatibility with State and Federal floodplain management policies; although facilities may, in most cases, be designed for lesser events. In these cases, the effect of the 100-year storm is considered when assessing the impact of the improvements.

The management strategies developed in this plan are for the major drainageways since they provide the basic outlet for both minor and major stormwater in each drainage basin. If the major drainageways are not preserved or improved, they will not be able to handle increased amounts of stormwater runoff, which can be expected as further urbanization takes place.

The drainage plan will incorporate the Comprehensive Plan policies and land use maps. With the extent of future development in mind, engineers and planners preparing the study can define both flood damage and risk and identify the actions needed to reduce these in the foreseeable future.

Throughout this planning process, input from citizens and City officials has been solicited to ensure compatibility of the recommended strategies with public wishes and the goals and objectives of the City of Oregon City.

Chapter 2
Physical Environment

CHAPTER 2

PHYSICAL ENVIRONMENT

2.1 GENERAL

Oregon City is located in Clackamas County, Oregon. Beyond the current city limits urban development will be contained within the City's established Urban Growth Boundary (UGB). The UGB for Oregon City consists of approximately 6,700 acres or 10.5 square miles. Figure 2.1 shows the UGB which represents the study area boundary for the storm drainage planning.

Oregon City is located at the confluence of the Clackamas and Willamette Rivers and is drained to those rivers primarily by a series of creeks and open ditches. The City is characterized by mild to relatively steep drainage courses with ground elevations ranging from a high of approximately 480 feet (National Geodetic Vertical Datum of 1929, NGVD) near the intersection of Warner Milne Road and Linn Avenue to a low of approximately 20 feet (NGVD) near the confluence of the Clackamas and Willamette Rivers.

2.2 CLIMATE

The study area's climate is moderate, primarily affected by humid maritime air masses with infrequent influxes of continental air masses from the east. Therefore, Oregon City has mild, wet winters and warm, relatively dry summers. Average minimum winter temperatures are in the mid-30's with extremes seldom dropping below 0 degrees Fahrenheit. Average maximum summer temperatures are in the low 80's with extremes seldom exceeding 100 degrees Fahrenheit.

The average annual precipitation is approximately 45 inches with 86 percent of the precipitation occurring from October to May. Snowfall constitutes only 2 percent of the annual precipitation in Oregon City. Winter snow does not accumulate in the area. However, quick snow melt can contribute to flooding problems throughout the Oregon City area.

2.3 SOILS

Soil associations in the Oregon City study area have been studied and classified by the Soil Conservation Service (Reference 1). These associations are classified according to their physical and chemical properties and indicate, in part, the problems associated with land use and drainage in individual areas. These

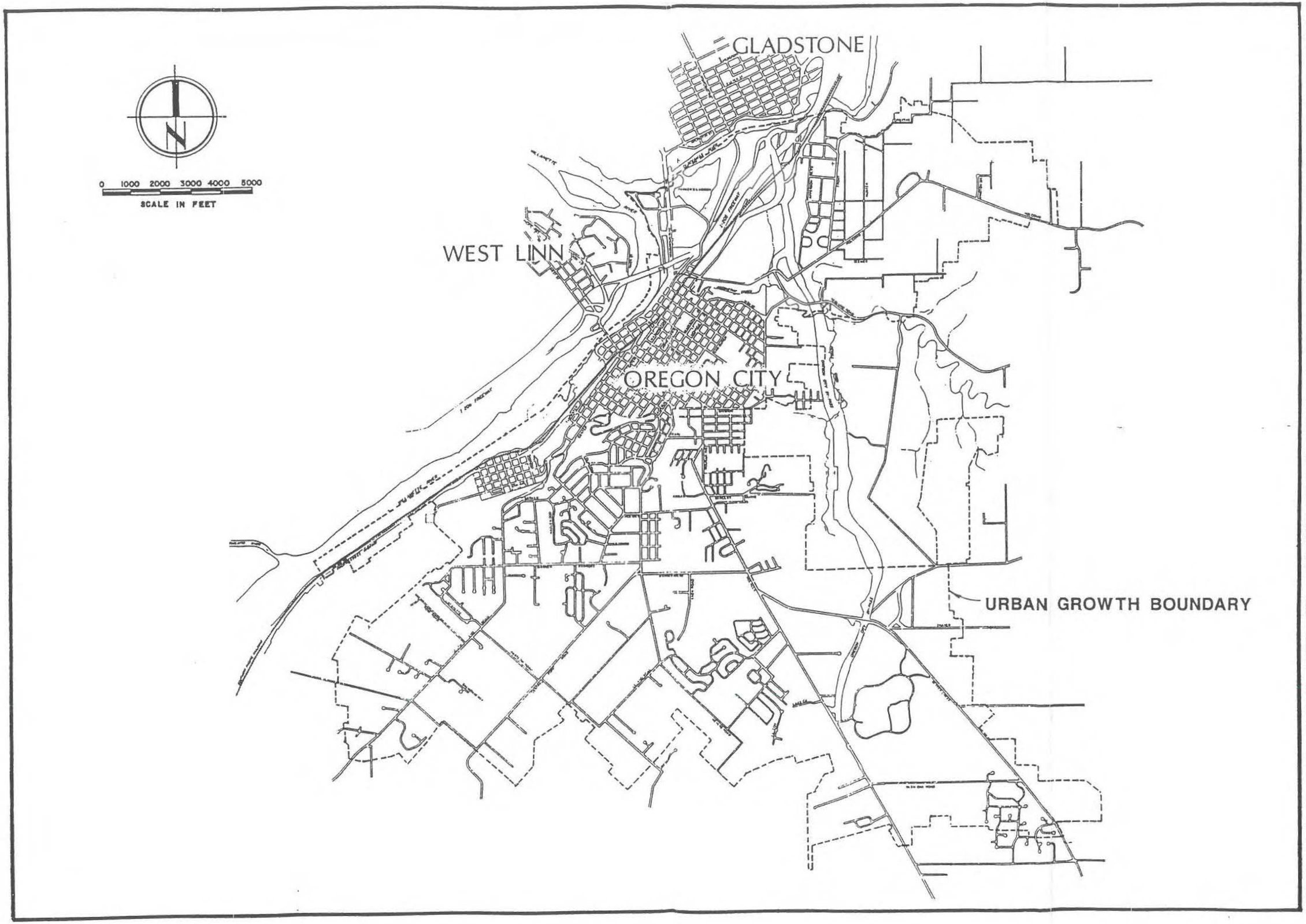

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FIGURE 2.1
OREGON CITY STUDY AREA

generalized data are also useful in predicting area-wide assessments of hydrologic response.

Soils in and around the City of Oregon City are predominately silt loams and silty clay loams on nearly level to very steep slopes. Drainage characteristics for these soils vary from good to poor. Table 2.1 summarizes the various soils found and their hydrologic grouping.

The hydrologic groupings are based on the rate of water transmission through the soil. Group A soils have a high rate of transmission and would have a low runoff potential. Thus, the drainage characteristics of A, B, C, and D soil groups are classified as excellent, good, fair, and poor, respectively.

The hydrological soils groups shown on Table 2.1 are very important. They are used to predict area wide hydrological responses. The exact hydrological techniques used in this study and their relationship to these soil groups and other important watershed characteristics will be described in Chapter 4.

2.4 LAND USE

Land use is an important factor in drainage planning because it affects the amount of rainfall that is transformed to runoff. Land use alone cannot be used to adequately predict rates of runoff and runoff volumes; however, it can be related to impervious or paved areas, which is one of the most important factors in urban hydrology.

2.4.1 Existing Conditions

Approximately two-thirds of the 6,700 acres contained within the City's Urban Growth Boundary (UGB) is already developed. The predominant land use throughout the developed area is residential. Approximately 70 percent of the residential land area is classified as low density with the remaining 30 percent classified as medium to high density.

Commercial development has occurred primarily in the downtown area and along Molalla Ave. with neighborhood commercial scattered throughout the study area. Existing industrial development is limited primarily to along the Willamette River and Interstate 205.

Approximately one-third of the study area is undeveloped with the majority of that land located outside the City's existing corporate limits to the south. Undeveloped land is located throughout the Newell Creek drainage and along the Willamette River southwest of the downtown area.

TABLE 2.1

HYDROLOGICAL GROUPINGS OF SOILS FOUND THROUGHOUT THE OREGON CITY STUDY AREA

Soil Classification	Hydrological Group	Runoff Potential
Aloha silt loam, 0 to 6% slopes	C	moderate
Amity silt loam	D	high
Borges silty clay loam, 0 to 8% slopes	D	high
*Bornstedt silt loam, 0 to 15% slope,	C	moderate
Camas gravelly sandy loam	A	very low
Chehalis silt loam	B	low
Clackamas silt loam	D	high
Cloquato silt loam	B	low
*Cottrell silty clay loam, 2 to 8% slopes	C	moderate
Delena silt loam, 2 to 12% slopes	D	high
Hard scrabble silt loam, 7 to 20% slopes	D	high
Helvetia silt loam, 8 to 30% slopes	C	moderate
*Jory silty clay loam, 2 to 30% slopes	C	moderate
Jory stony silt loam, 3 to 15% slopes		moderate
Laurelwood silt loam, 3 to 8% slopes	B	low
Nekia silty clay loam, 2 to 8% slopes	C	moderate
Newberg fine sandy loam	B	low
Gravel pits		very low
Salem silt loam, 0 to 7% slopes	B	low
Saum silt loam, 3 to 60% slopes	C	moderate
Wapato silty clay loam	D	high
Woodburn silt loam, 3 to 15% slopes	C	moderate
*Xerochrepts and Haploxerolls, 20 to 60% slopes	-	varies
Xerochrepts - Rock outcrop complex, 0 to 30% slopes	-	varies

* More predominate soil classifications density with the remaining 30 percent classified as medium to high density.

2.4.2 Ultimate Development Conditions

Oregon City has adopted an urbanization goal to "preserve and manage their scarce natural resources while building a livable urban environment". The Comprehensive Plan for the City (Reference 2) adopted in 1980 estimates that approximately 1,800 acres of land is available to accommodate anticipated development needs throughout the remainder of the century.

For this reason, drainage master planning must focus on the hydrological response of the various drainage basins under their ultimate development conditions. Since this will result in an increase in impervious land surface, an increase in storm runoff peaks, volumes, and pollutant loadings will also occur. Drainage master planning attempts to minimize these impacts though the identification of adequate drainage facilities.

The ultimate development conditions used throughout the Oregon City UGB were based on the City's existing zoning maps for areas within the current corporate limits. For those areas located outside of the existing corporate limits but within the City's established UGB, the Clackamas County Comprehensive Plan Maps were used to establish ultimate development conditions.

The relationships that were established between land use conditions and their associated imperviousness will be presented and discussed later in Chapter 4.

2.5 FLOOD PLAIN INFORMATION

2.5.1 Principal Flood Problems

The past history of flooding on the streams within Oregon City indicates that flooding occurs in the winter and spring seasons. The larger floods on the Willamette and Clackamas Rivers resulted from abnormally heavy or prolonged rainfall combined with snow melt, frozen and nearly saturated ground surfaces. Although floods on Abernethy Creek can result from rainfall on its drainage area, the largest floods along Abernety Creek result from highwater on the Willamette River.

During the flooding season of October through April, cyclonic storms from the Pacific Ocean sometimes produce intense rainfall on these three drainage basins. When these storm fronts move in a downstream direction, greater discharges are produced. If this condition is accompanied by rapid snow melt and frozen ground in the upper watersheds, large floods can occur.

There have been a number of major floods on the Willamette River (References 3, 4, and 5). The largest recent flood occurred as the result of a December, 1964 storm that dropped from 6 to 10 inches of rainfall over the watershed and caused the freezing level to rise to the 10,000-foot elevation. A peak discharge of 403,000 cubic feet per second (cfs) from this flood was observed

on December 25, 1964, at the Willamette Locks upper gage in Oregon City. This discharge exceeded that of the estimated 100-year flow of 341,000 cfs. The flood of December 1861 is believed to be the greatest historical flood, with a peak discharge at the same gaging station estimated at 590,000 cfs. The second largest historical flood, in 1890, had an estimated discharge of 510,000 cfs. Both of these floods exceeded the estimated 500-year flood flow of 469,000 cfs. The January 9, 1923, peak discharge of 357,800 cfs and the January 9, 1943 peak discharge of 324,800 cfs are the fourth and fifth largest recorded floods, respectively, for the Willamette River at the upper gage.

Major floods have also occurred on the Clackamas River. The largest recorded flood at the gage near Clackamas occurred on December 22, 1964, with a peak discharge of 120,000 cfs. Other major floods on the Clackamas River occurred in March, 1931, January, 1923, and November, 1960, with peak discharges of 82,000 cfs, 80,000 cfs and 73,000 cfs, respectively. The estimated 100-year discharge for the Clackamas River at the gaging station near Clackamas is 110,000 cfs.

Oregon City has sustained major damage from flooding of the Willamette and Clackamas Rivers. The 1861 Willamette River flood inundated the main streets of Oregon City with 4 feet of water. Although the 1890 Willamette River flood had a smaller discharge than the 1861 flood, water from the later flood rose to a level 2.1 feet above the earlier flood, due to the presence of buildings along the river which reduced the channel capacity. The December, 1964 flood also caused extensive damage to waterfront industry and shopping areas along the Willamette and Clackamas Rivers.

2.5.2 Flood Insurance Study

In 1977, the Federal Emergency Management Agency (FEMA) contracted with James M. Montgomery Engineers to conduct a Flood Insurance Study (FIS) for the City of Oregon City and the unincorporated areas of Clackamas County. As part of this study the Clackamas River, the Willamette River, and approximately 2.4 stream miles of Abernethy Creek from the Willamette River upstream to Holly Lane were studied by detailed hydraulic methods. The final version of the Oregon City FIS was published in August, 1979.

For the purposes of both insurance and the regulation of development within the floodplain, FEMA established the 100-year flood as the base, or regulatory, flood. The 100-year flood event by definition has a 1 percent chance of occurrence in any given year while over an indefinite period of time it will occur on the average of once every 100 years.

The published FIS maps show that during the 100-year flood event extensive overbank flooding will occur along the Willamette and Clackamas Rivers and the lower Abernethy Creek. This 100-year floodplain information has been reproduced on Figure 2.2 to show

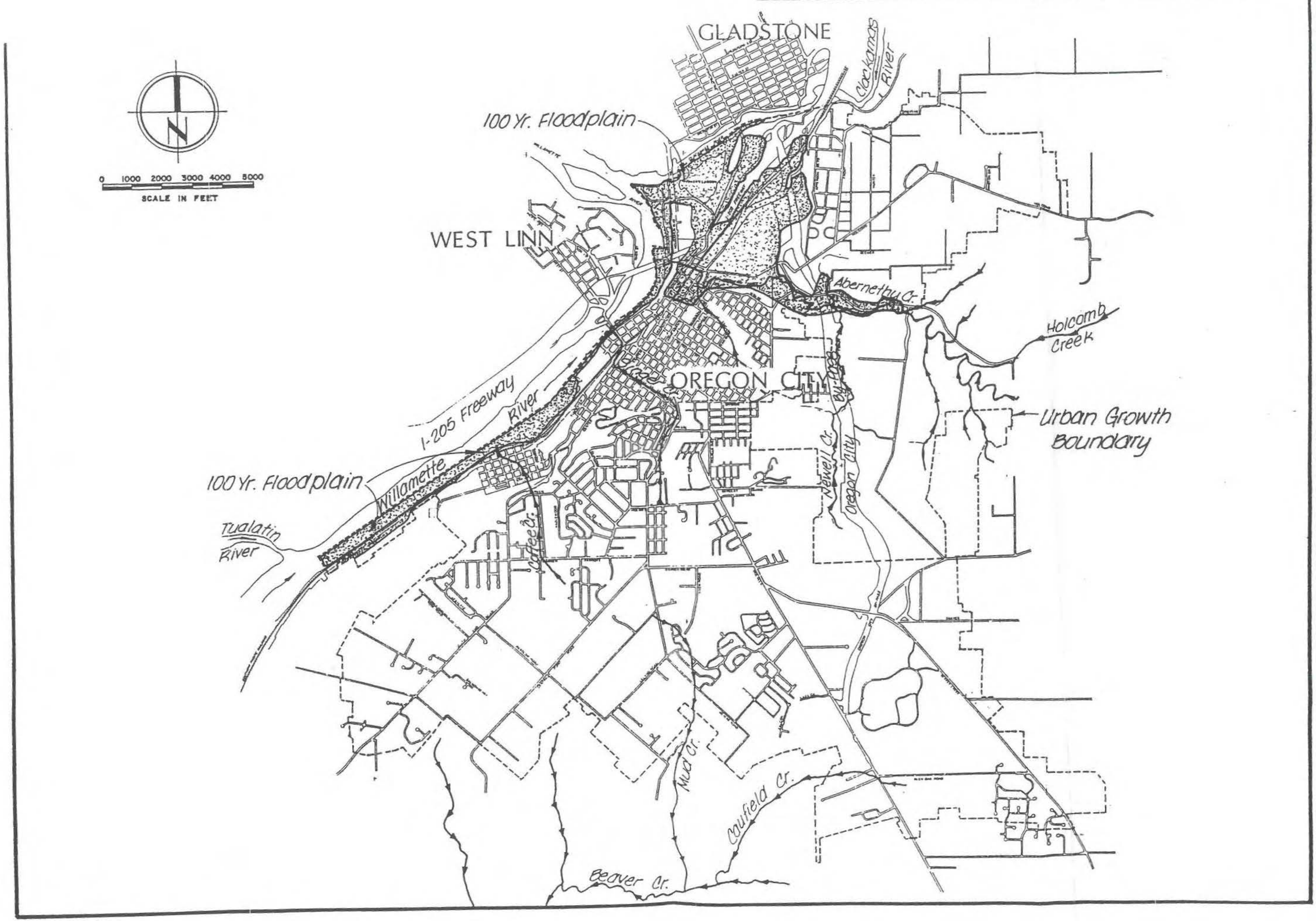

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FIGURE 2.2
100 YEAR FLOODPLAIN THROUGHOUT
THE OREGON CITY STUDY AREA

the extent of infrequent flooding from major drainage systems throughout the Oregon City study area. For detailed flood plain information, the reader should refer to the FIS maps for both Oregon City and unincorporated Clackamas County.

Chapter 3
Existing Drainage System

CHAPTER 3

EXISTING DRAINAGE SYSTEM

3.1 GENERAL

The transition of a drainage basin from rural to urban land uses can greatly alter its hydrological response to rainfall. Urban land development is usually characterized by a rapid conversion from farmland and natural vegetative cover to rooftops and pavement. This increase in impervious land surfaces can dramatically alter the quantity and quality of runoff from the land.

As urban development occurs, the amount of rainfall converted to surface runoff is increased and the amount of rainfall contributed to groundwater recharge is decreased. If urban development is accompanied by an efficient drainage system, the time needed for surface runoff to reach a stream is substantially decreased. This results in a concentration of storm water runoff that generally increases peak flow. Greater peak flows can create flooding problems, depending on the capacity of the drainage system and the downstream conditions.

The primary drainage problem in the Oregon City area is overbank flooding that occurs when the quantity of storm water exceeds channel capacity and/or the capacity of constrictions such as culverts and pipes. Overbank flooding can destroy property and endanger lives. High floodwater velocities can erode channels and banks, endangering bridge piers and approaches. Floodwaters can scour valuable topsoil from farmland and decrease normal channel capacity with mud and debris.

This chapter will describe the existing drainage system and its associated problems throughout the Oregon City area. The discussion is divided into two categories; major and minor drainage systems. The major drainage system is the drainage network that should be designed to pass runoff without damage from an infrequent, large flood such as the 10-year or 25-year flood.

3.2 MAJOR DRAINAGE SYSTEM

The exact distinction between major and minor drainage is difficult to make. It is generally based on drainage area; however, other conditions, such as the amount of impervious area, are also important. For the purposes of this study, the start of the major drainage collector system is defined as a system that drains an area of approximately 30 acres in the downtown area and about 100 acres if located throughout the remainder of the study area. This is an arbitrary distinction and should not be misconstrued as a definition of major drainage systems throughout Oregon City.

This distinction is based primarily on the contractual definition of the approximate area of the delineated subbasins that subdivide the major drainage basins and define the major drainage collector system.

The major drainage collector system is the network of open channels and/or closed pipes that connect the outlet of these subbasin areas to their respective major drainage basin outfall. The capital improvement projects presented in Chapter 6 are designed to solve drainage problems that occur along these major drainage collector systems.

3.2.1 Drainage Area Delineation

The Oregon City study area has been divided into 22 major drainage areas. Twelve of these areas have been defined as major drainage basins and have been further divided into subbasins. Hydrological computer models have been developed for these 12 basins. The capital improvement projects described in Chapter 6 address the major drainage collector systems for these 12 basins.

The remaining 10 drainage areas represent lateral drainage areas that are tributary to the Willamette River, the Clackamas River, and Abernethy Creek. These lateral drainage areas were identified and named in an effort to account for all of the 6,700 acres of land that exists within the Oregon City urban growth boundary. These lateral drainage areas are part of the minor drainage systems throughout the study areas. As a result, they will not be specifically addressed in the development of the capital improvement programs.

Figure 3.1 shows the areal extent of the 22 drainage areas delineated throughout the Oregon City urban growth boundary. The 12 major drainage basins that have been analyzed by hydrological computer models are as follows:

Symbol	Name
CA	Caufield
CLT	Clinton
CO	Coffee
CP	Central Point
JA	John Adams
L	Livesay
M	Mud
N	Newell
P	Park Place
S	Singer
SE	South End
T	Tumwater

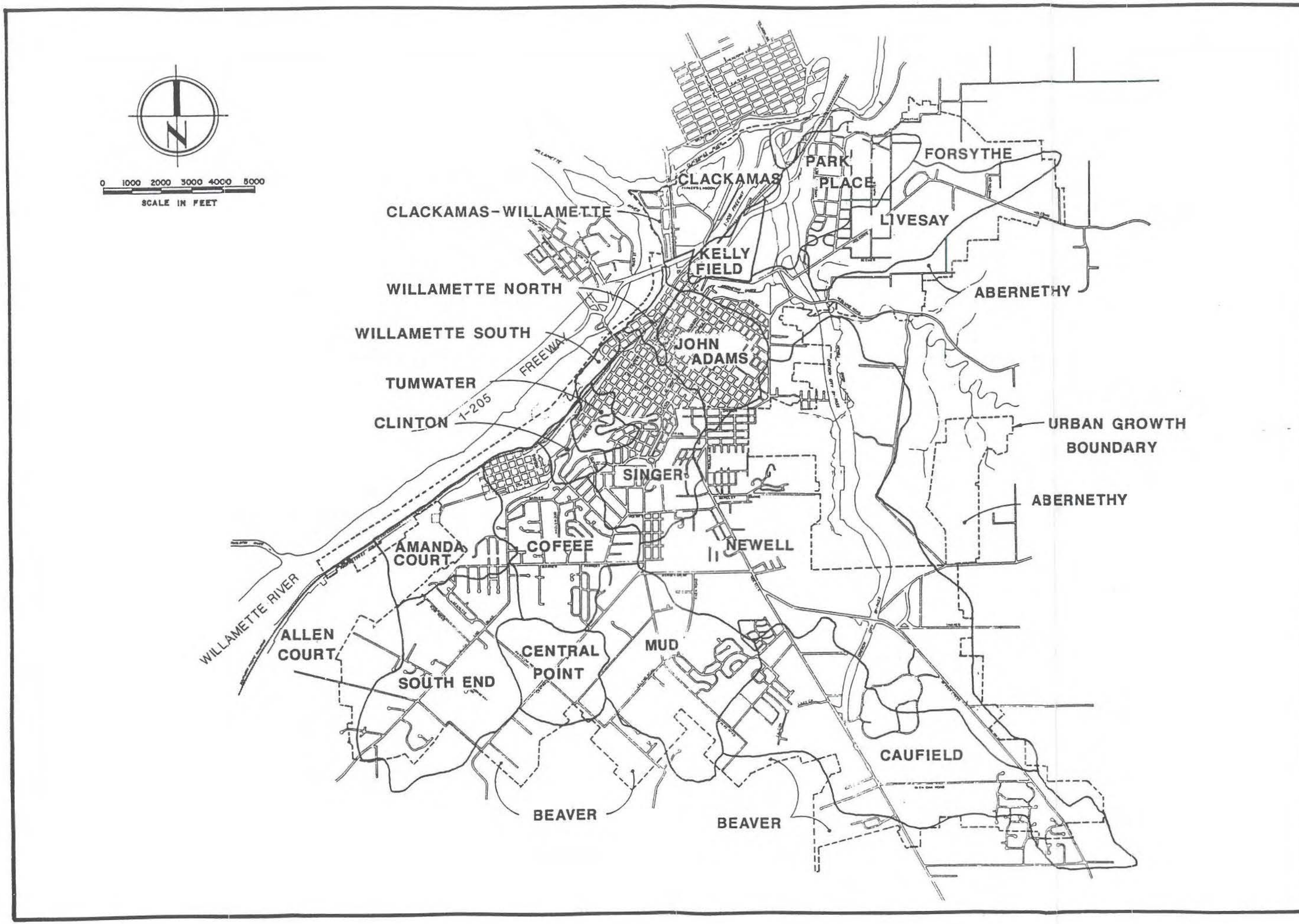

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FIGURE 3.1
DRAINAGE BASIN BOUNDARIES THROUGHOUT THE OREGON CITY STUDY AREA

The lateral drainage areas presented on Figure 3.1 are as follows:

Symbol	Name
A	Abernethy
AL	Allen Court
AM	Amanda Court
B	Beaver
CL	Clackamas
CW	Clackamas Willamette
FO	Forsythe
KF	Kelly Field
WN	Willamette North
WS	Willamette South

Each of these major drainage areas have been named and assigned a mapping or computer coding symbol. In several cases, the drainage area name corresponds to the name of the creek such as Newell, Coffee, and Singer. In some cases, the name corresponds to a street that either is a prominent feature throughout the basin or is located within the basin or near the outfall of the basin. In a few cases, the drainage area is named for the river or rivers to which it laterally drains.

It should be noted that the Newell, Livesay, Park Place, and Kelly Field drainage areas are all tributaries of Abernethy Creek. Combined, they represent approximately 13 percent of the 30 square mile area that drains to Abernethy Creek.

A hydrological model for Abernethy Creek was not developed as part of this study for several reasons. First, the portion of the Oregon City study area that drains to Abernethy Creek represents only 15 percent of its total drainage area. The effect of urbanization throughout this area will not significantly increase the existing flood flow estimates for Abernethy Creek. Second, Abernethy Creek was included within the City's Flood Insurance Study (FIS) which provides reasonable estimates of the peak discharges for the various flood return intervals. And finally, the overbank flooding along Abernethy Creek documented as part of the FIS is the result of high water on the Willamette River and not the flood flows estimated for Abernethy Creek itself.

3.2.2 Drainage Basin Descriptions

Each of the 12 major drainage basins will be presented and briefly described. Any known drainage or drainage related problems will be discussed.

3.2.2.1 Caufield

Caufield as a 936-acre drainage basin named for the creek to which it drains. Caufield Creek is a large northern tributary of Beaver Creek. Beaver Creek flows in a westerly direction and

eventually joins the Willamette River approximately four miles upstream of the Willamette Falls near the town of New Era.

The drainage basin boundary and the delineation of the Caufield basin into 15 subbasin areas is shown on Figure 3.2. These subbasins average about 62 acres in size. The subbasin delineation was based on the City's 1 inch equals 200 feet, 2 feet contour topographical maps that were originally flown in 1970. The maps in the southern portion of the study area which includes the Caufield basin were updated around 1984.

Figure 3.2 also shows the major drainage collector system throughout the Caufield basin along with the node points that define this system. Nodes that are numbered as even multiples of ten indicate that they are assumed to be the outlet or outfall of one or more contributing subbasin areas. These nodes define where the contributing subbasin areas are assumed to enter the major drainage collector system.

Note that both the subbasin and node numbers increase as one moves downstream. The numbering typically starts at the farthest upstream subbasin in the drainage basin. When a tributary system is encountered, the node point at its confluence with the mainstream is then numbered in sequence along with the contributing subbasin area at that point. The numbering sequence of both subbasins and nodes then continues at the farthest upstream subbasin for the tributary. This numbering sequence continues until the outlet or outfall of the drainage basin has been reached and all the contributing subbasin areas have been included. The subbasin and node numbers shown in each of the basin maps presented in this chapter are used in the hydrological models discussed in Chapter 4.

The location of the hydraulic structures such as culverts located along the major drainage collector system are also shown on Figure 3.2 along with their appropriate structure identification number. Each of the hydraulic structures has been given a unique number used to identify its location. The letters and numbers to the left of the decimal point refer to the subbasin in which the structure is located. The numbers to the right of the decimal point identify the node along the major drainage collector system where the structure is located. The characteristics of these hydraulic structures will be presented later in this chapter.

There are seven major hydraulic structures located throughout the Caufield basin. Six of these structures are culvert crossings of various public streets. One structure (i.e. CA50.45) is located on private property. Most of these structures have been placed at very shallow depths, which will severely reduce their hydraulic capacities.

Figure 3.2 shows an existing pond located between node points 70 and 75. The surface area of the pond is approximately 1-acre. The outflow from the pond is controlled by a dam with a low flow

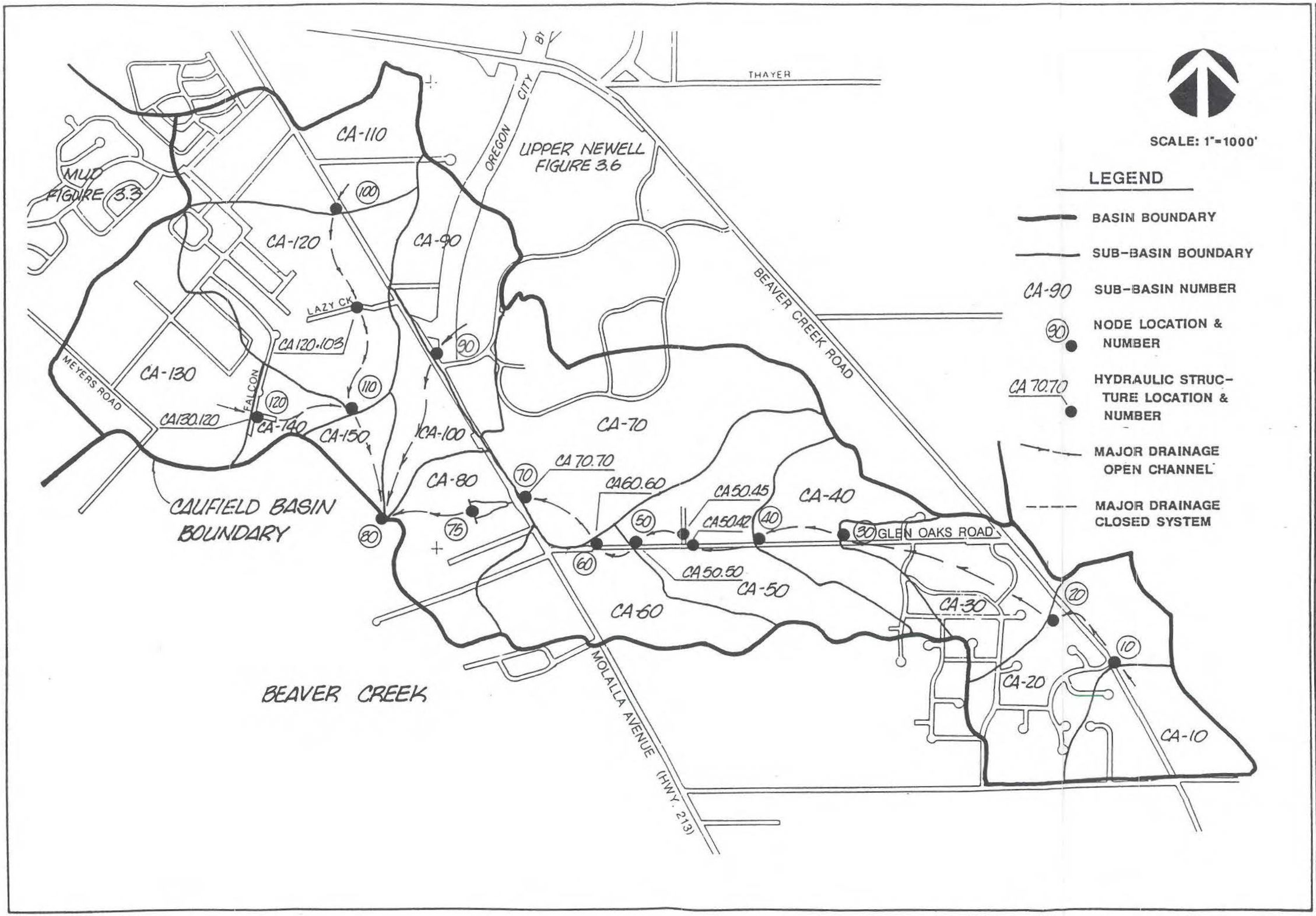

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FIGURE 3.2
CAUFIELD DRAINAGE BASIN

pipe located through the embankment and an overflow weir located along the top of the embankment. The pond may be providing some reduction in inflow flood peaks so it will be included in the hydrological model. The dam is privately owned and the safety of the structure comes under the jurisdiction of the Dam Safety Section of the Oregon Water Resources Department. Several small ponds are located downstream of CA120.103. However, these ponds are also on private property and they will not significantly affect flood peak flows. As a result, they were not included in the hydrological analysis.

The drainage conditions that generally exist throughout the Caufield basin seem to be characteristic of all the southern drainage basins within the study area (i.e. Central Point, Mud, and South End). These conditions include shallow, heavily vegetated drainage channels or ditches. As a result of these conditions, shallow and undersized culverts have generally been used to cross under nearly all of the major roadways in the basin. The existing development throughout the basin is single family residential with some commercial and institutional located along Molalla Ave. About 40 percent of the basin is undeveloped with open field, orchards and wooded land mixed throughout. Approximately one-third of the Clackamas Community College campus is included in the Caufield basin with portions located in subbasins CA70 and CA90.

3.2.2.2 Central Point and Mud

Central Point is a 230-acre drainage basin named for Central Point Road which runs through the center of the basin. Mud is a 533-acre drainage basin named for the creek to which it drains. Both of these basins are located along the southern boundary of the study area and they drain directly to Beaver Creek.

Figure 3.3 shows the major drainage basin boundaries and the subbasin delineations for both the Central Point and Mud basins. The average subbasin size in Central Point and Mud is 77 and 89 acres, respectively. The major drainage collector systems are shown along with the node points that define this system. In addition, Figure 3.3 shows the location of the major hydraulic structures throughout both basins. There are four major hydraulic structures located in the Central Point basin and six within the Mud basin. Nine of these structures are shallow culverted crossings of roadways. The remaining structure (i.e. M20.20) is a long storm sewer that passes under a subdivision located downstream of node point 20 within the Mud basin.

The existing development throughout these two basins is predominately single family residential. Approximately 40 percent of the Mud basin is currently undeveloped. The undeveloped area in Central Point represents approximately 30 percent of the basin area.

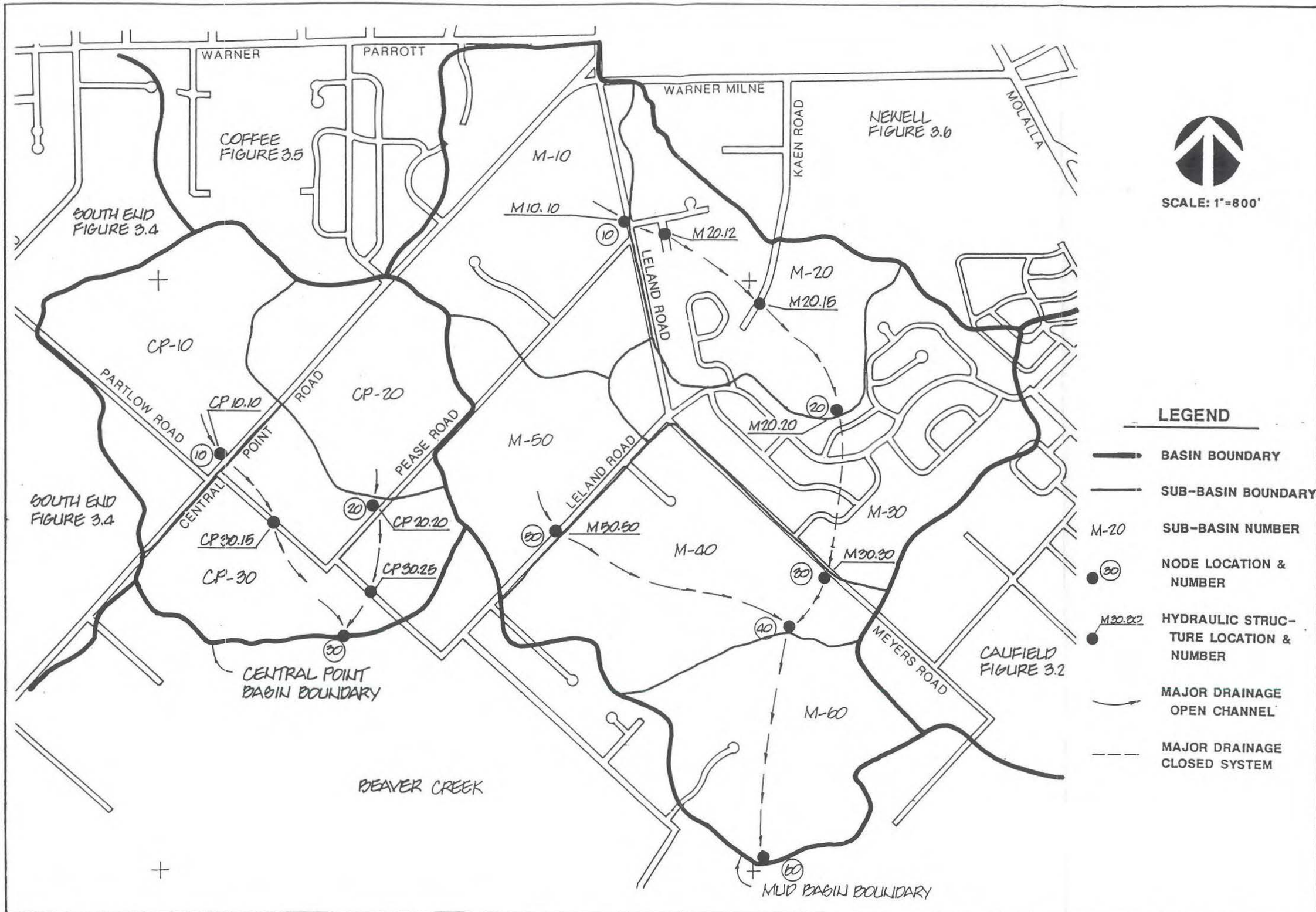

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FIGURE 3.3
CENTRAL POINT AND MUD DRAINAGE BASINS

3.2.2.3 South End

South End is a 607-acre drainage basin named for South End Road which runs through the center of the basin. South End is located in the southwest corner of the study area and is yet another tributary to Beaver Creek.

Figure 3.4 shows the major drainage basin boundary and the subbasin delineations for the South End basin. Eight subbasins have been delineated whose average size is approximately 75 acres. There are seven major hydraulic structures located throughout the South End basin as shown on Figure 3.4. All of these structures are culverts, two of which are located on private property. In addition to the shallow heavily vegetated channels and shallow undersized culverts characteristics of this and the other southern basins, the South End basin has relatively flat drainageway slopes which have resulted in excessively silted culverts.

Most of the South End basin is located outside of the City's current corporate limits. This compounds the existing drainage problems because the City is not obligated to maintain these structures and Clackamas County has not provided the needed maintenance.

Probably the most notable existing drainage problem in the South End basin involves the lack of adequate drainage for the Oaktree subdivision. The Oaktree subdivision is located in subbasin S-10 and is part of the minor drainage collector system for that subbasin.

The subdivision covering Oaktree Avenue and Lafayette Avenue has been built upon a natural water course by filling the right-of-way. This has left some lots low and subject to flooding. While most builders have tried to correct these type of conditions by importing fill, some houses were built low.

An attempt to drain such low lots is evident at 18790 and 18800 Oaktree Avenue, where a storm drain is extended to the backyards. This measure, however, produced no improvements because the storm drain in the street is rather shallow, making the drain in the low lying back yards even more shallow. In addition, the catch basins in the back yards are set up too high to drain properly. Also, the yards were left flat and uneven without a protective slope away from the houses at their perimeter. Such improper handling of drainage has resulted in soggy yards. During a prolonged rainfall, standing water is regularly observed and the two houses referenced above are often flooded.

Although the water course where the development is located has a very flat gradient, the storm drain has not been extended to the southwest, outside of the subdivision. This extension would have provided a steeper slope for the storm drain by lowering its depth.

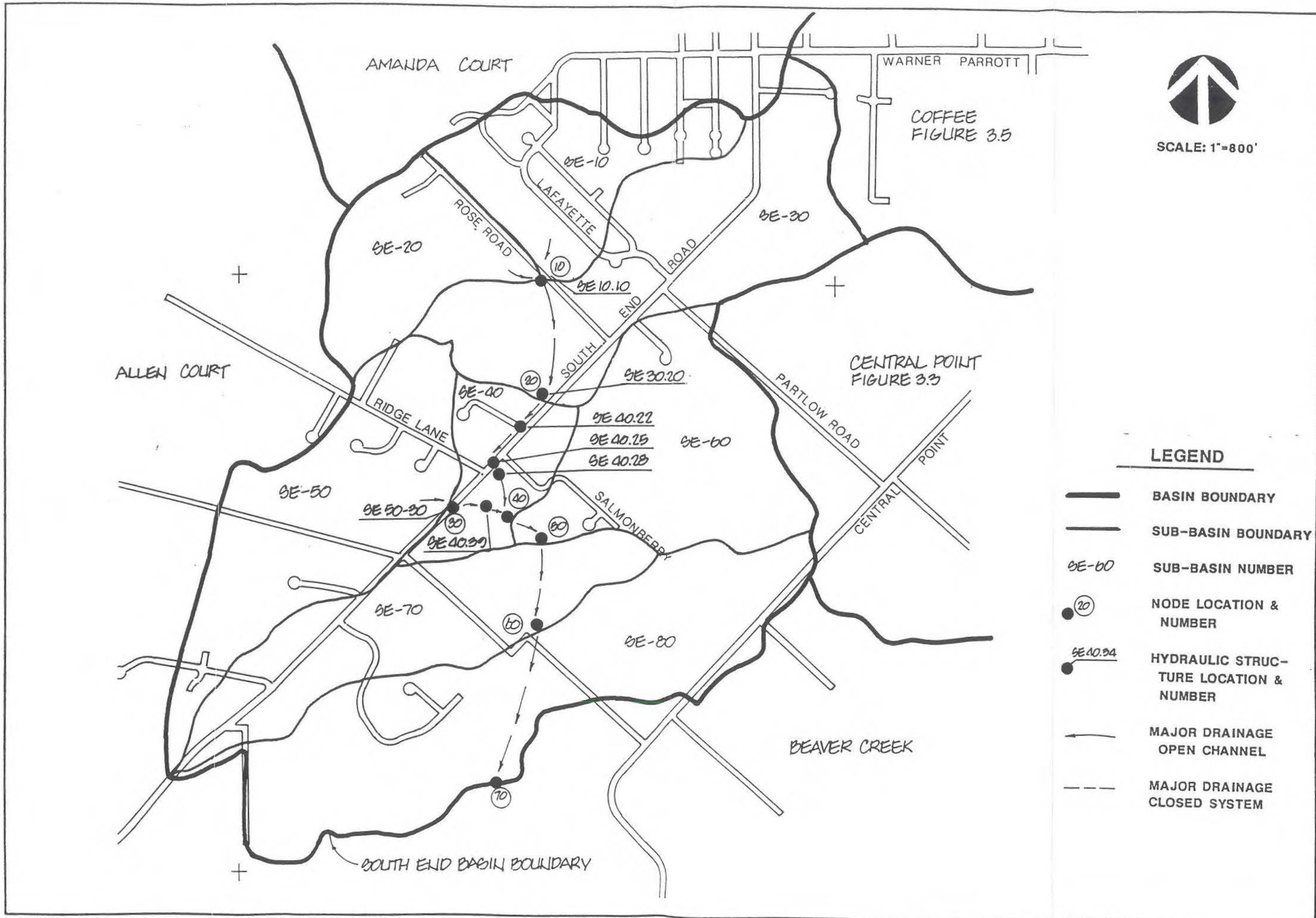

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FIGURE 3.4
SOUTH END DRAINAGE BASIN

Instead, the storm drain has been discharged at the surface at the southwest boundary of the subdivision. Between this point and Rose Avenue located to the south, there appears to be very little drop in ground elevation. The result is a swamp which during heavy and prolonged rainfall is likely to create a backwater effect upon the storm drain in Lafayette Avenue and Oaktree Avenue. This condition is made worse by the culvert in Rose Avenue (i.e. SE10.10) which is completely silted and does not function.

3.2.2.4 Coffee, Clinton, and Tumwater

Coffee is a 445-acre drainage basin named for the creek to which it drains. Clinton is a small 57-acre basin named for Clinton Street located on its northeastern boundary. Tumwater is a small 89-acre basin named for Tumwater Drive located near its outfall. All three of these basins are characterized by steep channels that drain directly to the Willamette River.

Figure 3.5 shows the major drainage basin boundaries and the subbasin delineations for the Coffee, Clinton, and Tumwater basins. The average subbasin is 111, 29, and 30 acres, respectively. The major drainage collector systems are shown along with the various hydraulic structures located along these systems. Ten major hydraulic structures are located in the Coffee basin and one is located in the Clinton basin.

Drainage from the Tumwater basin is currently being collected by a combined sewer system (i.e. sewer system for both sanitary sewage and storm water) that runs underneath Tumwater Drive. The combined sewer separation program currently underway within the City calls for a separate storm sewer system that will drain via a more direct route to the Willamette River. The Tumwater basin has been drawn to reflect that proposed improvement. However, Figure 3.5 also shows the general alignment of the existing combined sewer system that now serves the Tumwater basin.

The Coffee basin is distinctly divided into the plateau portion and the old residential area along the Willamette River. The plateau portion has been heavily developed and the minor collector systems throughout this area have been storm sewered. The major drainage course from node point 10 to 30 follows the back property lines of the subdivision lots east of the Hazelwood Drive. Along these lots the channel has been substantially obstructed by fences.

At the division between the plateau portion of the basin and its lower portion (i.e. upstream of node 30), the main course falls down the cliff to South End Road where it is directed through a rock channel to a culvert under the street (i.e. C030.30). Although the culvert seems to function adequately, it appears that some water at the waterfall may remain unintercepted during a minor flood event and seep through the road embankment under the South End Road. This is certainly a serious concern given the possibility of a potential slide.

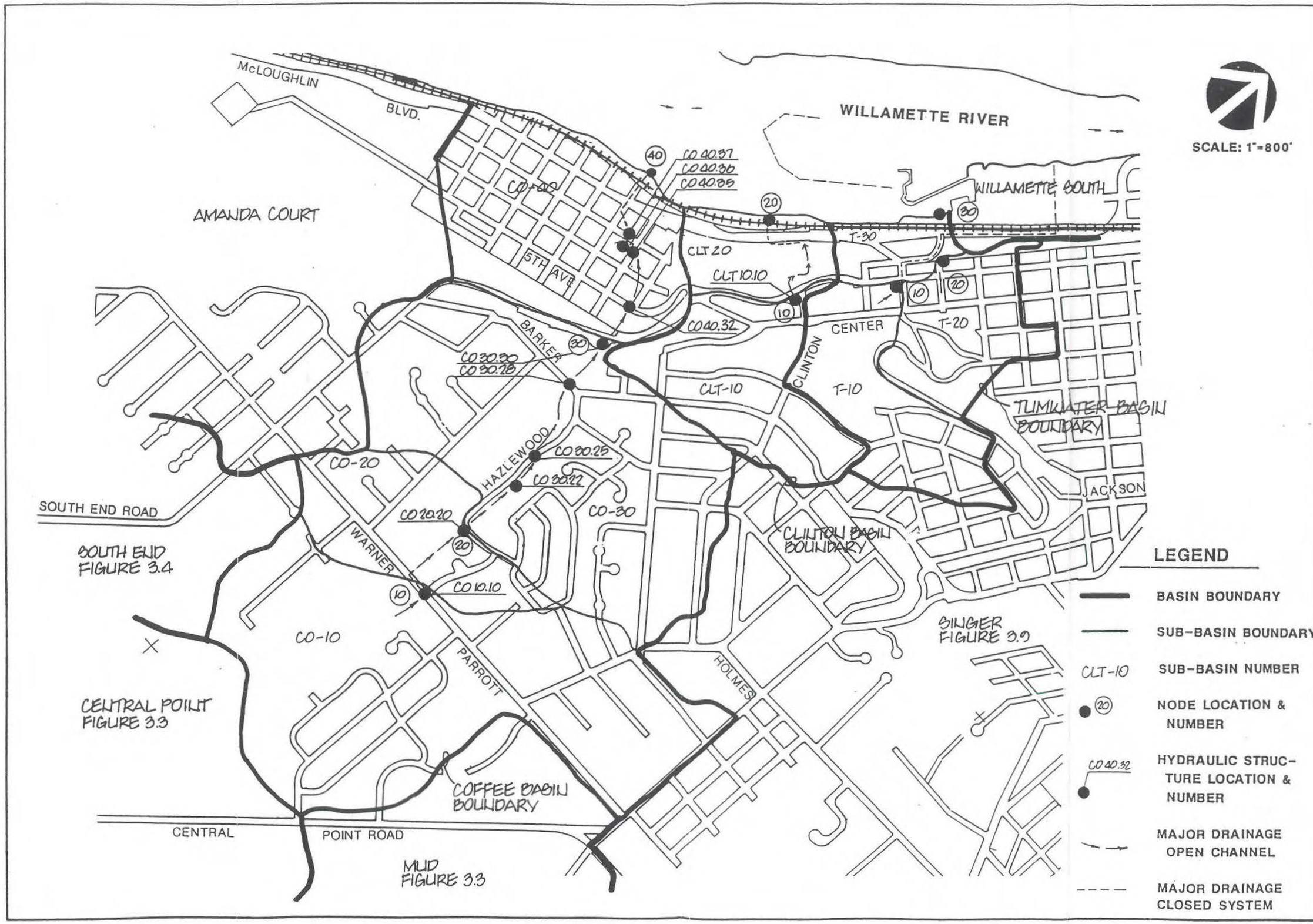

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FIGURE 3.5

COFFEE, CLINTON AND TUMWATER DRAINAGE BASINS

The lower portion of the course below South End Road while old in appearance seems to function well. At the corner of Ganong Street and 3rd Avenue the drainage course is located in an irregular rock channel that passes under a historical building (i.e. CA40.36) that is over 100 years old.

Clinton is an extremely steep basin whose drainage is primarily collected by shallow roadside ditches that intercept the various overland flow areas. The major drainage course crosses South End Road at Clinton Street (i.e. CLT10.10) and drops over the cliff. The channel then finds its way through the PGE substation and eventually crosses under McLoughlin Boulevard and the Southern Pacific Railroad tracks to the Willamette River. No drainage problems appear to exist within this steep well drained basin.

3.2.2.5 Newell

The 1790-acre Newell basin is the largest major drainage basin studied as part of the planning effort. It is named for the creek to which it drains. Newell Creek is a major tributary of Abernethy Creek. Newell Creek joins Abernethy approximately 300-feet upstream of Abernethy's crossing of the newly constructed Oregon City Bypass.

Figures 3.6 and 3.7 show the major drainage basin boundary and the subbasin delineations for the Newell basin. Figure 3.6 shows the upper Newell basin and Figure 3.7 shows the lower Newell basin. The average subbasin size in the Newell basin is 119 acres. Figure 3.6 also shows the location of the seven major hydraulic structures that are found within the Oregon City study area included in the Newell basin. Unlike the conditions found in the southern drainages, most of the major hydraulic structures in the Newell basin appear to have sufficient capacity due to the adequate headwater depths that generally exist throughout.

One notable problem area exists along the major drainage channel that connects node point 60 to 70. This channel has a series of culverted crossings located on private property that severely reduce its flow capacity around the Barclay Apartments. Power poles are also located in the center of the channel. Downstream of the apartments, the channel is wider and shallow and becomes choked with vegetation which further reduces its capacity.

Most of the Newell Creek channel is located outside of the Oregon City study area as it runs through a deep wooded canyon that will remain in its natural state. The existing development within the Newell basin occurs in the upland areas located in the southwest portion of the basin. The land use mix in these areas includes residential, commercial, and institutional. Approximately two-thirds of the Clackamas Community College campus is included in the Newell basin within subbasin N-20. Downstream of point 20 is an area of the campus called John Inskeep, which is an environmental learning center that supplements the flow of Newell Creek to breed fish.

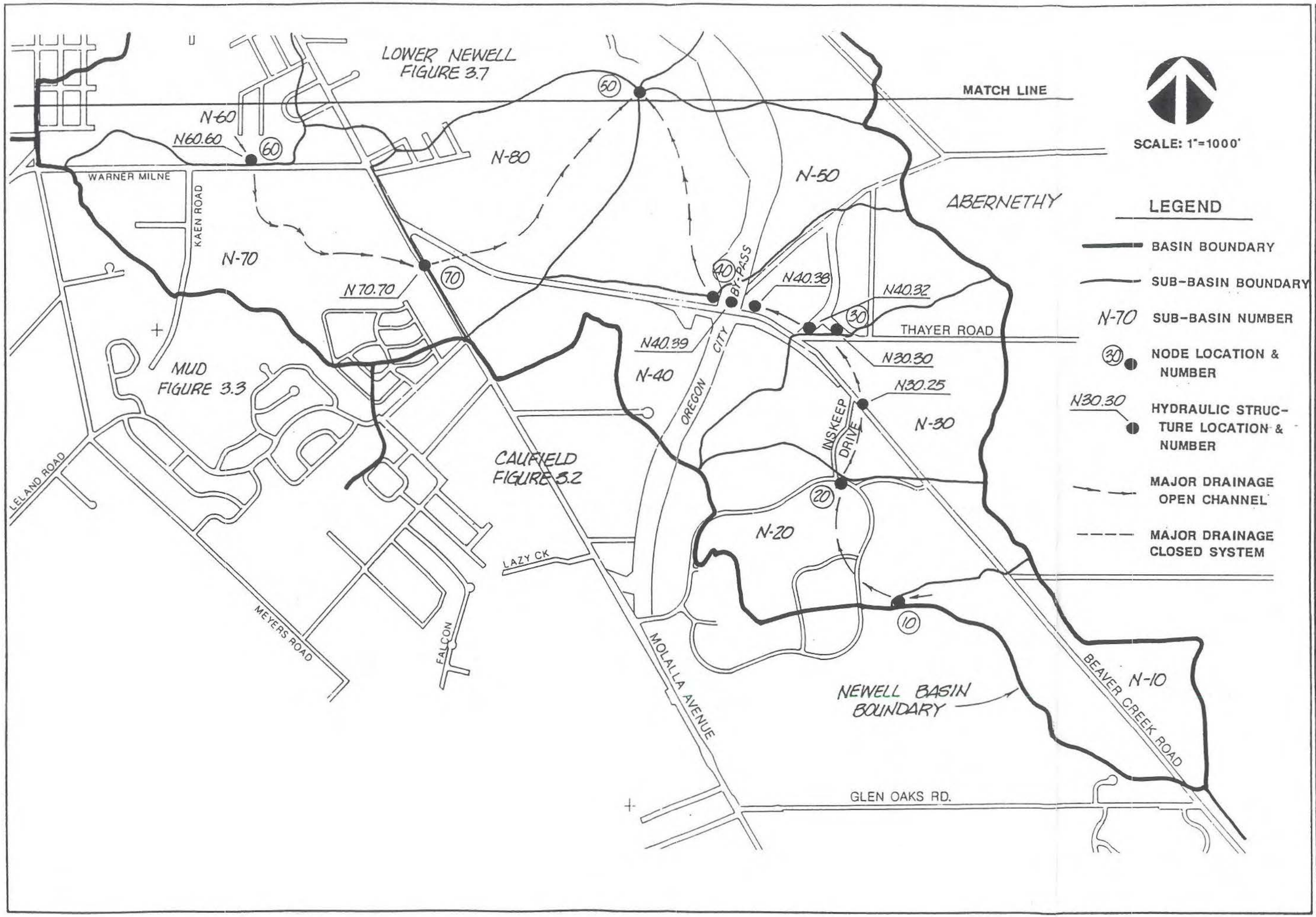

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FIGURE 3.6
UPPER NEWELL DRAINAGE BASIN

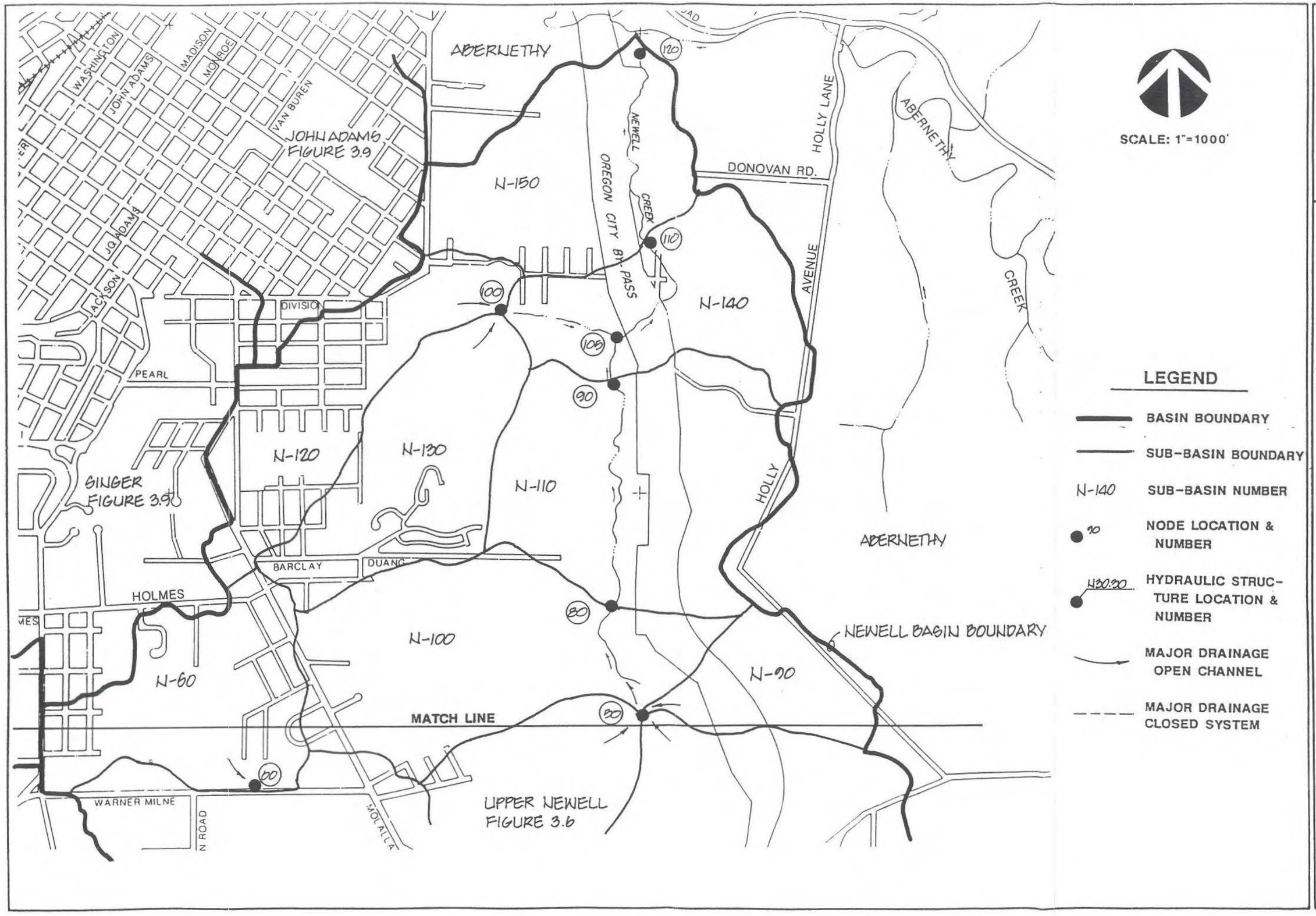

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FIGURE 3.7
LOWER NEWELL DRAINAGE BASIN

3.2.2.6 Park Place and Livesay

Park Place is a 323-acre drainage basin named for the neighborhood which occupies most of its area. Livesay is a 359-acre drainage basin named for Livesay Road located near its outlet. These two basins encompass the most northerly portion of the Oregon City study area. Both of these basins are currently outside of the City's corporate limits. Both of these basins drain directly to Abernethy Creek.

Figure 3.8 shows the major drainage basin boundaries and the subbasin delineations for both the Park Place and Livesay basins. The average subbasin size in the Park Place and Livesay basins is 53 and 90 acres, respectively. The major drainage collector systems are shown in Figure 3.8 along with the various hydraulic structures located along these systems. Eight major hydraulic structures are located in the Park Place basin and one is located in Livesay.

Both basins are characterized by adequate relief and natural water courses all of which appear to function well without signs of erosion. This may be partially explained by the relatively undeveloped nature of the area. The drainage in the area is generally overland flow being intercepted by roadside ditches with very few storm sewers.

3.2.2.7 John Adams and Singer

John Adams is a 234-acre drainage basin named for the street that defines its southern boundary. Singer is a 396-acre drainage basin named for the creek to which it now drains. Portions of the existing development in both Singer and John Adams are served by combined sewer systems. As a result, some of the John Adams basin drains directly to the Willamette River and some to Abernethy Creek. All of Singer basin drains to the Willamette River.

Figure 3.9 shows the major drainage basin boundaries and the subbasin delineations for the John Adams and the Singer basins. The average size of the subbasins for both drainage basins is 33 acres. The average subbasin size for the other ten drainage basin presented earlier is 80 acres. The smaller subbasin size found in the John Adams and Singer basins reflects the greater detail that was contractually specified for the analysis of the old town combined sewer areas. Because of the complex nature of the existing drainage systems throughout both of these basins they will be discussed separately.

John Adams

The most pronounced natural drainage course in the John Adams basin has been named High School Creek because it drains through the Oregon City High School athletic field. High School Creek begins near the intersection of 9th and Division Streets. The

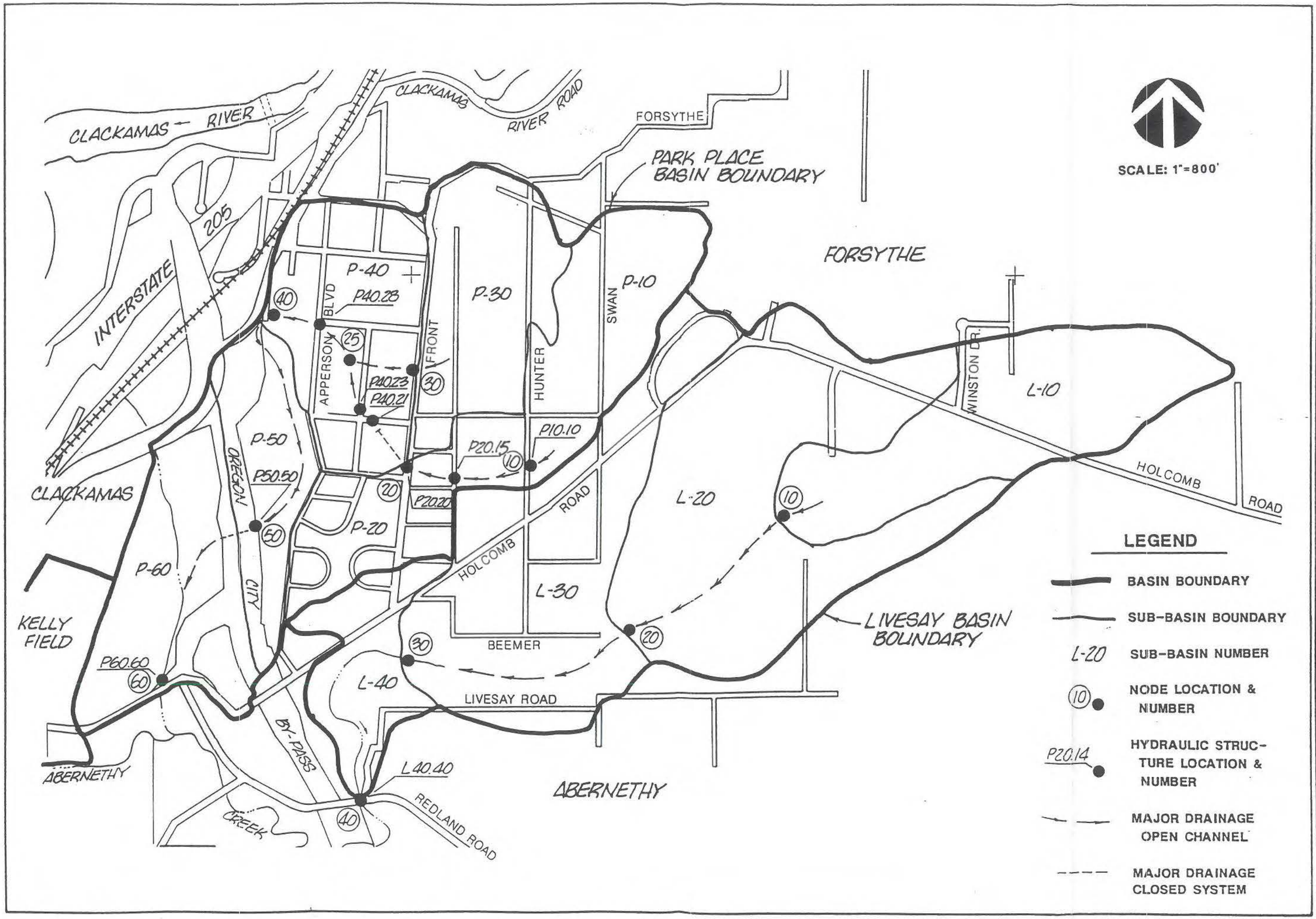

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FIGURE 3.8
PARK PLACE AND LIVESAY DRAINAGE BASINS

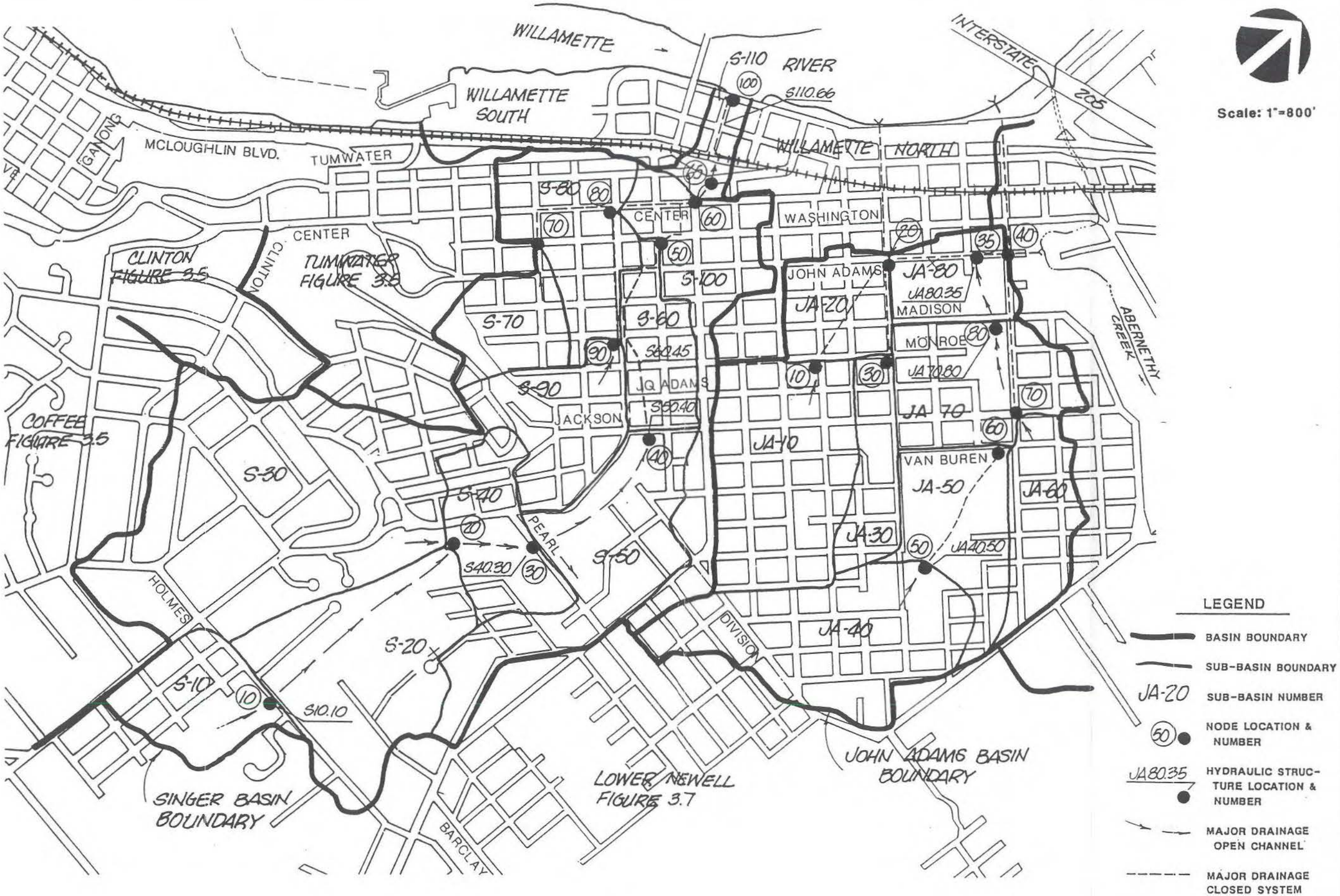

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FIGURE 3.9

JOHN ADAMS AND SINGER DRAINAGE BASINS

creek enters a long storm sewer (i.e. JA40.50) at node point 50 which passes its flow through the high school and under both Van Buren and Jackson Streets. The inlet to this long pipe system is grated to protect against debris entering the pipe. However, both the grated inlet and a chain link fence crossing the creek upstream of the inlet are substantially obstructed by trash, debris, and silt, which will significantly reduce the ability of higher flows to enter the pipe.

After exiting this long pipe, High School Creek travels approximately 600 feet through a deep wooded ravine where it enters a culvert under Madison Street (i.e. JA70.80). The inlet to this culvert has a broken grate where at least two-thirds of its inlet capacity has been clogged with debris and silt. During a heavy storm complete plugging of the culvert entrance appears to be possible. In such a case, the runoff would become detained upstream of Madison Street leaving the plugged inlet under up to 50 feet of water. Since the embankment of Madison Street is probably not designed to serve as a dam, it could collapse sending a wall of water and mud downstream to the downtown area. The damage and loss in a situation such as this would be catastrophic.

After exiting this culvert, the creek travels approximately 400 feet downstream through a deep wooded ravine where it enters yet another conduit. This long pipe (i.e. JA80.35) apparently makes a 90 degree turn to the north where it follows John Adams Street and eventually outfalls into Abernethy Creek.

As noted earlier, the John Adams basin is now served by a combined sewer system. The separation of the sanitary sewage flows and the storm runoff is now underway within the basin. Once the separation is completed, the existing combined sewer system will serve as a storm sewer system only. The existence of the old combined sewer system only adds confusion to the complete understanding of how the storm water runoff in this area drains. The major drainage boundaries and subbasin boundaries for the John Adams basin shown on Figure 3.9 are actually based on how we feel this portion of the existing combined sewer system should drain once the separation is completed and the capital improvements specified in this plan have been completed. Routing the entire John Adams basin to Abernethy Creek seems to be the most sensible way of solving the severe capacity and age problems that exist in the combined sewer lines located downstream of John Adams Street. A brief description of the existing combined sewer system will now follow.

All of the drainage from the residential development located in subbasin areas JA40, JA50 (i.e. north of High School Creek), JA60, and a small portion of JA70 (i.e. north of 15th Street) drain to the Willamette River via a combined sewer line located in 15th Street. Although this line appears to have adequate capacity at this time, the sewer is very old and has outdated small diameter brick manholes which do not allow for proper maintenance.

All of the drainage from the development located in subbasin areas JA10, JA20 (i.e. south of 11th Street), JA30 and JA70 (i.e. south of High School Creek) currently drain to the Willamette River via a combined sewer line located in 12th Street. The 12th Street sewer has a very flat gradient and it does not have adequate capacity. During heavy storms, the line develops excessive pressure (due to the flows from the John Adams basin) which causes flooding from several manholes located along 12th Street.

A small portion of the drainage for subbasin areas JA20 and JA80 drains to Abernethy Creek via a combined sewer line located in John Adams Street that apparently joins the High School Creek culvert (i.e. JA80.35) flow just downstream of node point 35.

Singer

The most pronounced natural drainage course in the Singer basin is Singer Creek. Singer Creek originates in the swampy grassed field upstream of the Holmes Lane culvert (i.e. S10.10). This culvert does not appear to function well because its outlet is obstructed by a poorly defined channel downstream. Approximately 300 feet downstream of Holmes Lane, Singer Creek begins its fall into a deep wooded ravine. The creek drops approximately 170 feet in the next 1900 feet where it enters the Pearl Street culvert (i.e. S40.30). This culvert is a recently constructed corrugated metal arch pipe with an invert elevation approximately 20 feet below the Pearl Street pavement. Presently the inlet of the culvert does not have any protective grating and may be easily plugged by the debris originating in the upstream wooded ravine.

Approximately 1,500 feet downstream of Pearl Street, Singer Creek enters the Jackson Street culvert (i.e. S50.40) which is also located some 20 feet below the top of the Jackson Street embankment. The culvert is old and has been constructed with rock. Most of the culvert opening has been obstructed due to the accumulation of trash, debris, and silt. The grating at the inlet to this culvert is in very poor condition and it is sloping in the wrong direction which worsens the existing debris problem. The complete plugging of this culvert during a heavy storm could result in severe flood damages both upstream and downstream of Jackson Street.

Approximately 150 feet downstream of the Jackson Street culvert outlet, Singer Creek enters what has been historically referred to as the Singer Creek Culvert (i.e. S60.45) which is primarily an old rock arch culvert that extends some 2500 feet downstream to the waterfall near the McLoughlin House. The inlet to this rock culvert is actually a corrugated metal pipe located 100 feet east of J.Q. Adams Street. The inlet is located in a depression within the backyard of a private dwelling. It has an old, inefficient grating made of pipes instead of thinner and less obstructive re-bar.

The access to the inlet is poor and difficult to maintain. Approximately one-third of the inlet capacity has been obstructed by silt. Traces of repeated ponding at the inlet are evident. High water elevations at the inlet endangers one dwelling where the basement floor is only 2 to 3 feet above the top of the inlet pipe. In addition to this, at least two buildings have their footings too close to the creek and they may eventually be undermined.

The old Singer Creek culvert dates back to the beginning of the 20th century. It apparently has been extended gradually during many years using varying standards and materials. Contrary to contemporary practices, the Singer Creek culvert does not have manholes at all the points where it changes grade or direction. Consequently, the exact location of the culvert often is not known. During the years, a number of houses have been constructed over the culvert, this has probably covered many access manholes whose exact location is now lost.

Nevertheless, the Singer Creek culvert is known to function adequately in spite of its age and the large area that it drains. There are several reasons why the existing culvert seems to function well. First, the runoff flows entering the culvert from the upper Singer basin travels in natural ditches, swales and the stream channel itself, which provides the maximum opportunity for infiltration into the ground. Second, there appears to be considerable amounts of detention storage occurring upstream of Holmes Lane, Pearl Street, Jackson Street, J.Q. Adams Street, and along its one upper tributary. This detention storage is reducing the peak flows that actually enter the culvert. And finally, most of the developed land located in subbasin areas S60, S70, S80, and S100 are now being served by combined sewer systems which do not substantially contribute any flow to the existing Singer Creek culvert. With the separation of the sanitary flows from these combined systems the additional storm water will be added to the existing Singer Creek culvert flow.

As mentioned earlier, the Singer Creek culvert outfalls at the top of the Singer Creek waterfall located near the intersection of Singer Hill Road and 7th Street. The waterfall itself has been carefully engineered and it does not seem to present any problems. Immediately downstream of the waterfall, Singer Creek enters yet another old rock culvert (i.e. S110.66) that carries its flow approximately 650 feet downstream to outfall into the Willamette River. Once again, this old rock culvert appears to function adequately under existing conditions. However, there are some concerns regarding its performance once the separation of the sanitary system is completed and the additional flow is added to its existing burden. The inlet to this lower culvert appears to have more than adequate capacity. However the lower 580 feet of this rock culvert is set at a gradient of approximately 1 percent and it may not have adequate capacity once the storm flow is returned to the Singer Creek system.

3.2.3 Major Hydraulic Structure Inventory

During the winter of 1986-1987, OTAK conducted an inventory of the major hydraulic structures found throughout the Oregon City study area. The objective of the inventory was to obtain data on all the accessible hydraulic structures located on the major drainageways. Major drainageways are the streams, ditches, and closed systems that connect the outlet of the subbasin areas to their respective major drainage basin outfall.

Table 3.1 presents the characteristics of the 59 major hydraulic structures that were inventoried. The approximate location of each of these hydraulic structures has been shown on the various drainage basin maps (i.e. Figures 3.2 through 3.9) presented earlier. The allowable headwater depths shown in Table 3.1 were generally based on estimates of threshold elevations for flow over the roadway the structure serves or for entry elevations of affected structures, whichever was less. Field survey elevations were not obtained. However, all estimates were checked against the available topographic maps and the various field photographs that were taken.

Structure lengths were either field measured or taken off the various topographical maps. The existing flow capacity was computed by hand calculations with inlet control assumed. Inlet control basically means that the capacity of the culvert or pipe is based on the size of the inlet, the allowable headwater at the inlet, and the characteristics of the inlet structure and pipe itself. Tailwater depth was not taken into consideration because almost all of the culverts or closed pipe systems shown in the table have adequate slope.

It should be emphasized that the existing flow capacities shown in the table were based on completely open conditions with no obstruction of the available pipe area or inlet capacity by trash, debris, or silt. Many of the culverts and closed pipe systems shown on Table 3.1 are either silted or their inlets are obstructed by silt or debris. In these cases, the actual capacity of the existing structure in the event of a high flow condition would only be a fraction of the flow capacity shown to exist in the table. The importance of maintaining the available capacity of these hydraulic structures will be discussed later in Chapter 6.

It should be noted that not all of the hydraulic structures that are now known to exist along the major drainageways presented on the drainage basin maps have been included in Table 3.1. Most of those omitted from the table were located on private property where access was not available. A few of the structures crossing public rights-of-way were assumed to exist based on the available topographical maps, however, they could not be located in the field for a variety of reasons. Also, none of the existing combined sewers that would qualify as major structures have been included in the table.

TABLE 3.1 MAJOR HYDRAULIC STRUCTURES INVENTORY

STRUCTURE ID	LOCATION	SIZE (IN)	MAT'L	STRUCT. TYPE	APPROX. LENGTH (FT)	INLET TYPE	AREA (ACRES)	HEAD-WATER (FT)	EXISTING CAPACITY (CFS)	(CFS/MI^2)	REMARKS
CA50.42	GLEN OAK 1600' E OF HWY 213	48	CMP	CULVERT	34	3	251	4.5	80	203.98	90DEG FLOW TURN @ENTRY
CA50.45	PRIV. DRVWY 150' N OF GLEN OAK	30X30	CONC	CULVERT	15	1	272	3.7	45	105.88	ON PRIVATE PROPERTY
CA50.50	GLEN OAK 1000'E OF HWY 213	24	CMP	CULVERT	34	3	314	4.0	21	42.80	90DEG FLOW TURN @ENTRY
CA60.60	GLEN OAK 500' E OF HWY 213	30	CONC	CULVERT	30	3	378	5.5	55	93.12	CULVERT TOO SHALLOW
CA70.70	HWY 213 700'N OF GLEN OAK	72	CMP	CULVERT	120	3	499	12.0	350	448.90	NEW CULVERT
CA120.103	LAZY CREEK LANE	24	CONC	CULVERT	10	3	137	3.0	22	102.77	CULVERT TOO SHALLOW
CA130.120	FALCON DRIVE	18	CMP	CULVERT	75	3	96	2.0	8	53.33	CULVERT TOO SHALLOW
M10.10	LELAND 1300'OF WARNER-MILNE	24	CONC	CULVERT	170	3	115	3.0	21	116.87	CULVERT TOO SHALLOW
M20.12	PRIV. DRVWAY 250' E OF LELAND	24	CONC	CULVERT	30	3	120	3.0	18	96.00	ON PRIVATE PROPERTY
M20.15	KAEN STREET	24	CMP	CULVERT	75	3	158	4.5	24	97.22	
M20.20	150'N OF WASSAIL	36	CONC	CLOSED	1120	1	192	5.5	64	213.33	INLET WITH GRATING
M30.30	MEYERS 1400'NW OF GAFFNEY	42	CONC	CULVERT	30	3	314	5.3	90	183.44	CULVERT TOO SHALLOW
M50.50	LELAND 800' S OF S HAVEN	12	CMP	CULVERT	80	3	58	2.0	4	44.14	SHALLOW SWAMPS UPST
CP10.10	CENTRAL 200'NE OF PARTLOW	30	CONC	CULVERT	45	3	77	5.0	50	415.58	CULVERT TOO SHALLOW
CP20.20	S PEASE 550' NE OF MCCORD	12	CMP	CULVERT	30	3	64	1.8	4	40.00	CULVERT TOO SHALLOW
CP30.15	PARTLOW 650' SE OF S.CENTRAL	15	CONC	CULVERT	30	3	102	1.75	6	37.65	CULVERT TOO SHALLOW
CP30.25	MCCORD 400' S OF S.PEASE	24	CMP	CULVERT	30	3	105	3.5	26	158.48	CULVERT TOO SHALLOW
SE10.10	ROSE ROAD	18	CMP	CULVERT	28	3	50	1.5	9	115.20	TOTALLY SILTED
SE30.20	250'N OF S END CT 50'W OF S END	24	CONC	CULVERT	134	3	109	3.0	22	129.17	INLET ON PRIV. PROP
SE40.22	S. END COURT @S. END	42	CONC	CULVERT	72	3	96	5.5	92	613.33	TWO THIRDS SILTED
SE40.25	S.END 100'W OF SALMONBERRY	48	ROCK	CULVERT	40	1	109	5.3	107	628.26	ENTRY IS A BOX
SE40.28	DRVWY S OF SALMONBERRY @S END	48	CMP	CULVERT	23	3	109	4.0	68	399.27	ON PRIVATE PROPERTY
SE40.39	300' DOWNSTREAM OF SE 50.30	18	CMP	CULVERT	45	3	95	2.0	8	53.89	ON PRIVATE PROPERTY
SE50.30	S.END 500'SW OF FOREST RIDGE	21	CONC	CULVERT	40	3	90	2.6	16	113.78	ONE HALF SILTED
N30.25	SW BEAVER CR RD @ JOHN INSKEEP	30	CONC	CULVERT	96	3	246	4.0	41	106.67	HAS JUNC. SECTION
N30.30	UNDER LOOP TO RESIDENTIAL	48	CMP	CULVERT	50	3	294	4.0	67	145.85	NO COVER
N40.32	RELOCATED MAPLE LANE	48	CMP	CULVERT	60	3	294	10.0	145	315.65	GOOD HEADWATER
N40.38	ROAD TO MAPLE LANE CHURCH	48	CMP	CULVERT	140	3	355	7.0	115	207.32	ON PRIVATE PROPERTY
N40.39	BYPASS @BEAVER CREEK	48	CMP	CULVERT	200	3	360	20.0	180	320.00	GOOD HEADWATER
N60.60	CROSSING WARNER-MILNE	36X22	CMP	CULVERT	50	3	109	4.8	37	217.25	CULVERT TOO SHALLOW
N70.70	HWY 213 400'S OF BEAVER CK	2-36	CONC	CLOSED	550	1	294	5.5	160	348.30	GOOD CONDITION
CO10.10	WARNER-PARROTT 800'E OF WDLN	2-24	CONC	CULVERT	175	1	110	4.0	50	290.91	GOOD CONDITION
CO20.20	HAZELWOOD 500'W OF HARTKE	42	CONC	CULVERT	80	3	211	6.5	110	333.65	SILTED TO 24" FROM TOP
CO30.22	BACK OF LOT 33 ON HAZELWOOD DR	24	CMP	CULVERT	100	1	264	2.5	17	41.21	POOR ACCESS.PRIVATE
CO30.25	HAZELWOOD 500' S OF VINE	42	CMP	CULVERT	50	3	317	6.5	90	181.70	ONE-THIRD SILTED
CO30.28	BARKER AVE @ HAZLEWOOD	2-30	CONC	CULVERT	54	3	369	6.5	120	208.13	GOOD CONDITION
CO30.30	S.END @ WATERFALL	36	CONC	CULVERT	80	1	378	8.0	85	143.92	GOOD-APPROACH VELOCITY
CO40.32	FIFTH ST.	36	CONC	CULVERT	39	3	399	4.5	60	96.24	
CO40.35	GANONG @ 3RD AVE.	42	CONC	CULVERT	60	3	413	7.0	113	175.11	
CO40.36	DS OF CO40.35,UNDER HOUSE	36X48	ROCK	CULVERT	60	1	417	4.0	210	322.30	ON PRIVATE PROPERTY
CO40.37	3RD AVE.100'W OF GANONG	36	CMP	CLOSED	900	1	424	6.5	75	113.21	ALIGNMENT UNKNOWN

EXISTING CAPACITY COMPUTATION IS BASED ON: 1)INLET CONTROL, 2)MAXIMUM ALLOWABLE HEADWATER,and 3)PIPE OPEN AND UNOBSTRUCTED BY SILT AND DEBRIS.

ENTRANCE TYPE :1-SQUARED EDGE WITH HEADWALL,2-GROOVE END WITH HEADWALL,3-GROOVE AND PROJECTING.

FOR CLOSED PIPE SYSTEMS THE CHARACTERISTICS OF THE INLET PIPE HAS BEEN SHOWN, HOWEVER THE LENGTH IS THE ENTIRE LENGTH OF THE MAINLINE CLOSED SYSTEM.

3.1 (CONT.) MAJOR HYDRAULIC STRUCTURES INVENTORY

STRUCTURE ID	LOCATION	SIZE (IN)	MAT'L	STRUCT. TYPE	APPROX. LENGTH (FT)	INLET TYPE	AREA (ACRES)	HEAD-WATER (FT)	EXISTING CAPACITY (CFS)	(CFS/MI^2)	REMARKS
P10.10	HUNTER 500' S CLEVELAND	18	CMP	CULVERT	60	3	51	3.5	12	150.59	
P20.15	CHARMAN 150' NORTH OF GAIN	42	CMP	CULVERT	30	3	70	3.5	47	429.71	NO COVER
P20.20	@ FRONT 200' NORTH OF GAIN	30	CMP	CLOSED	500	3	96	5.0	39	260.00	
P40.21	CLEVELAND 60'E OF HARLEY	24	CONC	CULVERT	45	3	96	3.0	20	133.33	
P40.23	HARLEY 60'N OF CLEVELAND	30	CMP	CULVERT	45	3	96	3.0	28	186.67	
P40.28	INTER. OF APPERSON & LA RAE	36	CONC	CULVERT	180	3	154	6.0	78	324.16	
P50.50	UNDER BYPASS	60	CMP	CULVERT	315	2	256	25.0	125	312.50	GOOD CONDITION
P60.60	TWIN PIPES UNDER ABERNETHY	57X38	CMP	CULVERT	138	3	320	13.0	340	680.00	ONE PIPE PART. SILTED
L40.40	REDLAND ROAD E OF BYPASS	50X31	CMP	CULVERT	50	3	358	7.4	95	169.83	
S10.10	HOLMES LANE 250' E OF LEONARD	21	CONC	CULVERT	50	1	19	4.4	24	808.42	DNST PART. BLOCKED
S40.30	PEARL EAST OF LINN	36	CMP	CULVERT	200	3	211	18.0	120	363.98	INLET BADLY OBSTRUCTED
S50.40	JACKSON 200' N OF LINN	24X42	ROCK	CULVERT	175	2	269	20.0	146	347.36	INLET BADLY OBSTRUCTED
S60.45	DOWNSTREAM OF JACKSON	36	CMP	CLOSED	2500	3	288	5.0	55	122.22	INLET ONE-HALF SILTED
S110.66	DOWNSTREAM OF WATERFALL	36X42	ROCK	CLOSED	650	2	390	20.0	100	164.10	GOOD INLET CONDITIONS
JA40.50	INLET @ ORECITY HIGH SCHOOL	36	CMP	CLOSED	990	3	58	13.0	100	1103.45	INLET 2/3 SILTED
JA70.80	SCHOOL CREEK @ MADISON ST	42	CMP	CULVERT	220	3	134	20.0	170	811.94	INLET 2/3 SILTED
JA80.35	JOHN ADAMS AND 14TH ST	42	CONC	CLOSED	670	2	237	5.5	110	297.05	LITTLE COVER
CLT10.10	UNDER S END @ CLINTON	18	CONC	CULVERT	60	3	45	3.5	14	199.11	GOOD CONDITION

EXISTING CAPACITY COMPUTATION IS BASED ON: 1)INLET CONTROL, 2)MAXIMUM ALLOWABLE HEADWATER,and 3)PIPE OPEN AND UNOBSTRUCTED BY SILT AND DEBRIS.

ENTRANCE TYPE :1-SQUARED EDGE WITH HEADWALL,2-GROOVE END WITH HEADWALL,3-GROOVE AND PROJECTING.
FOR CLOSED PIPE SYSTEMS THE CHARACTERISTICS OF THE INLET PIPE HAS BEEN SHOWN, HOWEVER THE LENGTH IS THE ENTIRE LENGTH OF THE MAINLINE CLOSED SYSTEM.

However, the approximate location of all of the various hydraulic structures that were omitted from the table have been shown on the 1 inch = 200 feet map overlays developed as part of this planning effort and provided to the City under separate cover. If any data exists on these structures, it too has been provided to the City on these map overlays.

3.3 MINOR DRAINAGE SYSTEM

The minor drainage system is defined as that system which should be designed to accommodate the storm runoff expected once every 5 to 10 years (i.e., the 5- or 10-year flood). However, minor drainage systems should also be evaluated under extreme flooding conditions, such as the 100-year flood, to ensure that major damage will not occur. Minor drainage systems consist of small swales, streets and gutters, storm sewers, and smaller open drainageways. They provide convenience drainage, reduce street maintenance costs, and directly affect the orderliness of an urbanized area.

For the purposes of this study, minor drainage systems are defined as the collector systems that convey runoff from within subbasin area to its point of outfall into the major drainage system. In addition, the collector systems that carry runoff from the portion of the Oregon City urban growth area that is located within the ten lateral drainage areas presented in Section 3.2.1 are also defined as minor drainage systems.

It is beyond the scope of this study to locate and describe in detail all of the problems of the minor drainage systems throughout the Oregon City area. However, some general comments can be made. The greatest problem with the minor drainage systems throughout the area appears to be ponding during runoff events. The cause of the ponding will vary with each specific problem. Some of the common reasons are: (1) blocked culverts or inlets caused by siltation and debris, (2) no drainage provided so runoff eventually percolates into the ground, and (3) backwater or backups caused by undersized downstream storm sewers or culverts.

Minor drainage system problems, such as ponding, are important because they create traffic hazards during times of poor visibility and road surface conditions. Every attempt should be made to resolve these minor drainage problems as soon as they are brought to the attention of the City.

Chapter 4

Hydrologic/Hydraulic Analysis Techniques

CHAPTER 4

HYDROLOGIC/HYDRAULIC ANALYSIS

4.1 GENERAL

Pivotal to any drainage master planning effort is the calculation of peak flows and runoff volume within the major drainage basins being studied. Peak flows are used to size culverts and pipe systems; runoff volume is used to size detention facilities.

Runoff simulation using computerized models is now an everyday aid to practicing engineers involved in drainage planning, management and design. Simulation has been found to provide an excellent representation of real-world conditions. Used properly, computer simulation can accurately predict runoff from large areas under both existing conditions of development and conditions of ultimately planned development. This simulation data can be used to identify inadequacies in the existing drainage system and afford the engineer an opportunity to test management alternatives and examine the effectiveness of the alternatives in solving both existing and future drainage problems.

A number of different approaches have been proposed for the simulation of the rainfall-runoff process, but most of them can be considered as somewhat comparable. Nearly all of the simulation techniques take rainfall as the driving variable and, using a time series of rainfall intensities (hyetograph), compute the amount of runoff into each drainage channel based on various characteristics of small basins (subbasins). A hydrograph (the runoff simulated from the driving hyetograph) is computed for each subbasin at specified time intervals. The computed hydrographs are marched downstream at each time interval with travel distance computed based on channel characteristics. Such routing results in a basin runoff hydrograph at the final downstream outlet.

There are numerous computer models available which perform this simulation. They vary, not so much in approach, as in the specific methodology (set of equations) used to model a portion of the hydrologic cycle. The value of the simulation is less dependent upon the methodology than on the data used as input to the program, and the skill of the user.

In preparing the Oregon City Drainage Master Plan, OTAK utilized the Hydrologic Engineering Center's HEC-1 Flood Hydrograph Package developed by the U.S. Army Corps of Engineers. The HEC-1 program can be obtained from the Corps of Engineers Hydrologic Engineering Center at Davis, California. OTAK used the most recent updated version of HEC-1 (February, 1987) that runs on

IBM-PC's or IBM compatible PC's. The HEC-1 program along with the final input files for the 12 major drainage basins analyzed as part of this planning effort have been provided to the City under separate cover.

The HEC-1 program and the input parameters estimated for the Oregon City study areas will be briefly described in Section 4.3. The drainage planning criteria that was established and utilized during the Oregon City drainage planning efforts will now be presented.

4.2 DRAINAGE PLANNING CRITERIA

The establishment of a specific set of drainage planning criteria to be used in the development of a drainage master plan is an important process. The drainage planning criteria is used to establish an acceptable level of protection against flooding throughout the study area. This acceptable level of protection is then compared against the estimated risk of flooding at various points throughout the major drainage system. When the estimated flood risk at any given point is greater than that which was established to be acceptable, the drainage engineer must analyze the various alternative available to either increase the level of protection such that it satisfies the criteria or to do nothing and consciously accept a lower level of protection than that specified in the criteria.

4.2.1 Storm Recurrence Interval

In hydrological simulation and drainage planning, both flood risks and level of protection against those risks are measured using the concept of storm recurrence interval and its reciprocal function, the probability of exceedance. If one designs a hydraulic structure using a 100-year storm recurrence interval, the probability that the design flow will be exceeded in any given year is quite low (i.e. 1 percent probability) so the level of protection against flooding would be very high. If the design was based on a 2-year storm recurrence interval, the probability of exceedance would be very high (i.e. 50 percent probability in any given year) and the level of protection would be quite low. The obvious trade-off in the planning and design of drainage facilities is the cost of the facility. A facility designed to withstand a 100-year flood peak will obviously cost considerably more than one designed to only pass the 2-year flood.

Table 4.1 present the storm recurrence intervals proposed for the planning and design of drainage improvements throughout the Oregon City study area. The proposed storm recurrence intervals shown in Table 4.1 are based on several factors that will help define the various levels of flood risks. The factors include drainage area, the type of construction and the type of drainage improvement being considered. The basic concept is to increase the recurrence interval to match increasing levels of potential flood damage risk. Each of these factors will now be discussed.

TABLE 4.1

STORM RECURRENCE INTERVALS FOR PLANNING AND DESIGN OF DRAINAGE IMPROVEMENTS

Drainage Area (acres)			Type of Construction		Type of Drainage Improvement				Design Storm Recurrence Interval[f] (Years)
							Culverts & Bridge		
							Type of Roadway		
<40	40 to 640	>640	Remedial[a]	New	Open Channel[b]	Closed Pipe[c]	Local Collectors[d]	Major Arterials[e]	
x			x			x			5
x			x		x		x		10[g]
x				x	x	x	x		10[h]
x			x	x				x	25
	x		x			x			10
	x		x		x		x		25[i]
	x		x	x				x	25
	x			x	x	x	x		25
		x	x	x		x[j]	x		25
		x	x	x	x			x	50
All drainage improvements located on a waterway that has a 100-year floodplain published by FEMA									100

a. Applies to capital improvement projects that are designed to correct existing drainage problems with approval by the City
b. Includes roadside ditches or drainage swales
c. Storm sewer systems or a closed conduit whose length exceeds that of a normal culverted crossing of a roadway
d. Includes local or residential street, local collectors, and any other roadway up to a major arterial
e. Major arterial or better within the City right of way maintenance
f. Assuming ultimately planned development conditions (i.e. impervious cover) within the City's urban growth boundary (UGB) and existing development conditions outside of the UGB
g. The 5-year recurrence interval can be used in unusual situations involving insufficient topographical conditions that result in an exceptionally high cost differential between the 10- and 5-year improvement design (e.g. 25%)
h. These conditions will fit most new development situations
i. The 10-year recurrence interval can be used in unusual situations that satisfy the special criteria discussed in (g) above but applied to the 25-year verse 10-year improvement
j. Closed pipe systems are not recommended for waterways that are draining more than 640 acres

Contributing drainage area is the most important factor since greater drainage areas will provide greater estimates of peak flow. Three classifications of contributing drainage area have been provided in the table. Almost all of the hydraulic structures inventoried throughout the Oregon City study area fall within the 40 to 640-acre category.

The type of drainage improvement is also important. As a general rule, open channels which include roadside ditches or swales should be designed for greater recurrence intervals when compared to closed pipe systems. In traditional practice, the closed pipe storm sewer designs are based on full flow conditions. This design practice is conservative in that extra capacity usually exists within the system when it is operating under infrequent pressure flow conditions. Culverts and bridges that cross major arterial roadways should be designed for a greater return interval than those that cross local collector streets. This provides a higher level of protection needed to keep the major transportation system accessible during a general condition of flooding.

The last factor is defined as the type of construction. Remedial construction applies to capital improvement projects that are designed to correct existing drainage problems. New construction applies to drainage systems designed to handle storm water from new development or a new crossing of a existing channel. The remedial construction category gives the City the opportunity to design drainage improvements for lesser storm recurrence intervals than those required by new construction provided the improvement will correct existing drainage problems and the cost differential of designing those improvements for the higher recurrence interval are disapportionally higher. The remedial construction category is provided in an attempt to satisfy the financial constraints that exist when trying to alleviate severe drainage problems that already exist.

4.2.2 Intensity Duration Frequency Curves

Once the storm recurrence interval or design frequency is established, the drainage planner or engineer must be able to estimate the rainfall intensity to be used in either drainage planning or design. Figure 4.1 presents the rainfall intensity, duration and frequency (IDF) relationships that exist for the Oregon City study area. The curves shown on Figure 4.1 were obtained from the Metropolitan Service District's Storm Water Management Design Manual published in 1980 (Reference 6).

The IDF curves shown on Figure 4.1 should be used with the flow estimation technique presented in Appendix A to develop peak flows estimates for the minor drainage systems throughout the study area. Data from Figure 4.1 were used to develop the hypothetical rainfall hyetographs utilized in the HEC-1 hydrographical modeling of the 12 major drainage basins presented in Chapter 3. The rainfall hyetographs used in the drainage master planning effort will be discussed in the next section.

IDF CURVES

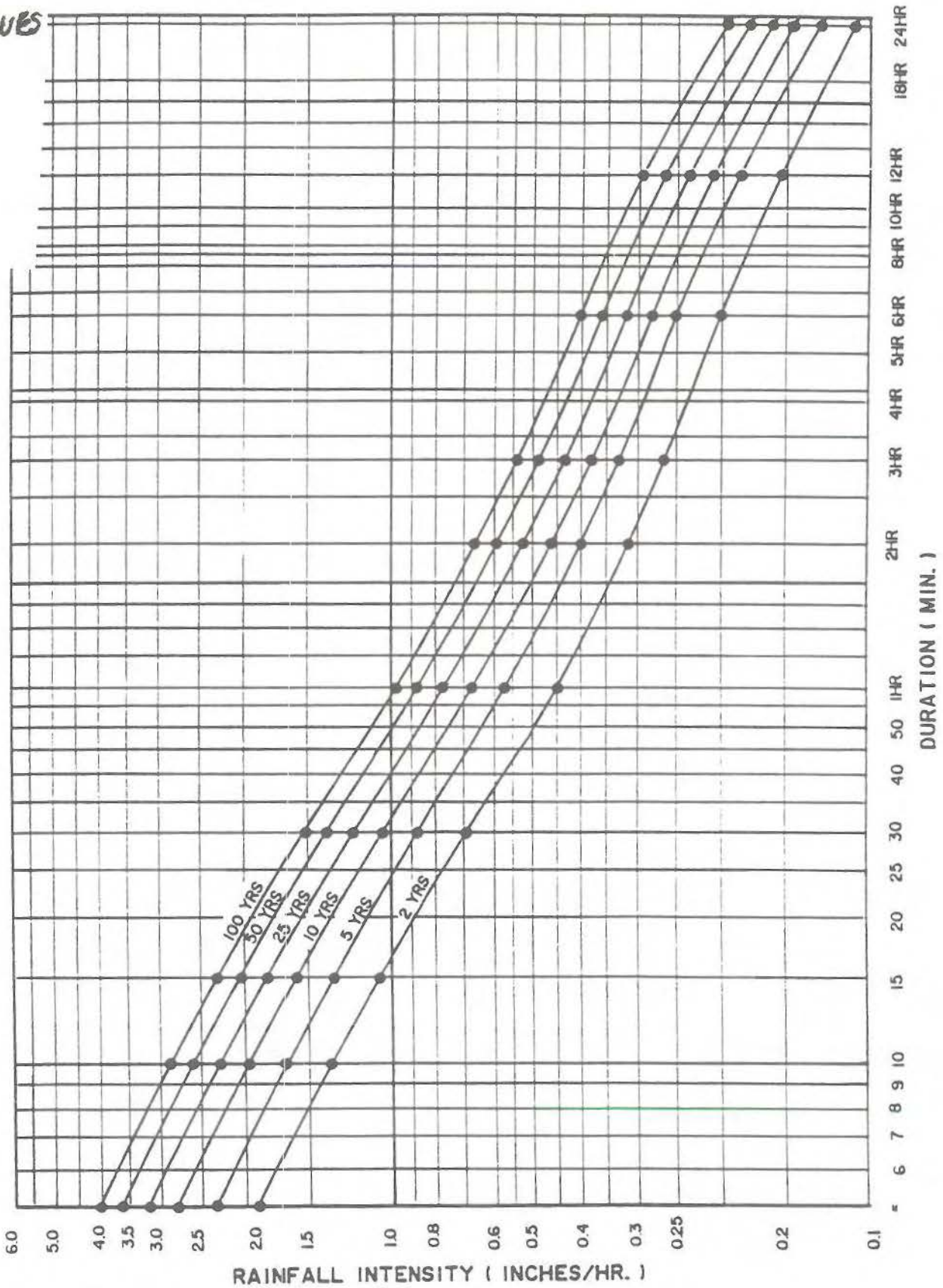

FIGURE 4.1

RAINFALL INTENSITY DURATION & FREQUENCY CURVES FOR OREGON CITY

otak incorporated

17355 S.W. Boones Ferry Rd. Lake Oswego OR 97
101 E. 8th Street Vancouver WA 98660 (206) 6
10655 Main Street #215 Bellevue WA 98004 (206) 455-3246

4.3 HEC-1 DESCRIPTION, INPUT PARAMETERS AND CALIBRATION

The HEC-1 model is designed to simulate the surface runoff response of a river or stream basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the rainfall-runoff process within a portion of the basin. A component may represent a surface runoff entity (subbasin), a stream channel or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component and mathematical equations which describe the physical processes. Details of the program can be found in the Corps of Engineers Hydrologic Engineering Center's User Manual for HEC-1 (Reference 7). The HEC-1 model was used to develop stormwater runoff hydrographs (time-series of flows) for designated design storm events at selected points throughout the 12 major drainage basins presented in Chapter 3. Data collection and assumptions for this analysis are described in the following sections.

4.3.1 Subbasin Runoff Computation

As described earlier in Chapter 3, the Oregon City study area was delineated into 22 major drainage areas. Twelve of these major drainage areas were established as major drainage basins. These major drainage basin were further subdivided into subbasins based on topography, land use, stream alignment and roadway alignments. Topographic maps and field reconnaissance were used to define the subbasin boundaries presented earlier in Chapter 3.

The appropriate drainage areas for each of the 85 delineated subbasins were determined by planimetering the topographic base maps. Although subbasin area varied from 7 to 188 acres, depending on unique topographic or land use features, the average subbasin area within these 12 basins was about 70 acres.

4.3.1.1 Rainfall Hyetographs

A hyetograph is a hypothetical time series of rainfall intensities used to simulate a rainstorm. This hypothetical rainstorm is input to the HEC-1 program, which in turn converts it to runoff for each of the subbasins throughout the major drainage basins.

The duration of the rainfall hyetograph used for drainage planning is extremely important. As a minimum the duration of the rainfall hyetograph should exceed the time of concentration of the entire basin that is being simulated. Depending on the size of the drainage basins being analyzed, the standard rainfall durations commonly used in drainage planning are 6, 12 or 24 hours.

The rainfall duration will effect both the estimate of peak flow and total runoff volume. A model sensitivity analysis was conducted on several drainage basins throughout the Oregon City study area. The results were used to establish the 24 hour

rainfall duration as the one to be used in the drainage master planning effort. Using the IDF curves shown on Figure 4.1, the 24 hour rainfall depths for the 5, 10, 25, 50 and 100-year storm recurrence intervals were estimated to be 3.05, 3.48, 3.86, 4.32 and 4.80 inches, respectively.

Working with the rainfall depths outlined above, the 24 hour Soil Conservation Service's (SCS) Type 1A rainfall distribution was used to distribute the rainfall depths at 30 minute time intervals. The 1A rainfall distribution was found by the SCS to be applicable to the storm patterns observed in the portion of Oregon and Washington located west of the Cascades. Appendix B presents the various SCS rainfall distributions used throughout the United States.

4.3.1.2 SCS Curve Number Method

The HEC-1 model offers the user many options in the method used to transform rainfall input into rainfall excess. Rainfall excess is the portion of rainfall that does not infiltrate into the soil-cover complex and is therefore available for runoff. The Soil Conservation Service's Curve Number method (Reference 8) was selected for use in this study.

Based on experimentation and experience, the SCS has been able to relate the drainage characteristics of soil groups to a curve number, CN. The SCS provides information on relating soil group type to the curve number as a function of soil cover, and antecedent moisture conditions.

The soil cover is classified by the SCS into various hydrologic soil groups (i.e. see Section 2.3). The curve numbers that were assigned to each of the hydrologic soil groups throughout the Oregon City study area are shown in Table 4.2.

TABLE 4.2

CURVE NUMBERS ESTABLISHED FOR EACH HYDROLOGIC SOIL GROUP

SCS Hydrologic Soil Group	Curve Number
A	65
B	78
C	85
D	89

An area-weighted average curve number was calculated for each subcatchment based on the percent of each soil group in the subcatchment.

4.3.1.3 Mapped Impervious Area

Mapped impervious area (MIA) is defined as that portion of a subbasin that is impervious to rainfall (e.g. does not allow rainfall to contact the soil). MIA is an extremely important parameter. In drainage planning, mapped impervious areas are usually estimated for both existing conditions of development and ultimately planned development conditions.

The amount of mapped impervious area in a subbasin will differ greatly depending on its land use. The more intense the development, the higher the percent impervious land cover. The typical percentages of mapped impervious area estimated for each land use category found in the Oregon City study area are shown in Table 4.3.

TABLE 4.3

MAPPED IMPERVIOUS AREA USED IN OREGON CITY BY LAND USE CATEGORY

Land Use Category	Mapped Impervious Area
Residential	
R2 (2,000 sq ft/dwelling unit)	60%
R4 (4,000 sq ft/dwelling unit)	55%
R6 (6,000 sq ft/dwelling unit)	50%
R8 (8,000 sq ft/dwelling unit)	40%
R10 (10,000 sq ft/dwelling unit)	35%
Commercial	90%
Industrial	70%
Open Space	0%

Estimates of mapped impervious area for existing conditions of development were determined from aerial photographs taken in 1984. The mapped impervious area was then computed based on an area-weighted average of typical impervious area percentages for each of the land use categories that exist within any given subbasin.

Estimates of mapped impervious area for ultimately planned development conditions were based on both the City's and Clackamas County's Comprehensive Plans. Mapped impervious areas were adjusted to slightly higher values in areas that the City felt would experience some pressure to increase existing planned densities.

4.3.1.4 Effective Impervious Area

Effective impervious area (EIA) is defined as that portion of a subbasin that is both impervious to rainfall and is also effectively connected to the drainage collector system. The HEC-1 model assumes that 100 percent of the rainfall that falls on the effective impervious area is available for runoff. EIA includes

street surfaces, paved driveways that connect to the street, sidewalks that are adjacent to curbed streets, rooftops if they are directly connected to the curb or connected driveways and parking lots. EIA is one of the most difficult and important parameters that must be estimated during the drainage master planning process.

The direct measurement of EIA is a tedious exercise that cannot be undertaken because drainage planning budgets cannot afford its excessive labor cost. Effective impervious area was assumed to be related to mapped impervious area by the following equation:

$$EIA = 0.1\ (MIA)^{1.5},\ MIA \geq 1$$

This equation was developed by Sutherland working with rainfall to runoff data collected by the U.S. Geological Survey (U.S.G.S.) throughout the metropolitan areas of Portland and Salem, Oregon. The development of the equation is described in Appendix C.

4.3.1.5 SCS Dimensionless Unit Hydrograph

The HEC-1 model offers the user many options in the method used to compute a subcatchment hydrograph from rainfall excess. The Soil Conservation Service's Unit Hydrograph method (Reference 8) was selected for use in this study for subbasins that were classified as natural or partially urbanized.

The unit hydrograph for a subbasin is defined as the hydrograph of direct runoff that will result from exactly 1-inch of rainfall excess on the subbasin generated uniformly within a specified unit duration. Thus, the 10-minute unit hydrograph from subbasin X is the hydrograph at the outlet of subbasin X that would result if exactly one inch of rainfall excess (i.e., rainfall minus infiltration losses) occurred uniformly over subbasin X and within a 10-minute period of time. The dimensionless unit hydrograph can be used to convert a time series of unit rainfall excesses into a subbasin hydrograph.

The input data for the SCS dimensionless unit hydrograph method consists of a single parameter, lag time. Lag time (TLAG) is the time (hours) between the center of mass of rainfall excess and the peak of the unit hydrograph. Lag time was computed from the following equation developed by Sutherland working with runoff data collected by the USGS throughout the metropolitan areas of Portland and Salem, Oregon.

$$TLAG = A(CL/CSL^{0.5})^{B}$$

Where:

CL = length of the main water course (i.e. most defined course) measured from the basin outlet upstream to the basin boundary (miles)

CSL = slope of the main water course (ft/mile) measured between points located at 10 percent and 85 percent of CL, respectively.

A and B are coefficients whose values are based on the mapped impervious area (MIA) within the basin and the characteristics of the drainage collector systems within the basin.

The complete documentation of this equation and its development has been provided in Appendix C.

4.3.2 Subbasin Parameter Estimates

Using the techniques described above, the parameters needed for the HEC-1 model simulations were estimated for each of the subbasins located within the 12 major drainage basins. Table 4.4 presents the subbasin parameters that were estimated for each of these basin models.

It should be noted that lag times were not estimated for the subbasins located in John Adams or several of the subbasins found in Singer. The SCS unit hydrograph technique was not used to estimate subbasin runoff from these areas. Instead the Kinematic Wave technique was used. The Kinematic Wave technique is briefly described in the next section. It is most appropriately used to estimate runoff from subbasins that are highly urbanized with well defined minor drainage collector systems.

4.3.3 Flood Routing Computation

The existing major drainage collector systems were presented earlier in Chapter 3. The HEC-1 program conceptualizes these collector systems as a series of routing reaches that link the subbasin runoff hydrographs and marches them downstream to the eventual outfall of the basin. These routing reaches can be simulated as either open channels, closed pipes or open detention ponds.

The HEC-1 model offers the user many options in the method used to simulate flood movement through stream reaches. The Modified Puls routing method (Reference 9) was used to simulate the storage detention occurring at several points throughout the Oregon City study area. The Kinematic Wave routing method (Reference 7) was used for the remaining routing reaches throughout the various major drainage networks.

4.3.3.1 Kinematic Wave

In the Kinematic Wave interpretation of the equations of motion, it is assumed that the channel bed slope and water surface slope are equal and acceleration effects are negligible. This assumption implies that depth changes gradually within channel distance and velocity changes gradually with distance and time, thus flow at any point in the channel can be computed from Manning's for

TABLE 4.4 SUBBASIN PARAMETERS USED IN HYDROLOGICAL MODELING

BASIN	SUBBASIN	DRAINAGE (ACRES)	AREA (MI^2)	IMPERVIOUS AREA (%) EXISTING MIA	EXISTING EIA	FUTURE MIA	FUTURE EIA	CURVE NUMBER	MAIN CHANNEL (MI.)	CHANNEL SLOPE (FT/MI)	EXISTING LAGTIME (HOURS)	FUTURE LAGTIM (HOURS
CAUFIELD	CA10	45	0.07	25	12	40	25	85	0.42	141	0.81	0.40
	CA20	63	0.10	18	7	28	15	85	0.35	207	0.93	0.57
	CA30	56	0.09	11	3	40	25	85	0.48	154	1.64	0.42
	CA40	62	0.10	25	12	34	20	85	0.36	213	0.68	0.44
	CA50	82	0.13	26	13	47	32	85	0.53	116	0.92	0.36
	CA60	67	0.10	18	7	40	25	85	0.42	83	1.28	0.47
	CA70	124	0.19	19	8	43	28	85	0.41	38	1.41	0.49
	CA80	52	0.08	18	7	40	25	85	0.29	134	0.95	0.34
	CA90	58	0.09	37	23	47	32	85	0.32	88	0.45	0.30
	CA100	26	0.04	28	15	48	33	85	0.39	51	0.86	0.36
	CA110	74	0.12	44	29	68	55	85	0.32	40	0.42	0.15
	CA120	98	0.15	40	25	50	35	85	0.42	66	0.51	0.33
	CA130	98	0.15	25	12	39	24	85	0.59	61	1.19	0.63
	CA140	18	0.03	14	5	37	23	85	0.23	24	1.52	0.54
	CA150	14	0.02	11	3	38	23	85	0.22	89	1.29	0.37
		936	1.46									
MUD	M10	112	0.17	41	26	50	35	85	0.45	26	0.62	0.42
	M20	77	0.12	26	13	52	37	85	0.67	54	1.23	0.40
	M30	120	0.19	28	15	40	25	85	0.70	65	1.09	0.66
	M40	91	0.14	11	3	36	22	85	0.50	75	1.99	0.62
	M50	59	0.09	14	5	40	25	85	0.45	86	1.55	0.48
	M60	75	0.12	7	2	40	25	85	0.39	305	1.53	0.32
		533	0.83									
CENTRAL PT	CP10	79	0.12	25	13	38	23	86	0.32	78	0.80	0.45
	CP20	66	0.10	28	15	39	24	85	0.33	144	0.61	0.38
	CP30	85	0.13	25	12	40	25	85	0.39	151	0.77	0.39
		230	0.36									
COFFEE	CO10	160	0.25	28	15	37	22	85	0.56	31	1.17	0.79
	CO20	51	0.08	29	16	37	23	86	0.35	96	0.66	0.46
	CO30	164	0.26	37	23	38	23	86	0.39	129	0.46	0.44
	CO40	70	0.11	47	32	48	34	82	0.30	930	0.16	0.15
		445	0.69									
TUMWATER	T10	45	0.07	39	24	44	29	81	0.44	731	0.28	0.22
	T20	32	0.05	29	15	52	37	80	0.30	38	0.79	0.28
	T30	12	0.02	39	24	46	31	80	0.16	592	0.18	0.13
		89	0.14									
CLINTON	CLT10	43	0.07	28	15	37	22	84	0.25	995	0.33	0.22
	CLT20	14	0.02	10	3	20	9	84	0.17	324	0.85	0.52
		57	0.09									
SOUTH END	SE10	50	0.08	36	21	37	22	86	0.44	52	0.65	0.63
	SE20	44	0.07	36	22	40	25	85	0.36	16	0.78	0.65
	SE30	107	0.17	34	20	37	23	85	0.76	44	0.96	0.83
	SE40	33	0.05	36	22	39	24	85	0.22	20	0.58	0.51
	SE50	87	0.14	32	18	36	22	85	0.37	12	1.04	0.85
	SE60	81	0.13	32	18	38	23	85	0.47	18	1.05	0.79
	SE70	75	0.12	25	12	39	25	85	0.47	107	0.92	0.48
	SE80	131	0.20	18	7	40	25	85	0.64	101	1.49	0.55
		607	0.95									

MIA = Mapped Impervious Area; EIA = Effective Impervious Area

TABLE 4.4 (CONT.) SUBBASIN PARAMETERS USED IN HYDROLOGICAL MODELING

BASIN	SUBBASIN	DRAINAGE (ACRES)	AREA (MI^2)	IMPERVIOUS AREA (%) EXISTING MIA	EXISTING EIA	FUTURE MIA	FUTURE EIA	CURVE NUMBER	MAIN CHANNEL (MI.)	CHANNEL SLOPE (FT/MI)	EXISTING LAGTIME (HOURS)	FUTURE LAGTIME (HOURS)
NEWEL	N10	82	0.13	42	27	52	37	85	0.54	111	0.45	0.29
	N20	107	0.17	28	15	40	26	85	0.39	33	0.96	0.56
	N30	107	0.17	28	15	48	33	85	0.47	123	0.76	0.32
	N40	139	0.22	46	31	61	48	85	0.54	140	0.36	0.18
	N50	128	0.20	17	7	46	31	81	0.53	466	0.97	0.26
	N60	107	0.17	57	43	60	46	85	0.63	51	0.31	0.27
	N70	188	0.29	65	52	73	62	85	0.73	33	0.27	0.19
	N80	87	0.14	12	4	51	36	81	0.58	604	1.20	0.21
	N90	53	0.08	11	3	40	25	82	0.47	564	1.19	0.30
	N100	170	0.27	36	21	45	30	82	0.64	615	0.42	0.27
	N110	176	0.28	11	3	39	24	85	0.53	75	2.05	0.56
	N120	96	0.15	41	26	53	38	82	0.91	452	0.42	0.26
	N130	118	0.18	35	21	44	29	81	0.63	495	0.45	0.30
	N140	108	0.17	11	3	40	25	80	0.36	183	1.38	0.35
	N150	124	0.19	39	24	41	26	81	0.55	34	0.71	0.64
		1790	2.80									
SINGER	S10	19	0.03	36	22	42	27	85	0.23	69	0.43	0.33
	S20	51	0.08	21	10	50	35	84	0.38	405	0.70	0.19
	S30	116	0.18	25	12	38	23	85	0.50	297	0.74	0.40
	S40	26	0.04	50	35	50	36	84	0.21	794	0.12	0.12
	S50	57	0.09	45	30	63	50	84		Kinematic Wave		
	S60	19	0.03	50	35	74	64	85		Kinematic Wave		
	S70	18	0.03	25	13	50	35	81		Kinematic Wave		
	S80	24	0.04	50	35	63	50	84		Kinematic Wave		
	S90	29	0.05	44	29	53	38	82		Kinematic Wave		
	S100	30	0.05	71	59	81	73	85		Kinematic Wave		
	S110	7	0.01	71	60	73	62	85	0.24	576	0.05	0.05
		395	0.62							Kinematic Wave		
JOHN ADAMS	JA10	49	0.08	50	35	54	40	85		Kinematic Wave		
	JA20	19	0.03	50	35	52	37	85		Kinematic Wave		
	JA30	15	0.02	50	35	50	35	85		Kinematic Wave		
	JA40	55	0.09	40	25	50	35	82		Kinematic Wave		
	JA50	27	0.04	38	23	50	35	82		Kinematic Wave		
	JA60	21	0.03	50	35	53	39	85		Kinematic Wave		
	JA70	32	0.05	47	32	51	36	83		Kinematic Wave		
	JA80	16	0.03	54	40	68	56	80		Kinematic Wave		
		234	0.37							Kinematic Wave		
LIVESAY	L10	83	0.13	9	3	28	15	84	0.80	385	1.84	0.75
	L20	132	0.21	4	1	18	8	84	0.79	450	2.37	1.11
	L30	111	0.17	32	18	38	23	84	0.75	297	0.63	0.50
	L40	33	0.05	18	8	29	15	81	0.37	101	1.12	0.69
		359	0.56									
PARK PLACE	P10	52	0.08	24	12	37	22	84	0.38	317	0.64	0.37
	P20	41	0.06	39	24	43	28	85	0.28	189	0.32	0.28
	P30	58	0.09	32	18	35	21	84	0.21	1006	0.25	0.22
	P40	65	0.10	35	20	40	25	86	0.38	204	0.44	0.35
	P50	42	0.07	33	19	70	59	79	0.38	40	0.72	0.14
	P60	65	0.10	24	12	56	42	78	0.35	19	1.25	0.31
		323	0.51									

MIA = Mapped Impervious Area; EIA = Effective Impervious Area

mula. Therefore, the only input data required is that to describe the geometry and roughness of the flow reach.

The HEC-1 model provides the user with five options in simulating the geometry of the routing reach. One is a closed circular pipe where the pipe length, pipe slope and pipe diameter must be specified. The remaining four are open channel shapes (i.e., triangular, square, rectangular, and trapezoidal). The open channel options require the specifications of the channel length, channel slope, channel bottom width and side slopes.

All of the above data used to describe the geometry of the routing reaches throughout the Oregon City drainage networks were obtained from field observation or aerial maps. The surface roughness of the routing reach was described by Manning's roughness factor "n".

It should be noted that the Kinematic Wave routing method works well in urban drainage networks where storage in the drainage system is not very significant. In areas where storage effects are significant, such as within detention facilities and ponding behind road culverts; a storage routing method, like Modified Puls should be used instead.

4.3.3.2 Modified Puls

As noted earlier, the Modified Puls routing method was used to simulate the storage detention occurring at several points throughout the area. The Modified Puls method is based on solving the continuity equation by using a unique relationship between storage volume and outflow. This unique relationship is usually obtained by working with the topographic maps of the storage area whether that area is the backwater area upstream of an existing culvert or an existing pond. For either of these areas, an elevation verse storage volume relationship is developed. Working with the proper culvert hydraulic charts or the weir equation to describe the elevation of flow over a roadway or dam, a relationship is developed for headwater elevation verse outflow. These two relationships are then combined to create the storage verse outflow curve required to use the Modified Puls technique.

This technique was applied to describe the existing pond on Caufield Creek between nodes 70 and 75. It was also applied to the backwater areas found upstream of Madison Street on John Adams (i.e. node 75 to 80) and the two backwater areas found along Singer Creek (i.e. at Pearl Street, node 25 to 30; and at Jackson Street, node 35 to 40).

4.3.4 Model Calibration

As part of a 2 to 3 year project, the U.S. Geological Survey (USGS) in cooperation with the City of Portland collected rainfall and runoff data for 24 different streamflow sites throughout the Portland metropolitan area (Reference 10). Singer Creek was

one of the sites that was included in this data collection effort.

The Singer Creek gauge was located at the Pearl Street culvert (i.e. S40.30). Both rainfall and streamflow data were collected for all of the storm events from January, 1976 through January, 1978. In an attempt to calibrate the HEC-1 model for Singer Creek, this data was obtained from the USGS in January, 1987.

The three largest storm events based on recorded peak flow were selected for the model calibration effort. Upon examination of this data, we found some serious problems that greatly diminished its usefulness for the calibration exercise. First, the Pearl Street culvert has apparently been replaced since the USGS data was collected. There was no accurate record available on the physical characteristics of the old culvert. And second, the model calibration exercise concluded that the flow through the culvert was severely obstructed by a possible debris accumulation. For example, the storm of December 2, 1977 had a recorded peak flow of 13 cfs. Whereas, the HEC-1 model computed a peak flow of 45 cfs.

Large discrepancies between observed and computed peak flows were found for all of the rainfall events simulated. Erratic oscillations found in the "observed" streamflow data that occurred during the time the simulated flows exceeded the "observed" values indicate that some sort of blockage was occurring. Although the data obtained from the USGS could not be used to calibrate peak flows, it was used to provide some confidence in the timing of the rising limb of the hydrograph which matched quite well in all three cases.

4.4 HYDROLOGICAL RESULTS

Working with subbasin parameters presented in Table 4.4 and the data estimated for the various major drainage collector systems throughout the drainage basins, the HEC-1 models were coded for each of the 12 drainage basins. Using the 5, 10, 25, 50 and 100-year 24-hour hypothetical storms discussed earlier, the HEC-1 models were used to estimate the resulting peak flows that would occur under both existing conditions of development and ultimately planned conditions of development.

4.4.1 Existing Conditions of Development

Table 4.5 presents the hydrological results obtained for each of the 12 major drainage basins under existing conditions of development. The 10, 25 and 100-year peak flows are shown for each of the subbasins located throughout the Oregon City study area. The simulated subbasin peak flows varied from 4 to 161 cfs depending upon the recurrence interval of the storm and the specific subbasin characteristics.

TABLE 4.5 SUBBASIN PEAK FLOWS FOR EXISTING CONDITIONS OF DEVELOPMENT

BASIN	SUBBASIN	DRAINAGE (ACRES)	AREA (MI^2)	CURVE NUMBER	EXISTING CONDITIONS IMPERVIOUS AREA MIA	EIA	LAGTIME (HOURS)	PEAK SUBBASIN FLOW Q10	Q25	Q100
CAUFIELD	CA10	45	0.07	85	25	12	0.81	15	17	24
	CA20	63	0.10	85	18	7	0.93	19	22	30
	CA30	56	0.09	85	11	3	1.64	13	16	22
	CA40	62	0.10	85	25	12	0.68	21	25	34
	CA50	82	0.13	85	26	13	0.92	26	30	41
	CA60	67	0.10	85	18	7	1.28	18	21	29
	CA70	124	0.19	85	19	8	1.41	31	37	51
	CA80	52	0.08	85	18	7	0.95	15	18	25
	CA90	58	0.09	85	37	23	0.45	25	28	38
	CA100	26	0.04	85	28	15	0.86	8	10	13
	CA110	74	0.12	85	44	29	0.42	33	38	51
	CA120	98	0.15	85	40	25	0.51	40	47	63
	CA130	98	0.15	85	25	12	1.19	27	32	44
	CA140	18	0.03	85	14	5	1.52	5	5	7
	CA150	14	0.02	85	11	3	1.29	5	5	7
MUD	M10	112	0.17	85	41	26	0.62	46	53	71
	M20	77	0.12	85	26	13	1.23	21	25	34
	M30	120	0.19	85	28	15	1.09	36	41	57
	M40	91	0.14	85	11	3	1.99	19	23	32
	M50	59	0.09	85	14	5	1.55	14	17	23
	M60	75	0.12	85	7	2	1.53	18	21	29
CENTRAL PT	CP10	79	0.12	86	25	13	0.80	27	31	43
	CP20	66	0.10	85	28	15	0.61	24	28	38
	CP30	85	0.13	85	25	12	0.77	28	33	45
COFFEE	CO10	160	0.25	85	28	15	1.17	46	54	73
	CO20	51	0.08	86	29	16	0.66	19	22	30
	CO30	164	0.26	86	37	23	0.46	71	83	111
	CO40	70	0.11	82	47	32	0.16	36	42	56
TUMWATER	T10	45	0.07	81	39	24	0.28	19	22	31
	T20	32	0.05	80	29	15	0.79	9	10	15
	T30	13	0.02	80	39	24	0.18	5	6	9
CLINTON	CLT10	43	0.07	84	28	15	0.33	18	21	29
	CLT20	14	0.02	84	10	3	0.85	4	5	7
SOUTH END	SE10	50	0.08	86	36	21	0.65	20	23	31
	SE20	44	0.07	85	36	22	0.78	16	18	25
	SE30	107	0.17	85	34	20	0.96	35	41	55
	SE40	33	0.05	85	36	22	0.58	12	14	19
	SE50	87	0.14	85	32	18	1.04	27	31	42
	SE60	81	0.13	85	32	18	1.05	25	29	39
	SE70	75	0.12	85	25	12	0.92	23	27	37
	SE80	131	0.20	85	18	7	1.49	32	28	53

MIA = Mapped Impervious Area; EIA = Effective Impervious Area

TABLE 4.5 (CONT.) SUBBASIN PEAK FLOWS FOR EXISTING CONDITIONS OF DEVELOPMENT

					EXISTING CONDITIONS					
BASIN	SUBBASIN	DRAINAGE (ACRES)	AREA (MI^2)	CURVE NUMBER	IMPERVIOUS AREA MIA	IMPERVIOUS AREA EIA	LAGTIME (HOURS)	PEAK SUBBASIN FLOW Q10	Q25	Q100
NEWELL	N10	82	0.13	85	42	27	0.45	36	41	55
	N20	107	0.17	85	28	15	0.96	33	39	53
	N30	107	0.17	85	28	15	0.76	36	42	57
	N40	139	0.22	85	46	31	0.36	67	77	103
	N50	128	0.20	81	17	7	0.97	31	37	53
	N60	107	0.17	85	57	43	0.31	57	65	86
	N70	188	0.29	85	65	52	0.27	109	124	161
	N80	87	0.14	81	12	4	1.20	19	22	32
	N90	53	0.08	82	11	3	1.19	12	14	20
	N100	170	0.27	82	36	21	0.42	64	76	104
	N110	176	0.28	85	11	3	2.05	37	44	60
	N120	96	0.15	82	41	26	0.42	38	44	60
	N130	118	0.18	81	35	21	0.45	42	50	69
	N140	108	0.17	80	11	3	1.38	20	25	36
	N150	124	0.19	81	39	24	0.71	39	46	64
SINGER	S10	19	0.03	85	36	22	0.43	8	10	13
	S20	51	0.08	84	21	10	0.70	16	19	27
	S30	116	0.18	85	25	12	0.74	38	45	62
	S40	26	0.04	84	50	35	0.12	15	17	23
	S50	57	0.09	84	45	30	K. WAVE	28	33	45
	S60	19	0.03	85	50	35	K. WAVE	9	11	15
	S70	18	0.03	81	25	13	K. WAVE	6	8	11
	S80	24	0.04	84	50	35	K. WAVE	10	12	18
	S90	29	0.05	82	44	29	K. WAVE	14	16	23
	S100	30	0.05	85	71	59	K. WAVE	18	21	27
	S110	7	0.01	85	71	60	0.05	5	5	7
JOHN ADAMS	JA10	49	0.08	85	50	35	K. WAVE	28	33	44
	JA20	19	0.03	85	50	35	K. WAVE	11	13	17
	JA30	15	0.02	85	50	35	K. WAVE	8	10	13
	JA40	55	0.09	82	40	25	K. WAVE	25	30	42
	JA50	27	0.04	82	38	23	K. WAVE	11	13	19
	JA60	21	0.03	85	50	35	K. WAVE	12	14	19
	JA70	32	0.05	83	47	32	K. WAVE	17	20	27
	JA80	16	0.03	80	54	40	K. WAVE	6	7	10
LIVESAY	L10	83	0.13	84	9	3	1.84	17	20	29
	L20	132	0.21	84	18	7	1.14	35	41	57
	L30	111	0.17	84	32	18	0.63	39	46	63
	L40	33	0.05	81	18	8	1.12	8	9	13
PARK PLACE	P10	52	0.08	84	24	12	0.64	17	21	29
	P20	41	0.06	85	39	24	0.32	19	22	30
	P30	58	0.09	84	32	18	0.25	26	31	43
	P40	65	0.10	86	35	20	0.44	28	33	44
	P50	42	0.07	79	33	19	0.72	11	14	19
	P60	65	0.10	78	24	12	1.25	13	16	23

MIA = Mapped Impervious Area; EIA = Effective Impervious Area

4.4.2 Ultimate Development Conditions

Table 4.6 presents the hydrological results obtained for each of the 12 major drainage basins under ultimately planned (future) development conditions. As the Oregon City study area continues to urbanize according to its comprehensive plan, mapped impervious area will increase which results in a decrease in the time it takes runoff to concentrate at a given point. All of these changes result in increases in the peak flows that would occur during each of these hypothetical storms.

The results shown on Table 4.6 indicate the dramatic increases in mapped impervious area that are planned at several subbasins throughout the Oregon City study area. These dramatic increases are planned to occur in the southern and eastern drainages which primarily include Mud, Caufield and Newell. These increases are the result of large tracts of undeveloped land that are expected to be developed.

Table 4.6 also indicates that these dramatic increases in MIA translate to large decreases in subbasin lag times and significant increases in the peak flows throughout these areas. The most dramatic increases in peak flow are generally occurring for the more frequent storms primarily the 10-year and 25-year events.

All of the hydrological results presented up to this point have concentrated on subbasins peak flows. Table 4.7 presents the hydrological results obtained for each of the major drainage system node points throughout the 12 modeled drainages. Both the existing development conditions and the ultimately planned (future) conditions peak flows have been included in Table 4.7.

The data presented in Table 4.7 provides a more consistent representation of how peak flows are estimated to change as the basins move toward their ultimately planned (future) development conditions. The dramatic increases in subbasin peak flows have been dampened somewhat as one moves downstream through the drainage basins. The greatest increases are still occurring in the undeveloped southern and eastern drainages. However, the increases are more in the order of 30 to 60 percent depending on the drainage and storm of interest.

It should be noted that relatively small increases (i.e. less than 20 percent) in peak flow generally are estimated to occur throughout the Coffee, Clinton, South End, Singer and John Adams basins. Each of these basins contain considerable amounts of existing development with limited opportunities for future growth.

TABLE 4.6 SUBBASIN PEAK FLOWS FOR ULTIMATELY PLANNED (FUTURE) DEVELOMENT CONDITIONS

				FUTURE CONDITIONS									
BASIN	SUBBASIN	DRAINAGE (ACRES)	AREA (MI^2)	MIA	%INCRS	LAGTIME (HOURS)	%DECRS	Peak Q10	%INCRS	Subbasin Q25	%INCRS	Flow Q100	%INCRS
CAUFIELD	CA10	45	0.07	40	60	0.40	51	20	33	23	35	32	33
	CA20	63	0.10	28	56	0.57	39	23	21	27	23	32	7
	CA30	56	0.09	40	264	0.42	74	25	92	29	81	39	77
	CA40	62	0.10	34	36	0.44	35	26	24	30	20	41	21
	CA50	82	0.13	47	81	0.36	61	40	54	46	53	61	49
	CA60	67	0.10	40	122	0.47	63	29	61	33	57	45	55
	CA70	124	0.19	43	126	0.49	65	53	71	61	65	82	61
	CA80	52	0.08	40	122	0.34	64	24	60	28	56	38	52
	CA90	58	0.09	47	27	0.30	33	29	16	33	18	45	18
	CA100	26	0.04	48	71	0.36	58	13	63	15	50	19	46
	CA110	74	0.12	68	55	0.15	64	47	42	54	42	70	37
	CA120	98	0.15	50	25	0.33	35	49	23	56	19	74	17
	CA130	98	0.15	39	56	0.63	47	37	37	43	34	58	32
	CA140	18	0.03	37	164	0.54	64	8	60	9	80	12	71
	CA150	14	0.02	38	245	0.37	71	8	60	9	80	13	86
MUD	M10	112	0.17	50	22	0.42	32	55	20	63	19	84	18
	M20	77	0.12	52	100	0.40	67	37	76	42	68	56	65
	M30	120	0.19	40	43	0.66	39	45	25	53	29	71	25
	M40	91	0.14	36	227	0.62	69	35	84	41	78	55	72
	M50	59	0.09	40	186	0.48	69	25	79	29	71	39	70
	M60	75	0.12	40	471	0.32	79	35	94	41	95	55	90
CENTRAL PT	CP10	79	0.12	38	52	0.45	44	34	26	40	29	54	26
	CP20	66	0.10	39	39	0.38	38	29	21	34	21	46	21
	CP30	85	0.13	40	60	0.39	49	38	36	44	33	60	33
COFFEE	CO10	160	0.25	37	32	0.79	32	55	20	65	20	87	19
	CO20	51	0.08	37	28	0.46	30	22	16	26	18	35	17
	CO30	164	0.26	38	3	0.44	4	72	1	84	1	113	2
	CO40	70	0.11	48	2	0.15	6	36	0	43	2	57	2
TUMWATER	T10	45	0.07	44	13	0.22	21	21	11	24	9	33	6
	T20	32	0.05	52	79	0.28	65	15	67	17	70	24	60
	T30	12	0.02	46	18	0.13	28	6	20	7	17	10	11
CLINTON	CLT10	43	0.07	37	32	0.22	33	21	17	24	14	33	14
	CLT20	14	0.02	20	100	0.52	39	5	25	6	20	8	14
SOUTH END	SE10	50	0.08	37	3	0.63	3	20	0	23	0	31	0
	SE20	44	0.07	40	11	0.65	17	17	6	20	11	27	8
	SE30	107	0.17	37	9	0.83	14	37	6	43	5	59	7
	SE40	33	0.05	39	8	0.51	12	13	8	15	7	20	5
	SE50	87	0.14	36	13	0.85	18	30	11	34	10	46	10
	SE60	81	0.13	38	19	0.79	25	28	12	33	14	44	13
	SE70	75	0.12	39	56	0.48	48	32	39	37	37	49	32
	SE80	131	0.20	40	122	0.55	63	53	66	61	61	82	55

MIA = Mapped Impervious Area; EIA = Effective Impervious Area

TABLE 4.6 (CONT.) SUBBASIN PEAK FLOWS FOR ULTIMATELY PLANNED (FUTURE) DEVELOMENT CONDITIONS

				FUTURE CONDITIONS									
BASIN	SUBBASIN	DRAINAGE	AREA			LAGTIME		Peak		Subbasin		Flow	
		(ACRES)	(MI^2)	MIA	%INCRS	(HOURS)	%DECRS	Q10	%INCRS	Q25	%INCRS	Q100	%INCRS
NEWELL	N10	82	0.13	52	24	0.29	36	43	19	49	20	65	18
	N20	107	0.17	40	43	0.56	42	43	30	50	28	67	26
	N30	107	0.17	48	71	0.32	58	53	47	61	45	81	42
	N40	139	0.22	61	33	0.18	50	85	27	96	25	126	22
	N50	128	0.20	46	171	0.26	73	59	90	68	84	93	75
	N60	107	0.17	60	5	0.27	13	60	5	69	6	90	5
	N70	188	0.29	73	12	0.19	30	122	12	137	10	176	9
	N80	87	0.14	51	325	0.21	83	43	126	50	127	67	109
	N90	53	0.08	40	264	0.30	75	23	92	27	93	37	85
	N100	170	0.27	45	25	0.27	36	79	23	92	21	124	19
	N110	176	0.28	39	255	0.56	73	70	89	82	86	110	83
	N120	96	0.15	53	29	0.26	38	48	26	55	25	73	22
	N130	118	0.18	44	26	0.30	33	51	21	59	18	81	17
	N140	108	0.17	40	264	0.25	82	42	110	49	96	68	89
	N150	124	0.19	41	5	0.64	10	42	8	49	7	67	5
SINGER	S10	19	0.03	42	17	0.33	23	9	13	11	10	14	8
	S20	51	0.08	50	138	0.19	73	28	75	32	68	43	59
	S30	116	0.18	38	52	0.40	46	51	34	59	31	80	29
	S40	26	0.04	50	0	0.12	0	15	0	17	0	23	0
	S50	57	0.09	63	40	Kinematic Wave		33	18	38	15	50	11
	S60	19	0.03	74	48	Kinematic Wave		11	22	13	18	17	13
	S70	18	0.03	50	100	Kinematic Wave		8	33	10	25	13	18
	S80	24	0.04	63	26	Kinematic Wave		12	20	15	25	19	6
	S90	29	0.05	53	20	Kinematic Wave		15	7	18	13	24	4
	S100	30	0.05	81	14	Kinematic Wave		20	11	23	10	29	7
	S110	7	0.01	73	3	0.05	0	5	0	5	0	7	0
JOHN ADAMS	JA10	49	0.08	54	8	Kinematic Wave		29	4	34	3	45	2
	JA20	19	0.03	52	4	Kinematic Wave		11	0	13	0	17	0
	JA30	15	0.02	50	0	Kinematic Wave		8	0	10	0	13	0
	JA40	55	0.09	50	25	Kinematic Wave		27	8	32	7	44	5
	JA50	27	0.04	50	32	Kinematic Wave		12	9	15	15	20	5
	JA60	21	0.03	53	6	Kinematic Wave		13	8	15	7	19	0
	JA70	32	0.05	51	9	Kinematic Wave		18	6	21	5	28	4
	JA80	16	0.03	68	26	Kinematic Wave		7	17	9	29	12	20
LIVESAY	L10	83	0.13	28	211	0.75	59	27	59	31	55	38	31
	L20	132	0.21	18	350	1.11	53	35	46	42	45	58	45
	L30	111	0.17	38	19	0.50	21	44	13	51	11	69	10
	L40	33	0.05	29	61	0.69	38	10	25	12	33	16	23
PARK PLACE	P10	52	0.08	37	54	0.37	42	23	35	26	24	36	24
	P20	41	0.06	43	10	0.28	12	20	5	24	9	32	7
	P30	58	0.09	35	9	0.22	12	28	8	32	3	44	2
	P40	65	0.10	40	14	0.35	20	31	11	36	9	48	9
	P50	42	0.07	70	112	0.14	81	25	127	29	107	37	95
	P60	65	0.10	56	133	0.31	75	29	123	34	113	45	96

MIA = Mapped Impervious Area; EIA = Effective Impervious Area

TABLE 4.7 SIMULATED PEAK FLOWS ALONG THE MAJOR DRAINAGE COLLECTOR SYSTEMS

BASIN	NODE	DRAINAGE (ACRES)	AREA (MI^2)	EXISTING PEAK FLOW Q10	Q25	Q100	FUTURE PEAK FLOW Q10	%INCRS	Q25	%INCRS	Q100	%INCRS
CAUFIELD	10	45	0.07	15	17	24	20	25	23	35	32	33
	20	108	0.17	33	39	54	43	23	50	28	68	26
	30	164	0.26	44	52	72	66	33	76	46	104	44
	40	226	0.35	62	73	101	91	32	103	41	143	42
	50	308	0.48	88	102	142	125	30	144	41	195	37
	60	375	0.59	105	122	169	153	31	176	44	239	41
	70	499	0.78	134	158	217	205	35	236	49	319	47
	80	936	1.46	253	296	406	373	32	431	46	576	42
	90	58	0.09	25	28	38	29	14	33	18	45	18
	100	74	0.12	33	38	51	47	30	54	42	70	37
	110	288	0.45	93	108	147	128	27	147	36	194	32
	120	98	0.15	27	32	44	37	27	43	34	58	32
MUD	10	112	0.18	46	53	71	55	16	63	19	84	18
	20	189	0.30	63	73	99	91	31	104	42	138	39
	30	309	0.48	96	112	152	133	28	153	37	204	34
	40	458	0.72	120	141	192	192	38	222	57	296	54
	50	59	0.09	14	17	23	25	44	29	71	39	70
	60	533	0.83	137	160	219	219	37	253	58	340	55
CENTRAL PT	10	79	0.12	27	31	43	34	21	40	29	54	26
	20	66	0.10	24	28	38	29	17	34	21	46	21
	30	230	0.36	77	91	124	100	23	116	27	156	26
COFFEE	10	160	0.25	46	54	73	55	16	65	20	87	19
	20	211	0.33	60	71	97	73	18	85	20	115	19
	30	375	0.59	115	135	183	136	15	159	18	215	17
	40	445	0.70	135	158	214	156	13	183	16	247	15
TUMWATER	10	45	0.07	19	22	31	21	10	24	9	33	6
	20	77	0.12	25	29	40	36	31	42	45	56	40
	30	89	0.14	29	35	48	41	29	48	37	65	35
CLINTON	10	43	0.07	18	21	29	21	14	24	14	33	14
	20	57	0.09	21	24	34	25	16	29	21	40	18
SOUTH END	10	94	0.15	35	41	55	37	5	50	22	58	5
	20	201	0.31	69	80	108	74	7	86	8	115	6
	30	87	0.14	27	31	42	30	10	34	10	46	10
	40	321	0.50	105	122	165	114	8	132	8	178	8
	50	402	0.63	130	151	204	142	8	164	9	222	9
	60	476	0.74	152	177	241	167	9	194	10	262	9
	70	607	0.95	182	212	288	213	15	247	17	334	16

TABLE 4.7 (CONT.) SIMULATED PEAK FLOWS ALONG THE MAJOR DRAINAGE COLLECTOR SYSTEMS

BASIN	NODE	DRAINAGE (ACRES)	AREA (MI^2)	EXISTING PEAK FLOW Q10	Q25	Q100	FUTURE PEAK FLOW Q10	%INCRS	Q25	%INCRS	Q100	%INCRS
NEWELL	10	82	0.13	36	41	55	43	16	49	20	65	18
	20	189	0.30	63	73	99	83	24	95	30	127	28
	30	296	0.46	99	115	157	131	24	151	31	201	28
	40	435	0.68	149	173	235	197	24	227	31	302	29
	50	998	1.56	332	385	520	474	30	546	42	726	40
	60	107	0.17	56	64	84	59	5	67	5	87	4
	70	295	0.46	152	174	226	164	7	187	7	238	5
	80	1168	1.83	392	457	617	549	29	632	38	839	36
	90	1344	2.10	407	474	647	617	34	710	50	941	45
	100	214	0.33	80	94	129	99	19	114	21	155	20
	105	1558	2.43	482	562	767	706	32	814	45	1081	41
	110	1666	2.60	494	577	789	740	33	855	48	1143	45
	120	1790	2.80	529	620	849	774	32	893	44	1195	41
SINGER	10	19	0.03	8	10	13	9	11	11	10	14	8
	20	186	0.29	62	73	101	82	24	95	30	129	28
	25	212	0.33	69	82	112	94	27	110	34	148	32
	30	212	0.33	66	76	95	81	19	89	17	104	9
	35	269	0.42	81	92	116	106	24	119	29	143	23
	40	269	0.42	77	88	104	91	15	97	10	113	9
	50	288	0.45	81	93	109	97	16	103	11	119	9
	60	389	0.61	115	132	172	138	17	156	18	192	12
	70	18	0.03	6	8	11	8	25	10	25	13	18
	80	71	0.11	29	35	49	34	15	41	17	54	10
	90	29	0.05	14	16	23	15	7	18	13	24	4
	100	396	0.62	117	135	176	140	16	158	17	195	11
JOHN ADAMS	10	49	0.08	28	33	44	29	3	34	3	45	2
	20	83	0.13	46	55	74	47	2	56	2	75	1
	30	15	0.02	8	10	13	8	0	10	0	13	0
	35	218	0.34	101	118	155	106	5	123	4	159	3
	40	234	0.37	107	124	165	113	5	131	6	170	3
	50	55	0.09	25	30	42	27	7	32	7	44	5
	60	83	0.13	35	43	60	40	13	47	9	64	7
	70	103	0.16	47	56	78	51	8	60	7	83	6
	75	135	0.21	63	75	105	68	7	80	7	111	6
	80	135	0.21	58	68	88	62	6	72	6	91	3
LIVESAY	10	83	0.13	17	20	29	27	37	31	55	43	48
	20	215	0.34	41	48	68	60	32	71	48	98	44
	30	326	0.51	59	69	96	94	37	110	59	152	58
	40	359	0.56	65	77	108	103	37	121	57	167	55
PARK PLACE	10	52	0.08	17	21	29	23	26	26	24	36	24
	20	93	0.15	33	39	53	42	21	49	26	66	25
	25	151	0.24	57	68	93	67	15	79	16	107	15
	30	58	0.09	26	31	43	27	4	32	3	44	2
	40	216	0.34	85	100	138	99	14	117	17	157	14
	50	258	0.40	94	111	155	115	18	134	21	179	15
	60	323	0.50	103	122	170	140	26	164	34	216	27

4.5 DRAINAGE SYSTEM DEFICIENCIES

Historically observed drainage problem areas throughout the 12 major drainage basins in Oregon City were described in Chapter 3. The hydrological/hydraulic analyses of existing development conditions generally confirm the existence of these and other problems.

4.5.1 Existing Development Conditions

Drainage system deficiencies are assumed if the existing capacities of the various channels, culverts and pipes along the major drainage courses are found to be significantly less than those specified in the appropriate drainage planning criteria. Table 4.8 presents the comparison of existing capacity verse estimated peak flows for the major hydraulic structures that were inventoried throughout the Oregon City study area. The peak flows shown on Table 4.8 are based on existing conditions of development.

Based on an examination of the data presented on Table 4.8, the existing flow capacities of approximately 33 of the 59 major hydraulic structures inventoried are providing less than 90 percent of the desired capacity specified in the appropriate planning criteria. This means that approximately 50 percent of the existing hydraulic structures appear to be deficient in effectively passing peak flows under existing conditions of development. Of the 33 structures that appear to be deficient, approximately 26 of them or 79 percent can not pass the estimated 5-year peak flow. This indicates that overbank flooding problems resulting from deficient culvert and pipe capacities probably occur very frequently at some locations throughout the study area.

4.5.2 Ultimately Planned Development Conditions

Table 4.9 presents the comparison of existing structure capacity versus the estimated peak flows for the ultimately planned development conditions. According to the data presented in Table 4.9, approximately 38 of the 59 hydraulic structures that were inventoried appear to be deficient under ultimately planned (future) flow conditions. It is interesting to note that the impact of continued urbanization of the culverts and pipes that were inventoried was not as great as what might have been expected. For example, only 5 more structures are classified as deficient under future flow conditions when compared to existing flow conditions. This means that 80 percent of the structures that are deemed adequate under existing development conditions will continue to function adequately under ultimate development conditions.

All of the discussion regarding drainage system deficiencies have generally centered around major structure hydraulics and their associated capacities. One should recognize that many of the existing open channels, ditches or swales throughout the study

TABLE 4.8 PERFORMANCE OF MAJOR HYDRAULIC STRUCTURES UNDER EXISTING CONDITION OF DEVLOPMENT

STRUCTURE ID	LOCATION	SIZE (IN)	MAT'L	STRUCT. TYPE	PEAK FLOWS(CFS) FOR EXISTING LAND USE CONDITIONS BY RETURN INTERVAL 5YR	10YR	25YR	50YR	100YR	EXISTING CAPACITY (CFS)	% OF DESIRED	REMARKS
CA50.42	GLEN OAK 1600' E OF HWY 213	48	CMP	CULVERT	58	72	85	100	117	80	94	
CA50.45	PRIV. DRVWY 150' N OF GLEN OAK	30X30	CONC	CULVERT	63	78	90	108	126	45	50	DEFICIENT
CA50.50	GLEN OAK 1000'E OF HWY 213	24	CMP	CULVERT	71	88	102	122	142	21	21	DEFICIENT
CA60.60	GLEN OAK 500' E OF HWY 213	30	CONC	CULVERT	84	105	122	145	169	55	45	DEFICIENT
CA70.70	HWY 213 700'N OF GLEN OAK	72	CMP	CULVERT	109	134	158	187	217	350	222	
CA120.103	LAZY CREEK LANE	24	CONC	CULVERT	47	57	66	77	89	22	33	DEFICIENT
CA130.120	FALCON DRIVE	18	CMP	CULVERT	22	27	32	38	44	8	25	DEFICIENT
M10.10	LELAND 1300'OF WARNER-MILNE	24	CONC	CULVERT	38	46	53	62	71	21	40	DEFICIENT
M20.12	PRIV. DRVWAY 250' E OF LELAND	24	CONC	CULVERT	38	46	53	62	71	18	34	DEFICIENT
M20.15	KAEN STREET	24	CMP	CULVERT	45	55	63	74	85	24	38	DEFICIENT
M20.20	150'N OF WASSAIL	36	CONC	CLOSED	52	63	73	86	99	64	102	
M30.30	MEYERS 1400'NW OF GAFFNEY	42	CONC	CULVERT	79	96	112	132	152	90	80	DEFICIENT
M50.50	LELAND 800' S OF S HAVEN	12	CMP	CULVERT	11	14	17	20	23	4	24	DEFICIENT
CP10.10	CENTRAL 200'NE OF PARTLOW	30	CONC	CULVERT	22	27	31	37	43	50	161	
CP20.20	S PEASE 550' NE OF MCCORD	12	CMP	CULVERT	20	24	28	33	38	4	14	DEFICIENT
CP30.15	PARTLOW 650' SE OF S.CENTRAL	15	CONC	CULVERT	29	35	41	49	57	6	15	DEFICIENT
CP30.25	MCCORD 400' S OF S.PEASE	24	CMP	CULVERT	27	32	38	45	51	26	68	DEFICIENT
SE10.10	ROSE ROAD	18	CMP	CULVERT	16	20	23	27	31	9	39	DEFICIENT
SE30.20	250'N OF S END CT 50'W OF S END	24	CONC	CULVERT	57	69	80	94	108	22	28	DEFICIENT
SE40.22	S. END COURT @S. END	42	CONC	CULVERT	58	70	81	95	110	92	114	
SE40.25	S.END 100'W OF SALMONBERRY	48	ROCK	CULVERT	61	74	86	101	116	107	124	
SE40.28	DRVWY S OF SALMONBERRY @S END	48	CMP	CULVERT	61	74	86	101	116	68	79	DEFICIENT
SE40.39	300' DOWNSTREAM OF SE 50.30	18	CMP	CULVERT	23	28	32	37	43	8	25	DEFICIENT
SE50.30	S.END 500'SW OF FOREST RIDGE	21	CONC	CULVERT	22	27	31	36	42	16	52	DEFICIENT
N30.25	SW BEAVER CR RD @ JOHN INSKEEP	30	CONC	CULVERT	67	81	94	113	128	41	44	DEFICIENT
N30.30	UNDER LOOP TO RESIDENTIAL	48	CMP	CULVERT	81	99	115	135	157	67	58	DEFICIENT
N40.32	RELOCATED MAPLE LANE	48	CMP	CULVERT	81	99	115	135	157	145	126	
N40.38	ROAD TO MAPLE LANE CHURCH	48	CMP	CULVERT	97	119	138	162	188	115	83	DEFICIENT
N40.39	BYPASS @BEAVER CREEK	48	CMP	CULVERT	97	119	138	162	188	180	130	
N60.60	CROSSING WARNER-MILNE	36X22	CMP	CULVERT	48	57	65	75	86	37	57	DEFICIENT
N70.70	HWY 213 400'S OF BEAVER CK	2-36	CONC	CLOSED	128	152	174	201	226	160	105	
CO10.10	WARNER-PARROTT 800'E OF WDLN	2-24	CONC	CULVERT	26	32	37	43	50	50	135	
CO20.20	HAZELWOOD 500'W OF HARTKE	42	CONC	CULVERT	50	60	71	83	97	110	155	
CO30.22	BACK OF LOT 33 ON HAZELWOOD DR	24	CMP	CULVERT	58	68	81	94	110	17	21	DEFICIENT
CO30.25	HAZELWOOD 500' S OF VINE	42	CMP	CULVERT	63	77	90	102	123	90	100	
CO30.28	BARKER AVE @ HAZELWOOD	2-30	CONC	CULVERT	86	104	122	143	166	120	98	
CO30.30	S.END @ WATERFALL	36	CONC	CULVERT	95	115	135	158	183	85	63	DEFICIENT
CO40.32	FIFTH ST.	36	CONC	CULVERT	100	121	142	166	192	60	42	DEFICIENT
CO40.35	GANONG @ 3RD AVE.	42	CONC	CULVERT	103	125	147	172	199	113	77	DEFICIENT
CO40.36	DS OF CO40.35,UNDER HOUSE	36X48	ROCK	CULVERT	104	126	148	173	200	210	142	
CO40.37	3RD AVE.100'W OF GANONG	36	CMP	CLOSED	106	128	150	175	202	75	59	DEFICIENT

TABLE 4.8 (CONT.) PERFORMANCE OF MAJOR HYDRAULIC STRUCTURES UNDER EXISTING CONDITION OF DEVLOPMENT

STRUCTURE ID	LOCATION	SIZE (IN)	MAT'L	STRUCT. TYPE	PEAK FLOWS(CFS) FOR EXISTING LAND USE CONDITIONS BY RETURN INTERVAL 5YR	10YR	25YR	50YR	100YR	EXISTING CAPACITY (CFS)	% OF DESIRED	REMARKS
P10.10	HUNTER 500' S CLEVELAND	18	CMP	CULVERT	14	17	21	25	29	12	57	DEFICIENT
P20.15	CHARMAN 150' NORTH OF GAIN	42	CMP	CULVERT	17	20	25	29	34	47	188	
P20.20	@ FRONT 200' NORTH OF GAIN	30	CMP	CLOSED	27	33	39	46	53	39	100	
P40.21	CLEVELAND 60'E OF HARLEY	24	CONC	CULVERT	27	33	39	46	53	20	51	DEFICIENT
P40.23	HARLEY 60'N OF CLEVELAND	30	CMP	CULVERT	27	33	39	46	53	28	72	DEFICIENT
P40.28	INTER. OF APPERSON & LA RAE	36	CONC	CULVERT	45	53	63	75	88	78	124	
P50.50	UNDER BYPASS	60	CMP	CULVERT	77	94	111	131	152	125	113	
P60.60	TWIN PIPES UNDER ABERNETHY	57X38	CMP	CULVERT	83	102	121	144	168	340	281	
L40.40	REDLAND ROAD E OF BYPASS	50X31	CMP	CULVERT	52	65	77	92	108	95	123	
S10.10	HOLMES LANE 250' E OF LEONARD	21	CONC	CULVERT	7	8	10	11	13	24	240	
S40.30	PEARL EAST OF LINN	36	CMP	CULVERT	56	66	76	86	95	120	158	
S50.40	JACKSON 200' N OF LINN	24X42	ROCK	CULVERT	66	77	88	95	104	146	166	
S60.45	DOWNSTREAM OF JACKSON	36	CMP	CLOSED	66	77	88	95	104	55	71	DEFICIENT
S110.66	DOWNSTREAM OF WATERFALL	36X42	ROCK	CLOSED	97	115	132	152	172	100	87	DEFICIENT
JA40.50	INLET @ ORECITY HIGH SCHOOL	36	CMP	CLOSED	20	25	30	36	42	100	400	
JA70.80	SCHOOL CREEK @ MADISON ST	42	CMP	CULVERT	51	63	75	89	103	170	227	
JA80.35	JOHN ADAMS AND 14TH ST	42	CONC	CLOSED	88	108	127	150	174	110	102	
CLT10.10	UNDER S END @ CLINTON	18	CONC	CULVERT	15	18	21	25	29	14	67	DEFICIENT

EXISTING CAPACITY COMPUTATION IS BASED ON: 1)INLET CONTROL, 2)MAXIMUM ALLOWABLE HEADWATER,and 3)PIPE OPEN AND UNOBSTRUCTED BY SILT AND DEBRIS.

ENTRANCE TYPE :1-SQUARED EDGE WITH HEADWALL,2-GROOVE END WITH HEADWALL,3-GROOVE AND PROJECTING.
FOR CLOSED PIPE SYSTEMS THE CHARACTERISTICS OF THE INLET PIPE HAS BEEN SHOWN, HOWEVER THE LENGTH IS THE ENTIRE LENGTH OF THE MAINLINE CLOSED SYSTEM.

TABLE 4.9 PERFORMANCE OF MAJOR HYDRAULIC STRUCTURES UNDER FUTURE CONDITIONS OF DEVELOPMENT

STRUCTURE ID	LOCATION	SIZE (IN)	MAT'L	STRUCT. TYPE	PEAK FLOWS(CFS) FOR FUTURE LAND USE CONDITIONS BY RETURN INTERVAL 5YR	10YR	25YR	50YR	100YR	EXISTING CAPACITY (CFS)	% OF DESIRED	REMARKS
CA50.42	GLEN OAK 1600' E OF HWY 213	48	CMP	CULVERT	86	97	119	141	164	80	67	DEFICIENT
CA50.45	PRIV. DRVWY 150' N OF GLEN OAK	30X30	CONC	CULVERT	92	11	128	151	174	45	35	DEFICIENT
CA50.50	GLEN OAK 1000'E OF HWY 213	24	CMP	CULVERT	103	125	144	169	195	21	15	DEFICIENT
CA60.60	GLEN OAK 500' E OF HWY 213	30	CONC	CULVERT	126	153	176	206	239	55	31	DEFICIENT
CA70.70	HWY 213 700'N OF GLEN OAK	72	CMP	CULVERT	168	205	236	276	319	350	148	
CA120.103	LAZY CREEK LANE	24	CONC	CULVERT	64	76	86	99	113	22	26	DEFICIENT
CA130.120	FALCON DRIVE	18	CMP	CULVERT	31	37	43	51	58	8	19	DEFICIENT
M10.10	LELAND 1300'OF WARNER-MILNE	24	CONC	CULVERT	46	55	63	73	84	21	33	DEFICIENT
M20.12	PRIV. DRVWAY 250' E OF LELAND	24	CONC	CULVERT	46	55	63	73	84	18	29	DEFICIENT
M20.15	KAEN STREET	24	CMP	CULVERT	64	77	88	102	116	24	27	DEFICIENT
M20.20	150'N OF WASSAIL	36	CONC	CLOSED	76	91	104	121	138	64	70	DEFICIENT
M30.30	MEYERS 1400'NW OF GAFFNEY	42	CONC	CULVERT	111	133	153	178	204	90	59	DEFICIENT
M50.50	LELAND 800' S OF S HAVEN	12	CMP	CULVERT	20	25	29	34	39	4	14	DEFICIENT
CP10.10	CENTRAL 200'NE OF PARTLOW	30	CONC	CULVERT	29	34	40	47	54	50	125	
CP20.20	S PEASE 550' NE OF MCCORD	12	CMP	CULVERT	24	29	34	40	46	4	12	DEFICIENT
CP30.15	PARTLOW 650' SE OF S.CENTRAL	15	CONC	CULVERT	38	45	53	62	71	6	11	DEFICIENT
CP30.25	MCCORD 400' S OF S.PEASE	24	CMP	CULVERT	33	40	47	55	63	26	55	DEFICIENT
SE10.10	ROSE ROAD	18	CMP	CULVERT	17	20	23	27	31	9	39	DEFICIENT
SE30.20	250'N OF S END CT 50'W OF S END	24	CONC	CULVERT	61	74	86	100	115	22	26	DEFICIENT
SE40.22	S. END COURT @S. END	42	CONC	CULVERT	62	75	87	101	117	92	106	
SE40.25	S.END 100'W OF SALMONBERRY	48	ROCK	CULVERT	66	79	92	108	124	107	116	
SE40.28	DRVWY S OF SALMONBERRY @S END	48	CMP	CULVERT	66	79	92	108	124	68	74	DEFICIENT
SE40.39	300' DOWNSTREAM OF SE 50.30	18	CMP	CULVERT	25	31	35	41	47	8	23	DEFICIENT
SE50.30	S.END 500'SW OF FOREST RIDGE	21	CONC	CULVERT	24	30	34	40	46	16	47	DEFICIENT
N30.25	SW BEAVER CR RD @ JOHN INSKEEP	30	CONC	CULVERT	89	107	123	143	164	41	33	DEFICIENT
N30.30	UNDER LOOP TO RESIDENTIAL	48	CMP	CULVERT	109	131	151	175	201	67	44	DEFICIENT
N40.32	RELOCATED MAPLE LANE	48	CMP	CULVERT	109	131	151	175	201	145	96	
N40.38	ROAD TO MAPLE LANE CHURCH	48	CMP	CULVERT	131	157	181	210	240	115	64	DEFICIENT
N40.39	BYPASS @BEAVER CREEK	48	CMP	CULVERT	131	157	181	210	240	180	99	
N60.60	CROSSING WARNER-MILNE	36X22	CMP	CULVERT	51	60	69	79	90	37	54	DEFICIENT
N70.70	HWY 213 400'S OF BEAVER CK	2-36	CONC	CLOSED	139	164	187	215	238	160	98	
CO10.10	WARNER-PARROTT 800'E OF WDLN	2-24	CONC	CULVERT	32	38	45	52	60	50	111	
CO20.20	HAZELWOOD 500'W OF HARTKE	42	CONC	CULVERT	61	73	85	100	115	110	129	
CO30.22	BACK OF LOT 33 ON HAZELWOOD	24	CMP	CULVERT	69	83	96	113	130	17	18	DEFICIENT
CO30.25	HAZELWOOD 500' S OF VINE	42	CMP	CULVERT	77	92	107	126	145	90	84	DEFICIENT
CO30.28	BARKER AVE @ HAZELWOOD	2-30	CONC	CULVERT	103	123	144	167	195	120	83	DEFICIENT
CO30.30	S.END @ WATERFALL	36	CONC	CULVERT	113	136	159	186	215	85	53	DEFICIENT
CO40.32	FIFTH ST.	36	CONC	CULVERT	118	142	166	194	225	60	36	DEFICIENT
CO40.35	GANONG @ 3RD AVE.	42	CONC	CULVERT	122	146	171	200	231	113	66	DEFICIENT
CO40.36	DS OF CO40.35,UNDER HOUSE	36X48	ROCK	CULVERT	123	147	172	201	233	210	122	
CO40.37	3RD AVE.100'W OF GANONG	36	CMP	CLOSED	124	149	175	204	236	75	50	DEFICIENT

TABLE 4.9 (CONT.) PERFORMANCE OF MAJOR HYDRAULIC STRUCTURES UNDER FUTURE CONDITIONS OF DEVELOPMENT

STRUCTURE ID	LOCATION	SIZE (IN)	MAT'L	STRUCT. TYPE	PEAK FLOWS(CFS) FOR FUTURE LAND USE CONDITIONS BY RETURN INTERVAL 5YR	10YR	25YR	50YR	100YR	EXISTING CAPACITY (CFS)	% OF DESIRED	REMARKS
P10.10	HUNTER 500' S CLEVELAND	18	CMP	CULVERT	19	23	26	31	36	12	46	DEFICIENT
P20.15	CHARMAN 150' NORTH OF GAIN	42	CMP	CULVERT	22	27	31	36	42	47	152	
P20.20	@ FRONT 200' NORTH OF GAIN	30	CMP	CLOSED	35	42	49	57	66	39	93	
P40.21	CLEVELAND 60'E OF HARLEY	24	CONC	CULVERT	35	42	49	57	66	20	41	DEFICIENT
P40.23	HARLEY 60'N OF CLEVELAND	30	CMP	CULVERT	35	42	49	57	66	28	57	DEFICIENT
P40.28	INTER. OF APPERSON & LA RAE	36	CONC	CULVERT	56	67	79	92	107	78	99	
P50.50	UNDER BYPASS	60	CMP	CULVERT	95	115	133	155	180	125	94	
P60.60	TWIN PIPES UNDER ABERNETHY	57X38	CMP	CULVERT	117	140	163	187	216	340	209	
L40.40	REDLAND ROAD E OF BYPASS	50X31	CMP	CULVERT	84	103	121	143	167	95	79	DEFICIENT
S10.10	HOLMES LANE 250' E OF LEONARD	21	CONC	CULVERT	8	9	11	12	14	24	218	
S40.30	PEARL EAST OF LINN	36	CMP	CULVERT	70	81	89	98	104	120	135	
S50.40	JACKSON 200' N OF LINN	24X42	ROCK	CULVERT	81	91	97	105	113	146	151	
S60.45	DOWNSTREAM OF JACKSON	36	CMP	CLOSED	81	91	97	105	113	55	60	DEFICIENT
S110.66	DOWNSTREAM OF WATERFALL	36X42	ROCK	CLOSED	120	138	156	173	190	100	72	DEFICIENT
JA40.50	INLET @ ORECITY HIGH SCHOOL	36	CMP	CLOSED	22	27	32	38	44	100	370	
JA70.80	SCHOOL CREEK @ MADISON ST	42	CMP	CULVERT	52	62	72	83	91	170	236	
JA80.35	JOHN ADAMS AND 14TH ST	42	CONC	CLOSED	89	106	123	142	159	110	104	
CLT10.10	UNDER S END @ CLINTON	18	CONC	CULVERT	17	21	24	29	33	14	58	DEFICIENT

EXISTING CAPACITY COMPUTATION IS BASED ON: 1)INLET CONTROL, 2)MAXIMUM ALLOWABLE HEADWATER,and 3)PIPE OPEN AND UNOBSTRUCTED BY SILT AND DEBRIS.

ENTRANCE TYPE :1-SQUARED EDGE WITH HEADWALL,2-GROOVE END WITH HEADWALL,3-GROOVE AND PROJECTING.
FOR CLOSED PIPE SYSTEMS THE CHARACTERISTICS OF THE INLET PIPE HAS BEEN SHOWN, HOWEVER THE LENGTH IS THE ENTIRE LENGTH OF THE MAINLINE CLOSED SYSTEM.

area are now deficient or could be deficient under ultimate development conditions. Many of these open channel problems were discussed earlier in Chapter 3. Most of these problems will be addressed in the development of the capital improvement projects presented in Chapter 6.

Chapter 5
Drainage Plan Development

CHAPTER 5

DRAINAGE PLAN DEVELOPMENT

5.1 PLANNING OBJECTIVES

Specific planning objectives have been developed for the Oregon City Drainage Master Plan. The planning objectives are:

- Provide technically acceptable and reliable plans that seek to provide adequate drainage and flood protection.
- Provide plans that minimize the amount of maintenance required.
- Provide economically acceptable plans that solve the drainage problems in the most cost-effective manner.
- Provide institutionally acceptable plans that can be implemented effectively.
- Provide environmentally acceptable and aesthetically pleasing plans.
- Provide flexible plans that can be easily adapted to changes in planned development as the area is developed.

5.2 ALTERNATIVE APPROACHES

There are essentially four basic approaches used in the control of stormwater:

- Preserve the natural or existing drainage patterns and their associated flood plains.
- Immediate discharge of stormwater.
- Reduce present levels of discharge (detention).
- Do nothing.

These approaches will be addressed in the following paragraphs.

5.2.1 Preservation of the Natural Drainage System

Stormwater will always seek the easiest route downhill. Over time, gullies, swales, creeks and rivers form along the paths of least resistance to flow. The natural drainage course sizes itself to respond to the varying amounts of runoff it must

discharge; low flow channels form for storms of about 2-year recurrence intervals or less, and flood plains form for the severe storm events.

5.2.2 Immediate Discharge--Piped Systems

Piped drainage systems are the most common method of conveying runoff to streams in urban settings. Piped drainage systems require the least land and are the easiest to maintain of all drainage system types. Since velocities of flow in pipes are usually greater than in other drainage systems, peak flows from a piped system tend also to be higher than with other types of drainage facilities.

Because of this, when the piped system approach is used to manage stormwater, the underlying premise is that stormwater should be discharged from the site as quickly as possible. In some instances such an approach is very appropriate. Most urban streets are neither safe nor adequate for the conveyance of large amounts of stormwater--use of a curb and gutter drainage to an underground pipe network is a sound, proven approach for stormwater collection.

5.2.3 Regional Detention

The concept of detention is simple: store the excess upstream stormwater that would cause flooding problems downstream, and release the detained water at a slower rate than would occur without the flow restriction. The rate of release from the detention pond may be based on the capacity of downstream drainage structures. Alternatively, in a drainage basin where development or other land use changes are occurring, the rate of release may be based on runoff from a fixed level of development; e.g., a detention pond would be sized to store excess runoff anticipated with future development, and to release no more than peak flows associated with present development. This is desirable where land use changes may cause flows that greatly overtax portions of an existing system, thus requiring system replacement or major improvements at costs which are not acceptable to the community or developers. If space exists for regional detention basins, or if on-site detention systems (roof-tops, parking lots, etc.) are appropriate, they can sometimes greatly reduce the cost of needed improvements to downstream pipes and channels. This technique seldom solves drainage problems when used alone but should be considered in combination with other drainage improvements.

On-site detention is defined as runoff detention installed with each development to reduce the peak runoff to a certain mandated value. Regional detention basins are defined as basins which receive runoff from a large area, usually tens to hundreds of acres, and are large enough to attenuate the peak in that runoff. A policy of requiring on-site detention results in numerous small detention basins throughout the community. These basins are difficult to maintain and thus often do not function properly. For this reason, we have not considered on-site detention in this

plan. Regional detention basins are small enough in number that they can be maintained. Often regional detention basins can have multiple uses, such as for parks or parking lots. When such additional uses of a detention facility require regular maintenance, the basin is more likely to be maintained and function properly when needed. Regional detention basins have been considered in this plan as possible elements in the drainage system.

5.2.4 No Action

A final approach to drainage management in the basin is the "no action" approach. This alternative implies that no changes will be made to the present drainage system. It is included in the analysis for comparative purposes. Simply maintaining the existing system could result in continued damage and inconvenience caused by inadequately sized portions of the system that increases with increasing development.

5.3 DRAINAGE PLAN DEVELOPMENT PROCESS

The development of specific plans for each of the study's 12 major drainage basins is a complicated process. Working with the four basic approaches described in Section 5.2, each reach of the major drainage system was examined. Based on a number of factors such as estimated flood flows, existing channel or hydraulic structure capacity, topographical constraints, downstream impacts and other concerns, alternative approaches are formulated. The alternative approaches are then evaluated for their effectiveness in achieving the stated objectives. Decisions are made at every point along the major drainage system as to what appears to be the best approach to use to satisfy the planning objectives presented in Section 5.2 and the planning criteria that was presented in Section 4.2. The results of this process is the recommended drainage plan for each basin being considered.

Chapter 6

Recommended Master Drainage Plan

CHAPTER 6

RECOMMENDED MASTER DRAINAGE PLAN

6.1 GENERAL

Drainage management strategies for each major drainage basin were developed and evaluated using the criteria described in Chapter 5. The overall goal was to identify the capital improvement projects necessary to reduce damages from the appropriate design flood (i.e. 10-year or 25-year) to an acceptable level while recognizing funding limitations for drainage projects.

No modifications to the drainage system have been proposed unless a serious and recurring flood problem is identified or is anticipated because of projected changes in land use. Wherever possible, preservation of the natural drainage easement is proposed as the recommended management strategy. This not only provides better land use planning, but also allows for more efficient use of public funds.

Early identification of drainage system improvements can provide the opportunity to combine them with other projects. For example, upgrading a culvert because of a road widening project can also include the removal of a constriction in the drainage system. Thus an improvement to the transportation system could be combined with an improvement to the drainage system with little or no additional expense to the City.

6.2 DRAINAGE SYSTEM DESIGN CRITERIA

The various drainage system design criteria used to improve the minor and major drainage systems throughout Oregon City needs to be expanded and standardized. The City established a set of interim drainage design criteria in July, 1985. The general design consideration section and the runoff calculation section were essentially identical to the City of Salem's Stormwater Management Design Standards published in March, 1984. The only portion of the existing interim requirements that creates concern on the part of the drainage consultant is the use of on-site detention as a blanket policy for all new development.

6.2.1 On-Site Detention Policy

On-site stormwater detention is a complex issue. Making on-site stormwater detention mandatory for all future developments throughout Oregon City without any regard for the site-specific circumstances would probably be a serious mistake. This type of requirement can easily result in an unmanageable maintenance

problem and, in some cases, could actually increase downstream flood elevations. However, the drainage master planning effort has identified some instances where on-site detention can be useful to help reduce the impact of new development on already undersized downstream drainage systems.

If the financial plan that is presented in Chapter 7 is implemented, the need for a blanket on-site detention policy will be eliminated since the funding source for constructing the recommended capital improvements presented in Section 6.4 will be available.

If the capital improvement funding is not made available, the need for a blanket on-site detention policy for new development would be the only prudent approach to reducing downstream peak flows. However, the existing detention policy could act as a major deterrent to new development unless some changes were made to increase its flexibility.

If the recommended financial plan (see Chapter 7) is not implemented, the following on-site detention policy is recommended for all proposed commercial and industrial developments and for all residential developments greater than 5 acres. The policy should be as follows:

> If (a) the 25-year peak runoff rate from the proposed development combined with any upstream contributions from existing watershed conditions exceeds the capacity of any existing downstream channels, culverts, bridges or storm sewers, and (b) the City believes an undesirable flooding problem will occur and a significant portion of the problem results from runoff from the proposed development, or the proposed development is compounding an existing flooding problem; then the City may require the developer to choose one of the following two options:
>
> - The developer and the City solve the downstream problem by increasing the capacity of affected channels, bridges, culverts or storm sewers. The developer's percentage of the downstream improvement costs could be defined as the absolute difference in the 25-year peak flow from the proposed project before and after development divided by the 25-year peak flow for the entire watershed at the point of the proposed downstream improvement after development. If more than one downstream improvement is required, then the above formula will be applied at approximately the midpoint between all necessary improvements.
>
> - The developer satisfies the requirements of the City's on-site detention criteria as amended in this plan.

The options outlined above provide both the City and the developer with some flexibility in handling stormwater problems. They provide the City with opportunities to share some of the costs associated with improving minor and major drainage systems. They also provide a limit to the cost the developer should incur. If the City wants the developer to share in the cost of downstream improvements but the developer's share of that cost appears to be more than the cost of providing on-site detention, then the City could increase its share of the downstream improvements costs so the developer will comply.

Implementation of the criterion could prove to be cumbersome since it will require staff time to deal with each problem. However, the proper application of this stormwater detention criterion should provide the City with unique and constructive solutions to downstream drainage problems.

The fee or charges that the City collects from the developer in these situations has been defined as the "in-lieu of on-site detention fee". As an alternative to basing this fee on the complicated hydrological criterion presented above, it could be established as a charge per square foot of new impervious area created. Based on this criterion, our experience indicates that approximately 20 to 30 cents an impervious square foot is the cost of reducing the 25-year after development peak to the 10-year before development condition.

6.2.2 On-Site Detention Design Requirements

The City's existing interim drainage design criteria for on-site detention facilities is impossible to satisfy for a reasonable cost.

Item 1 of the City's Detention Basin Design Criteria states:

> "Each detention/retention basin facility shall be designed to accommodate a range of potential storm conditions considering all frequencies and durations of storm events up to and including the 100-year frequency storm as determined by the Director of Public Works/City Engineer; controlling releases in all frequency events to discharge rates in the "after development condition" to values which are equal to or less than the "before development" (existing) conditions. Unless otherwise determined by the Director of Public Works/City Engineer 5, 10, 25 and 100-year frequency storms for the 3, 6, and 24-hour duration shall be analyzed."

Item 7 of the City's General Design Considerations states:

> "Detention basins will be designed so that release rates downstream of the development do not exceed the 10-year frequency design storm flows for existing land use conditions. These release rates cannot increase the flooding conditions downstream. The detention basins may be either

off-line as a separate basin or on-line and designed as part of a swale system."

The only way to technically satisfy both of these requirements is to design the detention facility to reduce the 100-year "after development condition" peak flow to the existing 10-year before development peak. The cost of building a facility designed to satisfy this criterion would be extremely high.

We would recommend that the design criteria be amended to read:

Each detention facility shall be designed to reduce the 25-year recurrence interval peak flow based on after development on-site conditions to that peak flow that would have occurred during a 10-year recurrence interval event based on the before development on-site conditions.

6.2.3 Other Design Criteria

Appendix A presents the standardized drainage system design criteria that is recommended for use in both the minor and major drainage systems throughout Oregon City.

6.3 CAPITAL IMPROVEMENTS COST ESTIMATING

The recommended capital improvement projects are presented and described in Section 6.4. Construction cost estimates have been prepared for each of the capital improvement projects. The estimated unit costs are based on projected price levels for March, 1987, ENR index of 4633 (i.e. Seattle, Washington). All costs are estimates of probable costs and do not reflect changes that could include increasing labor costs, materials costs and phased construction dates. Costs at any time can be estimated by applying the ratio of the prevailing ENR index for Seattle to the 4633 index used in this report.

The typical unit costs used for pipe construction include excavation and export of material, granular bedding and backfill, cutting of asphalt, repaving of street, traffic control, pipe placement and rip-rap. If rock excavation is assumed to be required, it was estimated and shown separately.

The total program cost is estimated at approximately 150 percent of the total estimated construction cost and includes a 15 percent contingency, 17 percent for engineering and surveying, 4 percent for legal and administrative and 9 percent for financing. The last three additional costs are shown as separate items figured on the sum of the individual capital improvement construction costs for each basin or groups of basins presented.

6.4 RECOMMENDED CAPITAL IMPROVEMENT PROGRAMS

The recommended capital improvement programs will now be presented for each of the twelve major drainage basins described in

Chapter 3. All of the capital improvement projects presented below have also been shown on overlays to the City's 1 inch = 200 feet base maps. These maps have been provided to the City under separate cover. These maps provide the City with more detail on the specific nature of any given capital improvement project.

6.4.1 Caufield

The location of the proposed capital improvement projects (CIPs) in the Caufield basin are shown on Figure 6.1. Table 6.1 presents the specific design criteria and descriptions for each of the 5 CIPs throughout the Caufield basin. Table 6.1 also documents our estimate of construction costs and identifies a priority ranking for the importance of the timing of the recommended improvements. The priority ranking represents which 5-year time frame the project should most likely be constructed (i.e. first 5-years, second 5-years, etc.). The specific criteria used to rank the construction priority of each project is described later in Section 6.4.11.

The proposed CIPs for the Caufield basin are designed to increase the existing hydraulic capacity of five culverts that were identified in Table 4.9 as being deficient. No improvement was proposed for the existing culvert at CA50.45 although it too was found to be undersized. CA50.45 is located on private property and the potential damage to its overtopping was determined to be minor. As a general rule, improvements to hydraulic structures located on private property were not proposed unless the potential flooding problems associated with the existing facility were determined to be unacceptable.

Table 6.1 shows the estimated CIP construction cost for the 5 projects to be $26,990. A contingency of 15 percent brings the total estimated construction cost to $31,040. Other additional costs are shown as a percentage of the total estimated construction cost. The result is a total estimated capital improvement program cost of $40,350. This represents a capital improvement program cost of approximately $43 per acre throughout the Caufield basin.

6.4.2 Central Point

The location of the proposed CIPs in the Central Point basin are shown on Figure 6.2. Table 6.2 presents the specific design criteria and descriptions for each of the five CIPs throughout the Central Point basin. Table 6.2 also documents our estimate of construction costs and identifies a priority rating for the importance of the timing of the improvement.

The proposed CIPs for the Central Point basin are designed to increase the hydraulic capacity of the culverts that were identified as deficient in Table 4.9. In addition, two channel improvements are proposed to increase the capacity of these

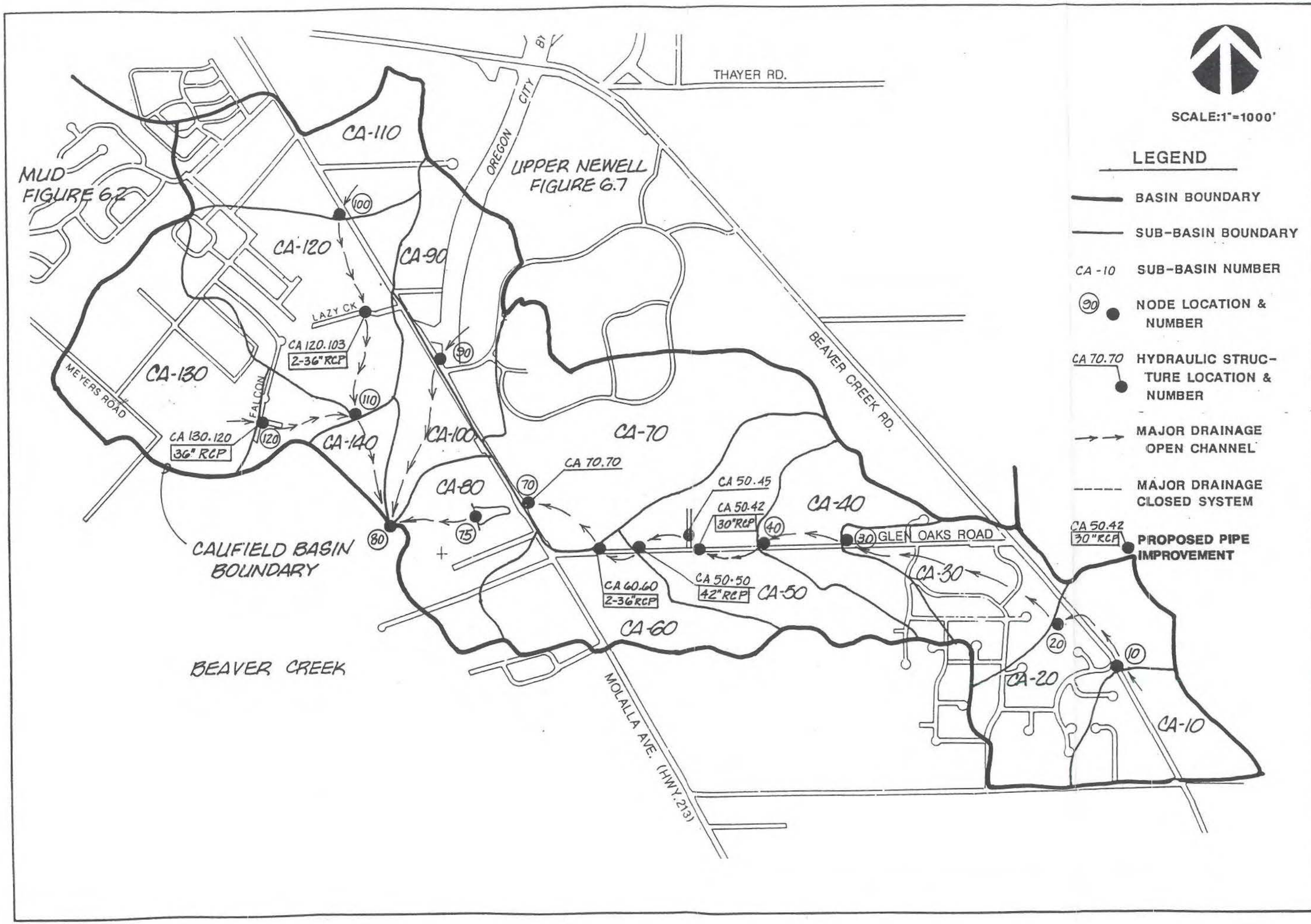

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FIGURE 6.1
CAPITAL IMPROVEMENT PROJECTS FOR THE CAUFIELD DRAINAGE BASIN

TABLE 6.1

PROPOSED CAPITAL IMPROVEMENTS FOR
THE CAUFIELD BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
CA50.42	Crossing at Glen Oaks Road, 1,600 ft. East of Molalla Avenue.	119	Minimum slope, 1.0% Headwater, 4.5 ft.	Extend the existing 48: CMP 8 ft. downstream and install a parallel 30" RCP				3
				48" CMP	8 L.F.	$109.82	$880	
				30" RCP	42 L.F.	$60.70	$2,550	
				Total			$3,430	
CA50.50	Crossing at Glen Oaks Road, 1,000 ft. East of Molalla Avenue.	144	Minimum slope, 1.0% Headwater, 4.5 ft.	Replace the existing CMP with 2 - 42" RCP. Improve Channel upstream and downstream				1
				42" RCP	84 L.F.	$83.50	$7,010	
				Grading	10 C.Y.	$12.00	$120	
				Total			$7,130	
CA60.60	Crossing at Glen Oaks Road, 500 ft. East of Molalla Avenue.	176	Minimum slope, 1.0% Headwater. 5.0 ft.	Extend the existing 30" RCP by 12 ft. downstream and install 2 - 36" RCP				1
				30" RCP	12 L.F.	$60.70	$730	
				36" RCP	84 L.F.	$71.94	$6,040	
				Grading	10 C.Y.	$12.00	$120	
				Total			$6,890	
CA120.103	Crossing at Lazy Creek Lane, 600 ft. West of Molalla Avenue	86	Minimum slope, 1.0% Headwater, 3.75 ft.	Replace the existing RCP with 2 - 36" RCP				3
				36" RCP	84 L.F.	$71.94	$6,040	
				Grading	30 C.Y.	$12.00	$360	
				Total			$6,400	

TABLE 6.1 (CONTINUED)

PROPOSED CAPITAL IMPROVEMENTS FOR
THE CAUFIELD BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
CA130.120	Crossing at Falcon Drive South of Vance Road	43	Minimum slope, 1.0% Headwater, 3.5 ft.	Replace the existing CMP with 36" RCP				3
				36" RCP	42 L.F.	$71.94	$3,020	
				Grading	10 C.Y.	$12.00	$120	
				Total			$3,140	
					CIP subtotal		$26,990	
					Contingency %15		$4,050	
				TOTAL ESTIMATED CONSTRUCTION COST			$31,040	
				Engineering and surveying @ 17%			$5,280	
				Legal and administration @4%			$1,240	
				Financing @9%			$2,790	
				TOTAL PROGRAM COST			$40,350	

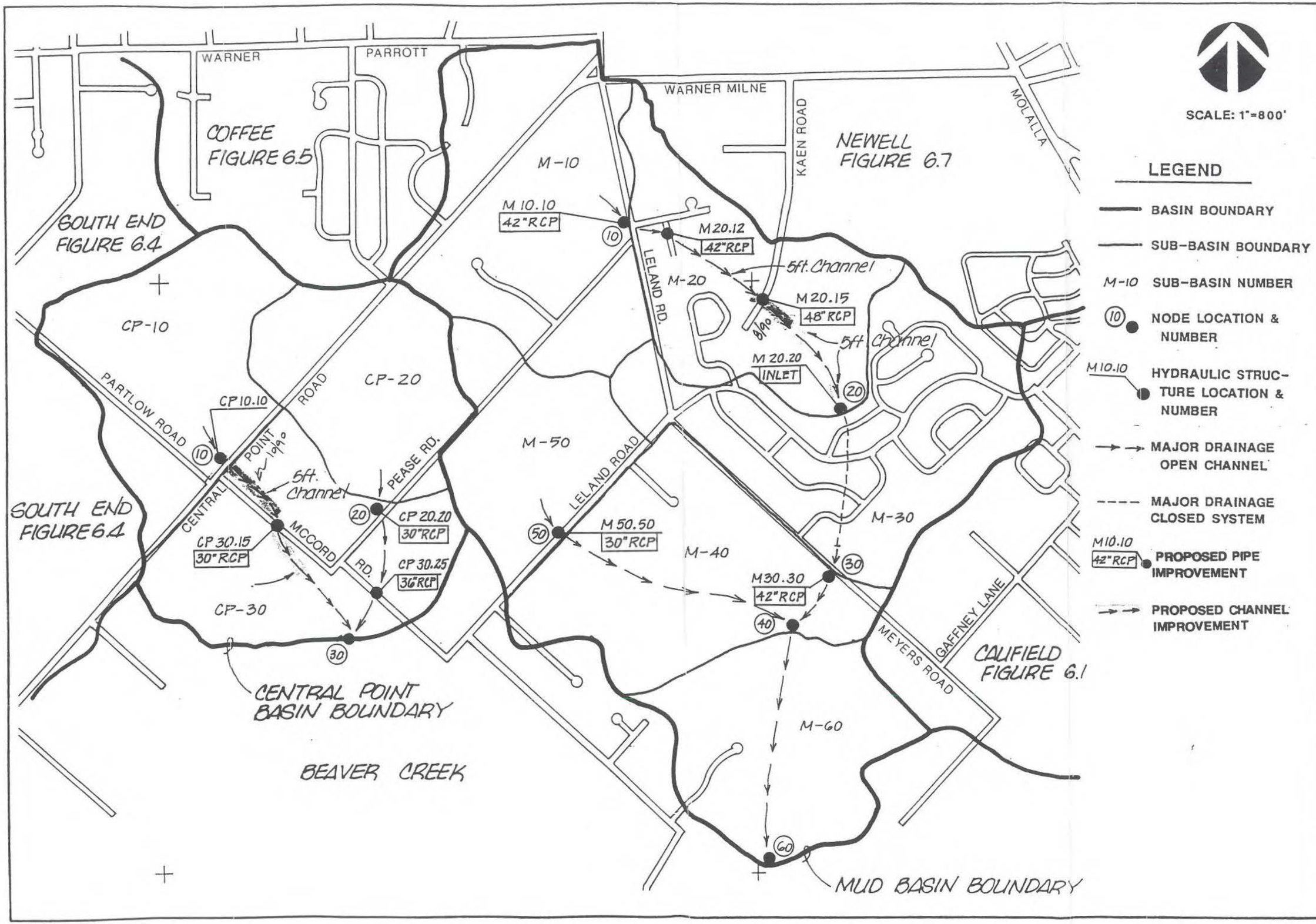

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FIGURE 6.2
CAPITAL IMPROVEMENT PROJECTS FOR THE
CENTRAL POINT AND MUD DRAINAGE BASINS

TABLE 6.2

PROPOSED CAPITAL IMPROVEMENTS FOR
THE CENTRAL POINT BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
CP10.10 to CP30.15	Major drainageway between Central Point Road and McCord Road.	40	Slope, 0.7%; Bottom width, 5.0 ft.; Minimum depth, 2.0 ft.; Side slope, 2:1	Construct 700 L.F. of grass lined channel.				3
				Channel	467 C.Y.	$10.00	$4,670	
CP30.15	Crossing at McCord Road, 650 ft. Southeast of Central Point Road.	53	Minimum slope, 0.7%; Headwater, 3.75 ft.	Replace the existing RCP with 36" RCP.				3
				36" RCP	42 L.F.	$71.94	$3,020	
CP30.15 to NODE 30	Major drainageway from McCord Road to 450 ft. downstream.	53	Slope, 0.3%; Bottom width, 5.0 ft.; Minimum depth, 2.0 ft.; Side slope, 3:1	Construct 450 L.F. of grass lined channel.				3
				Channel	300 C.Y.	$10.00	$3,000	
CP20.20	Crossing at Pease Road, 550 ft. Northeast of McCord Road.	34	Minimum slope, 1.0%; Headwater, 3.5 ft.	Replace the existing CMP with 30" RCP.				3
				30" RCP	42 L.F.	$60.70	$2,550	
CP30.25	Crossing at McCord Road, 400 ft. Southeast of Pease Road.	57	Minimum slope, 1.0%; Headwater, 4.0 ft.	Replace the existing CMP with 36" RCP.				3
				36" RCP	42 L.F.	$71.94	$3,020	
						CIP subtotal	$16,260	
						Contingency @ 15%	$2,440	
						TOTAL ESTIMATED CONSTRUCTION COST	$18,700	
						Engineering and surveying @ 17%	$3,180	
						Legal and administration @ 4%	$ 750	
						Financing @ 9%	$1,680	
						TOTAL PROGRAM COST	$24,310	

shallow channels. Table 6.2 shows that the total estimated construction cost with contingency for all five CIPs in Central Point is $18,700. The total estimated capital improvement program cost for Central Point is $24,310 or approximately $106 per acre.

6.4.3 Mud

Figure 6.2 also shows the location of the proposed CIPs in the Mud basin. The specific design criteria, descriptions, construction cost estimates and priority rankings for the seven CIPs throughout the Mud basin are shown in Table 6.3.

The proposed CIPs for the Mud basin are designed to increase the hydraulic capacity of the six culverts and one closed pipe that was identified as being deficient in Table 4.9. In addition, channel improvements along most of the major drainageway from node 10 to node 20 have also been proposed.

Several considerations have been given to increasing the existing capacity of the 1,120 foot long 36-inch RCP (i.e. M20.20) that begins approximately 150-feet north of Wassail. The pipe is only providing 70 percent of the desired capacity under future development conditions but it is considered adequate under existing conditions of development. The replacement of the pipe or the construction of a parallel pipe appear to be cost prohibitive. Detention could be provided upstream of the pipe but costly excavation would be required to achieve the desire reduction. The actual capacity problem for this structure is the result of poor hydraulic conditions at the inlet. Thus, the most cost effective solution is to reconstruct the inlet as shown in Figure 6.3. This funnel shaped design will increase the inlet capacity of the existing pipe and solve this potential flooding problem.

The total estimated construction cost with a contingency for all of the CIP's shown in Table 6.3 is $59,410. Engineering, administration and financing related costs bring the total estimated capital improvement program cost to $77,240 for the Mud Basin. This represents an average cost of approximately $145 per acre throughout the basin.

6.4.4 South End

The location of the proposed CIP's in the South End Basin are shown on Figure 6.4. Table 6.4 outlines the specific information associated with the six CIPs throughout the South End Basin.

Once again, all of the CIPs in the South End basin have been designed to increase the hydraulic capacity of several of the culverts that were identified in Chapter 4 as being deficient. Two channel improvements have also been proposed in the major drainageway that connects node 10 to node 40 via node 20.

No capital improvements have been proposed upstream of Rose Road (i.e. SE 10.10) because this area has been defined as a minor

TABLE 6.3

PROPOSED CAPITAL IMPROVEMENTS FOR THE MUD BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
M10.10	Crossing at Leland Road, 1,300 ft. South of Warner Milne Road.	63	Minimum slope, 1.0%; Headwater, 4.4 ft.; Upstream elevation, 450.5 ft. (Max.).	Replace existing RCP with 42" RCP, lower invert to maximize upstream development potential.				1
				42" RCP	170 L.F.	$83.50	$14,200	
M10.10 to M20.15	Major drainageway between Leland Road and Kaen Road.	75	Minimum slope, 0.5%; Bottom width, 5 ft.; Minimum depth, 2.3 ft.; Side slope, 2:1	Construct 1,300 L.F. of grass-lined channel. Replace existing RCP with 42" RCP at private drive-way (M20.12).				1
				42" RCP	30 L.F.	$83.50	$2,500	
				Channel	786 C.Y.	$10.00	$7,860	
				Total			$10,360	
M20.15	Crossing at Kaen Road, 2,000 ft. South of Warner Milne Road.	88	Minimum slope, 1.0%; Headwater, 4.5 ft.	Replace existing CMP with 48" RCP.				1
				48" RCP	75 L.F.	$109.82	$8,240	
M20.15 to M20.20	Major drainageway between Kaen Road and Wassail Lane.	88 to 104	Maximum slope, 1.0%; Bottom width, 5 ft.; Minimum depth, 2.5 ft.; Side slope, 2:1	Construct 600 L.F. of grass-lined channel downstream of Kaen Road and 250 L.F. up-stream of closed system inlet near Wassail Lane.				2
				Channel	514 C.Y.	$10.00	$5,140	
M20.20	At the closed system inlet, 150 ft. North of Wassail Lane.	104	NA	Construct and improved inlet (See Figure 6.3)				2
				Inlet		L.S.	$4,350	

TABLE 6.3 (CONTINUED)

PROPOSED CAPITAL IMPROVEMENTS FOR
THE MUD BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
M30.30	Crossing at Meyers Road, 1,400 ft. Northwest of Gaffney Lane.	153	Match slope of existing 42" RCP; Headwater, 5.3 ft.	Install a parallel 42" RCP.				1
				42" RCP	54 L.F.	$83.50	$4,510	
M50.50	Crossing at Leland Road, 900 ft. Northeast of Kalal Court.	29	Minimum slope, 1.0%; Headwater, 3.5 ft.	Replace the existing CMP with 30" RCP.				1
				30" RCP	80 L.F.	$60.70	$4,860	
						CIP subtotal	$ 51,660	
						Contingency @ 15%	$7,750	
						TOTAL ESTIMATED CONSTRUCTION COST	$59,410	
						Engineering and surveying @ 17%	$10,100	
						Legal and administration @ 4%	$2,380	
						Financing @ 9%	$5,350	
						TOTAL PROGRAM COST	$77,240	

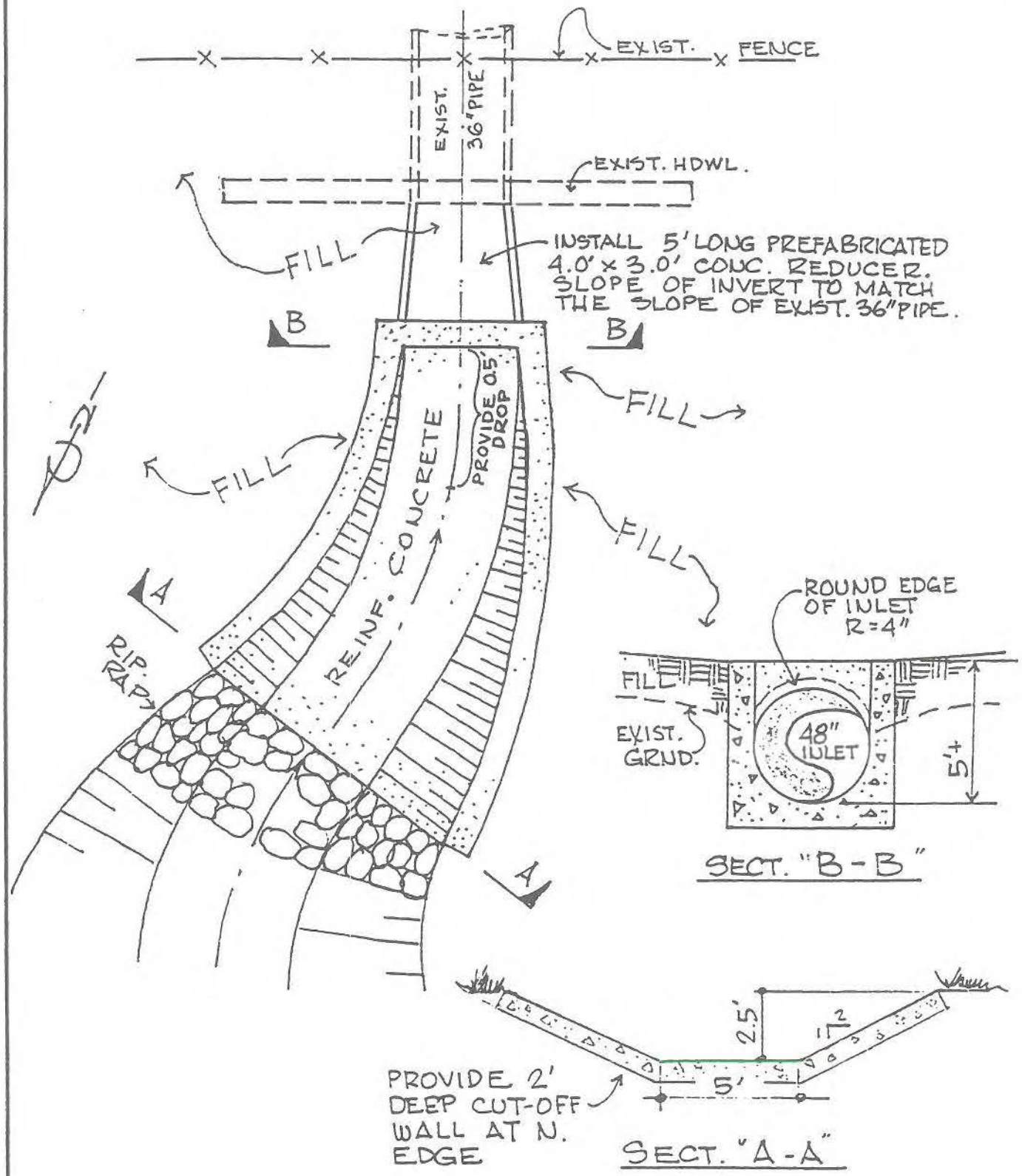

FIGURE 6.3

PROPOSED IMPROVEMENT OF EXISTING PIPE INLET TO M20.20

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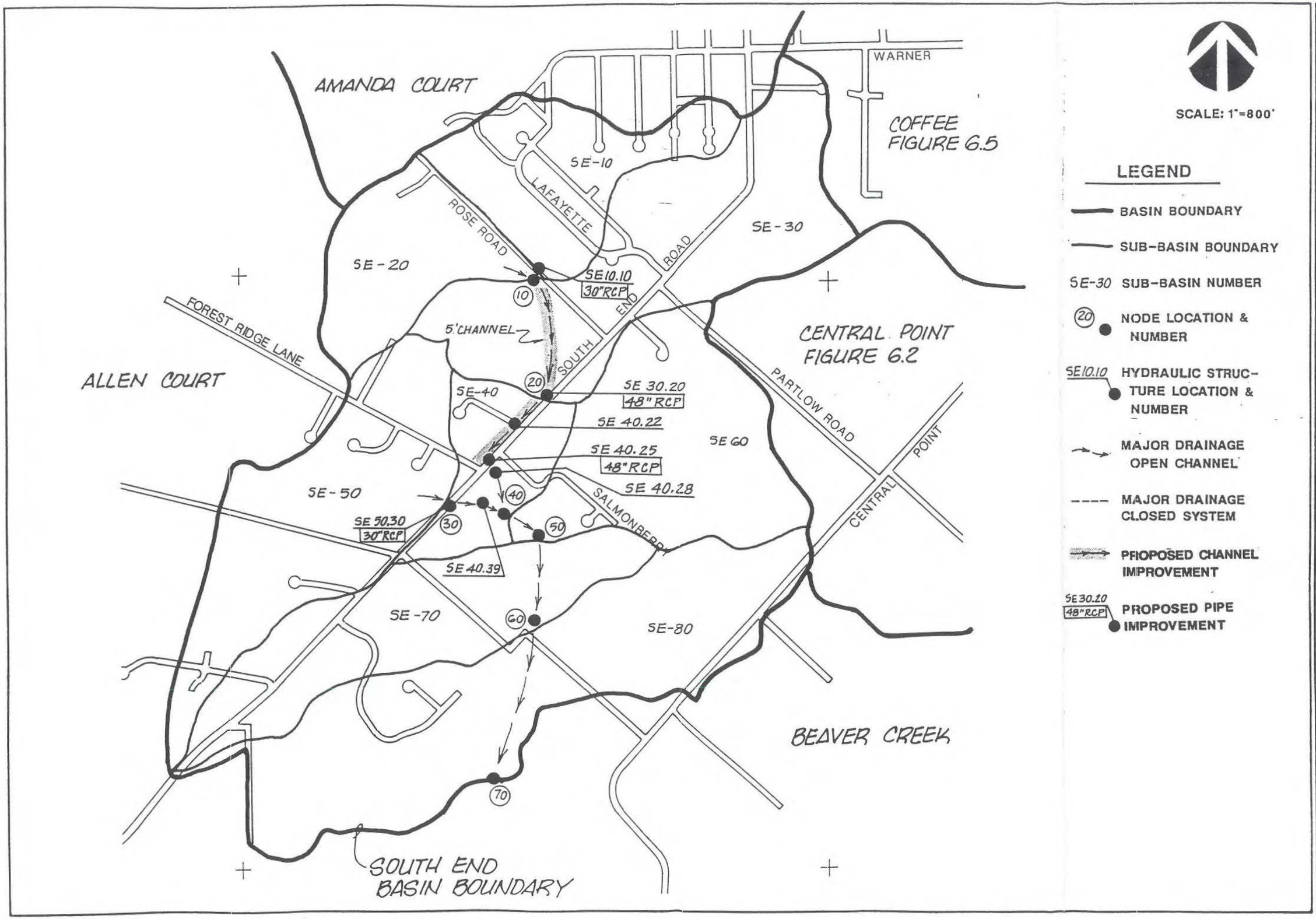

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FIGURE 6.4
CAPITAL IMPROVEMENT PROJECTS FOR THE SOUTH END DRAINAGE BASIN

TABLE 6.4

PROPOSED CAPITAL IMPROVEMENTS FOR
THE SOUTH END BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
SE10.10	Crossing at Rose Road, 900 ft. Northwest of South End Road	23	Minimum slope, 1.0%; Headwater, 3.5 ft.	Replace existing CMP with 30" RCP				1
				30" RCP	42 L.F.	$60.70	$2,550	
SE10.10 to SE30.20	Major Drainage Way between Rose Road and South End Road	50	Minimum slope, 0.33%; Bottom width, 5 ft.; Minimum depth, 2 ft.; Side slope, 2:1	Construct 1,450 L.F. of grass lined channel. Install a 36" RCP at private drive crossing				1
				36" RCP	30 L.F.	$71.94	$2,160	
				Channel	1,140 C.Y.	$10.00	$11,400	
				Total			$13,560	
SE30.20 to SE40.22	Crossing at South End Court to 350 ft. upstream along South End Road	86	Minimum slope, 0.33%; Headwater, 4.0 ft.	Construct grated inlet at SE30.20 and install 350 ft. of 48" RCP				4
				Inlet		L.S.	$3,000	
				48" RCP	350 L.F.	$109.82	$38,440	
				Total			$41,440	
SE40.22 to SE40.25	Major drainage way between South End Court and Forest Ridge Lane	92	Minimum Slope, 0.50%; Bottom width 2.0 ft.; Minimum depth, 3.3 ft. Side slope, 2:1	Construct 450 L.F. of grass-lined channel. Install 48" RCP at driveway crossings				4
				43" RCP	45 L.F.	$109.82	$4,940	
				Channel	400 C.Y.	$10.00	$4,000	
				Total			$8,940	

TABLE 6.4 (CONTINUED)

PROPOSED CAPITAL IMPROVEMENTS FOR
THE SOUTH END BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
SE40.25	Crossing at South End Road, North of Forest Ridge Lane	92	Minimum slope, 1.0%; Headwater, 5.0 ft.	Abandon the existing rock culvert and install 48" RCP at new location. Grade and rip-rap downstream to match existing				3
				48" RCP	42 L.F.	$109.82	$4,610	
				Channel		L.S.	$1,000	
				Total			$5,610	
SE50.30	Crossing at South End Road, 500 ft. SW of Forest Ridge Lane	34	Minimum slope, 1.0%; Headwater, 3.75 ft.	Replace the existing RCP with 30" RCP				3
				30" RCP	42 L.F.	$60.70	$2,550	
				CIP subtotal			$74,650	
				Contingency @ 15%			$11,200	
				TOTAL ESTIMATED CONSTRUCTION COST			$85,850	
				Engineering and surveying @ 17%			$14,590	
				Legal and administration @ 4%			$3,430	
				Financing @ 9%			$7,730	
				TOTAL PROGRAM COST			$111,600	

drainage system. However, the improvement of SE 10.10 and the drainageways and culverts downstream will certainly reduce the extent of the existing severe flooding problem that exists in the Oaktree Subdivision.

The total estimated construction cost with a contingency for all of the 4 CIPs shown in Table 6.4 is $85,850. The total program cost for this basin is estimated to be $111,600. This represents an areawide cost of approximately $184 per acre. This areal cost is the highest of all the southern drainage basins that contribute to Beaver Creek.

6.4.5 Coffee

Figure 6.5 shows the location of the proposed CIPs in the Coffee basin. The specific design criteria, descriptions, construction cost estimates and priority rankings for the seven CIPs throughout the Coffee basin are shown in Table 6.5.

The proposed CIPs for the Coffee basin are designed to increase the hydraulic capacity for five of the seven structures that were found to be deficient in Table 4.9. In addition, a minor channel improvement is proposed for the drainageway upstream of C010.10.

Special consideration has been given to improving the inlet characteristics of four existing culverts or pipes in the Coffee basin. Once again, this alternative appeared to be the most cost-effective in each of these cases (i.e. C030.30, C040.32, C040.35 and C040.37).

Figure 6.6 shows a sketch of the special inlet requirements for Coffee Creek's crossing of South End Road just downstream of the waterfall. The other three inlet designs should be similar to the one shown for M20.20 that was presented on Figure 6.3.

The total estimated construction cost for all of the CIPs shown in Table 6.5 is $168,430. Approximately 72 percent of this cost is associated with the proposed improvement to Coffee Creek's crossing of McLoughlin Boulevard which is a state highway. The high construction cost is due to the crossing of the railroad and the assumption that rock excavation will be required.

The total program cost for the Coffee basin is estimated to be $218,960. This represents an areawide cost of approximately $492 per acre. The areawide cost of improvements for the Coffee basin is the fourth highest in the study, exceeded only by Clinton, John Adams and Singer.

6.4.6 Clinton and Tumwater

Figure 6.5 also shows the location of the proposed CIPs for both the Clinton and Tumwater drainage basins. The improvements planned for the Tumwater basin are included in the proposed combined sewer separation program for this area. As a result, the costs of these drainage improvements have not been included in this

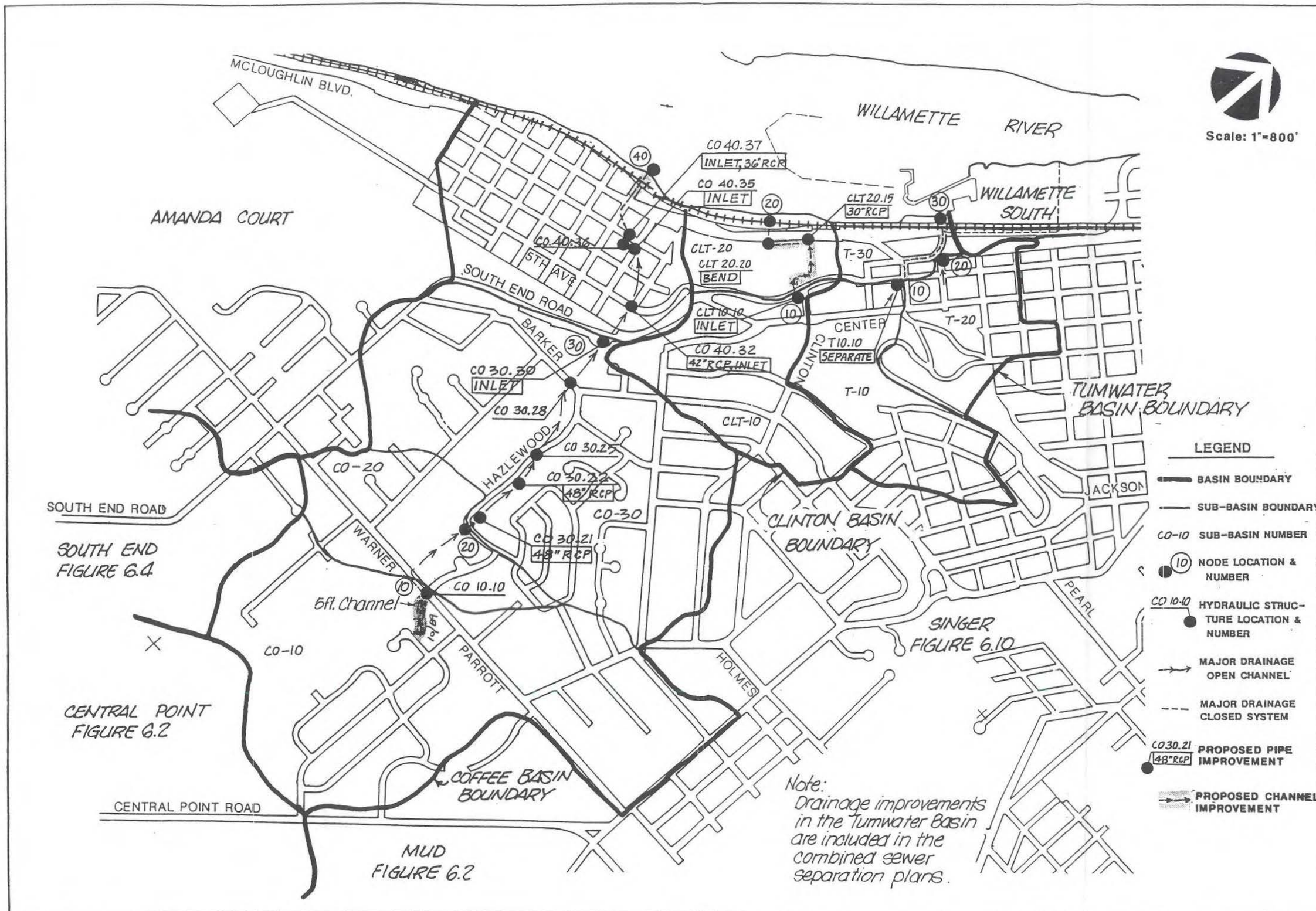

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FIGURE 6.5
CAPITAL IMPROVEMENT PROJECTS FOR THE COFFEE, CLINTON AND TUMWATER DRAINAGE BASINS

TABLE 6.5

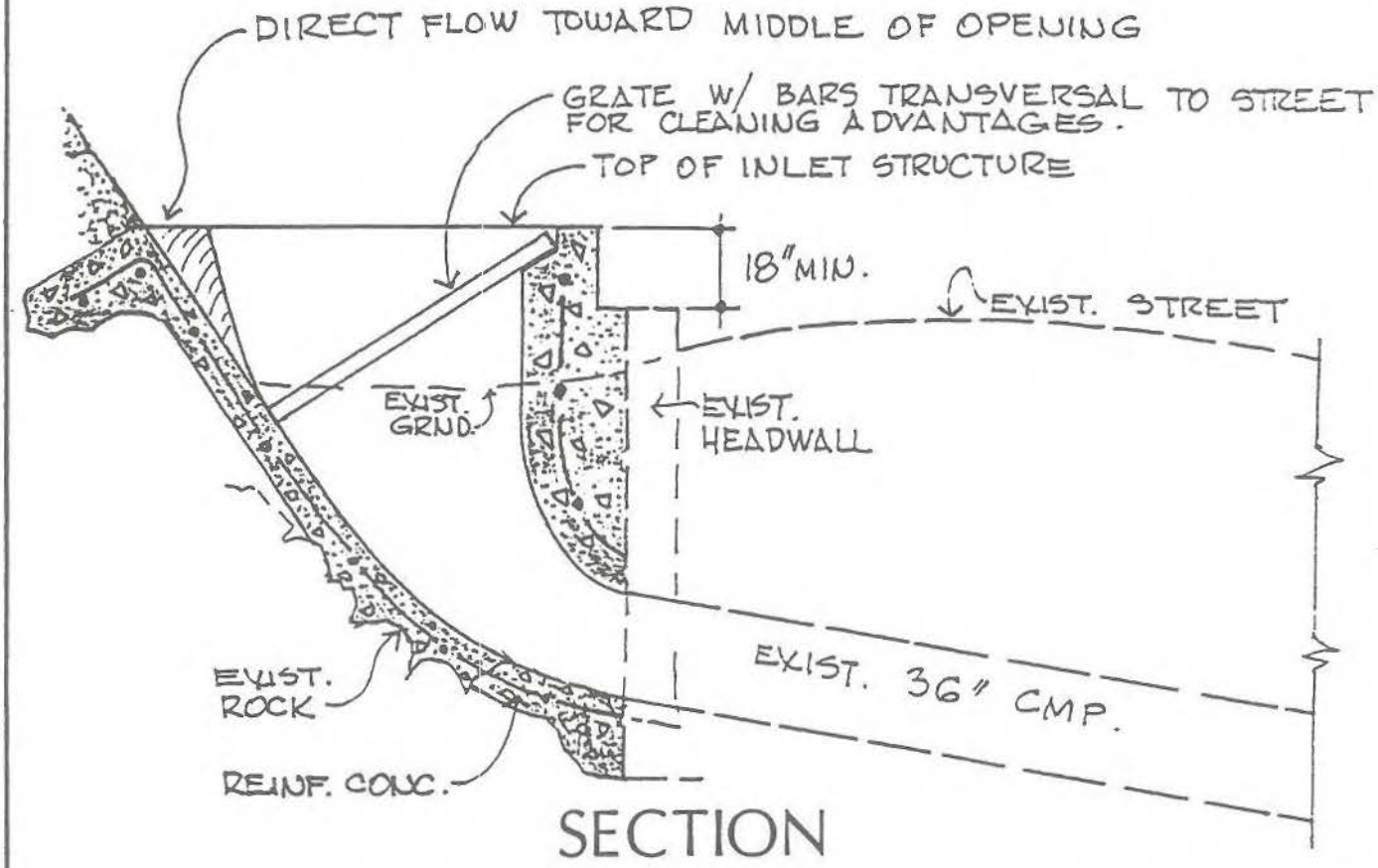
PROPOSED CAPITAL IMPROVEMENTS FOR THE COFFEE BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
Upstream of CO10.10	Drainage way between Joyce Court and Warner Parrott Road	45	Minimum scope, 0.3%; Bottom width, 5.0 ft.; Minimum depth, 2.3 ft.; Side slope, 2:1	Construct 450 L.F. of grass lined channel upstream of CO10.10				3
				Channel	417 C.Y.	$10.00	$4,170	
CO30.21	Constriction on private property, 100 ft. North of Hazelwood Drive	85	Minimum scope, 1.0%; Headwater, 4.0 ft.	Remove concrete walls and install 48" RCP				3
				48" RCP	40 L.F.	$109.82	$4,390	
				Restoration		L.S.	$1,000	
				Total			$5,390	
CO30.22	Constriction on private property, 500 ft. North of Hazelwood Drive	85	Minimum slope, 1.0%; Headwater, 4.8 ft.	Replace the existing CMP with 48" RCP				3
				48" RCP	100 L.F.	$109.82	$10,980	
				Restoration		L.S.	$1,000	
				Total			$11,980	
CO30.30	Crossing at South End Road	159	NA	Construct an improved inlet (See Figure 6.6)				1
				Inlet		L.S.	$6,500	
CO40.32	Crossing at 5th Avenue	166	Match slope of existing RCP Headwater, 4.5 ft.	Extend the existing 36" RCP 25 ft. downstream install a parallel 42" RCP and improved inlet (See Figure 6.3)				3
				36" RCP	25 L.F.	$71.94	$1,800	
				42" RCP	55 L.F.	$83.50	$4,590	
				Inlet		L.S.	$2,500	
				Total			$8,890	

TABLE 6.5 (CONTINUED)

PROPOSED CAPITAL IMPROVEMENTS FOR
THE COFFEE BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
C040.35	Crossing at Ganong Street, 50 ft. South of 3rd Avenue	171	NA	Construct an improved inlet (See Figure 6.3)				3
				Inlet		L.S.	$4,500	
C040.37 to Willamette River	Closed system from 3rd Avenue to the Willamette River	150	Match slope of existing 36" CMP Headwater, 6.5 ft.	Install a parallel 36" RCP from McLoughlin Blvd. to Willamette River, construct an improved inlet (See Figure 6.3)				4
				Inlet		L.S.	$4,500	
				36" RCP	300 L.F.	$62.60	$18,780	
				Rock excavation	300 C.Y.	$150.00	$45,000	
				Diversion manhole	1 ea.	$1,750.00	$1,750	
				Railroad crossing	50 L.F.	$700.00	$35,000	
				Total			$105,030	
						CIP subtotal	$146,460	
						Contingency @ 15%	$21,970	
						TOTAL ESTIMATED CONSTRUCTION COST	$168,430	
						Engineering and surveying @ 17%	$28,630	
						Legal and administration @ 4%	$6,740	
						Financing @ 9%	$15,160	
						TOTAL PROGRAM COST	$218,960	



SECTION

PROPOSED INLET STRUCTURE SHOULD BE A FUNNEL HAVING APPROX. 4'× MIN. 6' TOP OPENING, AND A SMOOTH TRANSITION TO 36" DIA. END OF EXIST. PIPE.
BARS SHOULD BE 6" O.C.
GRATE TO REST ON SHOULDERS ALONG 3 SIDES.

FIGURE 6.6

PROPOSED IMPROVEMENT TO EXISTING CULVERT INLET TO CO30.30

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study. However, the specific nature of both the existing combined sewer system and the proposed drainage improvements for this area can be found on Section 31 of the CIP map overlays provided to the City under separate cover.

Table 6.6 provides the specific detailed information on the four improvement projects proposed throughout the Clinton basin. Once again, the CIPs primarily addressed the need to increase the existing hydraulic capacity of several structures by improving their inlet characteristics. One culvert replacement was also proposed (i.e. CLT 20.15).

The total estimated construction cost for the four CIPs proposed in the Clinton basin is $23,440. Total program cost was estimated to be $30,470. The $530 per acre basin wide cost is the third highest of the twelve basins studied.

6.4.7 Newell

The location of the proposed CIPs in the Newell basin are shown on Figure 6.7. All three of the proposed improvements are located in the Upper Newell basin. Table 6.7 documents the nature and cost of these improvements.

The three CIPs deal with increasing the capacity of existing hydraulic structures that were found to be deficient. The improvements proposed at N70.70 are rather complicated and the reader should refer to Section 5 of the 1-inch to 200-foot overlay maps, if more detailed information is required.

Special concern should be given to the pathway that drainage is now following from node 10 to node 20. The drainageway is not well defined. It is a very shallow swale in the upland portion which enters a catch basin located on the Clackamas Community College campus. If development occurs in Sub-basin N10 or the eastern portion of N20, an alternative route to N30.25 may be desirable along Beaver Creek Road.

The total estimated construction cost for the three CIPs shown in Table 6.7 is $62,320. The total program cost is estimated to be $81,010 or approximately $45 per acre throughout the Newell basin.

6.4.8 Park Place and Livesay

The proposed CIPs for both the Park Place and Livesay drainage basins are shown on Figure 6.8 and tabulated on Table 6.8. Only one CIP is proposed for the largely undeveloped Livesay Basin. Three CIP's are proposed for the Park Place Basin.

Special attention has been given to the proposed improvements in the vicinity of the S. Cleveland and S. Harley intersection (i.e. P40.21 to P40.23). Figure 6.9 documents the complicated nature of the improvements for this area.

TABLE 6.6

PROPOSED CAPITAL IMPROVEMENTS FOR
THE CLINTON BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
CLT10.10	Crossing at South End Road, South of Clinton Street.	24	NA	Install a 24" x 18" reducer, minimum of 5 ft. long, and an entry structure with grate				1
				Inlet		L.S.	$3,000	
CLT10.10 to CLT20.20	Major drainageway near the PGE substation, 1,200 ft. South of Tumwater Street on McLoughlin Blvd.	24	Minimum slope, 0.5%; Bottom width, 5.0 ft.; Minimum depth, 2.0 ft.; Side slope, 1.5:1	Construct 350 L.F. of grass-lined channel				3
				Channel	148 C.Y.	$10.00	$1,480	
CLT20.15	Near the PGE sub-station, 1,200 ft. South of Tumwater Street on McLoughlin Blvd.	24	Minimum slope, 0.35%; Headwater, 4.0 ft.	Replace the existing 18" pipe with 30" RCP Install entry structure with grate				3
				Remove existing inlet structure		L.S.	$250	
				30" RCP	150 L.F.	$60.70	$9,100	
				Inlet		L.S.	$2,500	
				Rock excavation		L.S.	$1,000	
				Total			$12,850	
CLT20.20	Crossing at McLoughlin Blvd., 1,300 ft. Southwest of Tumwater Street	24	Minimum slope, 1.0%	Install a 30" x 24" reducer, and a 90 degree bend with a 5 ft. radius				3
				Remove existing junction structure		L.S.	$250	
				Manhole		$1,800.00	$1,800	
				90 degree bend & reducer		L.S.	$1,000	
				Total			$3,050	
						CIP subtotal	$20,380	
						Contingency @ 15%	$3,060	
						TOTAL ESTIMATED CONSTRUCTION COST	$23,440	
						Engineering and surveying @ 17%	$3,980	
						Legal and administration @ 4%	$940	
						Financing @ 9%	$2,110	
						TOTAL PROGRAM COST	$30,470	

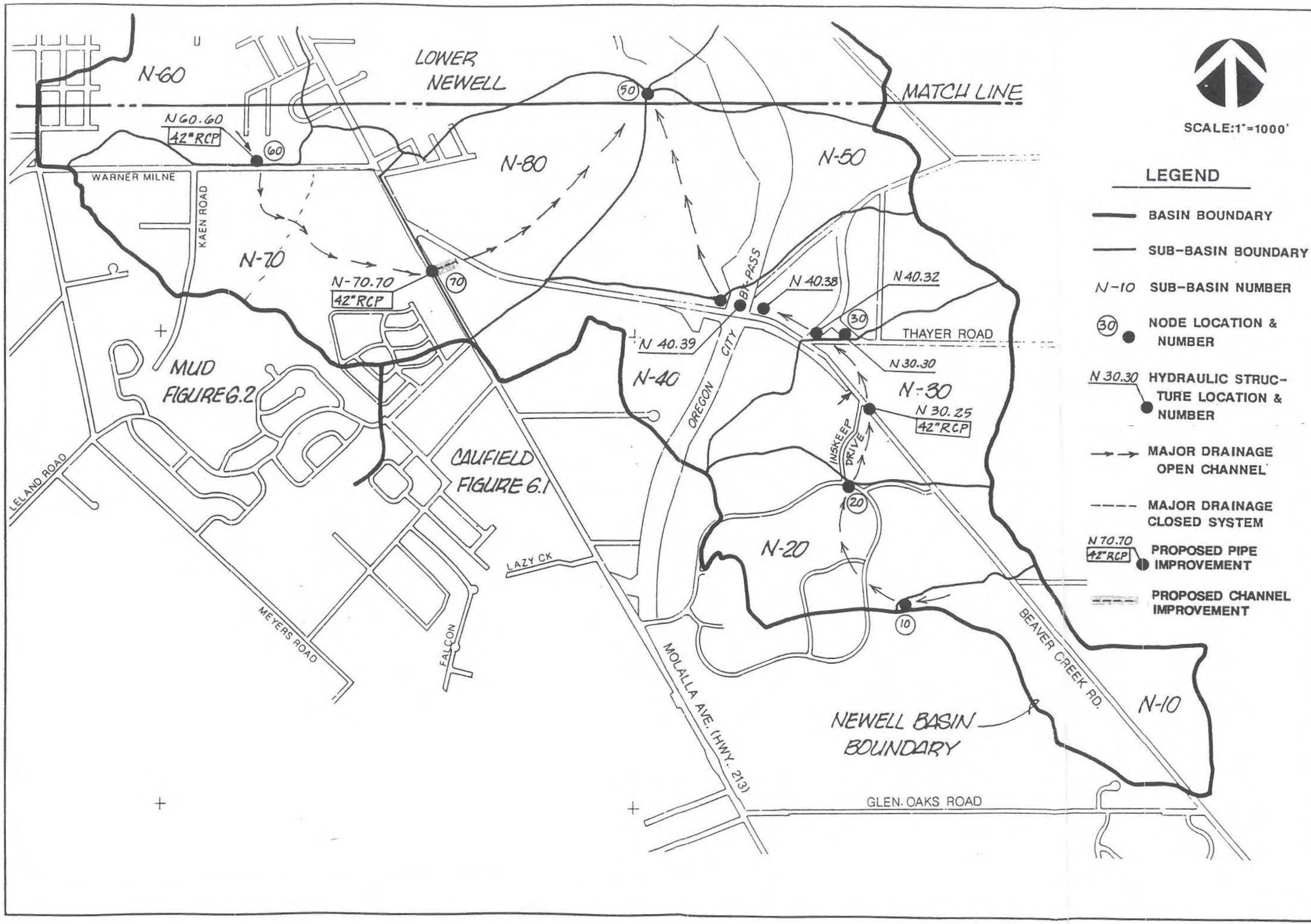

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FIGURE 6.7
AL IMPROVEMENT PROJECTS FOR THE
UPPER NEWELL DRAINAGE BASIN

TABLE 6.7

PROPOSED CAPITAL IMPROVEMENTS FOR
THE NEWELL BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
N30.25	Crossing at Beavercreek Road Southeast of Inskeep Drive	123	Minimum slope, 1.0% Headwater, 5 ft.	Install 42" RCP diagonally between the ends of the existing culvert.				1
				42" RCP	87 L.F.	$83.50	$7,260	
N60.60	Crossing at Warner Milne Road, 1,350 ft. West of Molalla Avenue	67	Minimum slope, 1.0% Headwater, 4.8 ft.	Replace the existing CMP with 42" RCP. Improve channel upstream and downstream.				1
				42" RCP	50 L.F.	$83.50	$4,170	
				Grading	30 C.Y.	$12.00	$360	
				Total			$4,530	
N70.70	Crossing at Molalla Avenue, 450 ft. South of Beavercreek Road.	187	Minimum slope, 1.0% Headwater, 6.0 ft. (Invert at 0.5 ft. below existing culverts)	Install 42" RCP from headwall to existing RCB at Beavercreek Road, extend the headwall and rip-rap at inlet.				4
				Headwall		L.S.	$1,000	
				42" RCP	470 L.F.	$83.50	$39,200	
				Manhole	1 ea.	$1200.00	$1,200	
				Traffic Control		L.S.	$1,000	
				Total			$42,400	
						CIP subtotal	$54,190	
						Contingency @ 15%	$8,130	
						TOTAL ESTIMATED CONSTRUCTION COSTS	$62,320	
						Engineering and surveying @ 17%	$10,590	
						Legal and administration @ 4%	$2,490	
						Financing @ 9%	$5,610	
						TOTAL PROGRAM COST	$81,010	

TABLE 6.8

PROPOSED CAPITAL IMPROVEMENTS FOR
THE PARKPLACE AND LIVESAY BASINS

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
Livesay Basin								
L40.40	Crossing at Redland Road, East of the Oregon City By-pass	121	NA	Construct an improved inlet (See Figure 6.3)				4
				Inlet		L.S.	$2,500	
Park Place Basin								
P10.10	Crossing at Hunter Avenue, 500 ft. South of Cleveland Street	26	Minimum slope, 1.0% Headwater, 3.5 ft.	Replace the existing CMP with 24" RCP				4
				24" RCP	42 L.F.	$47.00	$1,970	
				Grading	10 C.Y.	$12.00	$120	
				Total			$2,090	
P20.20	Crossing at Front Street, 200 ft. North of Gain Street	49	NA	Construct an improved inlet (See Figure 6.3)				4
				Inlet		L.S.	$2,500	
P40.21 to P40.23	Crossing at the Cleveland Street and Harley Street intersection	49	Minimum slope, 0.5%	Extend existing 30" culvert 90 ft. downstream, replace the Harley Street culvert with 36" RCP, install two inlets (See Figure 6.9)				4
				30" RCP	90 L.F.	$60.70	$5,460	
				36" RCP	50 L.F.	$71.94	$3,600	
				Ditch inlet	2 ea.	$500.00	$1,000	
				Grading		L.S.	$500	
				Total			$10,560	
						CIP subtotal	$17,650	
						Contingency @ 15%	$2,650	
						TOTAL ESTIMATED CONSTRUCTION COST	$20,300	
						Engineering and surveying @ 17%	$3,450	
						Legal and administration @ 4%	$810	
						Financing @ 9%	$1,830	
						TOTAL PROGRAM COST	$26,390	

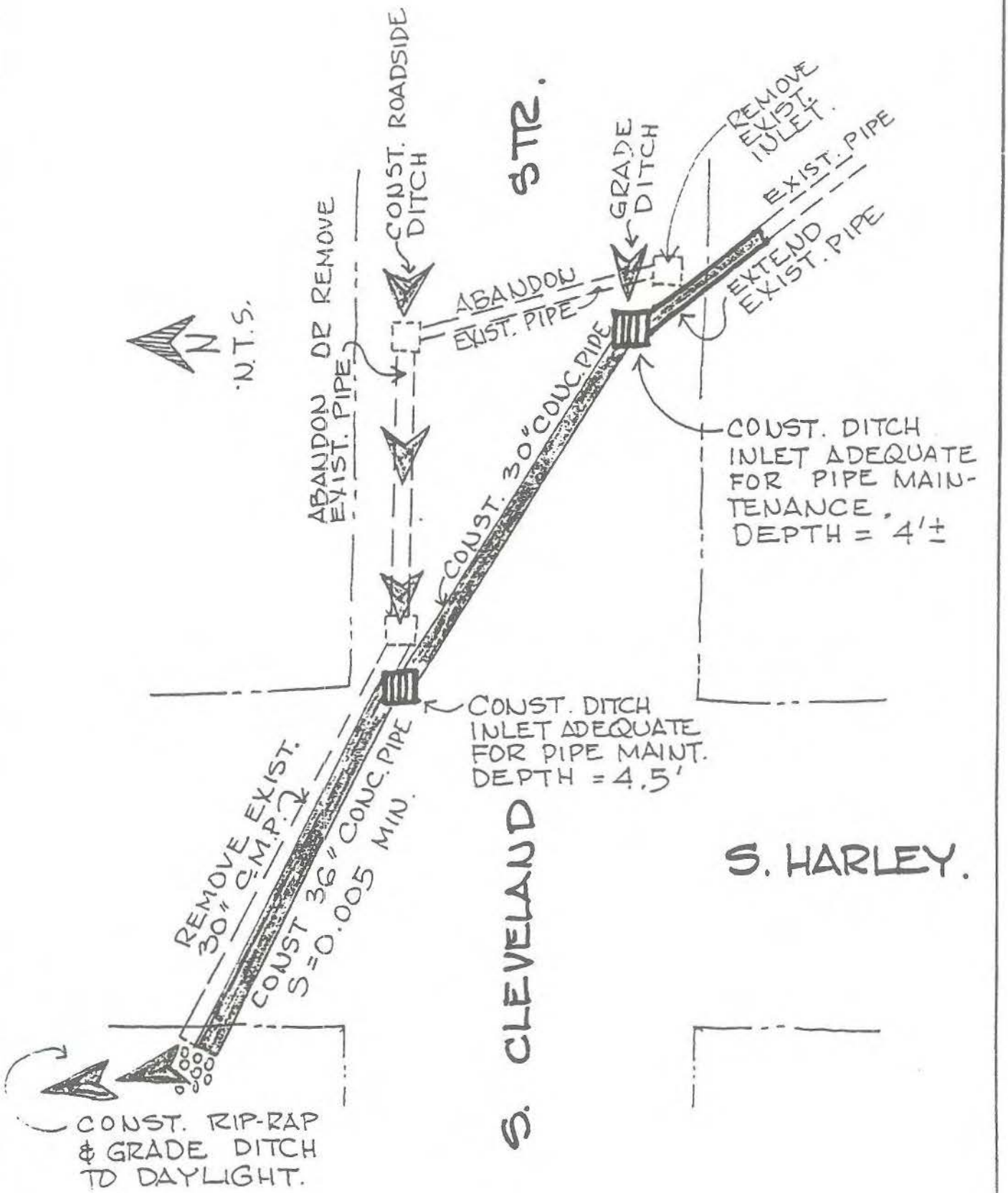

FIGURE 6.9

PROPOSED IMPROVEMENT AT THE INTERSECTION OF S. CLEVELAND & S. HARLEY ST. P40.21 TO P40.23

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Total CIP construction cost for both the Park Place and Livesay basins are estimated to be $20,300. The total program cost is $26,390 or approximately $39 per acre throughout both of these basins.

6.4.9 John Adams

Figure 6.10 shows the approximate location of the proposed CIP's in the John Adams basin. Table 6.9 provides a brief summary of the complex drainage improvements proposed for the John Adams basin.

The recommended drainage plan for the John Adams basin contains two major components. The first is the reduction of the peak flows on "High School" Creek by making use of the considerable detention storage available in the 50-foot deep ravine upstream of Madison Street. The second major component is to intercept the drainage from the old combined sewer system and divert it along John Adams Street to Abernethy Creek. This ensures that the severe hydraulic capacity problem in the existing 12th Street sewer will be solved. Each of these components will be discussed separately.

Figure 6.11 shows a sketch of a typical on-line detention outlet structure that is being proposed for use upstream of Madison Street. The primary engineering concern associated with this proposed improvement (i.e. JA70.80) is the ability of the Madison Street embankment to withstand the static force of approximately 12 feet of headwater during the hypothetical 100-year event. This level of water against a poorly placed fill of non-uniform material could be extremely dangerous. A geotechnical investigation of the nature of this old embankment is an absolute necessity before the planned detention improvement can be constructed.

Figure 6.12 shows a plan view of the complicated improvements proposed along John Adams Street from 12th Street to 15th Street. Note that the existing 18-inch pipe in 12th Street and the 21-inch pipe in 15th Street will be severed and plugged. Care must be taken during both the design and construction of these improvements to ensure that all the minor drainage systems throughout this area are intercepted and diverted to Abernethy Creek as planned.

The total estimated construction cost for the numerous improvements planned in the John Adams basin is $234,630. The total program cost is estimated to be $305,130. The basinwide cost is approximately $1,300 per acre which is the highest of the twelve basins studied.

6.4.10 Singer

Figure 6.10 also shows the approximate location of the proposed CIP's in the Singer basin. Table 6.10 provides a brief summary of the drainage improvements proposed for the Singer basin.

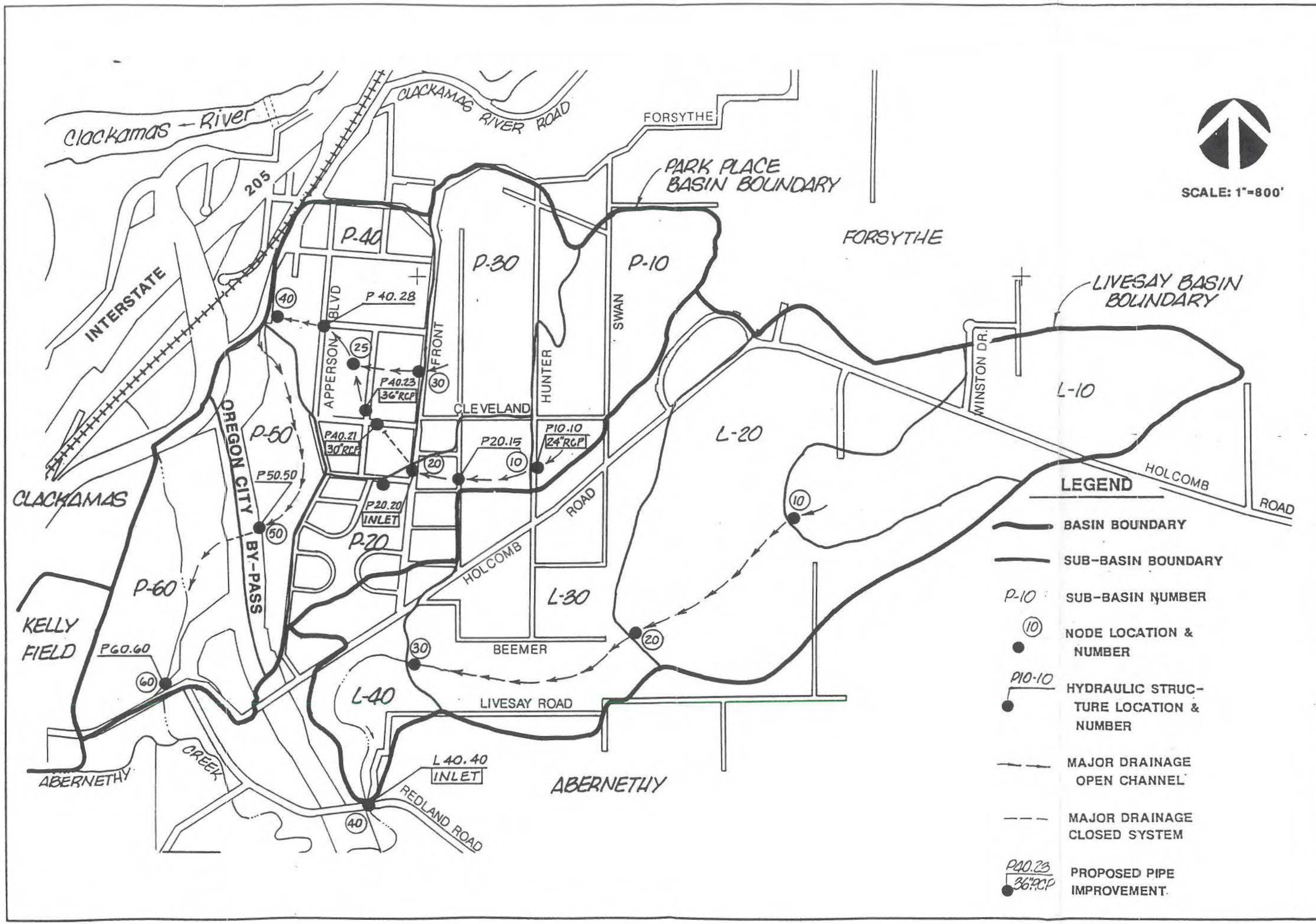

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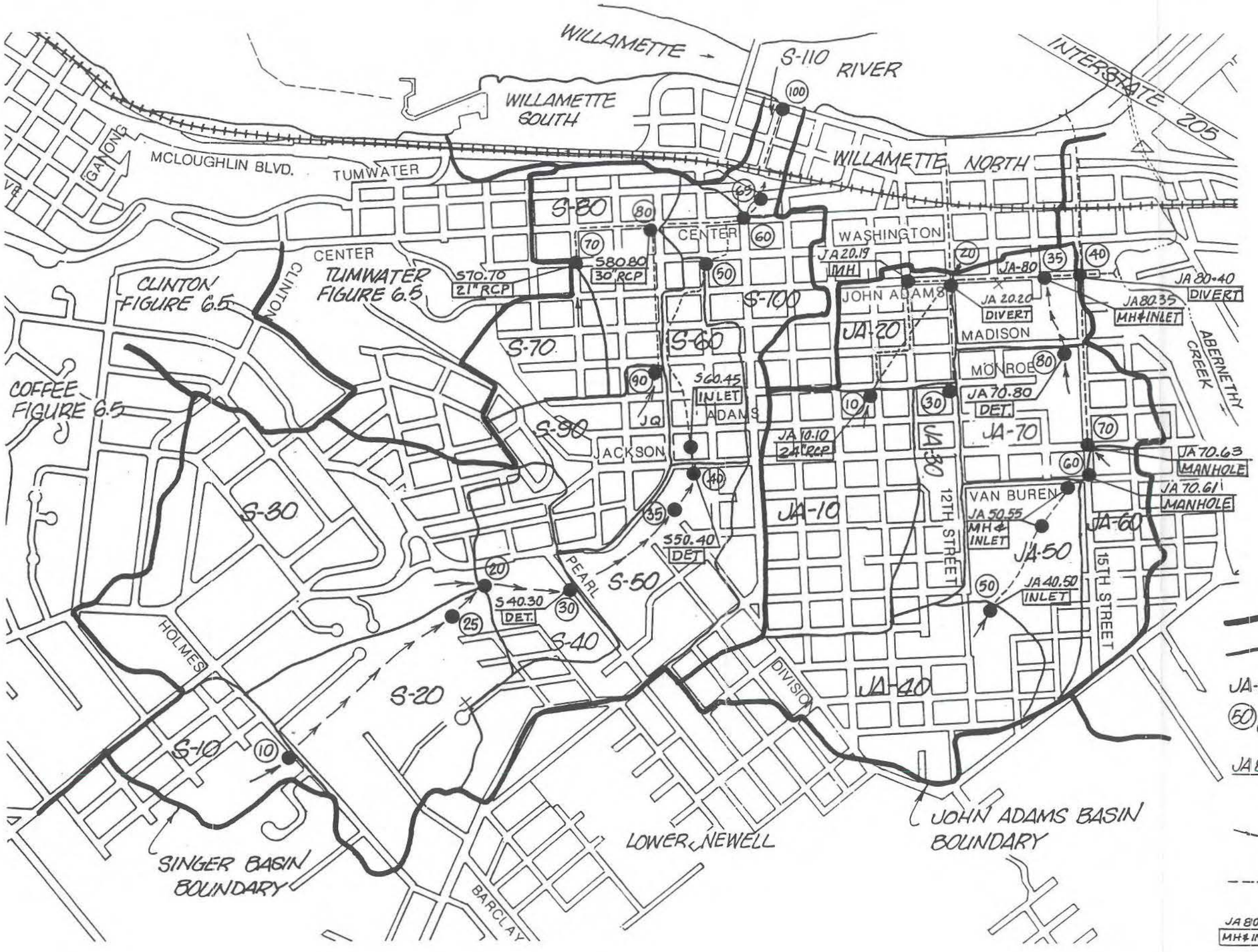
FIGURE 6.8
CAPITAL IMPROVEMENT PROJECTS FOR THE PARK PLACE AND LIVESAY DRAINAGE BASINS



Scale: 1"=800'

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otak man hanson kurahashi incorporated planners engineers surveyors 17355 sw boones ferry road lake oswego, oregon 97035 telephone (503) 635-3618

FIGURE 6.10
CAPITAL IMPROVEMENT PROJECTS FOR THE JOHN ADAMS AND SINGER DRAINAGE BASINS

TABLE 6.9

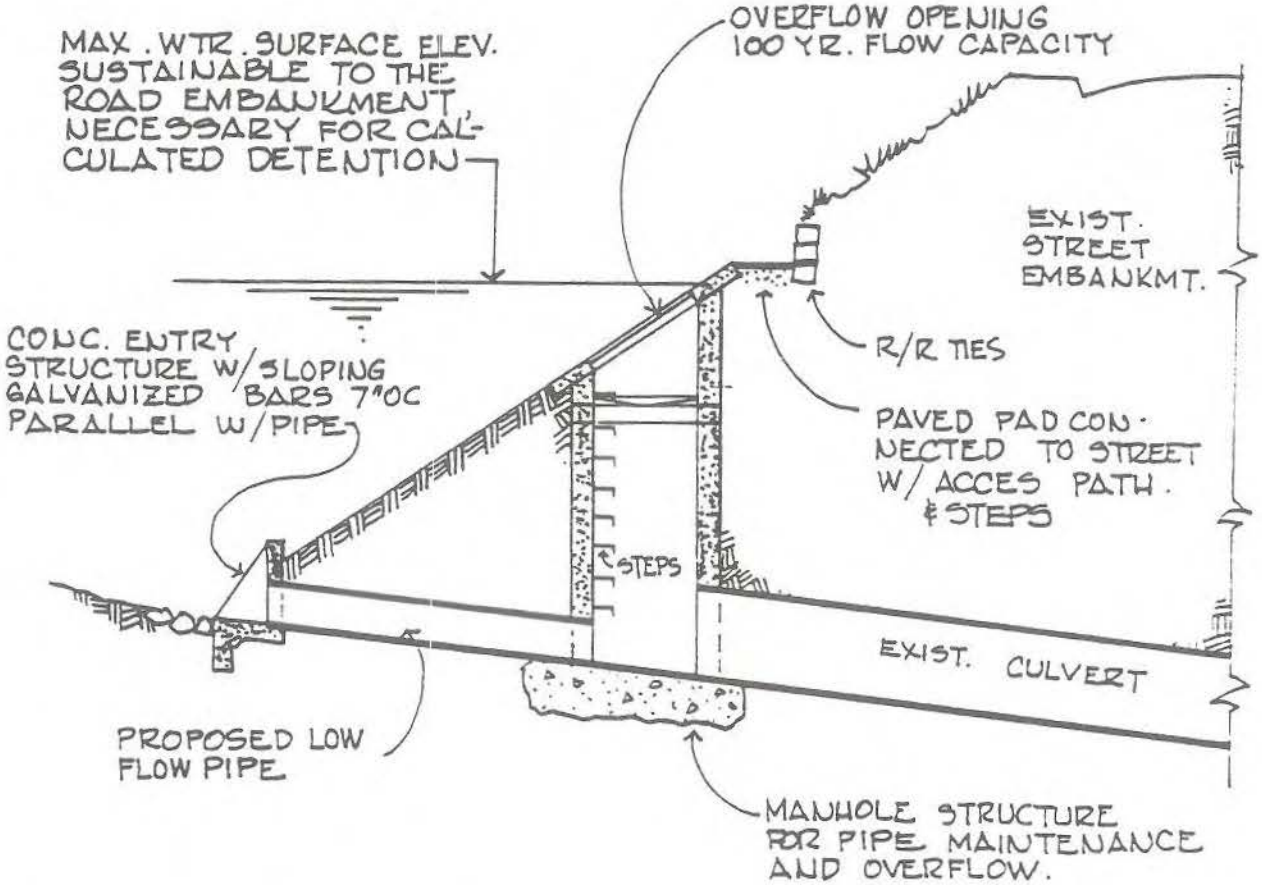
PROPOSED CAPITAL IMPROVEMENTS FOR
THE JOHN ADAMS BASIN

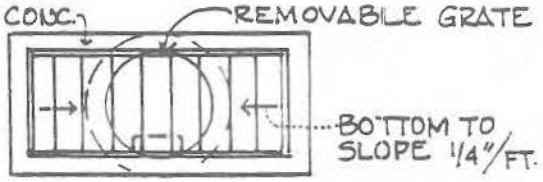
Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
JA40.50	Inlet at the South-east end of the school athletic field.	32	NA	Install an inlet with a sloping debris rack				1
				Inlet		L.S.	$2,500	
JA50.55	Crossing at Van Buren Street to 500 ft. upstream.	32	NA	Install catch basins and manholes at angle points of the existing 36" CMP				1
				Catch basin	3 ea.	$500	$1,500	
				Manhole	4 ea.	$1,800	$7,200	
				Total			$8,700	
JA70.61	At 15th Street, 70 ft. Northwest of Van Buren Street.	47	NA	Install a manhole at the angle in the exist-ing pipe system				1
				Manhole	1 ea.	$1,800	$1,800	
JA70.63	At 15th Street, 70 ft. Northwest of Jackson Street.	47	NA	Install a manhole at the angle in the exist-ing pipe system				1
				Manhole	1 ea.	$1,800	$,1800	
JA70.80	Crossing at Madison street, 200 ft. South-west of 15th Street.	72	Maximum detention Depth, 15 ft.; Overflow capacity, 110 cfs	Construct a detention structure (See Figure 6.11)				1
				18 ft. Manhole		L.S.	$6,500	
				Overflow inlet		L.S.	$4,500	
				24" Low flow pipe	25 L.F.	$100	$2,500	
				Low flow inlet		L.S.	$6,000	
				Access path			$2,500	
				Rip-rap			$1,000	
				Total			$23,000	

TABLE 6.9 (CONTINUED)

PROPOSED CAPITAL IMPROVEMENTS FOR
THE JOHN ADAMS BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
JA20.20	Intersection of John Adams Street and 12th Street.	46	Minimum slope, 4.0%	Construct the improvements to divert flow to the Northeast (See Figure 6.12)				1
				Plug N.W. outlet		L.S.	$200	
				24" RCP	300 L.F.	$47	$14,100	
				Manhole	1 ea.	$1,800	$1,800	
				Manhole connection		L.S.	$1,000	
				Existing utilities		L.S.	$2,000	
				Rock excavation	67 C.Y.	$150	$10,050	
				Total			$29,150	
JA80.35	Intersection of John Adams Street and 14th Street.	96	NA	Construct a structure with a debris rack at the inlet to the 42" pipe (similar to Figure 6.13), construct manholes to connect existing pipes (See Figure 6.12)				1
				Inlet		L.S.	$3,000	
				Grading		L.S.	$1,000	
				Oversize manhole	1 ea.	$3,000	$3,000	
				Standard manhole	1 ea.	$2,500	$2,500	
				Total			$9,500	
JA80.40	Intersection of John Adams Street and 15th Street.	105	NA	Construct the improvements to divert flow to the Northeast (See Figure 6.12)				1
				Oversize manhole	1 ea.	$3,500	$3,500	
				24" RCP	40 L.F.	$47	$1,880	
				Plug NW outlet		L.S.	$200	
				Total			$5,580	





NOT TO SCALE

PLAN VIEW OF OVERFLOW OPENING.

BARS TO BE 1 1/2" x 1/4" GALV. STEEL 7" O.C. PARALLEL W/ PIPE.

FIGURE 6.11

TYPICAL DETENTION OUTLET STRUCTURE

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[illegible] E. 8th Street Vancouver WA 98660 206-695-0357
10258 Main Street #215 Bellevue WA 98004 206-455-5340

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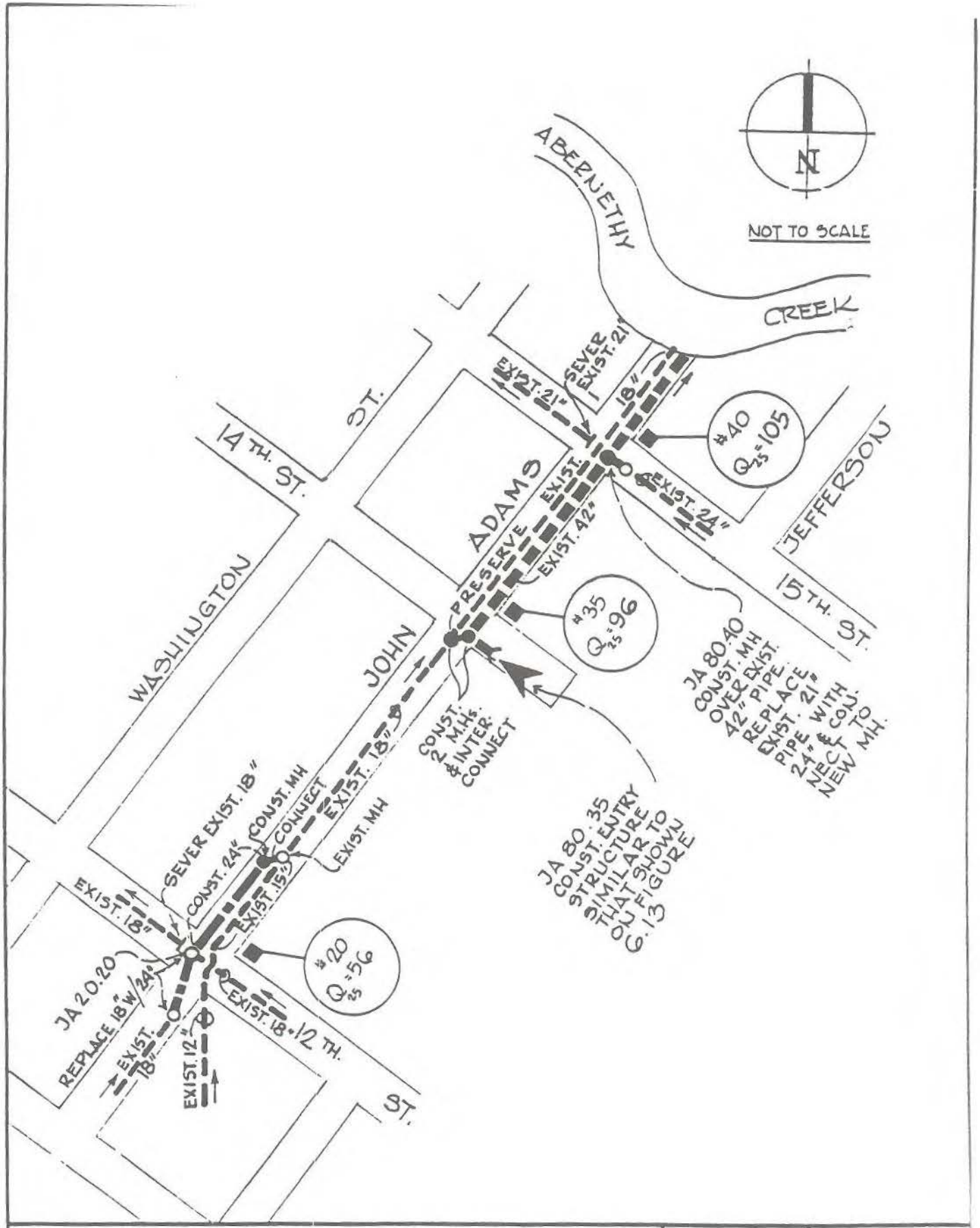

FIGURE 6.12

PROPOSED IMPROVEMENTS IN JOHN ADAMS STREET FROM 12TH ST. TO 15TH ST.

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TABLE 6.10

PROPOSED CAPITAL IMPROVEMENTS FOR
THE SINGER BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
S40.30	Crossing at Pearl Street, 150 ft. West of Linn Avenue	110	Maximum detention depth, 17.5 ft.; Overflow capacity, 120 cfs	Construct a detention structure (See Figure 6.11)				1
				18 ft. Manhole		L.S.	$5,000	
				Overflow outlet		L.S.	$3,500	
				24" Low flow pipe	25 L.F.	$75.00	$1,870	
				Low flow inlet		L.S.	$4,500	
				Access path		L.S.	$1,000	
				Rip-rap		L.S.	$750	
				Total			$16,620	
S50.40	Crossing at Jackson Street, 200 ft. Northeast of Linn Avenue	119	Maximum detention depth, 18.0 ft.; Overflow capacity, 150 cfs	Construct a detention structure (See Figure 6.11)				1
				18 ft. Manhole		L.S.	$5,000	
				Overflow inlet		L.S.	$3,500	
				24" Low flow pipe	25 L.F.	$75.00	$1,870	
				Low flow inlet		L.S.	$4,500	
				Access path		L.S.	$1,000	
				Rip-rap		L.S.	$750	
				Total			$16,620	
S60.45	Inlet of the closed pipe system, 100 ft. Southeast of J.Q. Adams Street	62	NA	Construct an entry structure (See Figure 6.13)				1
				48" x 36" Reducer	1 ea.	$1,000	$1,000	
				48" RCP	20 L.F.	$110	$2,200	
				5 ft. Transition and grate		L.S.	$2,500	
				10 ft. Transition		L.S.	$2,500	
				Rip-rap		L.S.	$500	
				Total			$8,700	

TABLE 6.10 (CONTINUED)

PROPOSED CAPITAL IMPROVEMENTS FOR
THE SINGER BASIN

Identification	Location	Design Flow (cfs)	Design Criteria	Improvements	Quantity	Unit Cost	Estimated Construction Cost	Priority
S70.70 to S80.80	Closed pipe system between 3rd Street and Linn Avenue, along Center Street	9	Minimum slope, 0.4%; Minimum depth, 3.5 ft.	Replace the existing pipe system with 21" RCP				3
				21" RCP	950 L.F.	$43.40	$41,230	
				10" CP	450 L.F.	$24.00	$10,800	
				Manhole	4 ea.	$1,500.00	$6,000	
				Connect roof drains		L.S.	$5,000	
				Existing utilities		L.S.	$7,500	
				Abandon Existing manholes	4 ea.	$350.00	$1,400	
				Rock excavation	355 C.Y.	$150.00	$53,250	
				Total			$125,180	
S80.80 to S100.60	Closed pipe system between Linn Avenue and 7th Street, along Center Street	34	Minimum slope, 0.7%; Minimum depth, 4.0 ft.	Replace the existing pipe system with 30" RCP				2
				30" RCP	650 L.F.	$60.70	$39,450	
				10" CP	110 L.F.	$24.00	$2,640	
				Manhole	1 ea.	$1,500.00	$1,500	
				Special manhole	1 ea.	$2,500.00	$2,500	
				Connect roof drains		L.S.	$4,000	
				Existing utilities		L.S.	$5,000	
				Abandon existing manhole	1 ea.	$350.00	$350	
				Rock excavation	260 C.Y.	$150.00	$39,000	
				Total			$94,440	
						CIP subtotal	$261,560	
						Contingency @ 15%	$39,230	
						TOTAL ESTIMATED CONSTRUCTION COST	$300,790	
						Engineering and surveying @ 17%	$51,130	
						Legal and administration @ 4%	$12,030	
						Financing @ 9%	$27,070	
						TOTAL PROGRAM COST	$391,020	

The major problem in the Singer basin is the potential lack of hydraulic capacity in the 2,500 foot-long Singer Creek "culvert". The actual capacity is not known since access to the structure is unavailable for approximately 850 feet downstream of its inlet. If this historic culvert has an adequate slope in this reach, its hydraulic capacity could be more than adequate for the existing 25-year flood peak. If the slope is relatively flat in this area, the capacity of the culvert would be less than the existing two-year flood peak. A survey of the culvert's actual alignment, elevation, size and roughness characteristics must be conducted before the issue of its capacity can be settled.

The drainage plan for the Singer basin includes the capital improvement projects that should be pursued to cost effectively minimize the potentially severe flooding problems that could materialize along Singer Creek once the combined sewer system is separated and the open drainageways are cleaned. The plan calls for the maximum utilization of the available storage detention volume located upstream of both Pearl and Jackson Streets to reduce peak flows upstream of the Singer Creek culvert entry. Once again, geotechnical analysis must be conducted to ensure the stability of these embankments under their designed headwater elevations. Detention outlet control structures similar to that shown on Figures 6.11 have been proposed.

As envisioned in the proposed plan, the combined detention facilities will reduce the 25-year future peak flow entering the Singer Creek "culvert" by 36 percent from 97 cfs to 62 cfs. The 100-year design flood headwater depths upstream of Pearl Street and Jackson Street would be 17.5 feet and 18.0 feet, respectively.

Also included in the plan is an improvement to the existing inlet to the Singer Creek culvert. The poor hydraulic characteristics of this inlet were discussed earlier in Chapter 3. Figure 6.13 shows a sketch of the type of improvement proposed to alleviate this potentially severe flood problem at the existing inlet.

The total estimated construction cost for the CIP's proposed throughout the Singer basin is $300,790. Approximately 85 percent of this cost is the result of the major drainage collector improvements proposed to increase the hydraulic capacity of the severely undersized old combined sewer pipes in this area. These expensive closed pipe improvements shown in the CIP plan address only the major drainage transport from node 70 to 60 via 80.

No improvements have been proposed at this time for the major drainage collector system between node 90 and 80. If the detailed study of the Singer Creek culvert concludes that adequate capacity exists for now and into the future, the drainage from node 90 may be connected directly to the culvert near its crossing of Madison Street. The cost of this potential connection would be a small fraction of the cost required to improve the existing 8-inch and 12-inch combined sewer pipes in 12th Street to handle the flow from sub-basin S90.

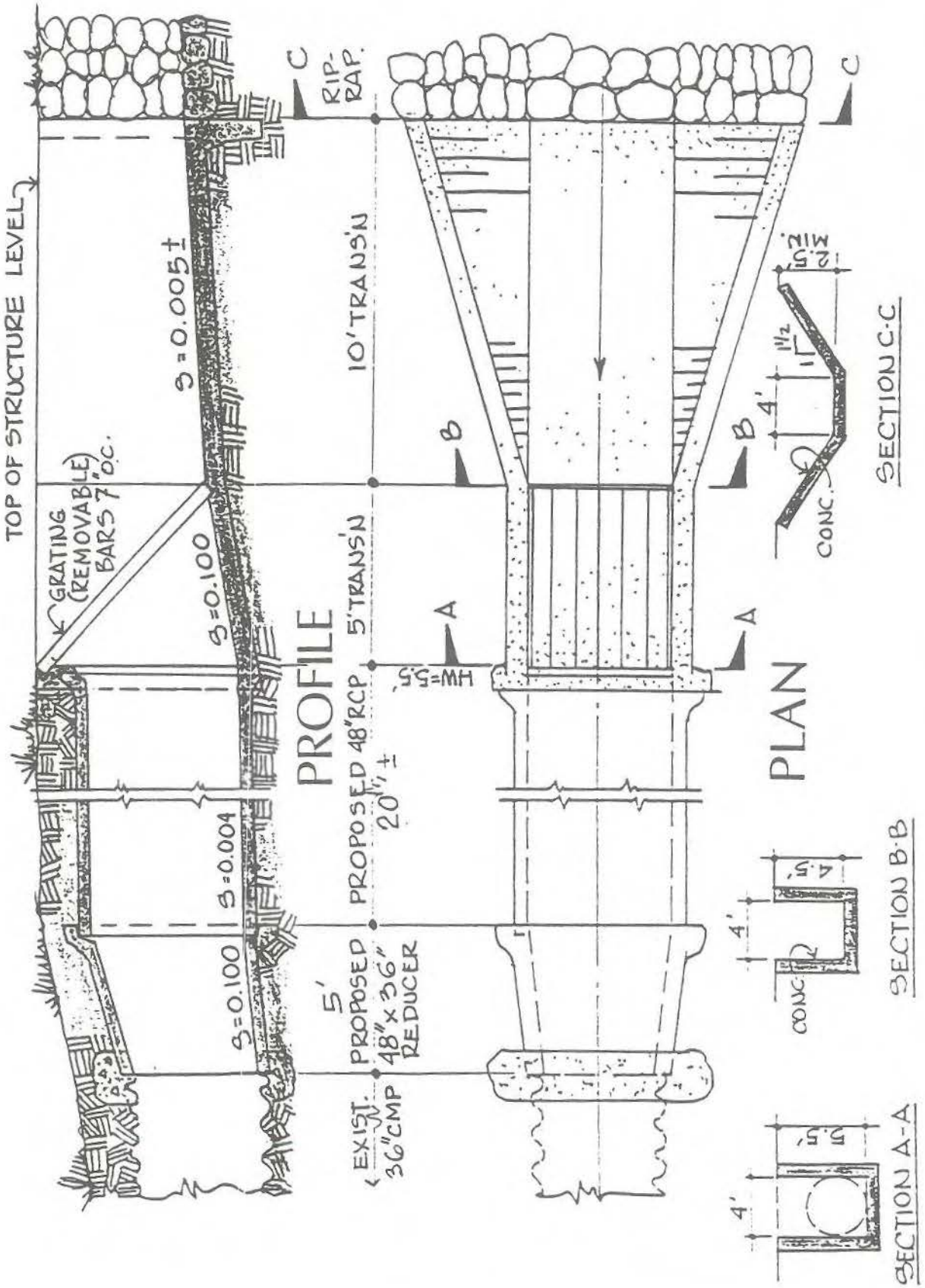

FIGURE 6.13

PROPOSED IMPROVEMENT OF EXISTING PIPE INLET TO S60.45

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The total estimated program cost for the Singer basin is now $391,020. This represents an areawide cost of approximately $987 per acre. These program costs could increase dramatically if the Singer Creek culvert capacity is determined to be significantly less than now estimated and additional improvements are required.

6.4.11 CIP Cost Summary and Priorities

A total of fifty-five capital improvement projects have been outlined and recommended above for the major drainage collector systems throughout the Oregon City study area. The total estimated capital improvement program cost for completing these construction projects is $1,306,480. This program cost includes construction with contingency at $1,004,910; engineering and surveying at $170,920; legal and administration at $40,200 and financing at $90,450.

The recommended capital improvement projects presented above are proposed to be implemented over a 20-year period. Each of the improvement projects were assigned a priority ranking from 1 to 4. The priority ranking represents which 5-year time frame the project should most likely be constructed (i.e. first 5 years, second 5 years, etc.). The priority rankings are based on several subjective criteria. First and foremost was the severity of the drainage problem the improvement would correct. Is the CIP addressing an existing drainage problem or one anticipated in the future? Another criterion was the location of the improvement in relationship to the City's existing corporate boundaries. Is the CIP located within the City now or is it anticipated to be located in the City later? In addition, the priority rankings were based on the assumption that CIP expenditures would be maintained at approximately $65,000 per year (i.e., 1987 dollars). Thus, the total program cost for all the projects with the same priority ranking would be approximately $325,000 (i.e. 1987 dollars).

6.5 RECOMMENDED DRAINAGE MAINTENANCE PROGRAM

Continued maintenance of the major drainage system is necessary if it is to act as an outfall for stormwater runoff. To maximize the use of the major drainageways during runoff and flood periods, and thereby reduce the damage potential, a good drainage maintenance program is an absolute necessity.

Based on our field observations during the collection of data for this study, it can be concluded that the actual hydraulic capacity of the existing major drainage systems throughout most of the Oregon City study area is only a small fraction of its potential capacity due to its poorly maintained condition. This statement is not intended as a criticism of existing maintenance practices. As a general rule, the monies have not been historically allocated for the maintenance of a city's drainage system. Maintenance personnel do the best they can with the budgets they

are given. Drainage usually receives the lowest priority in the competition for limited maintenance dollars.

Numerous studies throughout the country have shown that the most economical expenditure of drainage monies is for the continued maintenance of the drainage system. Given the tremendous investment the City currently has in their existing drainage systems, it only makes sense that it should be functioning at close to 100 percent of their available capacity when that infrequent flood event occurs.

The primary goal of the recommended drainage maintenance program could be stated as:

> "Within available funds, the City of Oregon City will provide preventive maintenance and rehabilitate drainageway facilities in a manner that will ensure reasonably adequate functioning of the drainageways and hydraulic structures during periods of stormwater runoff."

Drainageways should not have to be capable of passing all floods without damage to the drainageway, its facilities, or private properties along the drainageway.

The Urban Drainage and Flood Control District in Denver, Colorado, developed the following objectives and guidelines for a drainage maintenance program (Reference 11). Those objectives that could meet the stated goal in the City of Oregon City are listed in the following section.

6.5.1 Objectives of the Maintenance Program

To maintain the drainageway's function, it is recommended that the following two objectives be attained as a first priority. The third, fourth and fifth objectives should be met on a continuing basis.

- Objective 1. Develop Facilities Maintenance Inventory. An inventory system should be developed that will identify facilities by drainageway and basin and its location on the drainageway. The inventory should be used to identify the current status of facilities and their maintenance needs. The major hydraulic structures inventory presented in Chapter 3 would be a good starting point for the City in achieving this objective. The inventory will assist in estimating costs and developing budgets, and permit the City to keep track of maintenance activities and their effectiveness. The inventory should identify property ownerships and where access right-of-way will limit the type of maintenance procedure that may be used.

- Objective 2. Develop Maintenance Standards and Criteria. The City should develop measures for identifying when and what type of maintenance is needed. Those measures will establish the basis for an adequately maintained and functioning drainage

system. The City should develop maintenance criteria standards for each maintenance activity, such as cleaning obstructions from structures, repair of slope erosion, cleaning and repair of channel bottom, repair of structures, debris and trash removal, landscape upkeep and others. Maintenance criteria established by the Denver Urban Drainage and Flood Control District are listed later in this chapter. The standards and criteria should be reviewed annually to reflect experience gained by maintenance staff in the previous year.

- Objective 3. Establish and Implement Continuing Inspection Program. The City should set up an inspection program to:

  -- Identify current maintenance needs.

  -- Determine the effectiveness of ongoing maintenance.

  -- Check maintenance contract performance.

  -- Monitor floodplain zoning implementation.

The inspections should be performed by the public works or engineering staff.

- Objective 4. Prepare an Annual Maintenance Budget. The City should develop unit cost schedules for each annual budget. The maintenance program budget should be based on cost estimates for remedial maintenance (rehabilitation of damaged facilities and major erosion repair), regular maintenance (periodic mowing and vegetation removal, debris and trash removal, minor erosion repair), right-of-way acquisition, administration and contract services for engineering and inspection.

- Objective 5. Maintain the Drainageways. The City should provide remedial maintenance to restore damaged drainageway facilities to an adequate operational status. Whether that work is provided by City staff or contracted through bid or negotiation, a performance standard to meet established standards and criteria should be required.

These objectives should be reviewed annually and amended as required by experience and additional information.

6.5.2 Maintenance Criteria

The following maintenance criteria are recommended for inclusion in a maintenance program in the City of Oregon City. The criteria were first established in draft format by the Denver Urban Drainage and Flood Control District.

- Cleaning Drainage Structures

  In order to restore or maintain proper drainage through the structure, it is necessary to clean culverts and catch basins and check dams and bridge openings for accumulations of sedi-

ment and debris that are greater than one-fourth the depth of the drain.

- Side Slope Erosion

  To restore proper grades and slope configuration of channel side slopes it will be necessary to remove or add required material. Work should be conducted when the following conditions occur:

  -- Erosion or washout of side slopes behind inlets or outfalls greater than 6 inches.

  -- Erosion or washout behind bridge or culvert wing walls or saddle head walls that expose 6 inches of the concrete surface or 75 percent of the thickness of the slab or wall.

- Channel Bottom Cleaning and Repair

  Mechanical or hand cleaning of lined or unlined open channels to remove accumulations of silt, weeds and trees to restore proper drainage.

  Erosion and silt should be repaired or removed whenever low flows are diverted more than 2 feet from the designed channel centerline or when ponding remains after storm runoff.

  Vegetation that restricts flow should be removed according to the following criteria:

  -- Trees or shrubs in the drainageway with a main trunk diameter of more than 1 inch.

  -- Trees or shrubs within the floodway whose root structures are exposed or may be washed out including those species, such as blackberries, that could cause significant blockage should they be washed into culverts or bridge openings.

  -- Trees or shrubs within the floodway that exceed 10 plants per 100 square feet.

  Eroded material should be replaced to restore proper line and grade whenever erosion occurs next to a drainage structure and exposes 75 percent of the slab thickness or the eroded channel bed is 8 inches below designed grade.

- Debris and Trash Removal

  Hand or mechanical pickup and disposal of trash and debris should eliminate the potential blockages of culverts and ensure that health standards are maintained.

· Repair of Channel Structures

Channel structures that have deteriorated should be restored to acceptable operating condition. Concrete structures should be maintained at the direction of the street superintendent. Typical items that could require remedial maintenance include cracks greater than 1/4 of an inch wide, concrete work more than 3/4 of an inch out of alignment and reinforcing concrete that is exposed because of deterioration.

The recommended criteria listed above are suitable to an ongoing maintenance program. Remedial maintenance required during and after a major flood event will be more random and will include tasks like removing debris blockages. A maintenance program developed to meet the objectives and criteria listed above will reduce drainage problems during major storm events.

One of the more serious maintenance problems with the open drainageways in Oregon City is access for maintenance personnel and equipment. To maintain channels at a reasonable cost and level of effort, they must be accessible to maintenance equipment. High labor costs have dictated the use of sophisticated mechanical equipment, much of it large and cumbersome. While preservation of the floodplain in the future would ease the access problem, access along certain stream reaches will still be restricted. If access to the drainageway can be made only at street crossings, it will probably be necessary to continue manual maintenance procedures. However, we strongly recommend that the City take any opportunity (e.g. when capital improvements are being made) to secure a permanent right-of-entry to the drainageway.

6.5.3 Estimated Maintenance Costs

Natural drainageways require regular maintenance to be efficient hydraulic conduits for flood flows. Little information regarding maintenance of these natural flood channels is generally available. Required maintenance items were discussed above. They include debris removal, mulching, seeding, selective erosion control and selective vegetation clearing and sediment clearing.

Estimates of natural drainageway maintenance throughout Oregon City were based on data developed for Tulsa, Oklahoma (Reference 12) in September, 1978. These costs were adjusted to reflect costs expected in 1987.

Man-made channels also require regular maintenance if they are to function properly during floods. Estimates of maintenance costs for grass-lined channels are based on Corps of Engineers and Denver Urban Drainage and Flood Control District data. Annual maintenance costs include mowing, handcutting, weed/brush control, mulching/seeding and debris removal. Costs were also estimated for cleaning both closed pipe systems and culverts.

Based on this available cost data, Table 6.11 presents the estimated cost of maintenance for the major drainage collector systems located throughout the Oregon City study area. The table shows an estimate of both initial maintenance costs and subsequent annual maintenance costs based on existing drainage conditions throughout the twelve drainage basins that were studied.

The total estimated initial maintenance cost for the major drainage systems is $161,820. The total annual maintenance cost for subsequent years is estimated to be $41,270.

TABLE 6.11

ESTIMATED MAJOR DRAINAGE SYSTEM MAINTENANCE COSTS

Major Drainage System Characteristics

Basin	Open Channels (Feet)	Closed Pipe (Feet)	No. of Culverts	Maintenance Costs Initial	Maintenance Costs Annual
Caufield	13,480	0	8	$ 21,090	$ 5,490
Mud	5,660	1,400	6	8,960	2,150
Central Point	3,100	0	4	17,160	4,250
South End	4,140	0	7	14,560	3,760
Coffee	3,890	900	11	19,560	5,100
Clinton & Tumwater	750	830	3	5,180	1,240
Newell	10,130	1,525	11	28,400	7,620
Park Place & Livesay	13,770	0	8	11,940	3,030
John Adams	2,040	4,060	1	11,840	2,890
Singer	6,370	3,860	4	23,130	5,740
Totals	63,330	12,575	63	$161,820	$41,270

Chapter 7
Financing Plan

CHAPTER 7

FINANCING PLAN

7.1 GENERAL

The lack of adequate funding is a common frustration of all municipalities in dealing with drainage problems. Drainage system maintenance and capital improvements funding is basic to its successful operation as an efficient transporter of storm-water runoff. If funds are not available to address existing and potential drainage problems, the chance for further flood damage must become a recognized fact. The recommendations of the master drainage plan range from the preservation of natural drainageways to structural modifications of channels, and culverts. To prevent damages, funds must be invested.

7.2 COST APPORTIONING

The crux of the funding issue is the manner in which costs are apportioned. The method of apportioning costs should be equitable and, at the same time, spread the costs widely enough so the amount an individual property owner pays is reasonable. The principal methods of cost apportionment that are available are: assessed value, land area, and contribution of runoff.

7.2.1 Assessed Value

Assessed value is a method frequently used by cities and counties for other improvements and is the basis for general taxation. The advantage of the assessed value method is that it is simple to calculate and administer since tax rolls are already set up. Its major disadvantage is that the assessed value method does not really relate benefits to payments, or payments to the degree that each property contributes to the problem. The recent failure of METRO to obtain financing based on assessed value to solve the Johnson Creek flooding problem attests to the method's basic weaknesses.

7.2.2 Land Area

Costs could be apportioned on the basis of the land area of each property, which is also available in the tax records. The land area method attempts to relate payment to the degree of contribution to the problem. However, its major shortcoming is that land area alone is not an accurate measure of the contribution to runoff. A large rural area, such as a farm, may contribute less runoff than an urbanized area that is much smaller in size.

7.2.3 Contribution of Runoff

The best method of cost apportioning is one that relates the payment directly to the amount of runoff each property contributes to the drainage problem. Runoff volumes can be related to different land uses. Thus, a commercial property with a high degree of imperviousness would pay more than the same size residential development. This method has been used successfully for apportioning drainage improvement and maintenance costs in several communities in Oregon that have established drainage utilities.

In practice, the apportionment of costs for these drainage utilities has been based on a unit called an Equivalent Residential Unit (ERU). An ERU is the amount of runoff from a typical single-family house and lot located in the respective city. Other land types, such as commercial or industrial, are assigned an appropriate number of ERU based on their size and degree of imperviousness. Typical annual payments for an ERU will vary from city to city. Currently, the annual payment for an ERU in Corvallis is $19.20.

7.2.4 Other Issues

Another issue to consider in allocating costs is equitably distributing costs on the basis of benefits received. Although property located in the upland portion of the basin may be contributing runoff to a drainage problem, property located next to the waterway will certainly obtain a much greater benefit if flood protection is provided.

Flood benefits can be classified as direct or indirect. Indirect or general benefits are those received by the general public, such as the elimination of nuisance flooding of roadways or the ability for emergency equipment to have road access during a serious flood. Indirect benefits are another means of justifying cost allocations to all property owners. However, an equitable cost allocation procedure should have a means of assessing direct benefits and including them in a reasonable manner. In the drainage utility concept, this direct benefit assessment could be achieved with a surcharge on ERUs that have received a direct flood benefit from drainage-related activities.

A final issue that must be considered in allocating drainage costs is the equitable distribution of costs between the present residents and future residents. The recommended plans presented in Chapter 6 were based on future runoff conditions. When a culvert is replaced, it is sized to pass future runoff, not just the present runoff conditions. Thus, present property owners are paying higher present costs so future owners will not have to pay to have the culvert replaced again.

System development charges are a common means of obtaining drainage revenue from future development. A once only charge is made at the time of development. These development charges can

be based on land area and land use type. The revenue can be used to pay off bonds obtained for drainage improvements.

7.3 FUNDING OPTIONS

Municipalities have a variety of options available for raising funds to pay for drainage improvements and maintenance. Common funding options include: property taxes, user fees, local improvement districts, system development charges, general obligation bonding and revenue bonding.

7.3.1 Property Taxes

Property taxes provide revenue for the City of Oregon City and other public agencies. The taxes are based on the value of property. These taxes apply only to private property, with public properties and some charitable and religious holdings exempted.

A tax base system is used in Oregon which limits the annual amount of taxes which local governments may collect without specific voter approval. The base can be increased annually by 6 percent, but this increase has historically not been adequate in a rapidly developing community. The taxing situation is further complicated by the fact that local government taxes on owner-occupied residences are partially subsidized by state revenues. This system results in some remarkable strategizing and structuring of local excess tax levies in Oregon.

Oregon City could ostensibly increase the funding for stormwater management by committing additional property taxes. The question is what other functions would be scaled down or eliminated to make funds available or if an excess levy would have to be proposed. In the past, however, other needs have taken priority over drainage.

The property tax option does not appear to be a viable approach given local circumstances and public attitudes about property taxes in general.

7.3.2 User Fees

User fees are being used in several communities in Oregon (including Portland, Corvallis, Medford, and Tigard) to fund stormwater programs. This approach has the advantage of greater flexibility in defining the factors and considerations on which charges are based. Rate structures can be developed which emphasize contribution to runoff, service level, benefit, other factors, or even a mix of several of these to achieve an equitable and publicly acceptable funding method.

In Oregon, cities have the authority to form drainage utilities and bill property owners service charges or user fees for improvements or maintenance of the drainage system. As discussed

earlier, the charges are usually based on the amount of runoff that the property contributes. Voter approval is not required.

7.3.3 Local Improvement Districts

Experience has shown that Local Improvement Districts (LIDs) are not an effective mechanism for implementing drainage programs to meet area-wide needs. Numerous small water, sewer, street lighting, and drainage projects have been funded through LIDs, but their political success has clearly been inversely proportional to the area covered by the assessment district. One striking example of this recently occurred when the Johnson Creek LID (covering 54 square miles) proposed by the Metropolitan Service District was rejected by property owners in the assessment area.

Washington County has had extensive experience with LIDs for drainage in recent years, requiring that they be set up by developers to fund on-going maintenance of detention and retention systems. For lack of other funding, the County has been forced to rely on these LIDs for maintenance funds for those systems despite the fact that the proliferation of small districts (now over 200) presents an administrative burden. The ideal use of drainage LIDs would be for small local construction projects, remedial maintenance, and other onetime jobs.

7.3.4 System Development Charges

This type of charge, under various titles, has been increasingly used by cities and counties to help fund capital improvements necessary for new development to occur. They usually are applied to the development sector through fees associated with plat (subdivision) approvals, the issuance of permits, or connection with utilities. They are especially useful when improvements include excess capacity in anticipation of future growth. Their use for stormwater management would be to ensure financial participation in capital improvements by properties which develop after a project is partially or complete paid off. They would be most easily implemented as an element of a rate structure for stormwater management service charges, wherein the system development charge could be treated as one type of service charge.

7.3.5 General Obligation Bonds

General obligation bonds are used primarily for major capital improvements. They are subject to voter approval and are backed by the full credit of the government issuing them. Specific revenues may be pledged to paying off G.O. bonds, but all revenues are accessible to bond holders to ensure payment. For example, so-called "Bancroft" bonding enables property owners use the government's other revenues that may be tapped to pay off the bonds. General obligation bonds usually command a favorable interest rate because of the strong credit of local governments and the fact that interest income from municipal bonds is not federally taxed.

7.3.6 Revenue Bonds

Bonds may also be sold secured only by specific revenue sources which are earmarked for their payment. These are usually termed revenue bonds. They are most appropriate for funding construction improvements, land acquisition, and other major capital outlays. They are not backed by the full credit of the government issuing them. As a result, they usually involve higher interest rates than G.O. bonds. A "coverage" requirement may also be necessary to successfully market revenue bonds, in which rates are designed to generate more revenue than actually needed for repayment to ensure that delinquencies will not cause default. As a general statement, revenue bonds do not require voter approval in Oregon provided they are issued by a city.

Revenue bonding supported by service charges would provide an equitable mechanism for funding drainage improvements in Oregon City. A service charge rate structure can be designed to distribute costs in line with contribution to runoff, service, and/or benefit. It can include charges to both public and private property, and system development charges whereby developing properties "buy into" the other property owner's financial investment in the system.

7.3.7 Other Fees and Charges

Local governments assess a variety of special fees and charges related to specific functions, including plan review, field inspections, and permit administration. In the case of drainage the most appropriate fees are plan review and inspection charges related to drainage systems to be installed on private property, abatement charges for correction of public hazards or nuisances, and in lieu of construction fees. These fees can supplement major revenue sources and assign special costs to certain persons or properties, but should not be expected to generate a major proportion of drainage funding.

7.4 RECOMMENDATIONS

7.4.1 Drainage Utility

It is our recommendation that a drainage utility be established by the City of Oregon City. The service charges or user fees from the utility can be used to finance:

1. The cost of implementing and continuing the drainageway maintenance program described in Chapter 6.

2. The cost of administration, engineering design and construction of the capital improvements presented in Chapter 6.

3. The cost of administration, engineering, design and construction of anticipated capital improvements to the minor drainage systems which have not been identified as part of this drainage study.

4. The cost of obtaining drainage easements for the major drainageways throughout the study area (These costs have not been estimated as part of this drainage study).

7.4.2 System Development Charges

Undeveloped properties should not be billed as part of the recommended drainage utility. However, capacity will be included in the systems to accommodate their anticipated future contributions to runoff.

In the interest of equity, some method must be devised to ensure that all properties which necessitate and use the extra capacity of the systems (including those presently undeveloped) participate in their funding. At the same time, properties which are not developed and therefore do not discharge stormwater runoff in excess of natural flows should not have to pay for sizing of systems to accommodate development. That would amount to undeveloped properties subsidizing the development of other land.

The stormwater utility rate structure adopted by Oregon City should provide a vehicle for assessing such costs to property as it develops. A "systems development charge" could be included in the rate structure, so that financial participation in capital improvements can be deferred as long as a property remains undeveloped, but a fair share of the cost of improvements is borne as part of the cost of developing. The charge could be assessed at the time that development permits are issued, and should be structured consistent with the utility rate structure so that developing properties "buy into" the communities' previous investment in the systems.

In a sense, this charge merely allows undeveloped properties to defer paying their portion of capital improvement costs until they develop and use the oversizing previously designed and built into the systems. The charge would be scaled to increase over time to reflect the amount each parcel would have paid for capital improvement financing had it been developed when the facilities were built and financial payment began. The charge would also be scaled to accommodate eventual systems depreciation, causing the charge to have a peak value probably equivalent to 20 years of accumulated charges. This will allow for the fact that properties developed several years after construction of capital improvements will be buying into a system with less useful life remaining.

Properties which undergo substantial redevelopment should also pay a systems development charge at the time of redevelopment, since the drainage systems are being designed on the basis of ultimate land use reflected by the Comprehensive Land Use Plan,

not present site conditions. Any redevelopment which increase the effective "intensity of development" as defined in the adopted rate structure should be subject to a systems development charge reflecting the additional amount the property would have paid for capital improvements had it been developed in the "redeveloped condition" at the time financing of the system improvements began. This entire approach is geared to making each property pay for the peak load they actually impose on the facilities.

The systems development charge is intended to cover only capital improvement costs. Therefore, it should be based on the portion of the total charge which would go toward the administration, engineering, and construction of capital improvements.

7.4.3 Rate Structure

There are many options available for establishing the rate structure of the proposed drainage utility. The Financial Planning Element report for the Washington County Drainage Master Plan, published in February 1985 (Reference 13), identified 7 different structures that could be used to establish rates for a drainage utility. They were as follows:

1. Charges Based on Impervious Area.

2. Charges Based on Intensity of Development and Land Use.

3. Charges Based on Gross Area and Impervious Area.

4. Charges Based on Gross Area and Intensity of Development.

5. A Flat Rate for All Single Family Residences, Combined with a Variable Rate Charge for Other Properties Based on Gross Area and Intensity of Development.

6. A Flat Rate Charge to Each Account for "Uniform" Costs of Service, Combined with Variable Charges Based on Gross Area and Intensity of Development.

7. A Flat Rate for All Single Family Residences, Combined with a Variable Rate Charge for Other Properties Based on Impervious Area.

Each of these rate structures were evaluated using criteria like equity, cost of implementation and upkeep, revenue capacity and flexibility. The recommended rate structure that emerged from this detailed evaluation was a flat rate for all single family residences combined with a variable rate charge for other properties based on gross area and intensity of development (i.e. number 5).

The most important requirement of this type of rate structure is that a means be developed to relate the residential flat rate charge to the charge per billing unit applied to nonresidential

parcels through the rate algorithm using gross area and intensity of development. This is accomplished by defining a "typical" singlefamily residential parcel in terms of gross area and intensity of development, which then serves as the basis of comparison with nonresidential parcels.

This approach has been used in Tacoma, Washington and other communities, and the flat rate for residences is proving popular. The major advantage is an estimated 25% reduction in costs of implementation and upkeep resulting from a flat rate which eliminates more than 50% of the data measurements. The principal disadvantage is overcoming public uncertainty related to the use of a flat rate for all residences when most other utilities are moving away from flat rates.

It is recommended that Oregon City adopt rate structure number 7 outlined earlier. This rate structure offers good equity based on gross area and intensity of development plus a cost effective implementation process resulting from use of a flat rate for residential properties.

7.5 DRAINAGE UTILITY RATES

As described in Section 7.4, the recommended rate structures for the Oregon City drainage utility is a flat rate for all single family residences combined with a variable rate charge for other properties based on gross area and the intensity of development. The flat rate charge for all single family residences could be based on a typical residential property having a 8,000 to 10,000 square foot lot with approximately 2,000 to 3,000 square feet of impervious coverage.

7.5.1 Area Range Numbers

Ranges of gross area square footage could be used to classify non-single family residential properties into groups of similar parcel size. For example, these parcels could be grouped by gross area ranges of 2,500 square feet. All parcels with gross areas of 1 to 2,500 square feet would be assigned an area range number (ARN) of 1. Parcels with gross areas of 2,501 to 5,000 square feet would have an ARN of 2. ARNs equal to 3 represent parcels whose area ranges between 5,001 and 7,500 square feet and so on.

7.5.2 Development Intensity Factor

The development intensity factor (DIF) can be viewed as a runoff coefficient that is indicative of the land use or impervious coverage of each property. For Oregon City, the following factors are recommended.

Land Use or Zoning		Development Intensity Factor
	Undeveloped	0.0
R10	(10,000 sq. ft./dwelling unit)	0.25
R8	(8,000 sq. ft./dwelling unit)	0.30
R6	(6,000 sq. ft./dwelling unit)	0.40
RC-4	McLoughlin Conditional	0.50
RD-4	Two-Family	0.50
RA-2	Multi-Family	0.60
LOC	Limited Office Conditional	0.70
LO	Limited Office	0.80
NC	Neighborhood Commercial	0.80
HC	Historic Commercial	0.70
LC	Limited Commercial	0.70
C	General Commercial	0.90
CBD	Central Business District	0.90
M1	Light Industrial	0.70
M2	Heavy Industrial	0.80

7.5.3 Equivalent Residential Unit

To determine the variable rate charge for non-single family residential properties the representative number of equivalent residential units for that property must be determined. The equivalent residential units (ERU) of a non-single family residential property is computed by multiplying the property's area range number (ARN) by its development intensity factor (DIF) as follows:

$$ERU = ARN \times DIF$$

For example, a 25,000 square foot lot (ARN=10) which is developed as a convenience store (DIF=.9, general commercial) would have an equivalent residential unit of 9 (i.e. 10x.9). Thus, the recommended Oregon City drainage utility would charge this property owner 9 times more than the rate charged a single family homeowner. The residential flat-rate charge is based on a 10,000 square foot lot (ARN=4) and an R10 development intensity factor of 0.25 which results in one equivalent residential unit. If the residential user fee is $1.00 per month, the convenience store owner would pay 9 times that or $9.00 per month.

7.5.4 Preliminary Rate Estimate

Section 7.4 outlined the various services that the recommended drainage utility should provide. In an effort to estimate a preliminary rate for the utility the following allocation of collected revenue is proposed.

Resource Allocation	Percentage of Collected Revenue
1. Maintenance of Drainage System	40%
2. CIPs for Major Drainage System	35%

3.	CIPs for Minor Drainage System	15%
4.	Utility Administration	10%
		100%

If the scheduled annual allocation of monies for completion of the major CIPs (i.e. presented in Chapter 6) is $65,000 and this represents only 35% of the utilities resource allocation, then the annual revenue requirement for the utility is estimated to be approximately $185,000. To obtain a preliminary estimate of the utility rate or user fee, we must estimate the number of equivalent residential units within the City's current corporate limits and divide this ERU into the annual revenue requirement.

Based on the 1977 housing and land use data published in the City Comprehensive Plan with some minor adjustment included for subsequent increases, the total number of equivalent residential units throughout Oregon City is estimated to be approximately 7,700. This ERU includes both single family residential and non-single family residential properties. Therefore, the estimated user rate for a single family resident owner within the recommended Oregon City drainage utility would be approximately $24 per year or $2 per month (i.e. $185,000/7700 ERU's/12).

7.5.5 System Development Charge

50% increase in rev. [illegible]

As noted in Section 7.4, a system development charge could be established as part of the drainage utility rate structure. The system development charge could be assessed when a parcel is developed or substantially redeveloped. The charge is structured so that developing properties will pay their equitable share of the cost of drainage improvements that were designed to eventually accommodate their development. Therefore, developing properties must "buy into" the community's previous investment in these drainage systems.

The system development charge is based on the accumulated capital improvement costs the property owner would have paid if his property had been developed since the drainage utility was originally established. For example, say a developer requests permits to develop a 10-acre parcel into a 40-lot subdivision 5 years after the Oregon City drainage utility was established. The system development charge would be based on the capital improvement portion of the utility's resource allocation for their first 5 years of operation. Assume that these CIP costs were $12.00 per year per ERU. The system development charge for this hypothetical development would be $2,400 (i.e. $12/year x 5 years x 40 ERU's).

The system development charge would reach an accumulated maximum at 20 years. Thus, the charge for the same 40-lot subdivision developed 20 years after the utility was established would be only $9,600, assuming the capital improvement allocation was an average of $12.00 per year per ERU for the entire 20-year period.

7.5.6 Implementation

The City may want to conduct a study to further refine the user rates and revenue requirements outlined above. The study could define the drainage utilities' administrative procedures and management responsibilities. This study could be completed in-house or let to a qualified consultant. Consultant fees would probably range from $10,000 to $15,000 depending on the specific scope of services.

Regardless of whether a further study is undertaken or the estimates provided in Section 7.5.4 are acceptable, the City will need to pass an ordinance establishing the drainage utility, its rate structure, its rates and its budget. The services of an experienced bonding attorney should be secured so all procedures are correct and later bond issues, if required, can proceed smoothly.

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13. Kramer, Chin and Mayo, Inc. and Water Resource Associates, Drainage Master Plan - Financial Planning Element, Washington County, Department of Land Use and Transportation, February 1985.

Appendix B

SCS Rainfall Distributions

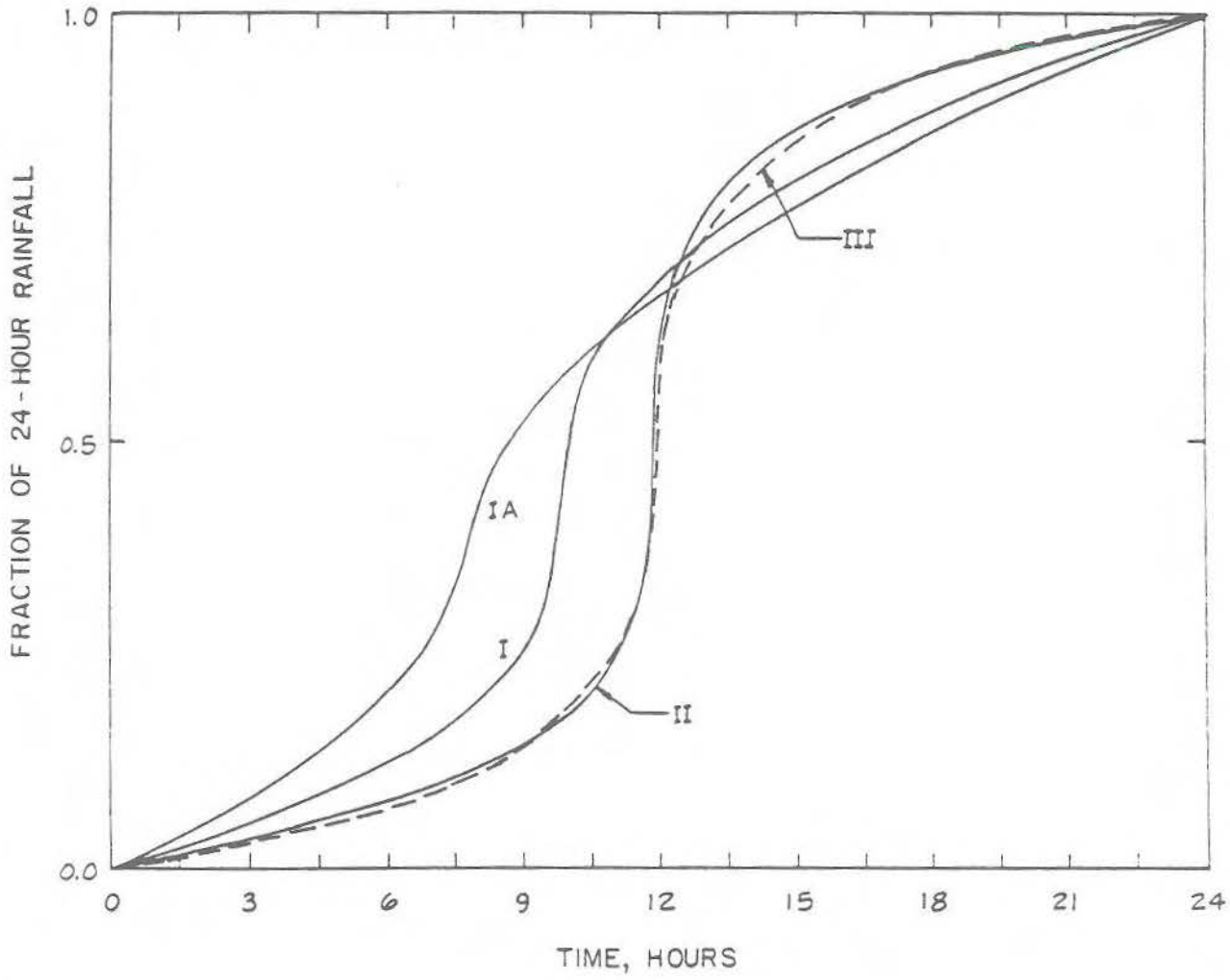

Figure B-1 - SCS 24-Hour Rainfall Distributions

otak INC. ARCHITECTS P.C.
incorporated
17355 S.W. Boones Ferry Rd. Lake Oswego OR 97035 (503) 635 3618
1011 E. 8th Street Vancouver WA 98660 (206) 695 0357

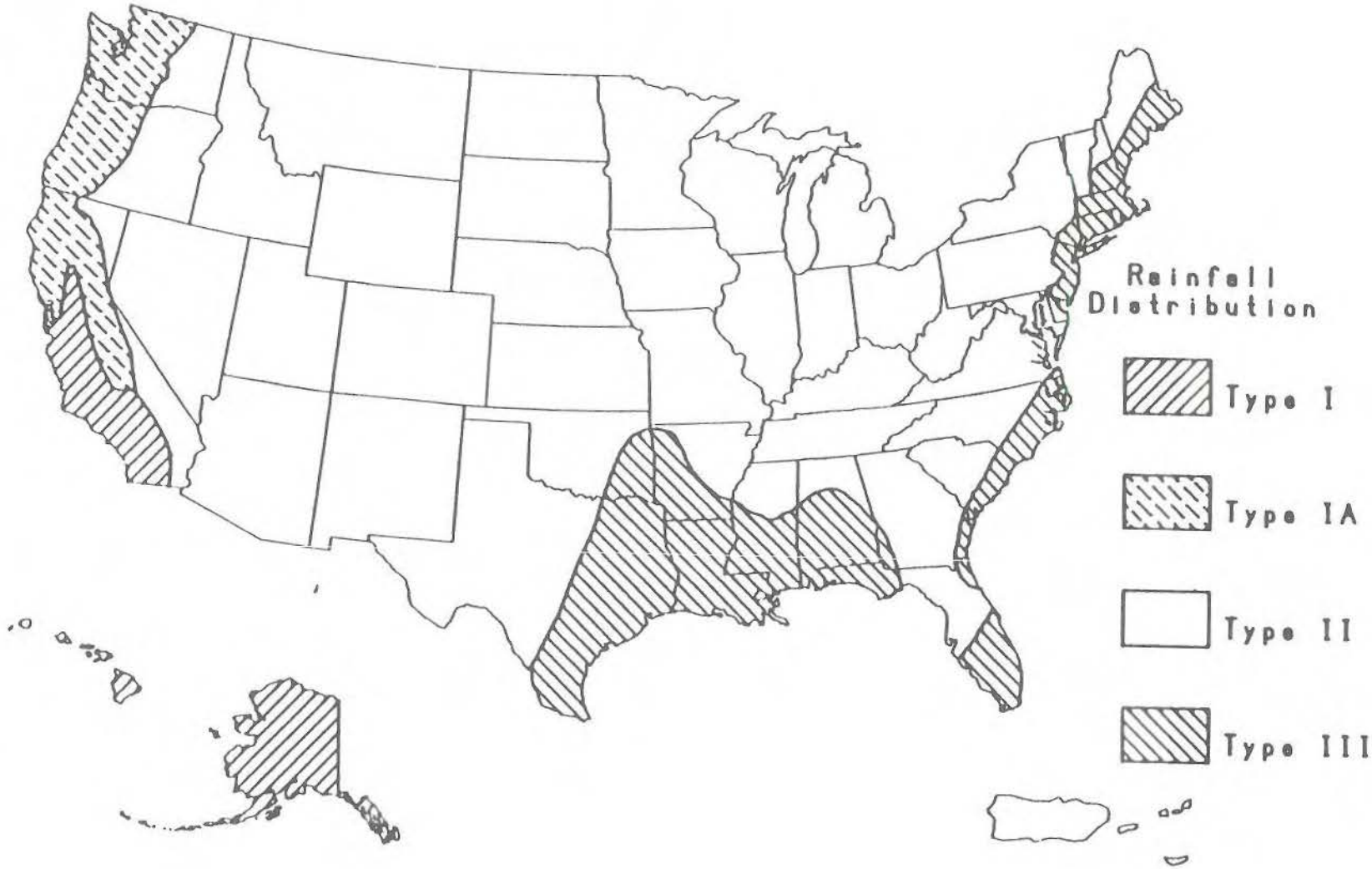

Figure B 2. - Approximate geographic boundaries for SCS rainfall distributions.

RAINFALL TABLE 1

STANDARD SCS 24-HOUR, TYPE I DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

```
     TABLE NO.        TIME INCREMENT
5 RAINFL 1                  0.5000

8                 0.0       0.0080    0.0170    0.0260    0.0350
8                 0.0450    0.0550    0.0650    0.0760    0.0870
8                 0.0990    0.1120    0.1260    0.1400    0.1560
8                 0.1740    0.1940    0.2190    0.2540    0.3030
8                 0.5150    0.5830    0.6240    0.6550    0.6820
8                 0.7060    0.7280    0.7480    0.7660    0.7830
8                 0.7990    0.8150    0.8300    0.8440    0.8570
8                 0.8700    0.8820    0.8930    0.9050    0.9160
8                 0.9260    0.9360    0.9460    0.9560    0.9650
8                 0.9740    0.9830    0.9920    1.0000    1.0000
9 ENDTBL
```

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0.
The format for this table is Form #271, Page F-7.

RAINFALL TABLE 2

STANDARD SCS 24-HOUR, TYPE II DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

	TABLE NO.	TIME INCREMENT				
5	RAINFL 2	0.2500				
8	0.0	0.0020	0.0050	0.0080	0.0110	
8	0.0140	0.0170	0.0200	0.0230	0.0260	
8	0.0290	0.0320	0.0350	0.0380	0.0410	
8	0.0440	0.0480	0.0520	0.0560	0.0600	
8	0.0640	0.0680	0.0720	0.0760	0.0800	
8	0.0850	0.0900	0.0950	0.1000	0.1050	
8	0.1100	0.1150	0.1200	0.1260	0.1330	
8	0.1400	0.1470	0.1550	0.1630	0.1720	
8	0.1810	0.1910	0.2030	0.2180	0.2360	
8	0.2570	0.2830	0.3870	0.6630	0.7070	
8	0.7350	0.7580	0.7760	0.7910	0.8040	
8	0.8150	0.8250	0.8340	0.8420	0.8490	
8	0.8560	0.8630	0.8690	0.8750	0.8810	
8	0.8870	0.8930	0.8980	0.9030	0.9080	
8	0.9130	0.9180	0.9220	0.9260	0.9300	
8	0.9340	0.9380	0.9420	0.9460	0.9500	
8	0.9530	0.9560	0.9590	0.9620	0.9650	
8	0.9680	0.9710	0.9740	0.9770	0.9800	
8	0.9830	0.9860	0.9890	0.9920	0.9950	
8	0.9980	1.0000	1.0000	1.0000	1.0000	
9	ENDTBL					

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0. The format for this table is Form #271, Page F-7.

RAINFALL TABLE 3

STANDARD SCS 24-HOUR, TYPE IA DISTRIBUTION
CUMULATIVE RAINFALL TABLE
(REVISED MAY 1982)

```
     TABLE NO.        TIME INCREMENT
5 RAINFL 3                 0.5000

8                 0.0      0.0100   0.0220   0.0360   0.0510
8                 0.0670   0.0830   0.0990   0.1160   0.1350
8                 0.1560   0.1790   0.2040   0.2330   0.2680
8                 0.3100   0.4250   0.4800   0.5200   0.5500
8                 0.5770   0.6010   0.6230   0.6440   0.6640
8                 0.6830   0.7010   0.7190   0.7360   0.7530
8                 0.7690   0.7850   0.8000   0.8150   0.8300
8                 0.8440   0.8580   0.8710   0.8840   0.8960
8                 0.9080   0.9200   0.9320   0.9440   0.9560
8                 0.9670   0.9780   0.9890   1.0000   1.0000
9 ENDTBL
```

Note: On Executive Control use Rainfall Depth in inches and Rainfall Duration of 1.0. The format for this table is Form #271, Page F-7.

Appendix C

Subbasin Parameter Estimation Techniques

METHODOLOGY FOR ESTIMATING EFFECTIVE IMPERVIOUS AREA FOR NATURAL, PARTIALLY URBANIZED AND URBAN WATERSHEDS BASED ON PUBLISHED U.S.G.S. DATA FOR WATERSHEDS THROUGHOUT THE METROPOLITAN AREAS OF PORTLAND AND SALEM, OREGON

By

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Introduction

One of the most difficult and important parameters that must be estimated during the drainage master planning process is the **effective impervious area, EIA**, of a basin or subbasin of interest. **Effective impervious area (EIA) is the portion of the mapped impervious area (MIA) within a basin that is directly connected to the drainage collection system.** EIA includes street surfaces, paved driveways that connect to the street, sideways that are adjacent to curbed street, rooftop is they are directly connected to the curb or if they drain to connected driveways, and parking lots.

EIA is usually **measured as a percentage of total subbasin or basin area.** And in traditional urban runoff modeling, the EIA is usually less than the subbasin's MIA. However, in highly urbanized subbasin's EIA values can approach and even equal MIA values.

The **EIA** of a basin is an **important parameter** in the rainfall to runoff process because it **directly affects the volume of runoff.** In hydrological models like HEC-1, the EIA of a subbasin is the impervious area value that should be used on the LU, LM, LS, and LH records. Models like HEC-1 assume that no precipitation losses occur on the impervious subbasin area that is specified. Thus, the portion of the precipitation that is assumed to land of the impervious area specified will be assumed to be direct runoff. That is why effective impervious area should be used and not mapped impervious area.

Determination of Effective Impervious Area

The direct measurement of EIA is a tedious exercise that is rarely undertaken because drainage planning budgets cannot afford its excessive labor cost. To actually measure the EIA of a basin, the hydraulic connections between the impervious areas and the major drainage collector systems would have to

be cataloged and then evaluated for their effectiveness. This is very time consuming and impractical for most drainage planning and design exercises.

If a basin is gaged, however, the effective impervious area can be estimated by using a rainfall to runoff model like HEC-1 and treating EIA as a calibration parameter. Once reasonable estimates of the precipitation loss components for the pervious areas of the basin are set, the EIA can be adjusted to match the observed runoff volumes. This calibration process would be undertaken for several observed rainfall to runoff events. And the final estimate if EIA would probably be some sort of average or weighted average of those values determined for each individual storm.

Unfortunately in the real world, observed rainfall to runoff data is rarely available so **empirical equations** must be developed. The U.S.G.S. developed estimates of EIA for the over forty watersheds throughout the metropolitan areas of Portland and Salem, Oregon (Laenen, 1980 and 1983). Working with this data, the **U.S.G.S.** also **developed an empirical equation used to estimate EIA** as a function of MIA.

U.S.G.S. EIA Equation

As stated earlier, the U.S.G.S. investigated the EIA parameter for the 42 drainage basins located throughout the metropolitan Portland and Salem area. As part of their rainfall to runoff modeling, the U.S.G.S. optimized the EIA parameter for each of the drainage basins. The measured mapped impervious area (MIA) and the optimized or modeled EIA are presented in Table 1.

It should be noted that the modeling technique used by the U.S.G.S. lumped all of the precipitation excess into a single optimized percentage of the basin area that was assumed to be contributing runoff. This optimized value was defined as the effective impervious area. A potential problem with this technique is that it will over estimate the "true" EIA for MIA's less than 10%. However, for MIA's greater than 10% and less than 50%, it will provide reasonable estimates of the EIA parameter.

Working with these optimized values of "EIA" and their corresponding value of MIA, the following equation was developed by the U.S.G.S. (Laenen, 1983):

$$EIA = 3.6 + 0.43(MIA) \qquad (1)$$

Watershed Characteristics for Basins throughout the Metropolitan Portland and Salem, Oregon area (Laenen, 1980 and 1983)

STATION NUMBER	STATION NAME	BASIN NO.	DRAINAGE AREA (MI^2)	MAP IMP AREA MIA	USGS MODEL EIA	TOTAL SEWERED AREA SA	TOTAL URBAN AREA UA	URBAN SEWERED AREA USA	UA STORAGE AREA ST	LOCAL DRAINAGE SYSTEM	MAJOR TRIBS	MAIN STEM	MIA + SA	BASIN CATEGORY
142580	KELLY CREEK	1	4.16	16	12	25.0	16	100	0.1	S	0	0	41	P1
144690	VANCOUVER SEWER OUTFALL	2	1.00	49	16	86.0	75	100	0.0	S	S	S	135	U2
206320	BEAVERTON CREEK	3	6.63	23	22	54.0	72	75	2.0	S	0	0	77	P2
206330	BEAVERTON CREEK TRIB.	4	0.21	19	15	75.0	58	100	1.1	S	S	0	94	U1
206470	BUTTERNUT CREEK	5	0.82	12	10	15.0	22	68	0.3	S	0	0	27	P1
206900	FANNO CREEK	6	2.37	32	17	57.0	87	66	0.0	S	0	0	89	P2
207800	SINGER CREEK	7	0.28	28	15	20.0	82	24	3.7	S	0	0	48	P1
210400	MOYER CREEK	8	2.04	6	6	1.0	7	14	0.0	0	0	0	7	N
211110	WILL R.(ROBINWOOD) TRIB.	9	1.03	10	4	0.0	31	0	0.0	0	0	0	10	N
211120	WILL R.(OAK GROVE) TRIB.	10	0.74	36	12	0.4	86	0	0.3	0	0	0	36	P1
211130	KELLOGG CREEK	11	2.42	22	15	5.0	65	8	7.0	0	0	0	27	P1
211301	TRYON CREEK TRIB.	12	0.36	32	25	29.0	88	33	0.6	S	0	0	61	P2
211450	JOHNSON CREEK TRIB.	13	0.21	16	7	14.0	41	34	0.0	S	0	0	30	P1
211500	JOHNSON CREEK	14	26.50	7	7	3.4	14	24	0.3	0	0	0	10	N
211604	N.W. 11TH-EVERETT SEWER	15	1.98	36	32	73.0	69	100	0.0	S	S	S	109	U1
211610	S.E. 9TH-MADISON SEWER	16	1.53	39	41	89.0	96	93	0.0	S	S	S	128	U2
211614	N.E. HANCOCK-FLINT SEWER	17	1.36	43	43	90.0	98	92	0.0	S	S	S	133	U2
211617	N.ALBINA-KIRKPATRICK SWR	18	0.95	44	22	88.0	94	94	0.0	S	S	S	132	U2
211618	N. VANCOUVER-OWRSN SWR	19	0.34	46	18	85.0	98	87	0.0	S	S	S	131	U2
211625	S.E. 27TH-BYBEE SEWER	20	0.77	26	28	91.0	93	98	0.0	S	S	S	117	U1
211630	S.E. 27TH-BELMONT SEWER	21	0.54	35	35	94.0	99	95	0.0	S	S	S	129	U2
211800	SALTZMAN CREEK	22	1.48	1	4	0.0	1	0	0.2	0	0	0	1	N
211950	VANCOUVER LAKE TRIB.	23	0.44	30	12	43.0	70	61	2.2	S	S	0	73	U2
213040	COUGAR CREEK	24	2.88	25	11	41.0	50	82	3.8	S	0	0	66	P2
190840	CROISAN CREEK	25	4.54	4	8	7.0	12	58	0.9	S	0	0	11	P1
190930	UPPER PRINGLE CREEK	26	2.93	2	4	0.0	0	0	0.9	0	0	0	2	N
190955	W.F. PRINGLE CREEK	27	3.16	30	18	44.0	73	60	2.5	S	0	0	74	P2
190960	CLARK CREEK	28	1.69	34	20	44.0	88	50	2.5	S	0	0	78	P2
190970	PRINGLE CREEK	29	12.60	22	15	21.0	45	47	4.5	S	0	0	43	P2
191440	BATTLE CREEK	30	5.56	2	3	0.0	0	0	3.9	0	0	0	2	N
191460	WALN CREEK	31	1.47	22	9	34.0	23	100	1.5	S	0	0	56	P2
192100	GLENN CR. @DOAKS FY.RD	32	2.51	8	6	8.0	12	67	0.3	S	0	0	16	P1
192120	GLENN CR. @ORCHARD HT.RD	33	3.31	10	6	11.0	20	55	0.3	S	0	0	21	P1
192150	GIBSON CREEK	34	0.54	2	3	0.0	0	0	0.1	0	0	0	2	N
192210	CLAGGETT CREEK	35	3.08	27	29	72.0	61	100	2.7	S	S	0	99	U1
192215	HAWTHORNE D. @D ST.	36	0.48	43	12	60.0	73	82	0.0	S	S	0	103	U2
192220	HAWTHORNE D. @SUNNYSIDE	37	0.80	53	28	76.0	82	93	0.1	S	S	0	129	U2
192225	HAWTHORNE D. @EASTGATE PK	38	1.40	45	25	86.0	84	100	0.4	S	S	0	131	U1
192230	HAWTHORNE D.@HYACINTH ST	39	1.68	43	23	88.0	84	100	0.4	S	S	0	131	U1
199655	L.PUDDING R.TR.@CORDON RD	40	0.79	15	12	36.0	51	71	9.9	S	0	0	51	P1
199855	L.PUDDING R.TR.@LARDON RD	41	0.27	1	2	0.0	0	0	1.5	0	0	0	2	N
200050	L.PUDDING R.TR.@KALE RD	42	0.75	20	8	66.0	60	100	7.4	S	0	0	86	P2

*NOTES O-OPEN S-SEWERED N-NATURAL P-PARTIALLY URBANIZED U-URBANIZED

Equation (1) has been found to work well for MIA values greater than 10% and less than 50%. The problem with Equation (1) is that it provides unrealistic EIA values for MIA's less than 10% and greater than 50%. In drainage master planning one commonly deals with small subbasins (i.e. 20 to 70 acres) where the ultimately planned mapped impervious area is estimated to be 60% to as much as 90%. The U.S.G.S. equation does not yield reasonable results for these cases.

Therefore, there is a need to develop a relationship between MIA and EIA such that if MIA = 1 then EIA = 0 and if MIA = 100 then EIA = 100. Working with the U.S.G.S. data several equations were developed to satisfy this need.

OTAK EIA Equations

The form of the equation chosen by OTAK, Incorporated to describe the relationship between MIA and EIA is as follows:

$$\mathbf{EIA = A\ (MIA)^{B}} \qquad (2)$$

Where A and B are unique combinations of numbers such that the following criteria is satisfied:

1. If **MIA = 1** THEN **EIA** $\simeq$ **0**

2. If **MIA = 100** then **EIA** $\simeq$ **100**

Working with the U.S.G.S. "modeled" values of EIA for all basins with MIA $\geq$ 4, several empirical equations (i.e. same form as Equation 2) were developed to describe various generalized conditions of subbasins that may be encountered in the drainage master planning process. The first equation presented below provided the best fit for all of the MIA verse EIA data used in the analysis. The remaining equations were based primarily on engineering judgment and experience as it relates to the various subbasin conditions that will affect EIA.

The **OTAK EIA equations** are as follows:

1. **Average basins** where the **local collector systems** for the urban areas within the basin **are predominately storm sewered** with curb and gutter inlets, **no dry wells or** other **drainage retention areas** are know to **exist** and the **rooftops** in the single family residential areas within the basin **are not connected** or piped directly **to the street curb.**

   $$\mathbf{EIA = 0.1\ (MIA)^{1.5},\ MIA \geq 1} \qquad (3)$$

2. **Highly connected basins** where everything in Condition 1 applies **except the residential rooftops are predominately connected to the streets** or storm sewer system.

$$EIA = 0.4\ (MIA)^{1.2},\ MIA \geq 1 \qquad (4)$$

3. **Totally connected basins** where **100%** of the urban area within the basin is **storm sewered** with **all impervious surfaces** appearing to be **directly connected** to the system.

$$EIA = MIA \qquad (5)$$

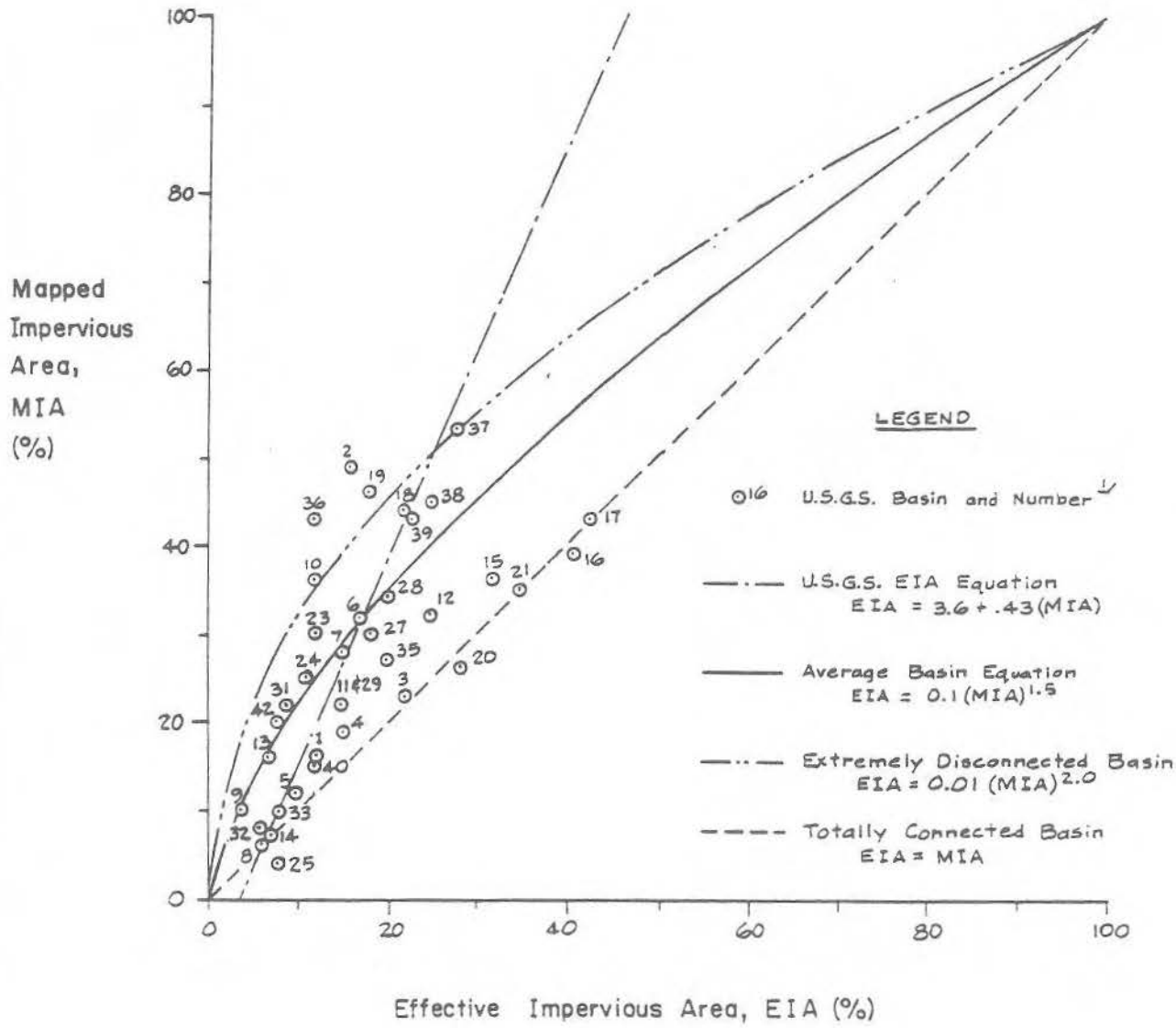
4. **Somewhat disconnected basins** where at **least 50%** of the urban areas within the basin **are not storm sewered** but are served by roadside ditches and the residential rooftops are not directly connected. Or condition 1 may apply but the basin is know to have **a few dry wells or other retention areas.**

$$EIA = 0.04\ (MIA)^{1.7},\ MIA \geq 1 \qquad (6)$$

5. **Extremely disconnected** basins were only a **small percentage** of the urban area within the basin is **storm sewered** or a **significant** portion of the basin area drains to **dry wells or other retention areas.**

$$EIA = 0.01\ (MIA)^{2.0},\ MIA \geq 1 \qquad (7)$$

Figure 1 presents the U.S.G.S. data used in the development of the OTAK EIA equations along with lines described by the U.S.G.S. Equation (1) and OTAK Equations (3), (5), and (7). The variation in the data presented in Figure 1 demonstrates the difficulty in properly estimating the EIA of a drainage basin. It is imperative that the drainage planner or engineer spends some time driving throughout the basin of interest to develop a feeling as to what EIA equation may apply. The greatest strength of the **OTAK EIA equations** presented above is that they can be used to provide reasonable estimates of EIA for all values of MIA. Thus, they can be used in the drainage planning process to **estimate the change in EIA that will occur as a basin becomes urbanized.**



1/ EIA values were based on a U.S.G.S. rainfall to runoff model study. Only points with MIA ≥ 4 were plotted (Laenen, 1980 and 1983).

Figure 1 - Relationships Between MIA and EIA Developed by the U.S.G.S. and OTAK

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METHODOLOGY FOR ESTIMATING LAG TIME OF NATURAL, PARTIALLY URBANIZED AND URBAN WATERSHEDS BASED ON PUBLISHED U.S.G.S. DATA FOR WATERSHEDS THROUGHOUT THE METROPOLITAN AREAS OF PORTLAND AND SALEM, OREGON

By

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Introduction

Lag time (t_L) of a watershed can be defined as **the time measured between the center of mass of the rainfall occurring on the watershed and the center of mass of the runoff** observed to occur at the outlet of the watershed. Numerous studies (Leopold, 1968; Anderson, 1970; Laenen, 1980; and Laenen, 1983 to cite only a few) have shown that as **a watershed urbanizes** (i.e. as the mapped impervious area, MIA, of a watershed increases) **the lag time of the watershed will decrease.**

Anderson (1970) analyzed the rainfall to runoff response of over 80 watersheds located throughout Northern Virginia, Baltimore, Maryland, and Louisville, Kentucky. The mapped impervious areas of these watersheds ranged from < 1% to 100%. His analysis concluded that the measured **lag time** of these watersheds could be directly **related to** their physical characteristics which included the **length and slope of** their **main channels and** their **mapped impervious areas** (see Figure 1).

Anderson (1970) developed several equations that could be used to estimate the lag time of a watershed based on the degree of urbanization that existed within the basin.

They are as follows:

For **natural basins** (Class N) with **MIA** $\leq$ 3%,

$$t_L = 4.64(L/S^{0.5})^{0.42} \qquad (1)$$

For **fully urbanized basins** (class U) with 33 $\leq$ MIA $\leq$ 100% and **fully sewered including main channels**

$$t_L = 0.56(L/S^{0.5})^{0.52} \qquad (2)$$

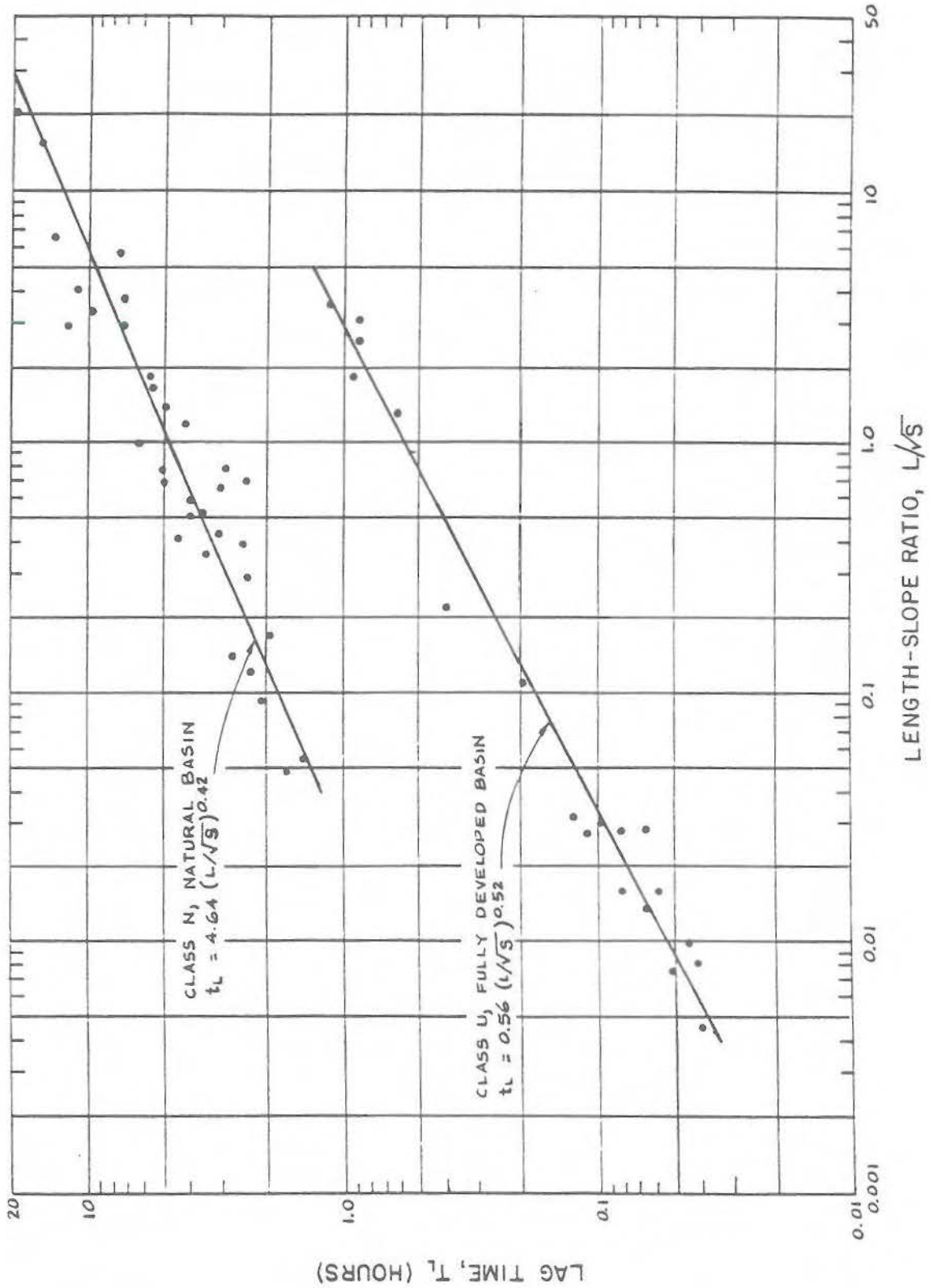

Figure 1 - Relationship Between Lag Time and Basin Length-Slope Ratio (From Anderson 1970)

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For **highly urbanized basins** (Class B) with **20 ≤ MIA ≤ 30%** and **main channels open**

$$t_L = 0.9(L/S^{0.5})^{0.50} \quad (3)$$

Where:

t_L = lag time of the basin (hrs)

L = length of the main water course (i.e. most defined course) measured form the basin outlet upstream to the watershed divide (mi)

S = slope of the main water course (ft/mile) measured between points located at 10% and 85% of L, respectively

All of Anderson's regression equations fit the observed data quite well with the standard error of estimates ranging from 17.7 to 26.0%. In addition, all of the equations have the same form which can be rewritten as:

$$t_L = A(L/S^{0.5})^B \quad (4)$$

Where **A** and **B** are coefficients whose **values** are **based** on the relative **amount of imperviousness** within the basin and the **characteristics of the drainage collector systems** within the basin. Note that the **"A"** coefficient **decreases as impervious area increases** whereas the **"B"** coefficient **increases with increases in impervious area.**

Study Objectives

Working with the form of the Anderson equations and the published U.S.G.S. data for metropolitan Portland and Salem, Oregon (Laenen, 1980 and 1983), the **major objective** of the study was to **develop a method that could be used to estimate the lag time of a watershed as a function of its physical characteristics.** The technique would have to be very simple in that the physical characteristics used to estimate the lag time would have to be easily measured or estimated. And the final requirement was that the method would provide reasonably good estimates of the actual measured lag time published by the U.S.G.S.

The final methodology, which has been presented below, can be used to quickly estimate the subbasin lag time as a function of easily measured physical characteristics. This technique can be **used in the drainage master planning process to**

provide an estimate of the change in a subbasin's lag time that will result from the ultimately planned development within the subbasin. This method allows the modeler to use as part of the drainage planning process the simple SCS unit hydrograph technique included in HEC-1. This lag time estimation technique will provide reasonable estimates of this single parameter required by the SCS unit hydrograph. And it will be especially helpful in subbasins where little or no urbanization has occurred to date but considerable urban development is planned for the future.

Lag Time Estimation Technique

The detailed description of the analysis methods used to develop this technique is not included herein. It will be the subject of a soon to be published technical paper.

The technique is based on the following equation:

$$t_L = K\ A\ (CL/(CSL)^{0.5})^B \qquad (5)$$

where:

t_L = lag time of the watershed (hrs)

CL = length of the main channel (i.e. most well defined water course) measured from the basin outlet upstream to the basin divide (mi)

CSL = the slope of the main channel (ft/mi) measured from .1 CL to .85 CL as follows:

$$CSL = \frac{EL85 - EL10}{.75\ CL} \qquad (6)$$

where:

EL10 = the flow line elevation (ft) of the main channel measured at a location which is approximately 10% of the distance CL from the outlet of the basin

EL85 = same as above (ft) with measurement take at approximately 85% of the distance CL from the outlet

"K" is a calibration factor used to uniformly adjust the computed subbasin lag times when observed rainfall and runoff data are available at the outfall of a multiple

subbasin watershed. Within each subbasin the "K" factor appears to be related to the percentage of the urbanized subbasin area that is served by storm sewer systems (i.e. urban sewered area, USA).

For partially urbanized drainage basins (P basins) whose **sewered areas** measured as a percentage of the urban area **(USA) is less than 40%** and whose **mapped impervious area (MIA) is between 6% and 50%**, the following equation can be used to estimate **K**.

$$K = 1.3 + .02(MIA) - .02(USA) \qquad (7)$$

if $K < 1.0$ use 1.0

where:

MIA = mapped impervious area measured as percentage of total basin area (%)

USA = urban sewered area measured as a percentage (%) of the urban area that is sewered as follows:

$$USA = 100\ (SA/UA) \qquad (8)$$

where:

SA = sewered area measured as a percentage (%) of the total basin area

UA = urban area measured as a percentage (%) of the total basin area

"B" is a coefficient whose value is **based on the mapped impervious area, MIA**, of the basin as follows:

$$B = 0.42\ (MIA)^{0.053} \qquad (9)$$

if $MIA < 1$, use 1

Equation (9) was **based on** the variation of this "B" parameter observed by **Anderson (1970)**. It was assumed that if $MIA = 1$ then $B = 0.42$ and if $MIA = 60$ then $B = 0.52$. There was not enough local data throughout Oregon to establish a different relationship for B. Equation (9) was used to set the "B" coefficient for any value of MIA so the "A" coefficient could be properly evaluated.

"A" is a coefficient whose value is based on the **mapped impervious area** and the **type of major drainage collector system** that serves the basin or subbasin as follows:

1. **Natural** (N), **partially urbanized** (P) and **urban basins** where the **major tributaries and main stem are open channels** (the local drainage collectors in the urban area can be storm sewered):

$$A = 12.0e^{-0.042(MIA)}, \quad MIA \geq 1 \qquad (10)$$

2. **Highly urbanized basins** (U) where **both the local drainage collectors and the major tributaries are closed pipes** (the main stem can be open channel):

$$A = 3.6e^{-0.031(MIA)}, \quad MIA \geq 1 \qquad (11)$$

3. A **single simplified equation** that can be used for all types of basins without concern for the type of drainage collector systems:

$$A = 13.2e^{-.055(MIA)}, \quad MIA \geq 1 \qquad (12)$$

Equation (12) will provide the best fit to all the data combined but it will **under estimate** the **"A"** values **for Condition 1 above** and **over estimate** the **"A"** values **for Condition 2 above. Equation (12) is not recommended for use** unless a quick and not as accurate planning level estimate is desired.

U.S.G.S. Published Data

The U.S.G.S. watershed characteristics collected for basins throughout the greater Portland and Salem, Oregon area are tabulated in Table 1. **Table 1** also **presents** the **estimated values of K, A, B, and t_L (computed)** based on the lag time estimation technique that was outlined above. The error of estimate in both absolute hours and percent of t_L measured is also shown.

It should be noted that **six drainage basins (i.e. Nos. 3, 7, 11, 14, 25, and 41) were eliminated from the analysis** for a number of reasons. The first and foremost was the **existence of excessive storage** within the basins which resulted in a dramatic increase in the measured lag time at the outlet of the basins. Basins that have excessive storage behind high embankment culverts and contain numerous natural depressions that provided considerable storage include Beaverton Creek (3), Singer Creek (7), Kellogg Creek (11), and Johnson Creek (14). The Little Pudding River tributary at Lardon Road (41)

Watershed Characteristics for Basins throughout the Metropolitan Portland and Salem, Oregon area (Laenen, 1980 and 1983)

STATION NUMBER	STATION NAME	BASIN NO.	DRAINAGE AREA (MI^2)	MAP IMP AREA MIA	USGS MODEL KIA	TOTAL SEWERED AREA SA	TOTAL URBAN AREA UA	URBAN SEWERED AREA USA	UA STORAGE AREA ST	LOCAL DRAINAGE SYSTEM	MAJOR TRIBS	MAIN STEM	MIA + SA	BASIN CATEGORY
142580	KELLY CREEK	1	4.16	16	12	25.0	16	100	0.1	S	O	O	41	P1
144690	VANCOUVER SEWER OUTFALL	2	1.00	49	16	86.0	75	100	0.0	S	S	S	135	U2
206320	BEAVERTON CREEK	3	6.63	23	22	54.0	72	75	2.0	S	O	O	77	P2
206330	BEAVERTON CREEK TRIB.	4	0.21	19	15	75.0	58	100	1.1	S	S	O	94	U1
206470	BUTTERNUT CREEK	5	0.82	12	10	15.0	22	68	0.3	S	O	O	27	P1
206900	FANNO CREEK	6	2.37	32	17	57.0	87	66	0.0	S	O	O	89	P2
207800	SINGER CREEK	7	0.28	28	15	20.0	82	24	3.7	S	O	O	48	P1
210400	NOYER CREEK	8	2.04	6	6	1.0	7	14	0.0	O	O	O	7	N
211110	MILL R.(ROBINWOOD) TRIB.	9	1.03	10	4	0.0	31	0	0.0	O	O	O	10	N
211120	MILL R.(OAK GROVE) TRIB.	10	0.74	36	12	0.4	86	0	0.3	O	O	O	36	P1
211130	KELLOGG CREEK	11	2.42	22	15	5.0	65	8	7.0	O	O	O	27	P1
211301	TRYON CREEK TRIB.	12	0.36	32	25	29.0	88	33	0.6	S	O	O	61	P2
211450	JOHNSON CREEK TRIB.	13	0.21	16	7	14.0	41	34	0.0	S	O	O	30	P1
211500	JOHNSON CREEK	14	26.50	7	7	3.4	14	24	0.3	O	O	O	10	N
211604	N.W. 11TH-EVERETT SEWER	15	1.98	36	32	73.0	69	100	0.0	S	S	S	109	U1
211610	S.E. 9TH-MADISON SEWER	16	1.53	39	41	89.0	96	93	0.0	S	S	S	128	U2
211614	N.E. HANCOCK-FLINT SEWER	17	1.36	43	43	90.0	98	92	0.0	S	S	S	133	U2
211617	N.ALBINA-KIRKPATRICK SWR	18	0.95	44	22	88.0	94	94	0.0	S	S	S	132	U2
211618	N. VANCOUVER-OWRSN SWR	19	0.34	46	18	85.0	98	87	0.0	S	S	S	131	U2
211625	S.E. 27TH-BYBEE SEWER	20	0.77	26	28	91.0	93	98	0.0	S	S	S	117	U1
211630	S.E. 27TH-BELMONT SEWER	21	0.54	35	35	94.0	99	95	0.0	S	S	S	129	U2
211800	SALTZMAN CREEK	22	1.48	1	4	0.0	1	0	0.2	O	O	O	1	N
211950	VANCOUVER LAKE TRIB.	23	0.44	30	12	43.0	70	61	2.2	S	S	O	73	U2
213040	COUGAR CREEK	24	2.88	25	11	41.0	50	82	3.8	S	O	O	66	P2
190840	CROISAN CREEK	25	4.54	4	8	7.0	12	58	0.9	S	O	O	11	P1
190930	UPPER PRINGLE CREEK	26	2.93	2	4	0.0	0	0	0.9	O	O	O	2	N
190955	W.F. PRINGLE CREEK	27	3.16	30	18	44.0	73	60	2.5	S	O	O	74	P2
190960	CLARK CREEK	28	1.69	34	20	44.0	88	50	2.5	S	O	O	78	P2
190970	PRINGLE CREEK	29	12.60	22	15	21.0	45	47	4.5	S	O	O	43	P2
191440	BATTLE CREEK	30	5.56	2	3	0.0	0	0	3.9	O	O	O	2	N
191460	WALN CREEK	31	1.47	22	9	34.0	23	100	1.5	S	O	O	56	P2
192100	GLENN CR. @DOAKS FY.RD	32	2.51	8	6	8.0	12	67	0.3	S	O	O	16	P1
192120	GLENN CR. @ORCHARD HT.RD	33	3.31	10	8	11.0	20	55	0.3	S	O	O	21	P1
192150	GIBSON CREEK	34	0.54	2	3	0.0	0	0	0.1	O	O	O	2	N
192210	CLAGGETT CREEK	35	3.08	27	20	72.0	61	100	2.7	S	S	O	99	U1
192215	HAWTHORNE D. @D ST.	36	0.48	43	12	60.0	73	82	0.0	S	S	O	103	U2
192220	HAWTHORNE D. @SUNNYSIDE	37	0.80	53	28	76.0	82	93	0.1	S	S	O	129	U2
192225	HAWTHORNE D. @EASTGATE PK	38	1.40	45	25	86.0	84	100	0.4	S	S	O	131	U1
192230	HAWTHORNE D.@HYACINTH ST	39	1.68	43	23	88.0	84	100	0.4	S	S	O	131	U1
199655	L.PUDDING R.TR.@CORDON RD	40	0.79	15	12	36.0	51	71	9.9	S	O	O	51	P1
199855	L.PUDDING R.TR.@LARDON RD	41	0.27	1	2	0.0	0	0	1.5	O	O	O	2	N
200050	L.PUDDING R.TR.@HALE RD	42	0.75	20	8	66.0	60	100	7.4	S	O	O	86	P2

*NOTES O-OPEN S-SEWERED N-NATURAL P-PARTIALLY URBANIZED U-URBANIZED

TABLE 1 (CONTINUED)

Watershed Characteristics for Basins throughout the Metropolitan Portland and Salem, Oregon area (Laenen, 1980 and 1983)

STATION NUMBER	STATION NAME	BASIN NO.	BASIN SLOPE (FT/MI) BSL	CHANNEL SLOPE (FT/MI) CSL	CHANNEL LENGTH (MILES) CL	CL/ SQRT CSL	"X"	"A"	"B"	LAG TIME (HOURS) TLAG MEASURED	COMPUTED	ERROR OF ESTIMATE (HOURS)	ERROR OF ESTIMATE (PERCENT)
142580	KELLY CREEK	1	260	48	4.70	0.678	1.00	6.13	0.49	4.96	5.07	0.11	2
144690	VANCOUVER SEWER OUTFALL	2	270	108	2.20	0.212	1.00	0.79	0.52	0.35	0.35	0.00	1
206320	BEAVERTON CREEK	3	400	150	5.70	0.465	1.00	4.57	0.50	14.00	3.13	-10.87	-78
206330	BEAVERTON CREEK TRIB.	4	380	160	0.62	0.046	1.00	2.00	0.49	0.43	0.44	0.01	3
206470	BUTTERNUT CREEK	5	510	240	2.40	0.155	1.00	7.25	0.48	3.00	2.97	-0.03	-1
206900	FANNO CREEK	6	860	200	2.50	0.177	1.00	3.13	0.50	1.87	1.31	-0.56	-30
207800	SINGER CREEK	7	580	310	0.77	0.044	1.86	3.70	0.50	4.20	1.43	-2.77	-66
210400	NOYER CREEK	8	320	175	1.90	0.144	1.42	9.33	0.46	4.75	5.41	0.66	14
211110	WILL R.(ROBINWOOD) TRIB.	9	890	400	2.10	0.105	1.50	7.88	0.47	4.55	4.06	-0.49	-11
211120	WILL R.(OAK GROVE) TRIB.	10	280	160	1.80	0.142	2.02	2.65	0.51	2.12	1.99	-0.13	-6
211130	KELLOGG CREEK	11	320	16	2.70	0.675	1.74	4.76	0.49	10.70	6.82	-3.88	-36
211301	TRYON CREEK TRIB.	12	440	210	0.88	0.061	1.94	3.13	0.50	0.93	1.48	0.55	59
211450	JOHNSON CREEK TRIB.	13	300	95	1.10	0.113	1.62	6.13	0.49	1.88	3.43	1.55	83
211500	JOHNSON CREEK	14	580	32	13.60	2.440	1.44	8.94	0.47	25.00	19.51	-5.49	-22
211604	N.W. 11TH-EVERETT SEWER	15	608	230	3.40	0.224	1.00	1.18	0.51	0.75	0.55	-0.20	-26
211610	S.E. 9TH-MADISON SEWER	16	136	66	2.30	0.283	1.00	1.07	0.51	0.51	0.56	0.05	11
211614	N.E. HANCOCK-FLINT SEWER	17	89	58	2.80	0.368	1.00	0.95	0.51	0.56	0.57	0.01	1
211617	N.ALBINA-KIRKPATRICK SWR	18	120	92	1.60	0.167	1.00	0.92	0.51	0.32	0.37	0.05	15
211618	N. VANCOUVER-OWRSN SWR	19	223	107	1.50	0.145	1.00	0.86	0.51	0.28	0.32	0.04	14
211625	S.E. 27TH-BYBEE SEWER	20	317	166	1.20	0.093	1.00	1.61	0.50	0.28	0.49	0.21	76
211630	S.E. 27TH-BELMONT SEWER	21	146	42	1.20	0.185	1.00	1.22	0.51	0.28	0.52	0.24	85
211800	SALTZMAN CREEK	22	1980	580	2.10	0.087	1.00	11.51	0.42	4.70	4.13	-0.57	-12
211950	VANCOUVER LAKE TRIB.	23	140	40	1.10	0.174	1.00	1.42	0.50	0.28	0.59	0.31	110
213040	COUGAR CREEK	24	270	50	4.00	0.566	1.00	4.20	0.50	3.83	3.16	-0.67	-17
190840	CROISAN CREEK	25	807	118	5.21	0.480	1.00	10.14	0.45	4.00	7.28	3.28	82
190930	UPPER PRINGLE CREEK	26	526	51	3.29	0.461	1.00	11.03	0.44	7.00	7.87	0.87	12
190955	W.F. PRINGLE CREEK	27	317	78	3.90	0.442	1.00	3.40	0.50	3.20	2.26	-0.94	-29
190960	CLARK CREEK	28	394	118	2.82	0.260	1.00	2.88	0.51	1.80	1.45	-0.35	-19
190970	PRINGLE CREEK	29	151	30	6.38	1.165	1.00	4.76	0.49	5.20	5.14	-0.06	-1
191440	BATTLE CREEK	30	712	86	4.71	0.508	1.00	11.03	0.44	9.70	8.21	-1.49	-15
191460	WALN CREEK	31	500	70	2.27	0.271	1.00	4.76	0.49	2.30	2.50	0.20	9
192100	GLENN CR. @DOAKS FY.RD	32	702	232	3.22	0.211	1.00	8.58	0.47	3.50	4.14	0.64	18
192120	GLENN CR. @ORCHARD HT.RD	33	926	180	4.43	0.330	1.00	7.88	0.47	3.20	4.66	1.46	46
192150	GIBSON CREEK	34	622	360	1.85	0.098	1.00	11.03	0.44	3.50	4.00	0.50	14
192210	CLAGGETT CREEK	35	68	11	4.82	1.453	1.00	1.56	0.50	2.20	1.88	-0.32	-15
192215	HAWTHORNE D. @D ST.	36	54	5	1.65	0.753	1.00	0.95	0.51	0.50	0.82	0.32	64
192220	HAWTHORNE D. @SUNNYSIDE	37	52	8	2.40	0.833	1.00	0.70	0.52	0.80	0.63	-0.17	-21
192225	HAWTHORNE D. @EASTGATE PK	38	55	10	3.40	1.086	1.00	0.89	0.51	1.40	0.93	-0.47	-34
192230	HAWTHORNE D.@HYACINTH ST	39	56	12	4.24	1.224	1.00	0.95	0.51	1.80	1.05	-0.75	-42
199655	L.PUDDING R.TR.@CORDON RD	40	61	15	1.86	0.480	1.00	6.39	0.48	5.30	4.48	-0.82	-15
199855	L.PUDDING R.TR.@LARDON RD	41	41	3	0.84	0.477	1.00	11.51	0.42	2.40	8.43	6.03	251
200050	L.PUDDING R.TR.@HALE RD	42	29	5	1.57	0.702	1.00	5.18	0.49	3.10	4.35	1.25	40
												0.03	11

was eliminated because approximately 80% of the basin **contained agricultural tiles** (i.e. underground drains) which **dramatically decreased the measured lag time.** Croisan Creek (25) in the Salem area was also eliminated for a dramatic decrease in measured lag time. The reason for its quick response to rainfall has not yet been determined. It may contain agricultural tiles or perhaps the predominately downstream location of its existing urban area could explain the rapid response of this long, narrow, and steep drainage.

The results of the computed lag time verse the published "measured" values are presented in Figure 2. The computed lag times in Figure 2 were based in "A" values obtained from Equation (10) for basins classified as natural (N) or partially urbanized (P) and Equation (11) for basins classified as urban (U). Basin Nos. 3, 7, 11, 14, 25, and 41 have been eliminated from Figure 2.

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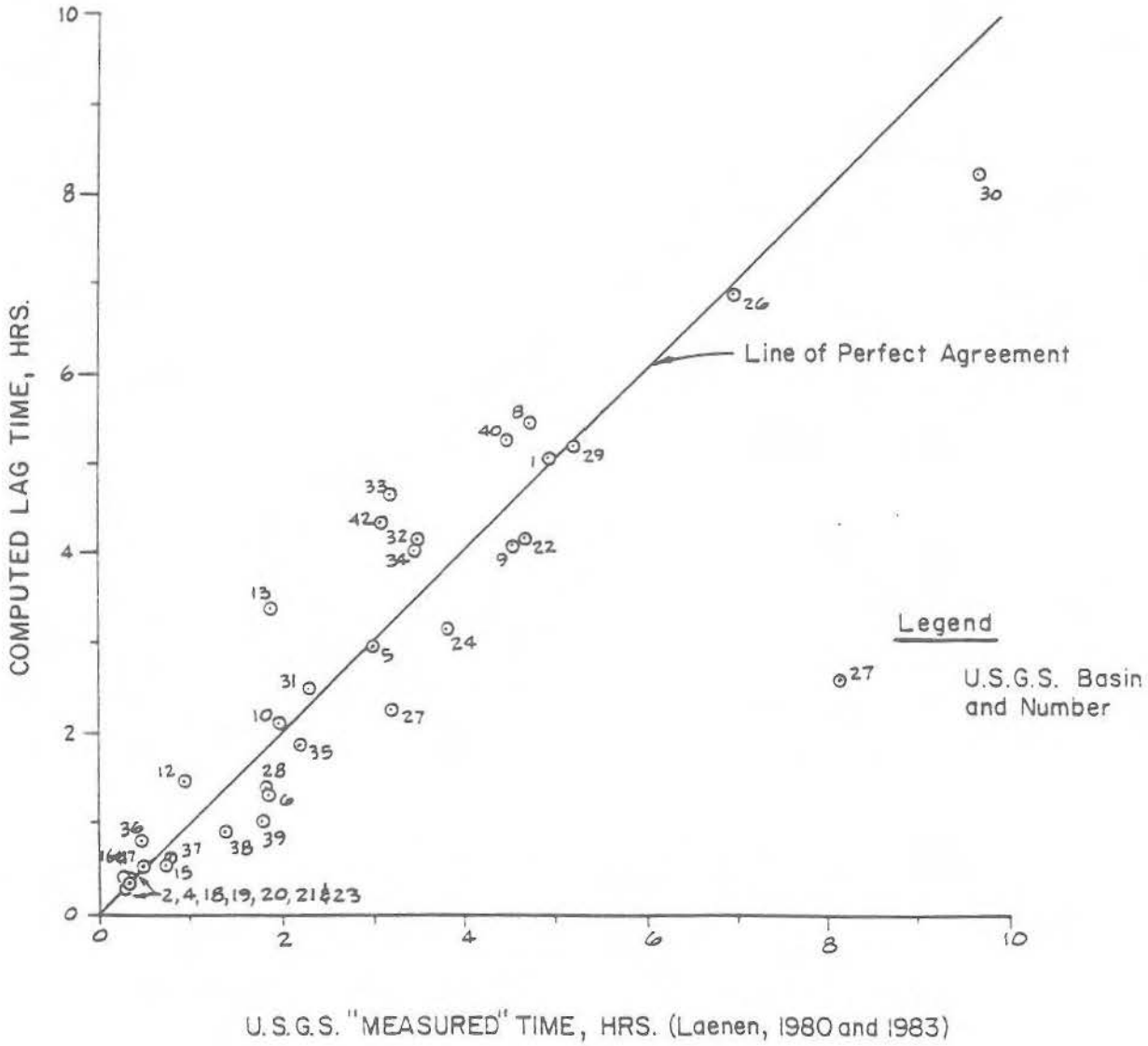

Figure 2 – Computed Versus Measured Lag Times for Metropolitan Portland and Salem, Oregon

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