# Chapter 1

# OBJECTIVES OF THE STUDY

## INTRODUCTION

This section of the Study presents the objectives and goals of the Woodburn Storm Drainage Study including issues pertinent to the City and its managers and decision-makers. This Study is to be used in context with the land use and planning decisions and documents in effect at the time of the Study's creation. As development and environmental conditions in the study area change and objectives concerning growth patterns and land use change, the recommendations of this Study should be revisited and either confirmed or modified to reflect the new situation. This Study was developed between 1996 and 2001. While the general information and proposed recommendations for this Study have been updated for the year 2001, the hydrologic and hydraulic modeling is based primarily on data and assumptions made for the existing conditions in 1996. Specific information about the storm water system can be obtained through the City Public Works Department including results of a storm system inventory conducted in 1999.

## REPORT FORMAT

This study outlines the storm drainage system capital projects that the City of Woodburn will need to implement soon. It also identifies capital projects that should be incorporated into the City's Capital Improvements Program (CIP) as development and growth require their implementation. All of these projects are needed to maintain a high level of runoff control while protecting public and private structures and overall public health. Since the City is served by two natural drainageways, Senecal Creek and Mill Creek and the two drainages have separate contributing areas, many of the report sections are formatted to discuss each drainage separately. A review of the detail of the table of contents will assist the reader in finding specific information about drainage facilities, runoff quantities and recommendations in each basin.

## OBJECTIVES OF THE STUDY

The following objectives outline the basis for this Study and are used to guide the analysis and decision process in arriving at the recommendations for drainage system modifications and improvements.

**1. To provide a methodical and documented guide for development of storm drainage facilities in the City**

a. by identifying problem areas that currently exist in the major drainage facilities.

b. by identifying potential problem areas that will become worse with development pressures.

c. by providing guidelines for future development and supporting storm drainage facilities.

**2. To protect public and private property from flood risk**

a. by identification of areas which frequently flood

b. by proposing system improvements to mitigate conditions which contribute to flooding conditions.

c. by selection of facilities which will convey storm water quantities deemed by the Study to be beyond the reasonable risk.

**3. To minimize the public and private costs of drainage facilities**

a. by optimizing use and conveyance capacity of existing facilities

b. by routing storm flows through natural channels, taking advantage of the capacities of those facilities.

c. by making recommendations that will provide maximum benefit to the public while minimizing cost.

d. by retaining natural drainageways in lieu of constructing structural systems in their place.

**4. To make optimum use of the existing drainage facilities**

a. by identifying natural flood storage areas and preserving and enhancing the capacity of those areas for similar use in the future.

b. by routing storm flow in a manner that takes advantage of the natural carrying capacity of existing drainageways.

c. by identifying critical elements of the existing manmade system and allowing for their continued preservation and maintenance.

**5. To create a system of drainage facilities that will convey design storm runoff safely**

a. by creation of a hydraulic model of the drainage system

b. by identifying those areas where storm runoff detention can safely be routed and stored during periods of high storm runoff.

**6. To create a drainage system that will minimize maintenance and repair costs**

a. by identifying opportunities to maximize the use of public rights-of-way for drainage routes.

b. by recommending drainage facilities that lend themselves to ease of maintenance and repair.

**7. To provide a sound planning document in the form of a Detention Policy to guide future development within the drainage area**

a. by creating a document for public review which will guide development of drainage facilities as the City grows and expands its service area.

**8. To provide flow and water surface elevations and related data for Senecal Creek and Mill Creek for the 25- and 100-year flood conditions**

a. by modeling the hydrologic conditions in each drainage way under estimated peak runoff conditions in each drainage watershed.

## DEFINITIONS

Build-out Impervious Area - The estimated impervious area associated with full and complete development of the parcel in question when the land is developed as permitted in the City's Comprehensive Planning Document. Parcels considered may be undeveloped or under-developed parcels of land or acreage within Urban Growth Boundary.

City - City of Woodburn, Oregon

Concentration of Flow - The natural or created diversion of overland flow into a pipe, ditch, channel or other stormwater conveyance.

Datum – All references to flood elevations are based on the National Geodetic Vertical Datum of 1929 (NGVD29). Use of other datum should be adjusted to the NGVD 29 base.

Drainageways – are located on public or private property and specifically classified as follows:

***Primary*** - A major drainage system serving an upstream area of 150 acres or greater, some or all of which are located within the City's Urban Growth Boundary. Specifically, Primary Drainageways include Mill Creek, the Stubb Road Tributary of Mill Creek, Senecal Creek and the East Tributary of Senecal Creek and other major drainageways as may be determined by the City.

***Secondary*** - A natural, or manmade, stormwater conveyance system which collects stormwater from a land area of between 50 acres and 150 acres including all upstream areas. Typically, secondary drainageways are small tributaries to Mill Creek and Senecal Creek or their Primary Drainageways.

***Local*** - All other water routes not classified as Primary or Secondary Drainageways which carry stormwater or natural runoff. Typically, these include culverts, storm drains, ditches and other facilities which carry flows originating in an area of 50 acres or less.

Floodplain - The area within and adjacent to the City's primary drainageways, which is inundated by the 100-year runoff event, also, referred to as "Special Flood Hazard Area".

Floodway - See City of Woodburn, Ordinance No. 2018, Section 2 and Section 3, dated March 27, 1989. (Ordinance No. 2018 can be found in Appendix 'B' of this Study.)

Floodway Fringe – The area between the floodway and the boundary of the 100-year flood is termed the floodway fringe. The floodway fringe encompasses the portion of the flood plain that could be completely obstructed without increasing the water surface elevation of the 100-year flood by more than 1.0 foot at any point.

Natural Drainageway - Any well-defined conveyance of stormwater which is not manmade. Alteration of any natural drainageways by diverting, culverting, armoring, deepening, or other similar activity does not change the category of such features, unless all, or nearly all, natural characteristics have been replaced with constructed facilities.

Overland Flow - Stormwater runoff which flows across sloping land in a diffused, non-concentrated manner.

Pre-Development - The existing, natural hydrologic condition of a site immediately prior to development or improvements.

Private Stormwater Facilities - Any stormwater facility which is not owned, maintained or operated by the City of Woodburn or any other public agency.

Post-Development - The hydrologic conditions of a site following development or improvements, including any new street or sidewalk construction.

Public Stormwater Facilities - All natural drainageways; also any stormwater facility which is owned by the City or located within a public utility easement, or public storm drainage easement that is granted to the City, but excluding any storm drainage pipe which crosses such easement for the specific purpose of serving private property or properties.

Stormwater Facility - Any man-made stormwater collection, conveyance or detention facility, including culverts, storm drains, manholes, catch basins, inlets, detention pipes and ponds, ditches, channels and swales, ownership of which may be public, private or shared.

Storm Return Frequency - The frequency that a storm of a given magnitude is statistically likely to occur, e.g., a 25-year storm event is a storm of such magnitude that has a statistical chance of 0.04 of occurring during a particular year.

System Development Charge – System Development Charges (SDC's) are fees charged in accordance with state law as implemented through Council-approved ordinances. Charges and fees are levied on new development in proportion to the impact on public facilities created by the development.

## SUPPORTING DOCUMENTS

Documents used as reference in the preparation of this Study include the following:

- Comprehensive Planning Document, City of Woodburn, latest revision
- Zoning and Land Use Maps, City of Woodburn, latest revision
- Storm Water Management Plan, City of Woodburn, Robert E. Meyer Consulting Engineers; October, 1978
- City of Woodburn Flood Insurance Study (FIS), Federal Emergency Management Agency (FEMA), 1974
- Storm Drain Inventory, June 1999, Crane & Merseth Engineering/Surveying (Appendix A)
- City of Woodburn, Ordinance No. 2018, Council Bill No. 1138, dated March 27th 1989 (Appendix B)
- City of Woodburn, Senecal Creek East Tributary Capacity Analysis, Oakley Engineering, Inc., March 1994 (Appendix C)
- Elevation and Cross Section Surveys along Mill & Senecal Creeks, DaHaas & Associates, various dates

# Chapter 2

# WATERSHED DEFINITION

## INTRODUCTION

Woodburn is situated primarily in the drainageway of Mill Creek and Senecal Creek. A very small portion of the City drains to the Pudding River, however, this portion is so small that the characteristics of the Pudding River watershed do not impact the analysis or findings of this Study. This section of the report describes the general characteristics of the watershed that contributes storm runoff to the Mill Creek and Senecal Creek drainageways as they cross the City's service area. It also includes areas within the City that contribute storm runoff directly to the two streams.

The City of Woodburn is the municipal agency with direct responsibility for storm water control and planning within the City's legally defined limits. The City of Woodburn does not have jurisdiction over all the lands located within the watersheds of the two major streams or that impact the drainage facilities within the City's control. The areas of the Senecal Creek and Mill Creek watersheds south of the Woodburn city limits are located within Marion County which has land use jurisdiction in the watershed.

Characteristics to be described include definition of the study area, annual and seasonal rainfall, topography, major natural features, soils conditions, current and projected land uses, and a description of the drainage facilities that serve the study area.

## STUDY AREA DEFINITION

The study area for the Storm Drain Master Plan is comprised of the area within the city limits, the Urban Growth Boundary and those areas immediately surrounding the City which contribute runoff flows to Mill Creek and Senecal Creek upstream of the City. This comprises approximately 9447 acres.

### Location

Woodburn is located in the French Prairie area of the north Willamette Valley approximately 25 miles south of the Portland metropolitan area and 12 miles north of the City of Salem. Two major highways traverse the City; Interstate 5 along the west side of the City and 99E along the east side of the City. Both routes run generally north-south through Woodburn. Oregon highway 214 is an east-west route through the City, separating the northerly third of the City from the south areas.

In addition, the City is bisected by the Southern Pacific Railroad main line. The railroad extends north-south through Woodburn and parallels Front Street through the City. These major transportation routes create a significant barrier to surface water flows.

The study area used to develop this Master Plan is shown on Figure 1, Senecal & Mill Creek Drainage Basin Boundaries. The major features mentioned above are noted on this figure.

### Natural Drainageways

Two natural drainageways cross the Woodburn study area from south to north. These are Senecal Creek along the west side of the study area and Mill Creek which bisects the City. The most easterly side of the City can be considered in the Pudding River watershed as small portions of the City generally (east of highway 99E) naturally drain to that watercourse. A few storm drainage facilities constructed and maintained by the City are routed to the Pudding River. These are small systems and are located so as to have little to no impact on the remaining natural and man-made drainageways located in the City.

Comprehensive Plan, for the land use plan as of June 2001.

2. Rainfall is assumed to be constant throughout the watershed for the design storm used in the calculations.

3. Existing impervious areas were computed from aerial photos of the Woodburn area taken in 1994. Estimates of future impervious areas are based on these aerial photos where similar land uses and full buildout of areas existed. (Note: More current aerial photos were taken in April, 2001 and are available at the City of Woodburn, Public Works Dept.)

4. Model parameters such as soil types within each basin and subbasin are taken from the relevant publications prepared by federal and state agencies.

5. The design storm used in this analysis is the 24-hour storm with recurrence intervals of 2-, 5- 25- and 100-years. Recurrence interval or annual probabilities of storm occurrence is as follws:

| Recurrence Interval (Frequency) (years) | Annual Probability of Occurrence |
|---|---|
| 2 | 0.50 |
| 5 | 0.20 |
| 25 | 0.04 |
| 100 | 0.01 |

It is apparent that the annual probability is the reciprocal of the recurrence interval indicating that large storms occur with less frequency than do smaller storms. A key assumption in development of the hydraulic analysis is the degree of protection from floodwaters that the community wishes to assure balanced against the ability of the community to pay for that added protection. It is also prudent, from a public works management point-of-view, to plan for maximum return on investment knowing that the community cannot practically afford to protect its property against all conceivable storm events.

Years of experience by communities like Woodburn has shown that planning drainageway facilities to convey storm runoff with a recurrence interval of 25 years is an affordable goal while providing reasonable flood protection.

## MODELING AND ANALYSIS BASIS AND ASSUMPTIONS

Figure 1, Senecal & Mill Creek Drainage Basin Boundaries, shows the entire study area including the portions of Senecal Creek and Mill Creek that flow through the City of Woodburn. The study area boundary is set by topographical features. Since one of the stated purposes of this Study is to determine the existing flow and water surface elevations in each stream during specified runoff conditions and, from that create a plan, definition of the study area itself is very important.

The most current hydraulic information currently available for Senecal and Mill Creeks includes the following reports:

1) "City of Woodburn Flood Insurance Study (FIS)", Federal Emergency Management Agency (FEMA), 1978.

2) "City of Woodburn Storm Water Management Plan", Robert E. Meyer Consultants, October, 1978.

Since publication of these two studies, several significant changes have occurred which affect hydrologic and hydraulic conditions in Senecal Creek and Mill Creek. The primary factors which account for differences between this Study and previous flow and flood profile information published for these streams are:

1. **Impervious area increases:** Flows calculated in this Study account for increases in impervious areas due to development which has occurred between 1978, when the previous drainage master plan was developed, and 1996.

2. **Additional downstream flow constrictions:** A privately-owned agricultural structure on Senecal Creek located about one mile north of Crosby Road restricts flows and causes water to back up into the city limits. Neither previous study calculated the impacts on upstream water levels resulting from this structure.

3. **Modifications to the Oregon Electric Railroad embankment:** Flood level calculations on Senecal Creek assumed there to be no breaks in the top of the berm at this abandoned crossing structure. Field observations indicate that a 60-foot wide portion of the earthen structure crossing Senecal Creek has been excavated to create a high water overflow channel at the eastern end of the structure.

4. **Senecal Drive Crossing at Senecal Creek Estates:** This culverted road embankment across Senecal Creek was not present during the time of the earlier studies.

5. **Field survey data:** In 1995 survey data was obtained at all road crossings and at a number of selected cross-sections for both Senecal Creek and Mill Creek for this Study. This information was obtained by DeHaas & Associates, Inc. using Global Positioning Systems methods and represents a higher standard of accuracy than that required by FEMA for the earlier studies. Modeled cross-sections were field surveyed to obtain more accurate data and elevations.

## HYDROLOGIC ANALYSIS METHODOLOGY

A hydrologic analysis was conducted to estimate peak flows and runoff volumes impacting drainage facilities within the study area. Computer models using the US Army Corps of Engineers HEC-1 methods were developed to compute flow hydrographs for existing and full buildout levels of development for each subbasin in the study area.

The HEC (Hydrographic Engineering Center) models are designed to simulate the surface runoff (HEC-1) and the hydraulic response (HEC-2) of a basin to that runoff. A design storm rainfall distribution is translated into a runoff hyetograph and this is used to develop the stream hydrograph for each subbasin and the main stem of the stream. Effective rainfall is computed by abstracting infiltration and surface storage using an infiltration function. Within each subbasin, rainfall and infiltration rates are assumed to be uniformly distributed across the subbasin. Flows from each subbasin are routed by the model to the point of confluence with other subbasins. When combined with the hydrograph from another subbasin, a composite hydrograph is computed by the model which accounts for any differences in the timing of the occurrence of the peak flow between the two hydrographs.

The processes used in the HEC-1 model are precipitation, interception/infiltration, transformation of effective precipitation into runoff, addition of base flows and flood hydrograph routing in the stream channels including reservoirs where they are present.

The HEC-1 and HEC-2 models use the following input parameters:

- <u>Subbasin Area</u> (acres)

  <u>Subbasin</u> area is the surficial watershed within which runoff can be assumed to flow to a single discharge point.

- <u>Design Precipitation Hyetograph</u>

The design hyetograph for the appropriate return interval is the bell-shaped relationship between precipitation intensity and time as the storm begins, then reaches peak intensity, then recedes. The SCS Type 1A 24-hour design storm distribution was used.

- Soil Conservation Service's (SCS) Runoff Curve Number

  The SCS runoff curve number is used to calculate the amount of rainfall that is absorbed by vegetation or the underlying soils. For impervious areas, a standard curve number of 98 was used. For grassed and wooded areas curve numbers were calculated based on the composite hydrologic group of the soils within each subbasin.

- Percent Impervious Area

  The percent impervious area is the percentage of the watershed surface over which no infiltration and only minor rainfall losses occur. Percent impervious for existing conditions was obtained from aerial photography taken in 1994. Percent impervious for future development conditions was computed based on typical percentages of impervious areas for the land use(s) planned for the subbasin.

- Percent Grass

  The percent grass parameter is used to represent the percentage of the basin for which runoff can be calculated using an SCS curve number typical of grass or pasture ground cover. For future full buildout conditions, pervious surfaces in fully developed areas were assumed to be primarily grass.

- Percent Woods

  The percent woods parameter is used to represent the percentage of the basin for which runoff can be calculated using an SCS curve number typical of wooded ground cover.

- Time of Concentration (Minutes)

  Time of concentration is the time lag between when the storm begins and when runoff from the most distant portions of the subbasin reach the discharge point. Time of concentration was estimated by calculating the time of travel from the most distant point in the subbasin to the subbasin discharge point, or node.

The result of the hydrologic modeling process is the computation of subbasin runoff hydrographs (runoff from each individual subbasin vs. time) and stream flow hydrographs (stream flow rates from all upstream subbasins vs. time). Peak flows at desired locations in the drainageways were estimated for both existing and full-buildout development conditions. Existing and future peak flows from individual subbasins were calculated for the 2-, 5-, 10-, 25- and 100-year events. Composite in-stream peak flows were calculated for the 5-, 10-, 25- and 100-year events.

Estimated peak flows are used in the hydraulic models described below to evaluate existing and future drainage facility capacities. The hydrologic model estimates "how much" flow is occurring and the hydraulic models estimate "how deep" the flow will get in the various types of conveyance structures.

## HYDRAULIC ANALYSIS

Hydraulic analysis involves calculating the water surface profile under existing and ultimate land use conditions based on the estimated peak flows generated from the hydrologic analysis. Hydraulic modeling was conducted to determine the maximum capacity of existing and future drainage facilities and to determine improvements required to meet existing and future drainage needs.

In-house hydraulic models were used to calculate hydraulic capacities for a majority of the piped, culverted and open channel drainage segments in the study area. These models implement Manning's equation for normal depth flow in pipes and open channels and the Federal Highway Administration (FHA) culvert equations for inlet control and submerged culvert conditions. For complex culvert configurations, the FHA utility program HY-8 was used. The Corps of

Engineer's HEC-2 hydraulic profile model was used to calculate hydraulic profiles along Senecal and Mill Creeks.

The HEC-2 and HY-8 models are described below:

HEC-2

The HEC-2 model computes water surface profile using several input parameters: (1) peak flows from the hydrologic analysis, (2) cross-sectional areas at regular intervals along the channel, (3) Manning's "n" friction coefficient, and (3) starting water surface elevations.

The HEC-2 model only models subcritical flow. Generally, subcritical flow is deep, slow flow, while super-critical flow is shallow, rapid flow. Starting at the water surface elevation at the outfall of the waterway, HEC-2 calculates water surface elevation at the next selected upstream point. The program then calculates the energy loss due to slope and friction between the points. Energy loss, or head loss, is expressed as loss in water surface elevation (relative to the streambed). This procedure is repeated for the length of waterway, resulting in a water surface profile. Water surface profiles were developed for flows anticipated under existing and future development conditions.

HY8

The Federal Highway Culvert Program HY8 model is a utility program which was used to supplement HEC-2's limited multiple culvert modeling capabilities. The HY8 program effectively models changes in the water surface profile due to multiple culverts and combination culvert/weir flow. The methodology of HY8 is identical to HEC-2, except that it requires the physical dimensions of the culvert/weir.

## ANALYSIS OF THE MODEL RESULTS

The primary objective of the analysis was to evaluate the adequacy of existing drainage facilities to accommodate both existing and future flows and to develop a phased capital improvements plan to upgrade inadequate facilities. The approach involved problem identification, identification of improvement alternatives, and selection of the appropriate system improvements. This approach was used first to analyze the drainage system's response to existing peak flows, and then its response to future peak flows.

Major drainage basins were defined for the major tributary creeks. Drainage subbasins were delineated by identifying areas which could be characterized as draining to one discharge point and which were relatively uniform as to slope and land use. Subbasins were delineated into relatively small areas (15-60 acres) within the City of Woodburn study area. Larger basin areas were used to calculate flows for portions of the watershed outside the Woodburn UGB and within Marion County. For each subbasin, hydrologic input parameters were estimated. (See Chapter 4, Model Definition and Basin Delineation.)

The design rainfall event for input into the hydrologic models was determined using the NOAA Atlas and the SCS Type 1A storm distribution. See Chapter 5, Analysis of Rainfall Runoff, for a more complete description of these runoff parameters.

The Santa Barbara Urban Hydrograph Method was used to compute individual flow hydrographs for each subbasin area. Peak undetained subbasin flows were determined for existing and future conditions.

In order to calculate peak in-stream flows for existing and future, unimproved conditions, hydrologic computer routing models were constructed using the branching configurations of the subbasins. The subbasin hydrographs were routed and combined using the "Hydrograph lag" and "Muskingum" routing methods described previously to determine "free-flowing" peak in-stream flows for the main open-channel drainageways. Locations where significant reduction in flow occurs due to existing culvert constrictions and storage of excess flows were identified and modeled separately using detailed storage routing computer spreadsheet models. The outflow hydrographs obtained from the storage routing models were then input back into the "free-flowing" models to obtain peak in-stream flows which account for storage. At key storage locations, existing culvert and storage-elevation curves were assumed for both existing and future, unimproved flow calculations.

Peak undetained subbasin flows and peak in-stream drainageway flows are calculated in Chapter 5, Analysis of Rainfall Runoff.

In-house hydraulic spreadsheet models were used to calculate the flow capacity of each significant drainage facility in the City. Drainage facilities evaluated include the major creeks and tributaries, piped segments 12 inches or larger in diameter and open channel drainage ditches which convey more than just local runoff from adjacent properties. For Senecal Creek and Mill Creek facilities, in-house models were used to estimate the individual capacity of the culvert and bridge crossings, while HEC-2 models previously developed by the Corps of Engineers were used to evaluate the capacity of the creek as a whole.

The capacity of each facility evaluated was compared to the calculated peak in-stream flows to determine the largest frequency event that could be conveyed by the existing facilities for existing and future flows. The frequency of excedence was then compared to the approximate risk to adjacent development. For example, if a parking lot were inundated every 5 years it would probably be considered tolerable. If a residential dwelling were at risk even at a 25-year frequency, it would probably be considered intolerable. If the frequency of system excedence was considered to be intolerable based on discussions with City staff, the segment would be identified for correction. The severity of each identified problem area was evaluated based on the extent of flooding hazards, such as inconvenience or property damage. The timing of improvements to correct an identified problem was dependent on the severity of risk associated with system excedence and the rate of upstream growth that could further aggravate the situation.

After the problem areas were identified, improvement alternatives for alleviating flooding were developed. The following alternatives were considered in this approximate order:

(1) Surcharge - Consider allowing pipes to be surcharged if the hydraulic grade line does not rise above the ground surface.

(2) Design Storm - Consider allowing a lesser design storm standard such as a 2 year or 5 year standard where the risk of damage is minimal or low.

(3) Bypass - Re-route the flow around or away from the problem area without further aggravating other areas.

(4) Detention - Consider the construction of detention facilities upstream of the problem area to hold back peak flows upstream in order to moderate downstream flows.

(5) Replacement - Replace the conduit with a larger diameter pipe and/or increase the slope of the pipe.

The effects of these alternatives were evaluated using the HEC-2 model for Senecal Creek and Mill Creek and in-house hydraulic spreadsheet models for other drainage systems. The alternative that minimized costs without compromising safety was the alternative recommended. (See Chapter 7, Recommendations for Drainage Improvements.)

When analyzing future system improvements, it was assumed that improvements required for the existing development conditions were in place. Costs were estimated for the recommended improvements. The costs, scope of improvement, and suggested phasing are summarized in Chapter 9, Recommended Capital Improvement Projects.

# Chapter 3

# STUDY METHODOLOGY

## INTRODUCTION

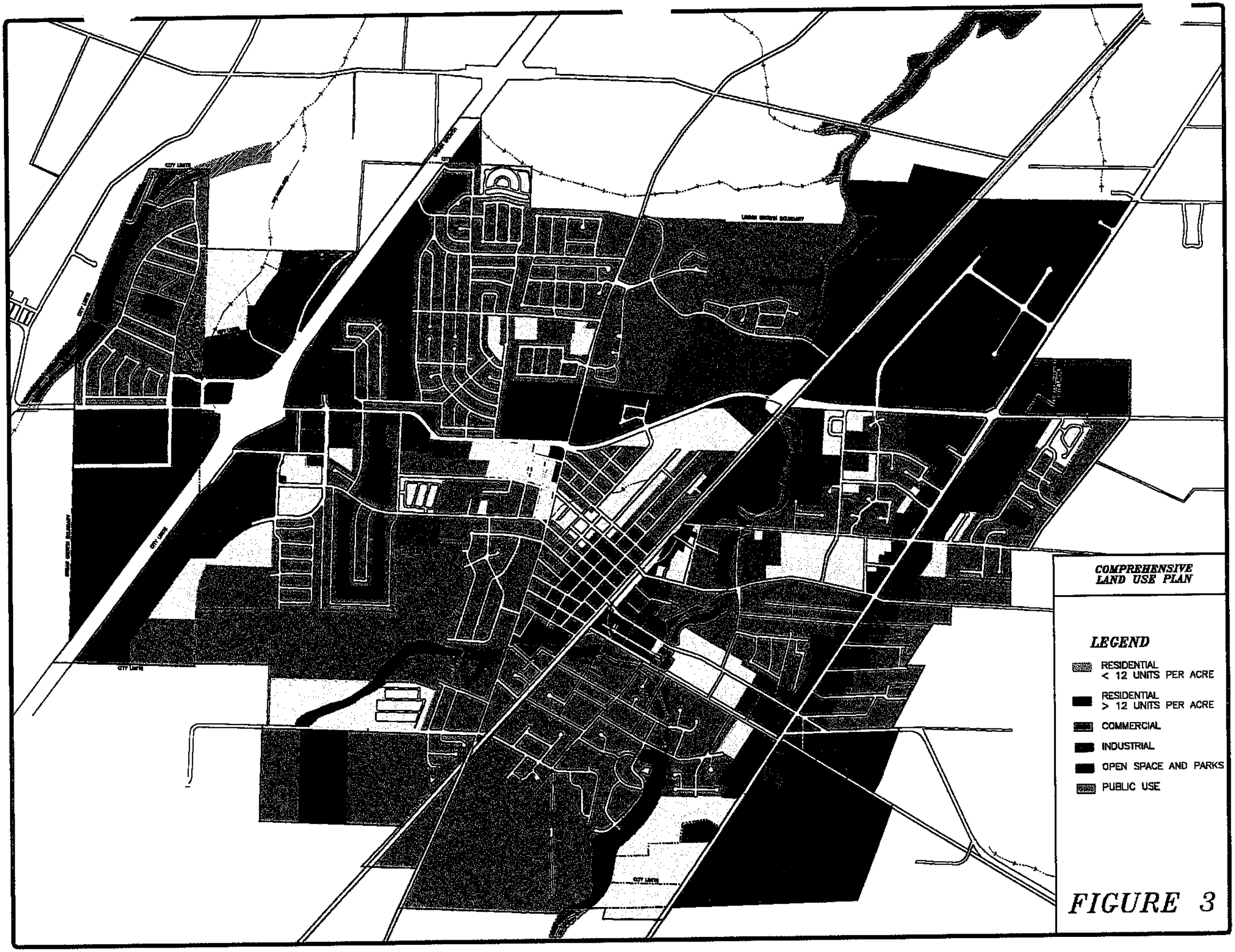
This section of the report outlines the approach and technical methods used to create the Storm Drainage Master Plan, analyze drainage system features and forecast the future needs of the system. Hydraulic and hydrologic calculations for this Study are based on 1996 land use and system layout. This Study includes both major drainageways that serve the City of Woodburn, Senecal Creek and its tributaries and Mill Creek and its tributaries. A part of Woodburn's jurisdiction is located east of Highway 99E and properties in this area generally drain to the Pudding River. These properties are relatively small and located at the fringe of the Pudding River catchment area. Consequently, these properties are not included in this Study as any analysis of this area would not provide a suitable basis for decisions about drainage system development within the City's Urban Growth Boundary in the area in question.

In order that the Study accurately reflect the existing system and hydraulic conditions in the Senecal Creek and Mill Creek drainage systems and to set the basis for conditions not easily measured, several assumptions were made and are outlined here. This section also presents the basis of the computer model used to calculate runoff quantities (hydrologic elements) and flow rates and conditions in the existing drainageway (hydraulic elements).

## ASSUMPTIONS USED IN THE ANALYSIS

The following assumptions were made in performing the hydrologic and hydraulic analysis:

1. Land uses for calculation of future development types and densities were based on the Woodburn Comprehensive Plan Land Use Map, dated 1996. See Figure 3, Woodburn



COMPREHENSIVE LAND USE PLAN
LEGEND
RESIDENTIAL < 12 UNITS PER ACRE
RESIDENTIAL > 12 UNITS PER ACRE
COMMERCIAL
INDUSTRIAL
OPEN SPACE AND PARKS
PUBLIC USE


FIGURE 3

infiltrates through the soil, the estimation of subbasin flows is more realistic if the subbasins are drawn to include areas of relatively uniform soils.

- Common Outfall - Generally, the subbasin should be drawn so that all flow from the subbasin discharges at one point, i.e., one storm drain outfall or a creek. For those subbasins which lie along major waterways, discharges into the waterway are often numerous and sometimes indistinct. However, these subbasins which discharge along drainageways can generally be considered as if they discharged at a single outfall into the waterway.

- Jurisdictional Boundaries - Where possible, subbasin boundaries were delineated to coincide with jurisdictional boundaries. This convention allows the Drainage Master Plan to more easily identify runoff impacts due to lands within other jurisdictions.

## SUBBASIN CODING

Subbasin coding is required as a means of referencing the branching relationships of the drainageways modeled in the hydrologic computer models. To facilitate subbasin coding and model identification, each of the major drainage basins in the Study area watershed was assigned a letter prefix. In this report, subbasins in the Senecal Creek drainage basin begin with an "S", an "ES" for the East Tributary of Senecal Creek and those located in the Mill Creek basin begin with an "M".

Portions of both stream basins are located outside the City's area of influence (the UGB). Runoff from these areas is simulated through use of a flow hydrograph located at the upper (south) segment of either Mill Creek or Senecal Creek where the two streams enter the Woodburn service area. On Mill Creek, an additional basin (M - 1) is located immediately north of the UGB and, while it has a minor impact on flood levels in Mill Creek, it does not have a direct effect on any of the tributary stream water levels or any piped system in the City. Subbasin M - 1 joins the main stem of Mill Creek just south of Crosby Road and because of this, subbasin M - 1 is included in the model calculations.

Within major basins of Mill Creek and Senecal Creek, smaller subbasins are identified and given discrete alphanumeric names. Each subbasin delineated has one node (defined as the subbasin discharge point) where the runoff collected within the subbasin is discharged into the next downstream subbasin. The downstream node number and the subbasin designation are the same. These smaller subbasins within the Mill Creek and Senecal Creek drainage basin were numbered, beginning with the furthest downstream node point. Moving upstream along the basin's main waterway, each node is numbered sequentially. For example, on Mill Creek, subbasins along the main creek are numbered M-1, M-2, M-3, M-4, etc.

In some cases, subbasins are further subdivided into smaller areas and these are numbered using a lower case letter following the major basin number. For example, the Mill Creek subbasin M - 7 has additional subbasins further away from the stream and these are designated M - 7a, M - 7b, etc. This convention was used where a subbasin was located on either side of the modeled stream or where the smaller areas within a subbasin have distinctly different characteristics and were modeled individually, Usually, the major subbasin designation (M-1, M-2, etc) were used to show the accumulated runoff in a defined tributary of either Mill Creek or Senecal Creek.

## HYDROLOGIC PARAMETERS

Chapter 3, Study Methodology, gives a brief overview of the hydrologic modeling process and the input parameters required to calculate peak drainageway flows. These required hydrologic input parameters were developed for each subbasin delineated in the Study area watershed and are discussed below. Table 4-1 and 4-2, Subbasin Runoff Parameters, for Mill Creek and the East Tributary of Senecal Creek respectively, summarizes the runoff parameters selected for use

# Table 4 - 1
## Subbasin Runoff Parameters
### Mill Creek

| | | | Soils / Infiltration | | | 1996 Level of Buildout | | | Full Buildout Conditions | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Sub-basin | Area (Acres) | Area (Sq. Miles) | % Class C Soils | % Class D Soils | Existing Use Curve No. | Mapped Impervious Area | Modelled / Effective Imperv. Area | Time of Conc. (Min) | Mapped Impervious Area | Modelled / Effective Imperv. Area | Time of Conc. (Min) |
| M-1 | 359 | 0.561 | 80% | 20% | 80.9 | 5% | 0% | 195 | 0% | 0% | 195 |
| M-1A | 390 | 0.610 | 90% | 10% | 80.0 | 5% | 0% | 70 | 0% | 0% | 70 |
| M-1B | 71 | 0.112 | 90% | 10% | 80.5 | 5% | 0% | 64 | 45% | 39% | 28 |
| M-1C | 53 | 0.083 | 65% | 35% | 76.1 | 39% | 24% | 26 | 39% | 24% | 26 |
| M-2 | 109 | 0.170 | 90% | 10% | 78.2 | 8% | 2% | 74 | 72% | 68% | 26 |
| M-3 | 76 | 0.119 | 92% | 8% | 74.5 | 5% | 0% | 73 | 0% | 0% | 73 |
| M-3A | 96 | 0.150 | 100% | 0% | 74.0 | 5% | 0% | 73 | 40% | 34% | 29 |
| M-4 | 40 | 0.062 | 90% | 10% | 74.4 | 25% | 19% | 26 | 37% | 30% | 26 |
| M-4A1 | 42 | 0.066 | 60% | 40% | 77.6 | 45% | 39% | 26 | 79% | 75% | 21 |
| M-4A2 | 16 | 0.025 | 40% | 60% | 77.6 | 56% | 50% | 15 | 56% | 50% | 15 |
| M-4A3 | 18 | 0.028 | 100% | 0% | 78.0 | 40% | 33% | 20 | 74% | 69% | 15 |
| M-5 | 59 | 0.092 | 90% | 10% | 74.4 | 30% | 24% | 25 | 55% | 49% | 20 |
| M-5A1 | 69 | 0.107 | 90% | 10% | 78.2 | 10% | 3% | 45 | 34% | 28% | 37 |
| M-5A2 | 65 | 0.102 | 75% | 25% | 77.0 | 15% | 6% | 60 | 46% | 39% | 27 |
| M-5A3 | 97 | 0.151 | 60% | 40% | 76.4 | 43% | 28% | 30 | 43% | 28% | 30 |
| M-5A4 | 30 | 0.047 | 50% | 50% | 74.5 | 23% | 11% | 30 | 23% | 11% | 30 |
| M-5B1 | 53 | 0.083 | 80% | 25% | 76.3 | 51% | 36% | 29 | 51% | 36% | 29 |
| M-5B2 | 37 | 0.058 | 60% | 40% | 76.4 | 35% | 29% | 35 | 56% | 50% | 20 |
| M-5B3 | 55 | 0.086 | 60% | 40% | 76.4 | 37% | 23% | 35 | 41% | 26% | 35 |
| M-6 | 42 | 0.066 | 90% | 10% | 71.7 | 0% | 0% | 70 | 0% | 0% | 70 |
| M-6A | 65 | 0.102 | 95% | 5% | 74.3 | 40% | 25% | 25 | 46% | 32% | 20 |
| M-6A2 | 69 | 0.108 | 95% | 5% | 74.3 | 33% | 19% | 40 | 66% | 61% | 30 |
| M-6B | 23 | 0.036 | 90% | 10% | 74.6 | 25% | 13% | 25 | 73% | 69% | 15 |
| M-7 | 99 | 0.154 | 95% | 5% | 74.3 | 40% | 25% | 35 | 55% | 41% | 35 |
| M-7A | 24 | 0.038 | 100% | 0% | 74.0 | 40% | 25% | 18 | 68% | 55% | 18 |
| M-7B1 | 49 | 0.076 | 90% | 10% | 74.6 | 25% | 13% | 24 | 39% | 25% | 24 |
| M-8 | 36 | 0.056 | 90% | 10% | 74.4 | 39% | 25% | 25 | 49% | 43% | 21 |
| M-9 | 31 | 0.049 | 85% | 15% | 74.7 | 15% | 6% | 40 | 33% | 27% | 28 |
| M-9A1 | 92 | 0.144 | 95% | 5% | 74.3 | 45% | 30% | 35 | 52% | 37% | 35 |
| M-9A3 | 57 | 0.089 | 100% | 0% | 76.0 | 50% | 44% | 30 | 68% | 63% | 25 |
| M-9B | 33 | 0.052 | 100% | 0% | 76.0 | 56% | 50% | 13 | 70% | 65% | 13 |
| M-10 | 100 | 0.157 | 90% | 10% | 74.5 | 38% | 24% | 27 | 43% | 28% | 27 |
| M-11 | 78 | 0.121 | 90% | 10% | 74.6 | 19% | 8% | 34 | 51% | 45% | 26 |
| M-11A | 43 | 0.067 | 90% | 10% | 74.6 | 30% | 16% | 25 | 41% | 35% | 20 |
| M-11B1 | 69 | 0.108 | 95% | 5% | 74.3 | 27% | 14% | 30 | 43% | 28% | 30 |
| M-11B2 | 28 | 0.044 | 90% | 10% | 78.0 | 0% | 0% | 63 | 39% | 32% | 38 |
| M-11C | 34 | 0.053 | 95% | 5% | 78.2 | 0% | 0% | 63 | 46% | 40% | 26 |
| M-11C2 | 39 | 0.060 | 60% | 40% | 79.6 | 5% | 0% | 75 | 45% | 39% | 23 |
| M-11D1 | 88 | 0.138 | 90% | 10% | 78.4 | 0% | 0% | 62 | 32% | 25% | 25 |
| M-11D2 | 48 | 0.075 | 60% | 40% | 76.4 | 49% | 34% | 25 | 49% | 34% | 25 |
| M-11E1 | 57 | 0.088 | 90% | 10% | 78.4 | 0% | 0% | 69 | 18% | 13% | 25 |
| M-11E2 | 77 | 0.120 | 90% | 10% | 78.4 | 0% | 0% | 69 | 45% | 39% | 25 |
| M-11E3 | 61 | 0.095 | 100% | 0% | 78.0 | 0% | 0% | 90 | 0% | 0% | 90 |
| M-11F | 161 | 0.252 | 50% | 50% | 80.0 | 0% | 0% | 180 | 0% | 0% | 180 |
| M-12 | 62 | 0.097 | 60% | 40% | 75.6 | 25% | 19% | 27 | 36% | 29% | 27 |
| M-12A1 | 23 | 0.036 | 95% | 5% | 72.0 | 5% | 0% | 75 | 45% | 39% | 26 |
| M-12A2 | 70 | 0.109 | 95% | 5% | 78.0 | 2% | 0% | 110 | 45% | 39% | 36 |
| M-12A3 | 12 | 0.018 | 90% | 10% | 78.0 | 0% | 0% | 45 | 45% | 39% | 20 |
| M-12B | 68 | 0.106 | 95% | 5% | 78.0 | 20% | 9% | 40 | 65% | 60% | 23 |
| M-12C | 134 | 0.209 | 95% | 5% | 78.0 | 2% | 0% | 120 | 45% | 39% | 36 |
| M-13 | 955 | 1.493 | 90% | 10% | 78.2 | 1% | 0% | 200 | 1% | 0% | 200 |

# TABLE 4 - 2

## East Tributary of Senecal Creek

## Subbasin Runoff Parameters

| Sub-basin | Area (Acres) | Area (Sq. Miles) | % Impervious MIA | % Impervious EIA | SCS Curve #, Pervious Areas | Time of Conc. (Minutes) | Lag Time (Hours) |
|---|---|---|---|---|---|---|---|
| **1993 Conditions** | | | | | | | |
| A | 124.5 | 0.195 | 8.0% | 1.0% | 78 | 102 | 1.02 |
| B-1 | 60.3 | 0.094 | 60.0% | 46.5% | 74 | 60 | 0.60 |
| B-2 | 49.4 | 0.077 | 2.0% | 0.5% | 74 | 31 | 0.31 |
| C | 49.0 | 0.077 | 38.0% | 23.4% | 74 | 19 | 0.19 |
| D | 78.3 | 0.122 | 3.0% | 0.5% | 80 | 35 | 0.35 |
| E | 50.2 | 0.078 | 22.0% | 10.3% | 80 | 28 | 0.28 |
| F | 30.0 | 0.047 | 35.0% | 20.7% | 80 | 20 | 0.20 |
| G | 76.6 | 0.120 | 5.0% | 0.5% | 80 | 127 | 1.27 |
| H | 26.7 | 0.042 | 60.0% | 54.4% | 77 | 20 | 0.20 |
| I | 33.6 | 0.053 | 57.0% | 51.2% | 77 | 20 | 0.20 |
| J | 40.6 | 0.063 | 2.0% | 0.5% | 80 | 130 | 1.30 |
| K | 83.1 | 0.130 | 5.0% | 0.5% | 80 | 160 | 1.60 |
| L | 121.0 | 0.189 | 3.0% | 0.5% | 81 | 190 | 1.90 |
| **Full Buildout Development Conditions** | | | | | | | |
| A | 124.5 | 0.195 | 8.0% | 1.0% | 78 | 102 | 1.02 |
| B-1 | 60.3 | 0.094 | 60.0% | 46.5% | 74 | 60 | 0.60 |
| B-2 | 49.4 | 0.077 | 2.0% | 0.5% | 74 | 31 | 0.31 |
| C | 49.0 | 0.077 | 45.0% | 30.2% | 74 | 19 | 0.19 |
| D | 78.3 | 0.122 | 70.0% | 65.5% | 77 | 22 | 0.22 |
| E | 50.2 | 0.078 | 67.0% | 62.1% | 77 | 16 | 0.16 |
| F | 30.0 | 0.047 | 66.0% | 61.0% | 77 | 18 | 0.16 |
| G | 76.6 | 0.120 | 72.5% | 68.3% | 77 | 37 | 0.37 |
| H | 26.7 | 0.042 | 68.0% | 63.3% | 77 | 16 | 0.16 |
| I | 33.6 | 0.053 | 72.0% | 67.5% | 77 | 14 | 0.14 |
| J | 40.6 | 0.063 | 75.0% | 71.1% | 77 | 18 | 0.18 |
| K | 83.1 | 0.130 | 45.0% | 38.5% | 77 | 24 | 0.24 |
| L | 121.0 | 0.189 | 3.0% | 0.5% | 81 | 190 | 1.90 |
| **New Development Areas to Be Routed (Full Buildout) *** | | | | | | | |
| M-C | 24.7 | 0.039 | 75.0% | 71.1% | 77 | 15 | 0.15 |
| S-I | 17.0 | 0.027 | 85.0% | 82.7% | 77 | 13 | 0.13 |
| M-HDR | 30.3 | 0.047 | 65.0% | 59.5% | 77 | 15 | 0.15 |
| M-LDR | 42.9 | 0.067 | 45.0% | 38.5% | 74 | 24 | 0.24 |

* NOTE: M denotes basins currently draining to Mill Creek.
S denotes basins currently draining to Main Senecal Creek.

in the hydraulic model. Parameters include size, percentage of various soils classes, mapped impervious and modeled impervious area for both existing and future conditions, and the time of concentration for both existing and future conditions. Descriptions and explanations of the key parameters follow.

## EFFECTIVE IMPERVIOUS AREA

The amount of runoff is increased substantially by increased impervious areas within the subbasins. Impervious areas, such as streets, parking lots, rooftops, sidewalks, and loading areas, increase the volume by preventing infiltration. Further, these impervious areas tend to concentrate the runoff into storm drains or ditches which more rapidly convey the runoff to the receiving stream. This decreased time of conveyance decreases the time of concentration and generally increases peak rates of runoff downstream. Transformation of agricultural lands to highly urbanized lands can increase the rates and volumes of storm runoff by a factor of 2 to 4 times. Impervious area is a very significant factor in the analysis of storm drainage systems.

Percent impervious areas for this Study were determined using aerial photography taken in 1994 supplemented with zoning maps. Using the aerial photographs, developed areas were delineated and categorized by the type of land use. Using zoning maps in conjunction with the aerial photographs, the approximate density and land use of each developed area were determined. For each developed area having a similar land use and density, a percent impervious area representative of existing conditions was computed. Within each subbasin, the ratio of impervious area was multiplied by the percent developed for each land use to determine a total impervious area for the subbasin.

The "mapped" impervious areas determined above were adjusted to an "effective" impervious area for input into the hydrologic models. For fully storm sewered basins, where a majority of the impervious surfaces drain directly to a storm drain system, the effective impervious area is equivalent to the mapped impervious area. In older areas of town, however, many of the streets do not have curb and gutter systems, roof drains may drain to individual drywells and many impervious surfaces may drain across pervious areas before runoff is collected by the storm system. In these cases, the mapped impervious area was adjusted to an effective impervious area in order to account for the additional rainfall losses which occur.

The following equation, which is based on regression analysis results published by OTAK, Inc. (1988), was used to estimate the effective impervious area (EIA) for use in the hydrologic models, given the mapped impervious (MIA) area determined from the aerial photographs:

$$EIA = 0.1 \times (MIA)^{1.5}$$

Future percent impervious areas were estimated using the Comprehensive Plan land use map and the Zoning Map for Woodburn. Lands which are currently undeveloped are assumed to be developed according to their Comprehensive Land Use designations. Developed, nonconforming parcels (different from what is specified in the Comprehensive Plan) are assumed to be re-developed and eventually reach the land use and density specified in the Comprehensive Land Use Plan.

Future developed areas are assumed to be fully served by a public storm sewer system, and have an effective impervious area equivalent to the mapped impervious area estimated from the land use maps. The final effective impervious areas for the Mill Creek basin used in the hydrologic models are summarized in Table 4-1, Subbasin Runoff Parameters, for existing and buildout conditions.

## SOIL LOSS PARAMETER

The reduction in runoff from pervious areas which occurs due to infiltration and evapotranspiration is estimated in the hydrologic models using a soil loss parameter developed by the Soils Conservation Service (SCS). This parameter, the Runoff Curve Number, depends on the soil type, ground cover and antecedent (pre-design storm) moisture of the watershed.

Chapter 2, Watershed Definition, discusses the infiltration characteristics of the surface soils found throughout the Study area watershed. In that chapter, soils were designated as belonging to Hydrologic Group A, B, C, or D as defined by the Soils Conservation Service (SCS). Hydrologic Group "A" soils typically include highly pervious soils with low potential for runoff, such as sands. Hydrologic Group "D" soils typically include fine grained impervious soils with high runoff potential such as clays. Table 4 - 1, Subbasin Runoff Parameters (Mill Creek), shows the distribution of surface soils in each Hydrologic Group throughout the Mill Creek Drainage area and Table 4 - 2 shows similar data for the East Tributary of Senecal Creek.

Given the hydrologic classification of the soil, a Runoff Curve Number can be determined using tables developed by the SCS. The SCS tables (published in SCS TR-55, "Urban Hydrology for Small Watersheds) list Runoff Curve Numbers for each Hydrologic Group for a variety of ground cover conditions. Values listed in the table are for average antecedent moisture conditions. Average antecedent moisture conditions, as defined by the SCS, apply to the Study area watershed and no adjustment to these values is necessary.

The hydrologic models require SCS Runoff Curve Numbers for two ground cover conditions. For this Study, pervious areas were classified as woods or grass/pasture. For existing conditions, the percentage of a subbasin classified as woods was determined by identifying undeveloped wooded areas using aerial photography. All other open areas were assumed to be grass/pasture. For future conditions, wooded areas were assumed to occur only in environmentally protected areas where development is not anticipated.

The SCS tables were used to determine Runoff Curve Numbers for grass/pasture and wooded ground cover conditions. The Runoff Curve Numbers selected for wooded areas are applicable to natural wooded areas with thick underbrush. Values selected for grass/pasture areas assume a

# Table 4 - 3

## Time of Concentration Assumptions

1. Channel, Gutter and Swale Velocity Assumptions

| Slope | Velocities in Feet per Second Channel/Pipe | Gutter | Swale |
|---|---|---|---|
| .002 - .005 | 2.0 | 1.0 | 0.5 |
| .005 - .010 | 3.0 | 1.5 | 1.0 |
| .010 - .015 | 4.0 | 2.0 | 1.5 |
| .015 - .020 | 5.0 | 2.5 | 2.0 |
| .020 - .030 | 6.0 | 3.0 | 2.5 |
| .030 - .060 | 7.0 | 4.0 | 3.0 |
| .060 + | 8.0 | 4.5 | 3.5 |

2. Manning's "n" for Overland Flow (OF)

| Surface | Manning's "n" |
|---|---|
| Paved or gravel with slope >2% | 0.02 |
| Lawns, natural grass areas, uncultivated fields | 0.24 |
| Cultivated fields | 0.18 |
| Trees with light underbrush | 0.40 |
| Trees with heavy underbrush | 0.70 |

3. Assumptions for Estimating Time of Concentration for Future Development

A. Single Family Residential

| | |
|---|---|
| Length of Overland Flow (OF) | 75 feet |
| Manning's "n" for OF | 0.13 (mix of lawn and pavement) |
| Slope for OF | Existing Grade |
| Length of Gutter | 250 feet |
| Channel/Gutter Slope | Existing Grade |

B. Multi-family Residential

| | |
|---|---|
| Length of Overland Flow (OF) | 100 feet |
| Manning's "n" for OF | 0.06 (primarily pavement) |
| Slope for OF | 0.005 - 0.02 |
| Length of Gutter | 250 feet |
| Channel/Gutter Slope | Existing Grade |

C. Commercial/Industrial

| | |
|---|---|
| Length of Overland Flow | 150 feet |
| Manning's "n" for OF | 0.02 (paved) |
| Slope of OF | 0.005 - 0.03 (depending on exst. grade) |
| Channel Slope | Existing Grade |
| Gutter Flow | None |

# Table 4 - 4

## Time of Concentration Equations

OVERLAND FLOW (Length < 300 feet)

$$Tc = \frac{.007\,(nL)^{0.8}}{(P_2)^{0.5}\,(S)^{0.4}}, \quad \text{where}$$

$Tc$ = time of concentration in hours
$n$ = roughness coefficient (dimentionless)
$L$ = length of flow (feet)
$S$ = slope (ft/ft)
$P_2$ = 2 year, 24 hour precipitation (inches)

GUTTER, CHANNEL AND GRAVITY PIPE FLOWS

$$Tc = \frac{L}{3600\,(V)}, \quad \text{where}$$

$Tc$ = time of concentration in hours
$L$ = length of flow (feet)
$V$ = velocity (feet/sec)

US Soil Conservation Service - Technical Release No. TR-55, "Urban Hydrology for Small Watersheds" (January, 1975) ground cover of only 50% to 75% to account for other landscape uses that may not have 100% ground cover.

The numeric Curve Number values for each ground cover were applied to the areas of mapped soil types within each subbasin and a composite weighted average (by area) for soil type within each subbasin was developed.

## TIME OF CONCENTRATION

The time of concentration is the travel time from the most hydraulically remote point in the subbasin to the subbasin outlet. These times of concentration are determined according to the specific type flow for that subbasin, i.e., overland flow, shallow concentrated flow, gutter flow, channel flow, or pipe flow.

The time of concentration was calculated for each subbasin for existing and future land use and building conditions. For existing conditions, overland flow, swale flow and channel flow conditions were estimated using aerial photography and existing topographic mapping. For future conditions, time of concentration parameters were estimated, based on typical values for each land use designated in the City's Comprehensive Land Use Plan (Figure 2).

Table 4 - 3, Time of Concentration Assumptions, lists the assumptions used to calculate the time of concentration for existing and future conditions. Table 4 - 4, Time of Concentration Equations, presents the equations used in the calculations.

# Chapter 4

# MODEL DEFINITION AND BASIN DELINEATION

## INTRODUCTION

The Study area watershed and major drainage basins within that area were delineated in Chapter 2, Watershed Definition. The major drainage basins delineated in Chapter 2, Watershed Definition, represent the overall basin areas draining to the two principle drainageways in the Study area, Senecal Creek and Mill Creek (See Figure 1, Senecal & Mill Creek Drainage Basin Boundaries). In this chapter, the major drainage basins are further divided into subbasins for the purpose of modeling drainage flows within the storm drainage system. This chapter describes the process used to delineate the subbasins and explains the conventions used for referencing the subbasins. The following sections describe and then quantify the hydrologic parameters for each subbasin used in the runoff calculation models. Chapter 3, Study Methodology, of the report describes the hydrologic runoff models used in this analysis and the input parameters required to adapt these models to the two major drainage basins and their subbasins.

## SUBBASIN DELINEATION METHODOLOGY

Using the criteria outlined below, for the Senecal Creek.basin delineation process resulted in 5 major basins, (S-1 to S-5). The East Tributary of Senecal Creek lies within major basin S-4. In the East Tributary of Senecal Creek basin 13 subbasins (A-L) were delineated within 4 management zones (ES-1 to ES-4). Areas of the Senecal Creek basin outside the East Tributary area were not further divided into subbasins. In the Mill Creek drainage basin 12 major basins (M-1 to M-12) and 51 subbasins were identified. Subbasins generally range in size from 15 to

100 acres with larger subbasins located at the extreme north and south ends of the Study area. The location of these subbasins are shown in Figures 4 and 5, Drainage Subbasins, for Senecal Creek and Mill Creek, respectively. The coding and drainage parameters of these subbasins are described below.

To refine the modeling analysis and facilitate identification of potential drainage problems and improvements, each primary basin delineated in Chapter 2, Watershed Definition, was further delineated into major basins and subbasins. The factors considered for each basin delineation include:

- Size between 20 and 120 acres - Subbasin areas within this range increase the modeling accuracy of peak flow analysis and are typically used in drainage master planning. Some subbasins smaller than 20 acres were necessary to account for highly varied land uses or topography, or to facilitate calculating flows at a specific key point of interest. Subbasins larger than 120 acres were used for large undeveloped areas with uniform basin characteristics and minimal existing drainage facilities.

- Similar existing and/or future land uses within subbasin - Since runoff rates and amounts are significantly impacted by impervious surface areas and since the amount of impervious surfaces is largely a function of land use intensity, delineating subbasins with relatively uniform land uses allows more meaningful runoff parameters to be estimated.

- Consistent topography - Since the time for runoff to reach the outfall of a subbasin from the furthest reaches of the subbasin is an important factor in the determination of peak flows and since flow time is related directly to slope, accuracy is improved if the subbasin is drawn to include areas of relatively uniform topography.

- Consistent soil type - Since runoff is that portion of precipitation that is not absorbed by the soil or otherwise retained and since the type of soil is directly related to how much water

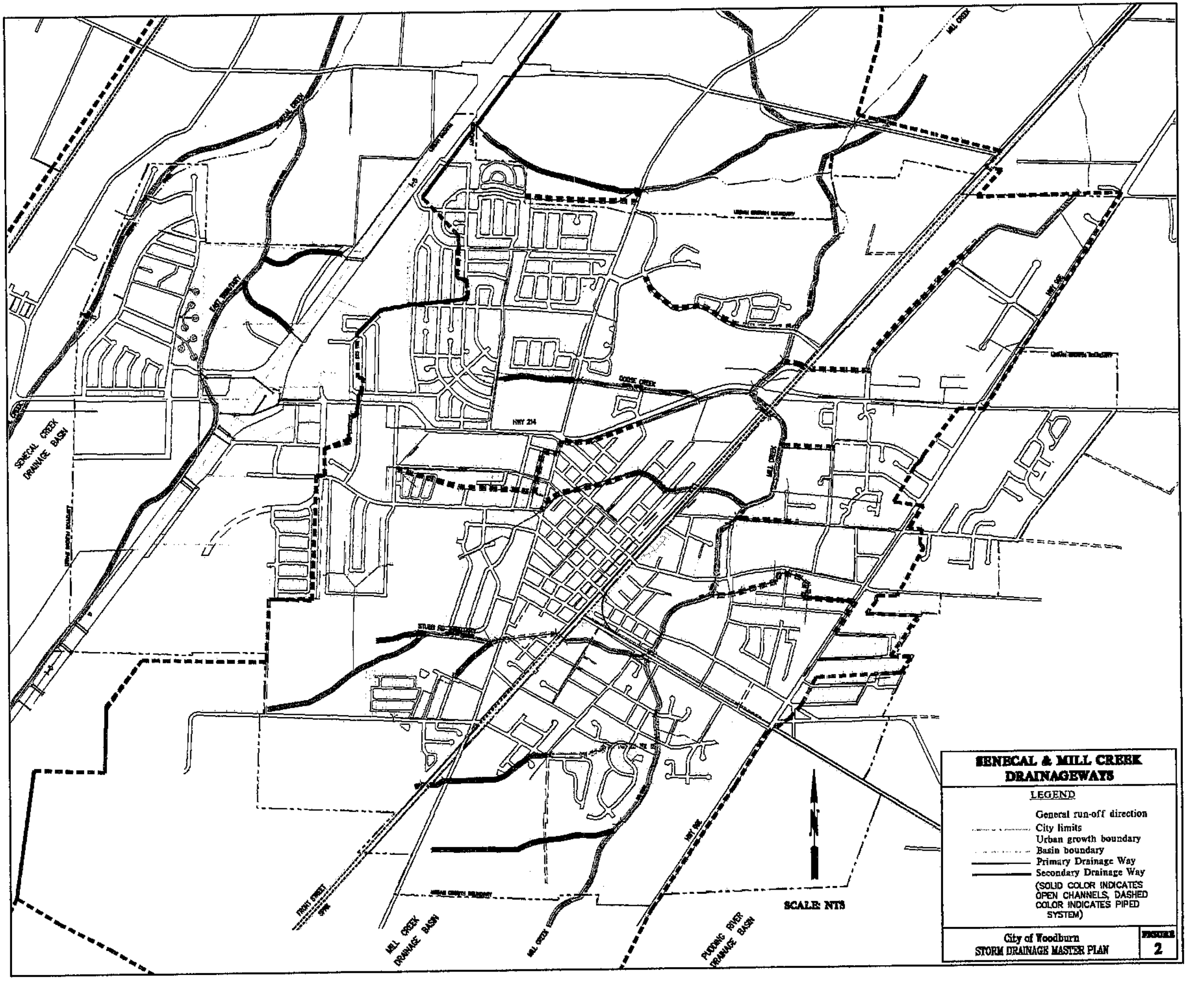
Each of the major drainageways, Senecal Creek and Mill Creek, has tributaries that originate within or just outside the study area. Figure 2, Senecal & Mill Creek Drainageways, displays the primary and secondary drainageways within the City boundaries. Some of these are named; others are small enough that no common name is associated with them. Senecal Creek and Mill Creek have their confluence north of Aurora approximately 4 miles north of Woodburn. Mill Creek continues to the Pudding River northeast of Aurora and the Pudding River joins the Molalla River near its confluence with the Willamette River. The gradient of both Senecal Creek and Mill Creek is steep enough that it is not necessary to model the drainageways to the north of a line approximately one mile north of Crosby Road.

The Corps of Engineers conducted a flood study for the Federal Emergency Management Agency (FEMA) in 1973. This Study provided a forecast of potential flood conditions along both Senecal Creek and Mill Creek outside the Woodburn city limits. Portions of this Study were used in preparation of this Master Plan.

## Mill Creek Watershed Area

Mill Creek originates in a broad area south of Woodburn, west and north of the City of Gervais. Most of the area upstream from Woodburn is in agricultural use and has been terraced for this use. The Mill Creek drainage area within the Study area is approximately 5017 acres. Mill Creek flows northerly to its confluence with the Pudding River near the town of Aurora. In addition to Senecal Creek, several small, unnamed tributaries contribute to the Mill Creek flows.

Near Cleveland Street in the south part of Woodburn, Mill Creek divides into two main branches. One drains the areas to the southwest, generally east of I-5; the other continues south toward Gervais where it collects drainage and runoff from farmlands between the two cities.



SENECAL & MILL CREEK
DRAINAGEWAYS
LEGEND
General run-off direction
City limits
Urban growth boundary
Basin boundary
Primary Drainage Way
Secondary Drainage Way
(SOLID COLOR INDICATES OPEN CHANNELS, DASHED COLOR INDICATES PIPED SYSTEM)
City of Woodburn
STORM DRAINAGE MASTER PLAN
FIGURE
2
SCALE: NTS
SENECAL CREEK DRAINAGE BASIN
MILL CREEK DRAINAGE BASIN
PUDDING RIVER DRAINAGE BASIN
HWY 214
GOOSE CREEK
MILL CREEK
STUBB RD
I-5
FRONT STREET
HWY 99E

## Senecal Creek Watershed Area

Senecal Creek is a tributary of Mill Creek and originates in the area west of the Mill Creek drainage. A relatively small watershed of approximately 4430 acres within the Study area Senecal Creek flows from south to north and abuts the Woodburn city limits near the Butteville Road intersection with Highway 214 west of I-5. It serves as a drainageway for area both within and outside the city limits. The main stem of Senecal Creek enters the City passing under a bridge under Highway 214 just east of the Butteville Road intersection and extends from there along the west side of the West Woodburn development area (the former Nazerene Estates). From there the stream flows north, in open water crossing an abandoned railroad embankment through a 48" culvert. Near the north end of West Woodburn, it passes under a new bridge structure entering Senecal Creek Estates. North of that point, it exits the City and continues in a northerly direction.

A small tributary of Senecal Creek termed the East Tributary in this Study provides a drainageway for the areas east of West Woodburn, most of the I-5 runoff and a portion of Woodburn proper east of I-5 and south of Highway 214. This tributary serves an area of 883 acres inside the City including properties both east and west of I-5.

### Major Topographic Features

The storm drainage study area is located on a reasonably flat, terraced area crossed by the two major drainageways described above. These two streams provide the greatest vertical relief in the study area itself, however south of the study area each becomes rather poorly defined and ends in a broad reach of agricultural land extending from highway 99E to I-5 and south from Parr Road to the City of Gervais. At the upper end of the two streams, the drainageway is poorly defined.

## Soils

Soils and their characteristics to either retain or absorb precipitation is a prime factor affecting runoff and subsequent water levels in the streams receiving runoff flows. Soils in the Woodburn area have been classified by the Soils Conservation Service (SCS) in a report titled, "Soil Survey of Marion County Area, Oregon." This report indicates that Woodburn and the surrounding areas are located in what is termed a "Young, silty terrace alluvium." This material was deposited on flood plains probably in the late Pleistocene epoch.

The SCS classifies the runoff characteristics of a soil as belonging to one of four hydrologic groups. These groups, classified as A, B, C and D are used to set the characteristics of the hydrologic model. Group A soils are typically sands and gravels, having a well-drained profile, high infiltration and result in low runoff rates. Conversely, Group D soils are very poorly drained, and have low infiltration rates, resulting in high runoff rates. In areas immediately adjacent to the streams in the area, other soil types such as the Concord (Group D) and Dayton (Group D) are found. These make up such a small portion of the soils in the area that their effect can be ignored for purposes of creating an accurate model.

The predominant soil series found in the drainage areas of Mill Creek and Senecal Creek are the Amity Series and the Woodburn Series. The Amity and Woodburn soils are described as silt loam and silty clay loam and are classified by the SCS as Group 'C' soils. These two series are moderately well drained and moderately fine textured. Like other soils found in the area, the Amity and Woodburn soils are usually moist but may be dry for more than 60 consecutive days following the summer solstice. These two soil types make up over 20 percent of soil classifications found in the northern Willamette Valley.

## Climate

The climate in the study area is a modified marine climate, typical of the northern Willamette Valley. The Pacific Ocean, the Coast Range and the Cascade Mountains have a substantial influence on the climate in the Woodburn vicinity as air masses move inland from the Pacific Ocean. Usually, the air is moist and near the ocean temperature. The coastal range serves as a buffer, moderating the most severe weather during the winter months.

On the floor of the Willamette Valley, those elevations between 100 and 500 feet, rainfall averages about 41 inches per year and one year in ten will exceed 51 inches in precipitation. The wettest months are usually November, December and January with almost 20 inches of rainfall occurring during that time. Lingering snowfall on the floor of the valley in Woodburn is unusual and contributes little to runoff and stream flows.

## EXISTING AND FUTURE LAND USE

The Woodburn Urban Growth Boundary (UGB) encompasses the land within and immediately surrounding the existing city limits which has been designated within the jurisdiction of the City for the purposes of providing future planning. Within the UGB, the City has created and adopted a Comprehensive Land Use Plan which delineates the general future land uses and the average density of development within each land use area. The Comprehensive Land Use Plan is indicated in Figure 3, Woodburn Comprehensive Plan. This Plan has been used as the basis for projecting future conditions in the areas that are currently unserved.

In Woodburn the remaining undeveloped acres will be developed as economic and social conditions dictate and regulation allows. Woodburn's location on Interstate 5, located midway between the urban areas of Salem and Portland make it a desirable area for both residential and commercial development. Woodburn has shown reasonably consistent growth over the past 10 years and is expected to continue that growth into the future.

Growth in Woodburn is expected to closely match historic growth in terms of the type and density.

**Future Increases in Impervious Areas**

The quantity of stormwater that reaches public waterways is a direct function of the location, amount and type of impervious areas in the drainage basin. Impervious areas are those surfaces such as rooftops, parking areas, streets, sidewalks which minimize and even prevent percolation of stormwater into the soil. Impervious areas may be in the City or outside the UGB and may be publicly or privately owned. It is important therefore, to estimate the location, type and runoff parameters of impervious areas in the drainage basins and base future hydraulic capacities on these projections.

Based on the 1996 Comprehensive Plan the estimated increase in development to achieve full build out is 617 acres of new development. In preparing the estimate of future impervious areas, the following assumptions were made:

A. Open Space and Natural Areas - Areas within the study area which are currently designated as Open Space or Natural Areas are assumed to remain as natural areas and are excluded from the impervious area calculations. Examples include public parks and designated wetlands.

B. Public Areas - Areas which are in public ownership and used as public facilities or set aside for future use as a public facility. Impervious area calculations will consider these parcels based on their current or intended use and assume runoff based on that use. For example, a future public park will have a negligible amount of impervious area while a school site will have an amount similar to a light industrial site.

C. Private Developable Areas – Privately held lands planned for residential (single or multifamily), light and heavy commercial and industrial uses were assumed to develop to the fullest extent allowed by the Comprehensive Plan. Areas not fully built as of 1996 were assumed to have the same percentage of impervious area as fully built areas with similar zoning.

D. Land Available for Development - Private lands planned for eventual development were assumed to have a full compliment of public streets, sidewalks and other amenities as required by development regulations. These impervious areas are assumed to be developed to the fullest extent allowed by the Plan.

E. Development Efficiency - Each category of land use was assumed to develop to 100 percent of the density allowed by the Comprehensive Plan. This is a conservative assumption as there will usually be parcels that, for a variety of reasons, are not developed at all or develop to densities less than that allowed by the Plan.

# Chapter 5

# ANALYSIS OF RAINFALL RUNOFF

## INTRODUCTION

Runoff from each of the subbasins identified in the study area was calculated using hydrologic models running on a PC computer model. This model requires input data in the form of a design storm rainfall distribution, commonly called a rainfall hyetograph. Earlier work for the City provided an Intensity-Duration-Frequency (IDF) chart which was based on data gathered from nearby rain gauge stations. Subsequent rainfall and runoff calculations were based on the IDF analysis figures.

This chapter of the report outlines the 24-hour precipitation figures that will be used to calculate the runoff flows in the drainage basins within the study area watershed. Rainfall hyetographs were calculated for the 2-, 5-, 10-, 25-, and 100-year storm events. These projections of storm events are typically referred to as the "return frequency" for a storm of a given magnitude. The resulting design storm hyetographs are then used in conjunction with the subbasin parameters. The combination of the subbasin parameters and the design hyetographs provides an estimate of the undetained, peak flow runoff hydrographs at specific points in the drainage system. Estimates of runoff rates and volumes can be calculated for both undeveloped and developed areas within the study area.

## DESIGN STORM PARAMETERS

In developing runoff rates for the subbasins in the study area, a design-storm duration of 24 hours was used. This design storm duration has been found to produce the greatest peak flows for basins with characteristics similar to those found in Woodburn. Shorter durations may be used where basins are smaller, have highly impervious areas or are much more steeply sloped than those found in Woodburn and the surrounding areas. At the other extreme, longer design storms would be used to produce peak runoff rates when the basins are much larger and have highly pervious surfaces.

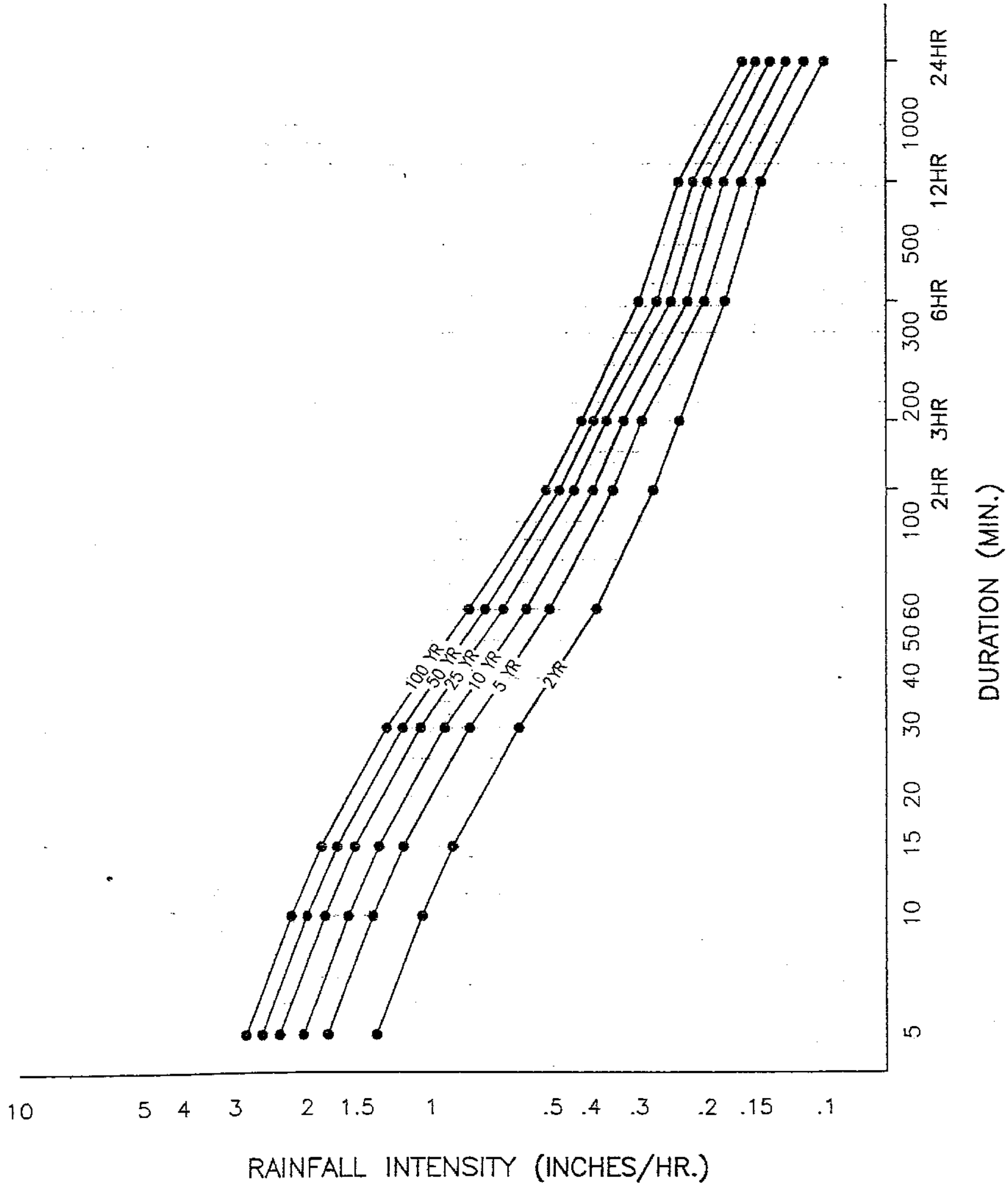
Total 24-hour rainfall depths for the 2-, 5-, 10-, 25-, and 100-year storm events were determined from isopluvial maps published by NOAA in the Precipitation-Frequency Atlas of the Western United States, Volume X - Oregon, 1973. These values were applied to the standard Soils Conservation Service (SCS) Type IA Rainfall Distribution to develop design storm rainfall hyetographs. The SCS has developed rainfall distribution curves to represent the depth vs. time relationship observed for large storms in various regions of the country. The Type IA distribution is typical of large storm events in Western Oregon.

The IDF curves are used to determine the rainfall intensity in inches-per-hour when associated with a storm event of a particular frequency and duration. This information is critical when using the Rational Method of predicting rainfall runoff. This method may be used by designers when estimating runoff for very small parcels in the study area. It is not typically used, and can become overly conservative when used to predict runoff for parcels exceeding 10 acres.

The IDF curves computed for this study are shown in Figure 6, Intensity, Duration and Frequency Curves. In addition, Figures 7 and 8, the SCS Type IA Rainfall Distribution Curve and SCS Type IA Rainfall Hyetograph are shown for reference.

## PEAK SUBBASIN FLOWS

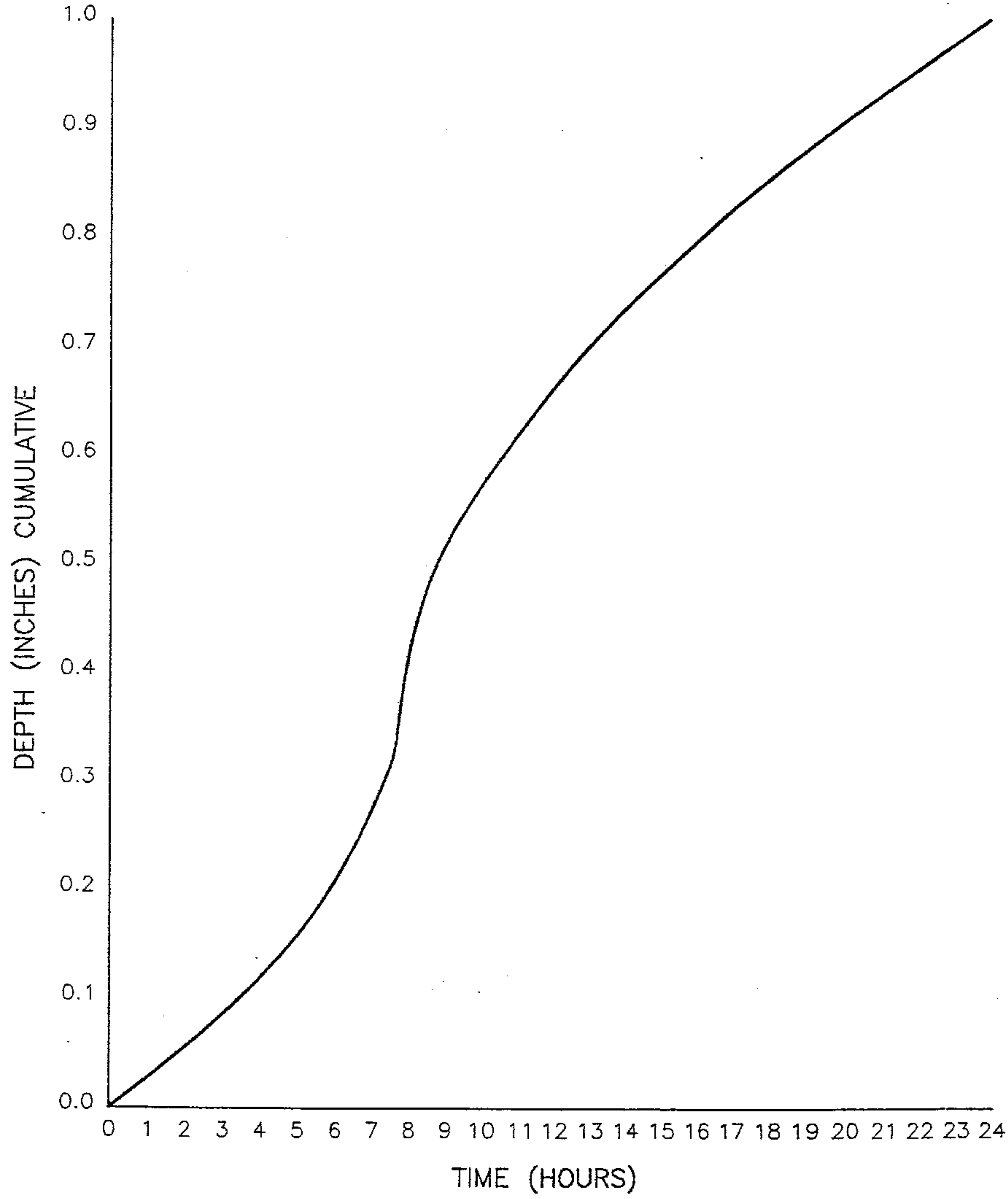
Flow hydrographs for each of the subbasins in the study area were computed for the 2-, 5-, 10-, 25-, and 100-year storm events. The results of these calculations are shown in Tables 5-1 and 5-2, Undetained Individual Subbasin Flows, for Mill Creek and the East Tributary of Senecal Creek




DECEMBER, 1992

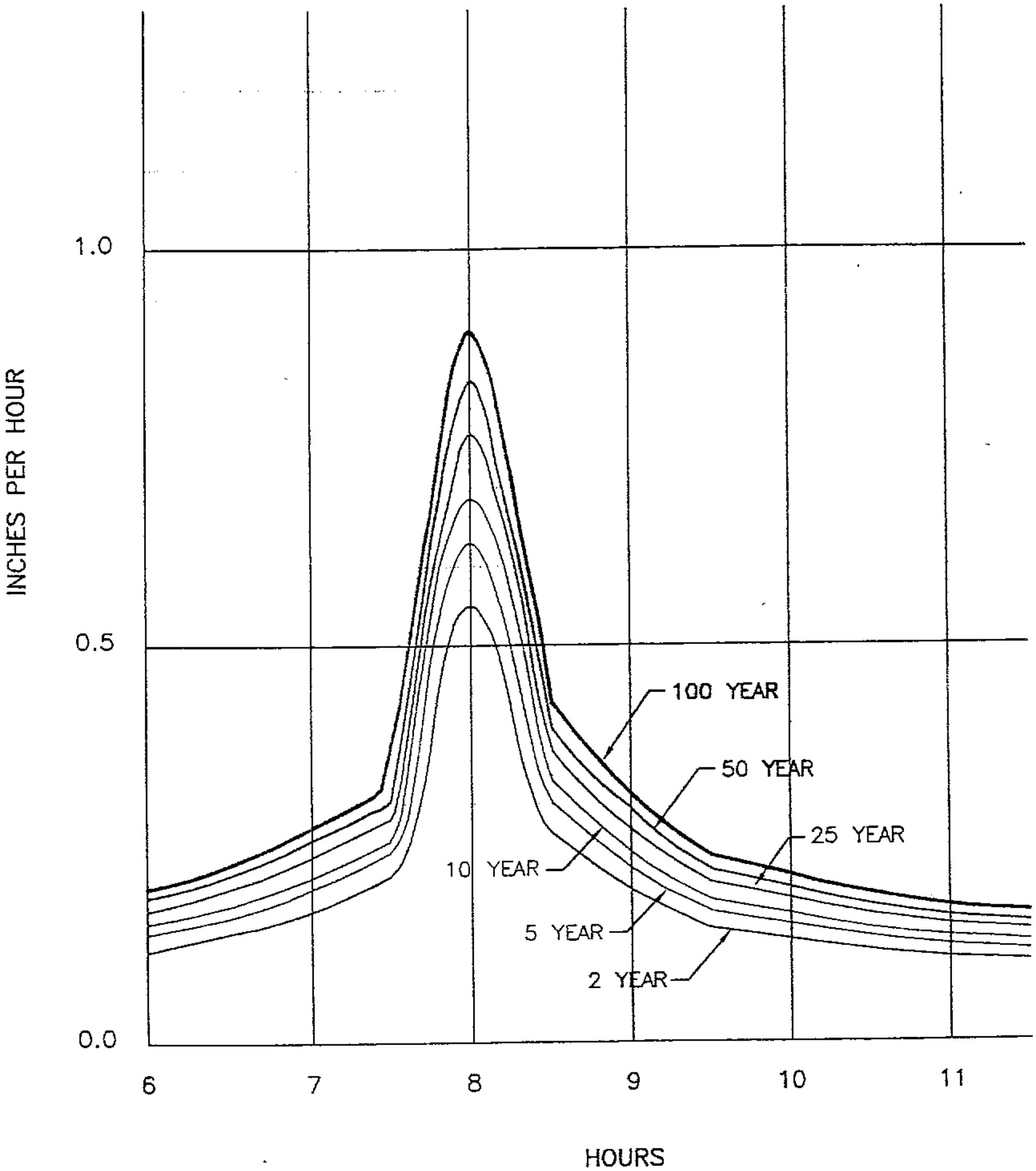
| DATE | | | FIGURE |
|---|---|---|---|
| July 2001 | *CRANE & MERSETH*<br>*Engineering/Surveying*<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | **INTENSITY DURATION AND FREQUENCY CURVE**<br>**City of Woodburn**<br>STORM DRAINAGE MASTER PLAN | **6** |

K/cad/112-005c/dwg/112-f6.dwg




| DATE | | | FIGURE |
|---|---|---|---|
| July 2001 | CRANE & MERSETH<br>Engineering/Surveying<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | SCS TYPE 1A RAINFALL DISTRIBUTION CURVE<br>City of Woodburn<br>STORM DRAINAGE MASTER PLAN | 7 |

K:/coc/112-005c/dwg/112-F7.dwg




| DATE | CRANE & MERSETH Engineering/Surveying | SCS TYPE 1A 24 HOUR RAINFALL HYETOGRAPH | FIGURE |
|---|---|---|---|
| July 2001 | 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | City of Woodburn STORM DRAINAGE MASTER PLAN | 8 |

K:/cad/112-005c/dwg/112-f8.dwg

# Table 5 - 1
## Undetained Individual Subbasin Flows (CFS)
### Mill Creek

| Sub-basin Name | Area (Ac) | Existing Conditions 2 YR | 5 YR | 10 YR | 25 YR | 50 YR | 100 YR | Full Buildout 2 YR | 5 YR | 10 YR | 25 YR | 50 YR | 100 YR |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***M-1*** | 359 | 26 | 36 | 43 | 53 | 63 | 71 | 26 | 35 | 43 | 53 | 63 | 71 |
| ***M-1A*** | 390 | 37 | 52 | 64 | 81 | 96 | 110 | 37 | 52 | 64 | 81 | 96 | 110 |
| ***M-1B*** | 71 | 12 | 15 | 18 | 22 | 25 | 28 | 16 | 19 | 22 | 26 | 30 | 33 |
| ***M-1C*** | 53 | 9 | 12 | 14 | 17 | 19 | 22 | 9 | 12 | 14 | 17 | 19 | 22 |
| ***M-2*** | 109 | 9 | 13 | 16 | 20 | 24 | 28 | 13 | 19 | 23 | 29 | 34 | 39 |
| ***M-3*** | 76 | 4 | 6 | 8 | 10 | 13 | 15 | 4 | 6 | 8 | 10 | 13 | 15 |
| ***M-3A*** | 96 | 4 | 7 | 9 | 12 | 15 | 18 | 19 | 24 | 27 | 33 | 37 | 42 |
| ***M-4*** | 40 | 5 | 7 | 9 | 11 | 13 | 14 | 7 | 9 | 11 | 13 | 15 | 17 |
| ***M-4A1*** | 42 | 10 | 13 | 15 | 17 | 19 | 21 | 17 | 20 | 22 | 25 | 28 | 30 |
| ***M-4A2*** | 16 | 5 | 6 | 7 | 8 | 9 | 10 | 5 | 6 | 7 | 8 | 9 | 10 |
| ***M-4A3*** | 18 | 4 | 5 | 6 | 7 | 8 | 9 | 7 | 8 | 9 | 11 | 12 | 13 |
| ***M-5*** | 59 | 9 | 12 | 14 | 18 | 20 | 23 | 16 | 20 | 23 | 26 | 29 | 32 |
| ***M-5A1*** | 69 | 7 | 10 | 12 | 15 | 18 | 21 | 13 | 16 | 19 | 23 | 26 | 29 |
| ***M-5A2*** | 65 | 6 | 8 | 10 | 13 | 15 | 18 | 15 | 18 | 21 | 25 | 28 | 31 |
| ***M-5A3*** | 97 | 18 | 23 | 27 | 32 | 37 | 41 | 18 | 23 | 27 | 32 | 37 | 41 |
| ***M-5A4*** | 30 | 3 | 4 | 5 | 7 | 8 | 9 | 3 | 4 | 5 | 7 | 8 | 9 |
| ***M-5B1*** | 53 | 12 | 14 | 17 | 20 | 22 | 25 | 12 | 14 | 17 | 20 | 22 | 25 |
| ***M-5B2*** | 37 | 7 | 9 | 10 | 12 | 14 | 15 | 11 | 13 | 15 | 17 | 19 | 21 |
| ***M-5B3*** | 55 | 9 | 11 | 13 | 16 | 19 | 21 | 11 | 13 | 16 | 19 | 21 | 24 |
| ***M-6*** | 42 | 1 | 2 | 3 | 4 | 6 | 7 | 1 | 2 | 3 | 4 | 6 | 7 |
| ***M-6A*** | 65 | 10 | 14 | 16 | 20 | 23 | 26 | 13 | 17 | 19 | 23 | 26 | 30 |
| ***M-6A2*** | 65 | 3 | 5 | 7 | 10 | 12 | 14 | 18 | 22 | 24 | 28 | 31 | 34 |
| ***M-6B*** | 23 | 3 | 4 | 4 | 6 | 7 | 8 | 9 | 11 | 12 | 14 | 15 | 16 |
| ***M-7*** | 99 | 15 | 19 | 23 | 28 | 32 | 36 | 21 | 26 | 30 | 35 | 40 | 44 |
| ***M-7A*** | 24 | 4 | 5 | 6 | 8 | 9 | 10 | 7 | 9 | 10 | 12 | 13 | 14 |
| ***M-7B1*** | 49 | 4 | 6 | 7 | 9 | 10 | 12 | 7 | 8 | 10 | 12 | 14 | 15 |
| ***M-8*** | 36 | 6 | 8 | 10 | 12 | 14 | 16 | 11 | 13 | 15 | 18 | 20 | 22 |
| ***M-9*** | 31 | 2 | 4 | 5 | 6 | 7 | 8 | 5 | 7 | 8 | 10 | 11 | 13 |
| ***M-9A1*** | 92 | 13 | 17 | 20 | 24 | 28 | 31 | 16 | 20 | 22 | 27 | 30 | 34 |
| ***M-9A3*** | 82 | 20 | 25 | 28 | 33 | 37 | 41 | 29 | 34 | 38 | 43 | 48 | 52 |
| ***M-9B*** | 33 | 10 | 12 | 14 | 16 | 18 | 20 | 13 | 15 | 17 | 19 | 21 | 23 |
| ***M-10*** | 100 | 16 | 21 | 24 | 30 | 34 | 39 | 17 | 22 | 26 | 32 | 36 | 41 |
| ***M-11*** | 78 | 7 | 10 | 12 | 16 | 19 | 22 | 19 | 24 | 27 | 31 | 35 | 39 |
| ***M-11A*** | 43 | 5 | 7 | 9 | 11 | 13 | 15 | 9 | 12 | 13 | 16 | 18 | 20 |
| ***M-11B1*** | 69 | 8 | 11 | 13 | 17 | 20 | 22 | 12 | 15 | 18 | 21 | 24 | 27 |
| ***M-11B2*** | 28 | 2 | 3 | 4 | 5 | 6 | 7 | 5 | 6 | 7 | 8 | 10 | 11 |
| ***M-11C*** | 34 | 3 | 4 | 5 | 6 | 8 | 9 | 8 | 9 | 11 | 13 | 14 | 16 |
| ***M-11C2*** | 39 | 5 | 7 | 8 | 10 | 12 | 14 | 13 | 16 | 18 | 21 | 24 | 26 |
| ***M-11D1*** | 88 | 7 | 11 | 13 | 17 | 20 | 24 | 14 | 19 | 22 | 27 | 31 | 35 |
| ***M-11D2*** | 48 | 12 | 15 | 17 | 20 | 22 | 25 | 10 | 13 | 15 | 18 | 20 | 23 |
| ***M-11E1*** | 57 | 4 | 7 | 8 | 10 | 13 | 15 | 6 | 9 | 11 | 14 | 16 | 19 |
| ***M-11E2*** | 77 | 6 | 9 | 11 | 14 | 17 | 20 | 17 | 21 | 24 | 29 | 33 | 36 |
| ***M-11E3*** | 61 | 4 | 6 | 8 | 10 | 12 | 14 | 4 | 6 | 8 | 10 | 12 | 14 |
| ***M-11F*** | 161 | 11 | 15 | 19 | 23 | 28 | 32 | 11 | 15 | 19 | 23 | 28 | 32 |
| ***M-12*** | 62 | 9 | 12 | 14 | 18 | 21 | 23 | 12 | 15 | 17 | 21 | 24 | 26 |

# Table 5 - 1
# Undetained Individual Subbasin Flows (CFS)
## Mill Creek

| Sub-basin | Area | Existing Conditions | | | | | | Full Buildout | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Name | (Ac) | 2 YR | 5 YR | 10 YR | 25 YR | 50 YR | 100 YR | 2 YR | 5 YR | 10 YR | 25 YR | 50 YR | 100 YR |
| ***M-12A1*** | 23 | 1 | 1 | 2 | 2 | 3 | 4 | 5 | 6 | 7 | 9 | 10 | 11 |
| ***M-12A2*** | 70 | 5 | 7 | 8 | 11 | 13 | 15 | 14 | 18 | 20 | 24 | 27 | 30 |
| ***M-12A3*** | 12 | 1 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 4 | 5 | 5 | 6 |
| ***M-12B*** | 68 | 6 | 8 | 10 | 13 | 16 | 19 | 22 | 26 | 29 | 33 | 37 | 40 |
| ***M-12C*** | 134 | 9 | 12 | 15 | 20 | 24 | 28 | 27 | 34 | 39 | 46 | 52 | 58 |
| ***M-13*** | 955 | 52 | 75 | 92 | 118 | 140 | 161 | 52 | 75 | 92 | 118 | 140 | 161 |

# TABLE 5 - 2
## East Tributary of Senecal Creek
## Undetained Individual Subbasin Flows

| Sub-basi | Area (Acres) | Area (Sq. Miles) | Existing 25 YR (CFS) | 100 YR (CFS) | Full Buildout 25 YR (CFS) | 100 YR (CFS) |
|---|---|---|---|---|---|---|
| **Natural Watershed** | | | | | | |
| A | 124.5 | 0.195 | 20 | 28 | 20 | 28 |
| B-1 | 60.3 | 0.094 | 24 | 29 | 24 | 29 |
| B-2 | 49.4 | 0.077 | 7 | 10 | 7 | 10 |
| C | 49.0 | 0.077 | 15 | 20 | 17 | 22 |
| D | 78.3 | 0.122 | 20 | 27 | 42 | 50 |
| E | 42.7 | 0.067 | 16 | 20 | 28 | 33 |
| F | 31.1 | 0.049 | 11 | 15 | 17 | 20 |
| G | 76.6 | 0.120 | 13 | 17 | 38 | 45 |
| H | 26.7 | 0.042 | 13 | 16 | 15 | 18 |
| I | 33.6 | 0.053 | 16 | 20 | 20 | 24 |
| J | 40.6 | 0.063 | 7 | 9 | 24 | 28 |
| K | 83.1 | 0.130 | 13 | 17 | 33 | 41 |
| L | 121.0 | 0.189 | 18 | 24 | 18 | 24 |
| **Potential Re-routing of Future Development** | | | | | | |
| M-C | 24.7 | 0.039 | - | - | 15 | 18 |
| S-I | 17.0 | 0.027 | - | - | 12 | 14 |
| M-HDR | 30.3 | 0.047 | - | - | 16 | 19 |
| M-LDR | 42.9 | 0.067 | - | - | 11 | 14 |

* NOTE: M denotes basins currently draining to Mill Creek.
S denotes basins currently draining to Main Senecal Cree

respectively. These flows represent the runoff flows from each subbasin and its specific natural parameters as identified earlier.

## PEAK CUMULATIVE FLOWS

Hydrographs from each subbasin were routed and flows combined using the routing methods described in Chapter 3, Study Methodology. The models take relative timing of flow patterns into account when peak flows from tributary subbasins arrive at a specific node at different times and allows for the attenuation of the peak flows when storage occurs in the runoff channels and flood ways of Senecal Creek and Mill Creek.

Storage locations exist throughout both areas and are not specifically identified in this report. However, Senecal Creek exhibits significant storage in the area north of Highway 214 and south of the new crossing at Senecal Estates. No areas of unusual storage were identified along the East Senecal Creek Tributary except for the small natural detention area located in the southwest quadrant of the intersection of Interstate 5 and Highway 214.

Along Mill Creek several areas of natural storage exist between the major crossings. These provide storage during high flow events and extend from the man-made pond near the south city limit on the main stem of Mill Creek to the large area south of the Front Street/SPRR crossing near Highway 214. Storage also exists along the west tributary of Mill Creek that extends from the Cleveland Street crossing west to Settlemier Avenue.

Peak instream undetained flows for the main stem of Mill Creek are shown on Table 5-3, Peak In-stream Flows at Key Points.

# Table 5 - 3
# Peak In-Stream Flows at Key Points
# Mill Creek

| Node | Description | Upstream Drainage Area | | | Peak In-stream Flows (CFS) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Incremental | Cumulative Upstream | | 1996 Conditions | | | Full Buildout | | |
| | | (Acres) | (Acres) | Sq. Miles) | 10 YR | 25 YR | 100 YR | 10 YR | 25 YR | 100 YR |
| *M-1* | Mill Cr. @ Crosby Rd. | 874 | 4,567 | 7.14 | 390 | 447 | 542 | 445 | 565 | 640 |
| *M-2* | Mill Cr., downstream of M-2 | 109 | 3,693 | 5.77 | 289 | 340 | 418 | 370 | 476 | 519 |
| *M-3* | Mill Creek @ Private Drive | 172 | 3,584 | 5.60 | 280 | 331 | 400 | 369 | 460 | 500 |
| *M-4* | Mill Creek @ Hazlenut Dr. | 580 | 3,412 | 5.33 | 280 | 331 | 400 | 369 | 453 | 490 |
| | | | | | | | | | | |
| *M-5* | Goose Creek Outlet | 59 | 465 | 0.73 | 100 | 122 | 161 | 131 | 156 | 198 |
| *M-5A* | Goose Cr. Trib. A @ Hwy 214 | 260 | 260 | 0.41 | 51 | 64 | 86 | 69 | 83 | 107 |
| *M-5B* | Goose Cr. Trib. B below Hwy 214 | 146 | 146 | 0.23 | 37 | 45 | 58 | 42 | 52 | 66 |
| | | | | | | | | | | |
| *M-6* | Mill Cr. @ Front St. / RR | 341 | 2,832 | 4.42 | 217 | 248 | 292 | 241 | 276 | 318 |
| | | | | | | | | | | |
| *M-7* | Side tributary @ Front St. | 210 | 210 | 0.33 | 30 | 33 | 37 | 65 | 76 | 96 |
| | | | | | | | | | | |
| *M-8* | Mill Cr. @ Gatch / Hardcastle | 43 | 2,491 | 3.89 | 199 | 230 | 272 | 218 | 252 | 290 |
| *M-9* | Mill Cr. @ Lincoln St. | 282 | 2,448 | 3.83 | 196 | 226 | 268 | 218 | 252 | 290 |
| | | | | | | | | | | |
| *M-9A1* | 99E tributary @ outfall to Mill Cr. | 218 | 218 | 0.34 | 53 | 65 | 84 | 82 | 95 | 117 |
| | | | | | | | | | | |
| *M-10/11* | Mill Cr. @ Cleveland St., downstream | 754 | 2,166 | 3.39 | 180 | 213 | 271 | 194 | 262 | 288 |
| *M-10* | Mill Cr. @ Cleveland St., upstream | 100 | 1,412 | 2.21 | 116 | 140 | 182 | 123 | 149 | 194 |
| | | | | | | | | | | |
| *M-11* | Stubb Rd. Tributary @ Brown St. | 78 | 754 | 1.18 | 65 | 75 | 89 | 83 | 96 | 113 |
| *M-11A* | Stubb Rd. @ Front St. | 43 | 677 | 1.06 | 60 | 69 | 81 | 66 | 74 | 84 |
| *M-11B* | Stubb Rd. @ Settlemeir | 634 | 634 | 0.99 | 57 | 65 | 77 | 62 | 70 | 79 |
| | | | | | | | | | | |
| *M-12* | Mill Cr. @ Wilson St. | 155 | 1,312 | 2.05 | 111 | 140 | 176 | 116 | 141 | 184 |
| | | | | | | | | | | |
| *M-12A* | Brown Street Tributary "12A" | 93 | 93 | 0.15 | 10 | 13 | 18 | 27 | 32 | 41 |

## Table 5 - 3
## Peak In-Stream Flows at Key Points
## Mill Creek

| Node | Description | *Upstream Drainage Area* Incremental (Acres) | Cumulative Upstream (Acres) | Cumulative Upstream Sq. Miles) | *Peak In-stream Flows (CFS)* 1996 Conditions 10 YR | 1996 Conditions 25 YR | 1996 Conditions 100 YR | Full Buildout 10 YR | Full Buildout 25 YR | Full Buildout 100 YR |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | |
| ***M-12B*** | Mill Cr. @ Deer Run | 202 | 1,157 | 1.81 | 109 | 140 | 176 | 112 | 142 | 192 |
| ***M-13*** | Mill Cr. @ UGB | 955 | 955 | 1.49 | 92 | 118 | 161 | 92 | 118 | 161 |

# Chapter 6

# EXISTING SYSTEM DESCRIPTION

## INTRODUCTION

This chapter presents a description and partial inventory of the existing public storm water systems found in the Mill Creek and Senecal Creek drainages as these drainageways traverse the City of Woodburn. These descriptions do not encompass the smaller portions of the systems such as catch basins, inlets and pipelines less than 12 inches in diameter. Likewise, small ditches and culverts that convey storm water across private lands are not included in this description and inventory.

## MILL CREEK

Mill Creek is the major natural drainage route serving most of the incorporated area of Woodburn, extending from north to south through the center of the City. Its drainage area within the Study area as indicated on Figure 1, Senecal & Mill Creek Drainage Basin Boundaries, contains about 5017 acres (7.84 square miles). The drainage system that has been constructed over time generally follows the natural topography with open channels branching into a number of areas of the City, supplemented by constructed drainage facilities using storm sewer systems serving residential and commercial areas, major culverts and bridges crossing the main stem of Mill Creek and some of the more major tributaries. Figure 9, Mill Creek Drainage, Existing Facilities, shows the existing storm water facilities in the Mill Creek basin within Woodburn.

Several major tributaries flow to the main stem of Mill Creek. In the urbanized areas of the City, many of these tributaries have been converted to piped systems and now carry much of the storm runoff. Several tributaries still convey storm flows via open channel. Most of these tributaries are unnamed and will be referred to by their subbasin designation.

At the north city limit, a shallow tributary extends west from Mill Creek near its crossing of Crosby Road. This tributary extends through subbasin M-1 and collects runoff from that subbasin. It is included in the model to incorporate any backwater effects caused by this system. The tributary primarily serves undeveloped lands including portions of the Tukwila Golf Course and a small, developed area labeled M-1a.

A tributary to the east serves the M-4 system of subbasins. This tributary is now entirely piped and discharges through two major storm drain lines; one along Highway 214, the other just north of the highway which serves the light industrial areas in the Woodburn Industrial Park. These lines, an older, 24-inch diameter system along the south right-of-way of Highway 214 and a newer 30-inch line serving the industrial properties convey all storm water to Mill Creek. Each system crosses the Southern Pacific Railroad (SPRR) main line.

Goose Creek is a steep channel extending westerly from near the intersection of Mill Creek and Highway 214. It provides drainage to the undeveloped area north of the highway, the high school property and collects piped flows where it crosses Boones Ferry Road. Subbasins M-5 flow through the piped and open channels of Goose Creek. Flows in Goose Creek are increased by drainage collected in a 48-inch diameter line immediately south of and parallel to Highway 214. Stormwater originating in the M-5 subbasins concentrates in either Goose Creek or the 48-inch line before discharging to Mill Creek. This sizeable system of subbasins extend to the Senecal Creek drainage and includes much of Senior Estates and all the commercial areas along Highway 214. A large undeveloped area north of Highway 214 and east of Boones Ferry Road contains the last remaining large parcels that are in the process of being developed in the these subbasins

Subbasins titled M-6a and M-6c are collected in 21- and 30-inch lines respectively. The larger line is aligned along Hardcastle Avenue and provides service to the residential areas both north and south of this major street and extends to the commercial areas along Highway 99E. The 21-inch line generally provides drainage to residential areas east of Mill Creek.

The oldest storm drainage system in the City serves the M-7 subbasins. This system of lines ranging from 8- to 30-inch lines generally follows an existing low area that meanders through the residential areas west of Front Street and extend to the west to Cascade Drive. Portions of this system are still open channel flow, however most of the system is conveyed in closed conduits. This system is probably the most hydraulically restricted system in the City, contains the oldest pipes and likely cannot be expanded to include any new flows from the remaining undeveloped area east of Senior Estates. Runoff from infill lots will be piped into this system and, with possible expanded capacity in the culverts on 1st and 2nd Streets, overall system capacity should be adequate. A small city park located along the west side of Front Street just north of Hardcastle Avenue serves as a detention facility during high flows.

Storm flows from subbasin M-7a are conveyed through a 16-inch diameter storm line northerly to the area of the city park. At this point, a leaping-weir manhole diverts storm flows to a surface ditch. This surface system conveys storm flows around the park area where they re-enter a storm drain line before crossing Front Street and the SPRR embankment.

Subbasins M-9a1 and M-9a3 generally follow a natural stream channel flowing from east to west, crossing Gatch Street south of Lincoln Street before discharging to Mill Creek. This system is a combination of open channel and closed conduit routes and provides storm runoff service to several residential areas and a number of commercial properties along Highway 99E. Property owners served by this system and located along Gatch Street have reported flooding problems in the past and the City has responded through extension of storm sewers in some areas.

Immediately south of Cleveland Street, Mill Creek divides into two major channels. The smaller of these channels turns west, crossing Brown Street, Front Avenue, the SPRR right-of-way and Settlemier Avenue. It is primarily an open channel system, fed by small lateral storm sewers. In the area of the city park east of Settlemier Avenue, the tributary has been routed through a 48-inch closed conduit storm sewer. This major tributary serves much of the developed and undeveloped property in the southwest quadrant of the Woodburn UGB. Large diameter culverts are used to cross Cleveland Street, Brown Street, Front Street/SPRR and Settlemier Avenue. A privately-owned detention facility is located west of Settlemier Avenue near the west end of Ben Brown Lane and serves the homes in the adjoining subdivision. More detention facilities have been constructed with the largest serving the new school property on Parr Road. Storm flows tributary to this system originate in the M-11 subbasins.

The main channel of Mill Creek crosses Cleveland Street. It progresses upstream through open channels, enclosed only by culverts where it crosses city streets. Portions of Mill Creek are located in City-owned parks between Cleveland Street and the south city limit. A constructed pond is located along the main stem of Mill Creek, east of Hermason Street. Control and maintenance of this pond is currently not a part of the City's responsibilities. The major piped system located in subbasin M-12 is a 48-inch line serving new subdivisions north and south of Warren Way. This line has been sized to serve developing areas west of Mill Creek and will be extended as development dictates. An off-line detention facility has been constructed in this subbasin to serve the Meadow Wood subdivision.

South of the City, Mill Creek progresses upstream into farmland where it terminates before reaching the neighboring town of Gervais.

Table 6-1, Mill Creek Tributary and Subbasin Storm Drain Capacity Inventory, provides an outline of the major tributaries, an inventory of the major storm water facilities in the subbasin including their diameter, type and length and their hydraulic adequacy compared to modeled

# Table 6 - 1
# Mill Creek Tributary and Sub-basin
# Storm Drain Capacity Inventory

(continued)

| **Pipe/Channel Segment** **Description** | **Flow Node/ Subbasin** | Size / Diam. (Inches) | Type | Approx. Length (FT) | **ADEQUACY Design Event Carried (YR)** 1996 Conditions. | Full Build. |
|---|---|---|---|---|---|---|
| | | | | | | |
| **SUB-BASIN M-6A2** | | | | | | |
| Hardcastle Ave. 30" Outfall Line | M-6A2 | 30" | CSP | 2800 | 100 | 25 |
| | | | | | | |
| **TRIBUTARY M-7 (includes M-11C2)** | | | | | | |
| SETTLEMEIR TO FRONT ST. | | | | | | |
| Front St. Crossing & Leaping Weir | #7 | 30 | CMP | 230 | 100 (Ponded) | 2 (Ponded) |
| Open Channel, 1st to Front | #7 | | DITCH | 250 | 25, Storage Area | Maintaing as Storage or Convey 100 CFS |
| 1st Street Crossing | #7 | 30 | CMP | 150 | 2 | <2 |
| Open Channel, 2nd to 1st | #7 | | DITCH | 200 | 100, out of bank | Convey 100 CFS |
| 2nd St. Crossing | #7 | 36 | CMP | 70 | 5 | 2 |
| 72", 3rd to 2nd St. Crossing | #7 | 72 | CMP | 350 | 100 | 100 |
| 42" Lincoln to 3rd St. | #7b | 42 | CMP | 1390 | 100 | 25 |
| 24" Settlemeir to Lincoln | #7b | 24 | RCP | 280 | 25 | <2 |
| | | | | | | |
| **HAYES ST. LINE** | M-7B1/B2 | 18 | RCP | 390 | 10 | (No additional capacity) |
| | | | | | | |
| **AUSTIN CT. /HAYES ST. LINE** | M-7B1 | 18 | RCP | 750 | 10 | (No additional capacity) |
| | M-7B1 | 15 | RCP | 440 | 10 | (No additional capacity) |
| | M-7B1 | 18 | RCP | 520 | 10 | (No additional capacity) |
| | | | | | | |
| **TRIBUTARY M-9A, McKINLEY / 99E** | | | | | | |
| HWY 99E TO OUTFALL | | | | | | |
| 48" CMP Gatch St. Crossing | #9a | 48 | CMP | 375 | 100 | 100 |
| Open Channel, Gatch to Bryant | #9a | | DITCH | 800 | 100 , Ponded | Convey 75 CFS |
| 48" Outfall @ Bryant | #9a | 48 | CMP | 150 | 25 | 25 |
| 48" CMP, Bryant to McKinley | #9a | 48 | CMP | 550 | 50 | 50 |
| McKinley St. 24", Conf. 48" to 99E | M-9A3 | 24 | CMP | 600 | <2 | <2 |
| | | | | | | |
| | | | | | | |
| **SUB-BASIN M-10** | | | | | | |
| 12" Collector, Outfall to Jana Ave. | M-10 | 12 | CMP | 470 | 2 | (No additional capacity) |
| 12" Collector, Jana Ave. to Hawley | M-10 | 12 | CMP | 650 | 2 | (No additional capacity) |
| | | | | | | |
| **TRIBUTARY M-11** | | | | | | |
| CLEVELAND ST. OUTFALL TO SETTLEMEIR | | | | | | |
| Outfall Culvert, Brown to Cleveland | #11 | (2) 42" | RCP | | 100 | 5 (Undetained) |
| Open Channel, Front St. to Brown St. | #11 | | DITCH | | 50 | 2 (Undetained) |
| Front St. Crossing | #11a | 48" | RCP | 200 | 50 | 2 (Undetained) |
| Park pipe, Settlemeir to Front | #11b | 48" | RCP | 1160 | 50 | 2 (Undetained) |
| Settlemeir Crossing | #11b | 54" | CMP | 50 | 50 | 2 (Undetained) |
| | | | | | | |
| 18" A Street Collector | M-11 | 18" | I | 1300 | 5 | <2 |
| | | | | | | |
| **SPUR M-11B / PARR ST. TO CONF.** | | | | | | |
| Open Channel, Brown St. to Conf. Main Trib. | M-11B1/B2 | | DITCH | | 100, Backwater Ponding | Convey 30 CFS |

flows. Table 6-2, Mill Creek Main Stem Existing Culvert Inventory, shows additional detail on major structures on Mill Creek including surveyed data on overflow elevations and flood elevations.

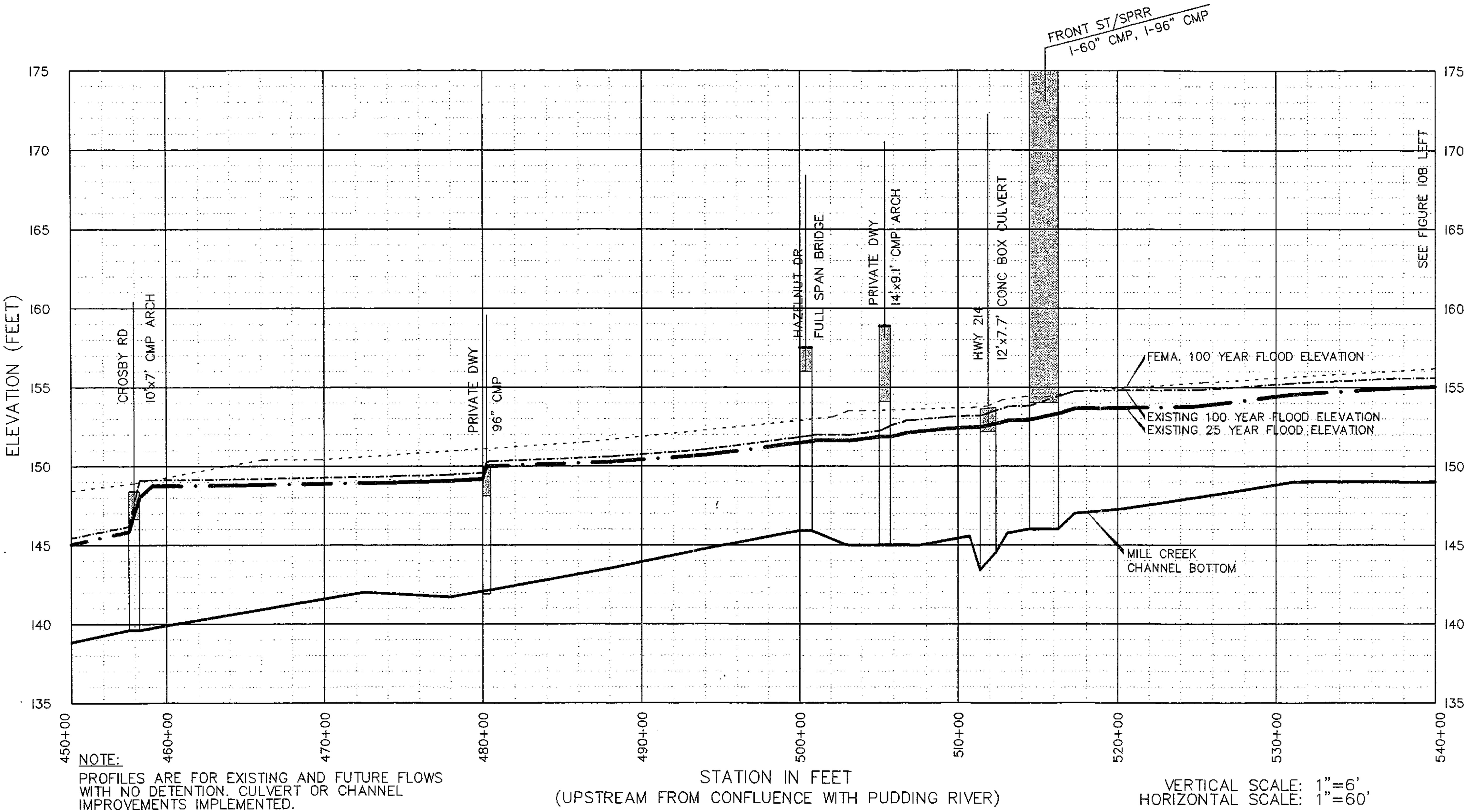
Figures 10A and 10B, Mill Creek Undetained Flows Structures and Flood Profiles, show the water surface profiles for 1996 conditions for the 25- and 100-year storm events. Figures 11A and 11B, Mill Creek Detained Flows, Structures and Flood Profiles, show future water surface profiles for the 25- and 100-year storm events assuming that detention measures as outlined in this Study are implemented.

**TABLE 6-2**

**Mill Creek Main Stem**
**Existing Culvert Inventory**

| Crossing Description | Flow Node | 1995 SURVEY DATA Size/ Diameter | Type | Length (FT) | Top of Road Overflow Elev. (FT) | Target Flood Elev. (FT) | APPROXIMATE CAPACITY Flow (CFS) | Event (YR) 1996 | Buildout |
|---|---|---|---|---|---|---|---|---|---|
| Crosby Road Arch Culvert | M-1 | 7'x10' | CMP Arch | 69 | 148.4 | 148.0 | 340 | 5 | 2 |
| Private Drive | M-2 | 8.3'x7.8' (96") | CMP | 26 | 149.1 | 149.0 | 280 | 2 | <2 |
| Hazelnut Ave. Bridge | M-4 | Natural Section | NA | 80* | 157.1 | 152.0 | >500 | 100 | 100 |
| High School Entrance Drive | M-4 | 9.1'x14.0' | CMP Arch | 66.8 | 158.9 | 153.4 | 490 | 100 | 100 |
| Hwy 214 - Box Culvert | M-5/6 | 12'X7.7' | Conc. Box | 73 | 154.4 | 154.0 | 500 | 100 (Backwater Flooding) | |
| Front St. and SPRR Culverts | M-6 | 96" | CMP | 185 | 180.6(RR) | 156.0 | 430 | 100 | 100 |
| Hardcastle Avenue - 72 " CMP | M-8 | 72" (deformed outlet) | CMP | 182 | 163.6 | 161.5 | 250 | 50 | 25 |
| Lincoln Street Culvert | M-9 | 84" (deformed) | CMP | 130* | 169.3 | 163.5 | 290 | 100 | 100 |
| Young Street Box Culvert | M-10/11 | 8'x6' | Conc. Box | 100* | 174.0 | 164.3 | 290 | 100 | 100 |
| Cleveland Street Arch Culvert | M-10 | 9.3'X16.4' | CMP Arch | 150* | 168 (street) | 164.4 | 210 | 100 | 100 |
| Marshall Street Culvert | M-10 | 48" | RCP | 57 | 165.5 | 165.5 | 82 | 10 | 5 |
| Stark Street Culverts | M-10 | (2) 48" | RCP | 62 | 167.9 | 167.0 | 200 | 100 | 100 |
| Wilson Street Culverts | M-12 | (2) 52" | RCP | 74 | 169.0 | 169.0 | 200 | 100 | 100 |

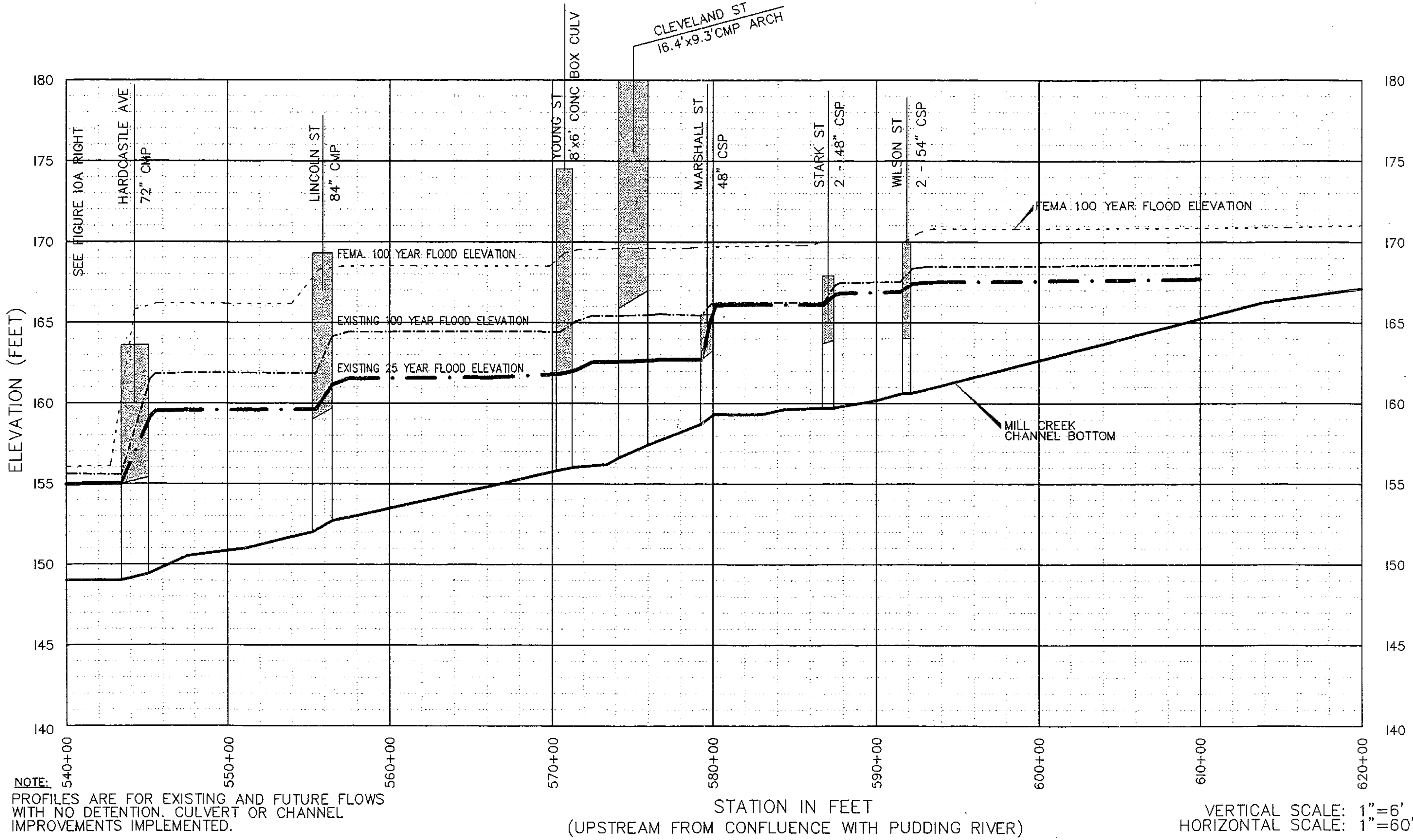
NOTE: * Indicates approximate length only, no field survey data.




NOTE:
PROFILES ARE FOR EXISTING AND FUTURE FLOWS WITH NO DETENTION. CULVERT OR CHANNEL IMPROVEMENTS IMPLEMENTED.

| DATE: July 2001 | CRANE & MERSETH Engineering/Surveying 6586 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | CLIENT LOGO | CITY OF WOODBURN SYSTEM DRAINAGE MASTER PLAN | MILL CREEK UNDETAINED FLOWS STRUCTURES AND FLOOD PROFILES | FIGURE 10A |
|---|---|---|---|---|---|

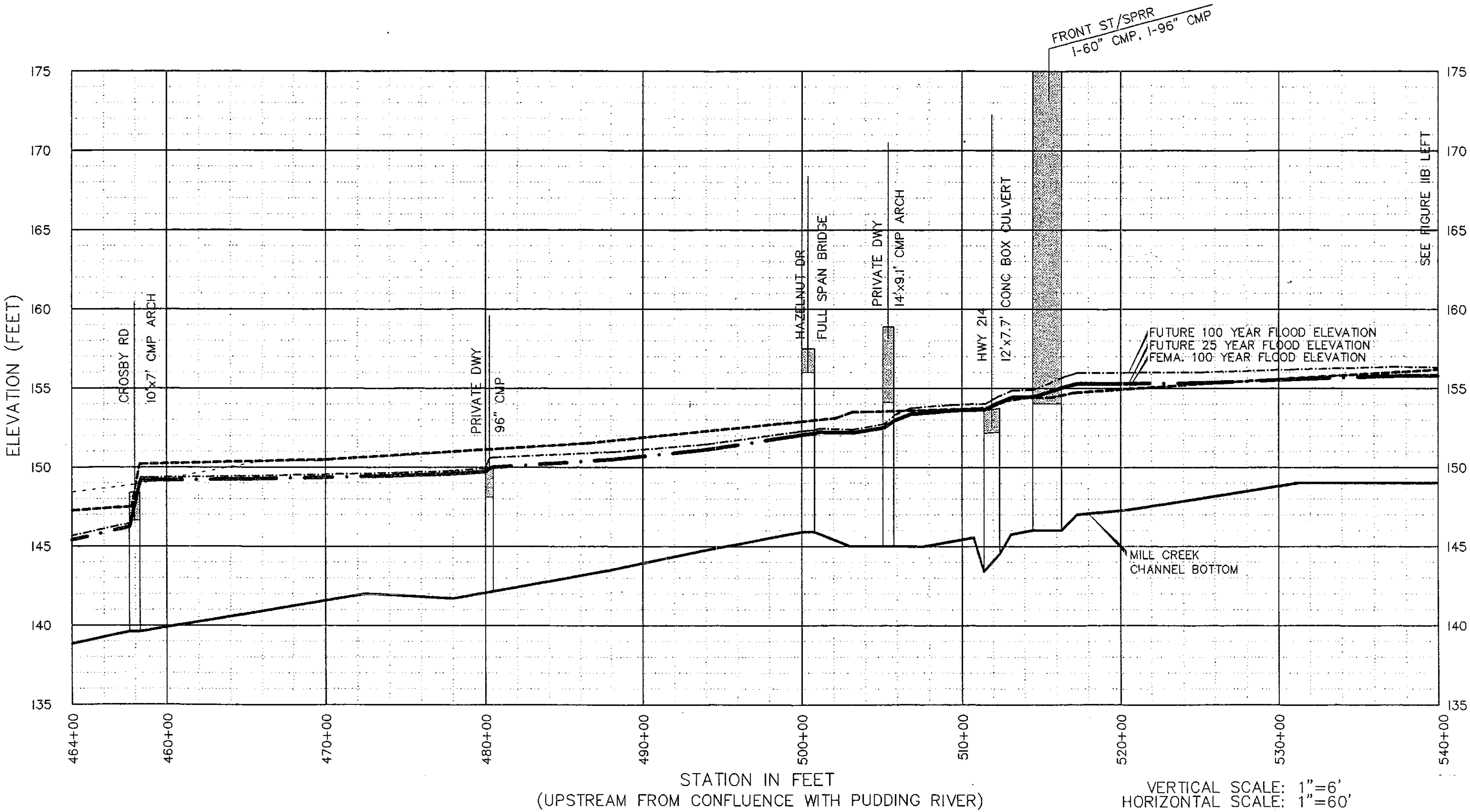
K:/CAD/112-005C/dwg/112-prof.DWG




NOTE: PROFILES ARE FOR EXISTING AND FUTURE FLOWS WITH NO DETENTION, CULVERT OR CHANNEL IMPROVEMENTS IMPLEMENTED.

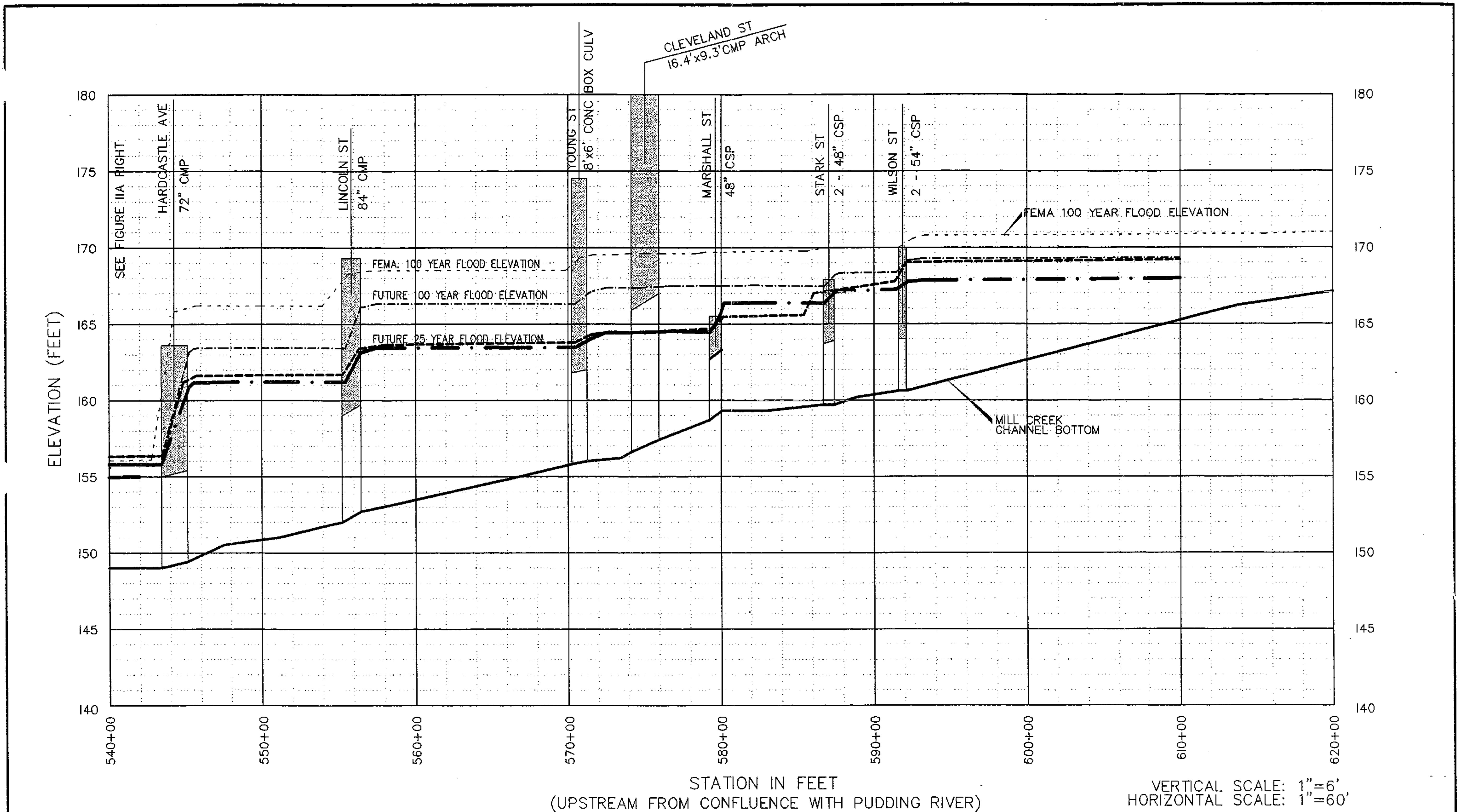
| DATE: July 2001 | CRANE & MERSETH Engineering/Surveying 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | City of Woodburn STORM DRAINAGE MASTER PLAN | MILL CREEK UNDETAINED FLOWS STRUCTURES AND FLOOD PROFILES | FIGURE 10B |
|---|---|---|---|---|

K:/CAD/112-005C/dwg/112-prof.DWG




| DATE: July 2001 | CRANE & MERSETH Engineering/Surveying 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | CLIENT LOGO | CITY OF WOODBURN SYSTEM DRAINAGE MASTER PLAN | MILL CREEK DETAINED FLOWS STRUCTURES AND FLOOD PROFILES | FIGURE 11A |
|---|---|---|---|---|---|

K:/CAD/112-005C/dwg/112-prof.DWG




| DATE: July 2001 | CRANE & MERSETH Engineering/Surveying 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | City of Woodburn STORM DRAINAGE MASTER PLAN | MILL CREEK DETAINED FLOWS STRUCTURES AND FLOOD PROFILES | FIGURE 11B |
|---|---|---|---|---|

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## SENECAL CREEK

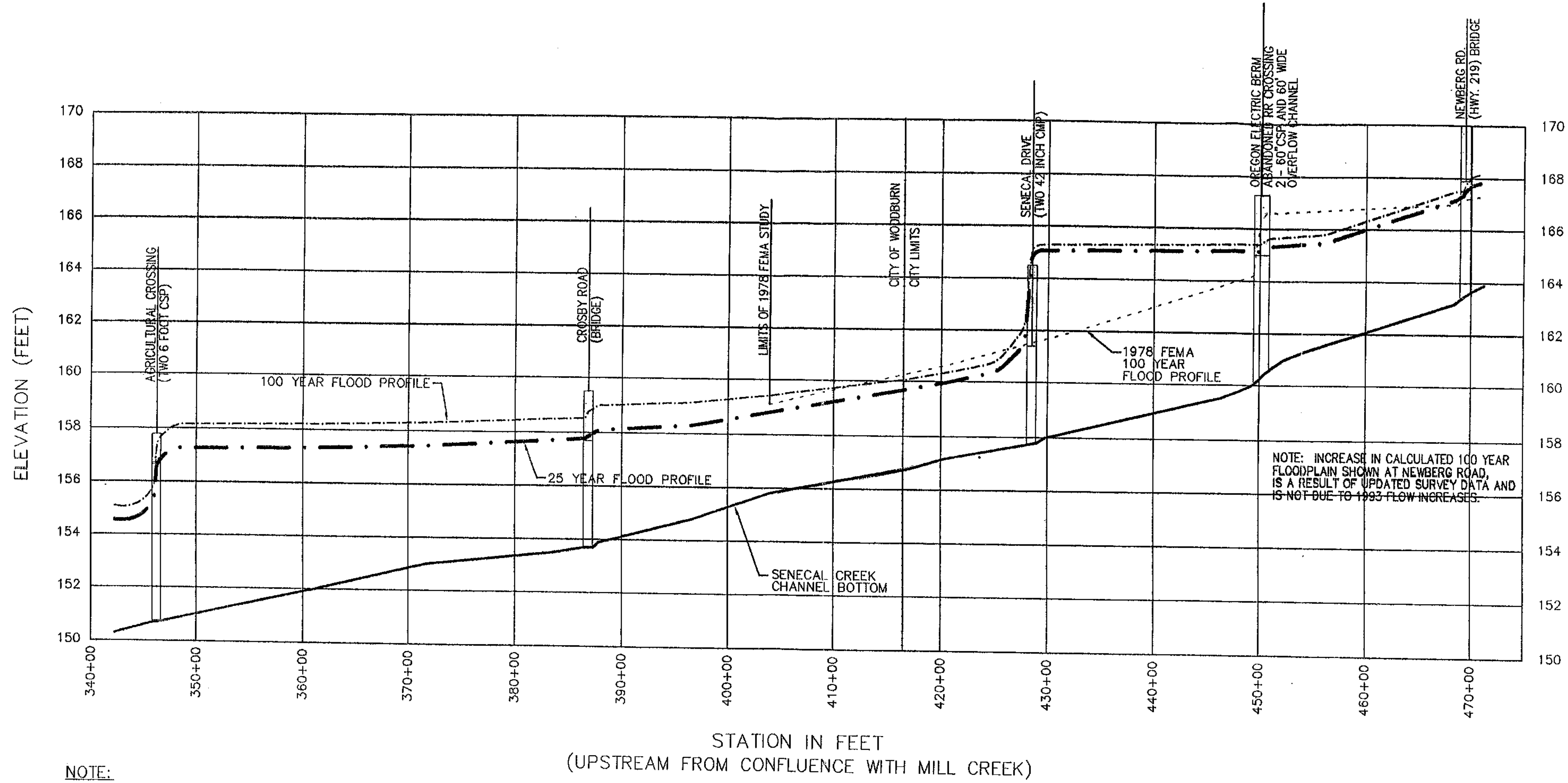
### Senecal Creek Main Stem

Senecal Creek is a shallow stream that begins southwest of Woodburn where it provides drainage to farmlands. The total drainage area of Senecal Creek within the Study area as indicated in Figure 1, Senecal & Mill Creek Drainage Basin Boundaries, is approximately 4,430 acres. The main stem of Senecal Creek skirts the western side of the City, paralleling the UGB and west city limit. In the city, Senecal Creek follows a course from the bridge crossing of Highway 214 near Butteville Road, through a wooded area west of West Woodburn, crosses an abandoned railroad embankment and flows through a crossing of the roadway into Senecal Estates. Figure 12, Senecal Creek Undetained Profile, shows the water surface profiles for the 25- and 100-year storm events. It is assumed that no detention facilities will be built along these segments of Senecal Creek. Most of the areas in the City served by Senecal Creek are served by the East Tributary of Senecal Creek. Additional modeling of the East Tributary has been completed and a separate set of recommendations has been compiled for this portion of Senecal Creek.

### East Tributary of Senecal Creek

Basin Delineation

The East Tributary of Senecal Creek drains approximately 823 acres along the Interstate 5 route corridor and a large portion of the City's future industrial and commercial development area. Within this watershed, approximately 210 acres of new impervious area is expected to be constructed between 1996 and 2016. In addition it is recommended that about 96 acres of developable lands outside the topographical watershed in the upper (southerly) end of the basin be drained to the East Tributary. Figure 13, Senecal Creek Drainage, Existing Facilities, shows the existing facilities in the East Tributary of Senecal Creek basin within Woodburn. Figure 14,



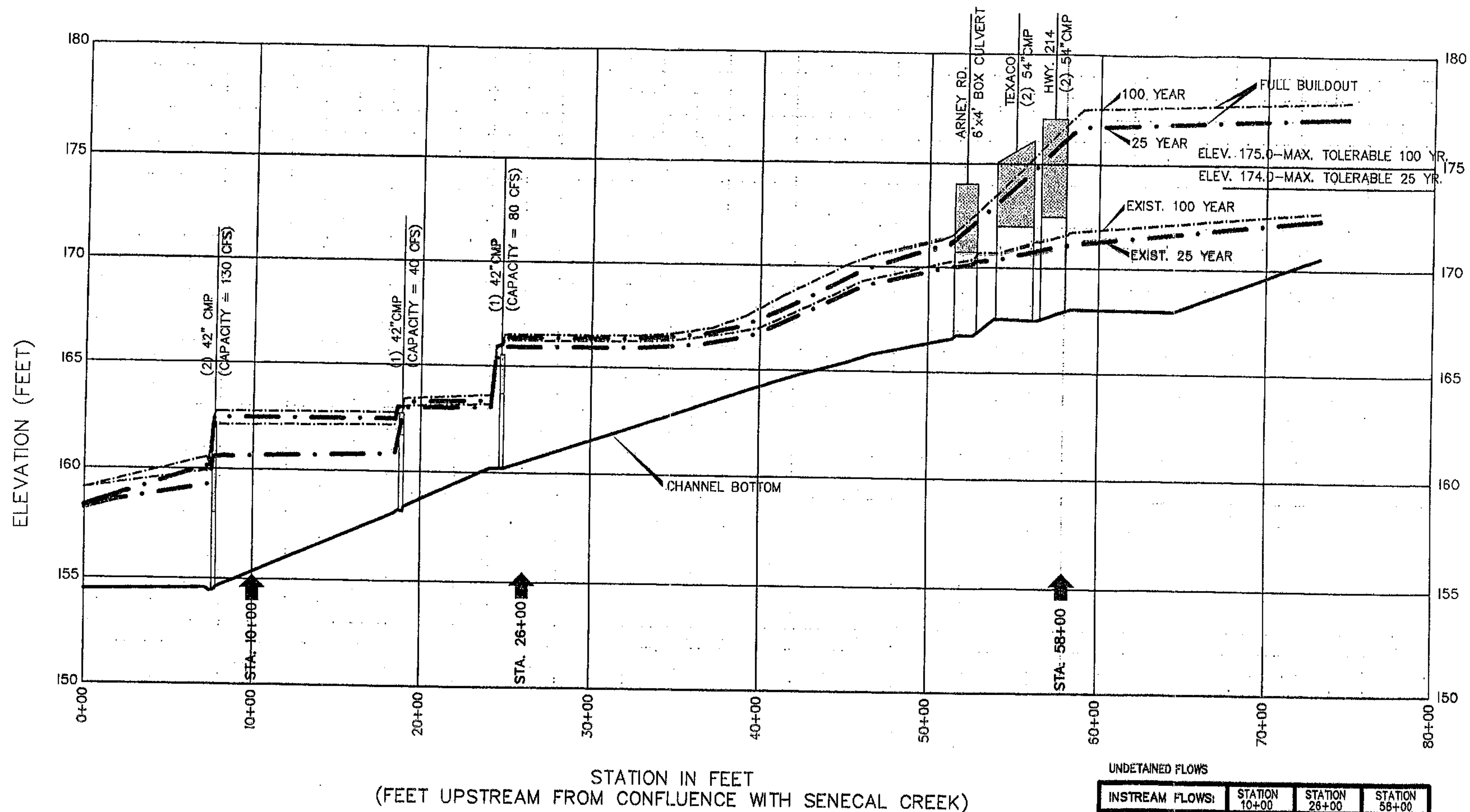

NOTE:

PROFILES ARE FOR EXISTING AND FUTURE FLOWS WITH NO DETENTION. CULVERT OR CHANNEL IMPROVEMENTS IMPLEMENTED.

VERTICAL SCALE: 1"=4'
HORIZONTAL SCALE: 1"=1000'

| DATE: July 2001 | CRANE & MERSETH Engineering/Surveying 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | City of Woodburn STORM DRAINAGE MASTER PLAN | SENECAL CREEK UNDETAINED FLOWS STRUCTURES AND FLOOD PROFILES | FIGURE 12 |
|---|---|---|---|---|

K:/CAD/112-005C/dwg/112-prof.DWG




UNDETAINED FLOWS

| INSTREAM FLOWS: | STATION 10+00 | STATION 26+00 | STATION 58+00 |
|---|---|---|---|
| EXIST. 25 YR | 115 CFS | 85 CFS | 58 CFS |
| EXIST. 100 YR | 155 CFS | 112 CFS | 76 CFS |
| FUTURE 25 YR* | 250 CFS | 230 CFS | 180 CFS |
| FUTURE 100 YR* | 350 CFS | 280 CFS | 220 CFS |

* EXPANDED AREA

DATE: July 2001

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6566 SE LAKE Rd., SUITE D
MILWAUKIE, OREGON 97222
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City of Woodburn
STORM DRAINAGE MASTER PLAN

EAST TRIBUTARY OF SENECAL CREEK
STRUCTURES & DETAINED FLOOD PROFILES

FIGURE 14

K:/CAD/112-005C/dwg/112-prof.DWG

East Tributary of Senecal Creek Structures and Detained Flood Profiles, shows the water surface elevations for the 25- and 100-year storm events.

The "Senecal Creek Capacity Analysis" (1994) evaluated the drainage capacity in terms of the existing channel and drainage structures. The "Senecal Creek Capacity Analysis" is included in Appendix "C" of this Study. In summary, the following key features of the basin were identified:

- The East Tributary provides an essential drainageway for Woodburn's development along I-5.

- Existing agricultural access crossings north (downstream) of Highway 214 will be overtopped by undetained runoff flow from anticipated future upstream development.

- The twin 54-inch culverts crossing Highway 214 immediately west of the I-5 intersection represent a constriction to stormwater flows from future development if no upstream detention is provided.

- There is limited culvert capacity available to convey stormwater from the future development area south of the Walmart complex on the east side of I-5 to the East Tributary on the west side of I-5.

- In the upper (southerly) reaches of the watershed, the topography becomes very flat leaving no well-defined drainageway to transport stormwater into the East Tributary.

In order to provide orderly and cost-effective urban drainage services to Woodburn's East Tributary watershed, the conveyance of increased stormwater flows to the East Tributary must be planned and drainage structures in the stream must be protected using upstream detention facilities or similar alternatives to mitigate stormwater flows. In the absence of such actions, significant capacity improvements must be made to a series of major culverts conveying the peak flows in the East Tributary in the vicinity of the Highway 214 and Interstate 5 intersection.

The upper (southerly) reaches of the East Tributary basin are so flat that the exact surficial basin boundary may be easily modified, thereby allowing the City to determine which properties should appropriately drain toward the East Tributary and which should drain eastward toward Mill Creek. Figure 4, Senecal Creek Drainage Subbasins, shows the natural basin boundary based on topography and the expanded basin boundary based on parcel limits and land use planning considerations. These expanded basin boundaries were analyzed in the "Senecal Creek East Tributary Capacity Analysis" (February, 1994) and will be assumed to hold throughout the following discussions.

## Management Zone Definition

Four management zones are defined for the East Tributary's expanded watershed. The boundaries for these zones are designated as ES-1 through ES-4 as shown on Figure 4, Senecal Creek Drainage Subbasins. The zones are delineated by the four quadrants created by the intersection of Interstate 5 and Highway 214.

## Culvert Capacity at Highway 214

The primary existing constraint in the East Tributary drainage system is in the vicinity of the Highway 214 culvert crossing. Water is conveyed through a series of culverts under Highway 214, through the Texaco station and under Arney Road. The two 54-inch diameter corrugated metal pipe culverts crossing Hwy 214 convey approximately 138 cubic feet per second (cfs). The maximum tolerable water surface elevation at the upstream (south) side of Hwy 214 is 174.0'.

This is due to the fact that water levels higher than 174.0' will adversely impact the existing I-5 culverts and other existing public stormwater systems up stream from the Hwy 214 crossing. Therefore, total stormwater flows generated south of Highway 214 should not be allowed to exceed 138 cfs to prevent surcharging of the 54-inch culverts. The following undetained flows are projected to occur using the runoff/routing model of the basin:

| EVENT | PEAK FLOW (CFS) |
|---|---|
| Existing (1993) 100-Year Event | 76 |
| Full Development, Undetained 25-Year Event | 180 |
| Full Development, Undetained 100-Year Event | 220 |

These future flow rates indicate that significant detention will be required to limit flows at Highway 214 to 138 cfs.

New I -5 Culvert Crossing to serve Zone ES-4

There are about 222 acres of prime future development land located in detention zone ES-4 south of the Walmart facility. Provisions have been made to convey runoff from this acreage across I-5 and into the East Tributary. At the time modeling for this Study was being completed there was no available capacity in the culverts crossing under I-5 into to convey additional runoff flows from ES-4. But, in order to prevent future flow rates above the 138 cfs at the Hwy 214 crossing it was recommended to limit the total flow capacity of the I-5 culverts to 78 cfs for detention zone ES-4. Preliminary analysis of the runoff flows indicated that an additional 42-inch diameter concrete sewer pipe (CSP) was required to convey the future modeled flow of 78 cfs from the east side of I-5 to the west side. In 2000 the 42-inch culvert was installed and presently serves zone ES-4. Approximately 3.0 feet of head loss has been incorporated into the crossing.

Near the upstream end of the East Tributary at Sta 73+00 as shown on Figure 14, East Tributary of Senecal Creek Structures and Detained Flood Profiles, the ditch bottom elevation is approximately 170.5'. This is the approximate location where existing 24" and 30" culverts that cross I-5 discharge into the East Tributary. At this location if the surface water elevation for the 100-year event in the East Tributary is maintained at 174.5', an upstream water surface elevation below 177.5 feet on the east side of I-5 can be maintained. A maximum 100-year water surface elevation of 177.5 on the upstream side of I-5 will allow sufficient hydraulic head to convey the 78 CFS, while low enough to avoid flooding on the east side of I-5.

In Zone ES-4, the following head losses in the future drainage system can be presumed:

| | |
|---|---|
| Prevailing ground surface elevation | 184.0 feet (MSL) |
| Less Minimum cover over the storm drain pipeline | 2.5 feet |
| Less Head Loss for 3,600 feet of storm drain pipe | 4.0 feet |
| Resulting water surface at I-5 culvert (upstream) | 177.5 feet (MSL) |

Detention in Zone ES-4

The undetained flow from future development in Zone ES-4 is 120 CFS. In order to reduce this flow to the capacity of the recommended 42-inch culvert (78 CFS), 7 acre-feet of detention is required on the east side of I-5. As of July 2001, this proposed detention facility is under construction as part of the Montebello Subdivision and is shown on Figure 13, Senecal Creek Drainage Existing Facilities. 7 acre-feet of storage volume will provide sufficient detention capacity in this zone. When combined with the mitigation policies listed above, and construction of the 7 acre-feet detention lagoon, the flow rate at the Hwy 214 crossing resulting from a 100-year event is not expected to exceed the target flow of 138 CFS.

# Chapter 7

# RECOMMENDATIONS FOR DRAINAGE IMPROVEMENTS

Continued improvements of the drainage system in Woodburn will rely on a number of activities and attention to the condition of the existing facilities. Specific recommendations for projects are found in Chapter 9, Recommended Capital Improvement Projects, of this report. These projects should be implemented as soon as funds, permitting and the public review process allow. However, these projects will only mitigate current shortcomings in the systems capacity and should not be relied upon to provide a drainage system which will adequately serve the City in the long term. Projects of growth and development patterns made in this study are based on the best understanding available at the time. Experience shows that trends change over time and forecasts of future conditions should be revisited periodically.

In light of this, the following general recommendations are provided to guide the City managers in their planning for future use of the drainage system.

## Detention Policy Implementation

A city-wide Stormwater Flow Management Program including policies regarding detention has been developed concurrently with this study. It addresses on-site detention for individual parcels of land and identifies several locations in the City where a public detention facility may be sited. For the past number of years, the city has utilized a guide presented in Table 7-1, "Volumes for Different Intensity Storms for 10-Acre Site". This guide is presented here to document the city's recent position on detention facility sizing. The new guidelines are provided in the Stormwater Flow Management Program document and these should be used for analysis and

# Table 7 - 1
## VOLUMES FOR DIFFERENT INTENSITY STORMS
### FOR 10-ACRE SITE

| Storms | Results (Rates) | i<br>(INTENSITIES) | A = 435,600<br>=10 acres | Developed<br>C = 0.71<br>(UN)developed<br>C = 0.25 | ft$^3$<br>Sec<br>(cfs) | Volumes ft$^3$ 3600sec hrs<br>storms sec hrs storm | |
|---|---|---|---|---|---|---|---|
| 100 yr. | 1.26"<br>2.7 hrs | 0.467 in<br>hr | 435,600 ft$^2$<br>or 10 acres | 0.71<br><br>0.25 | 3.313<br><br>1.167 | 32,205 ft$^3$<br><br>11,340 ft$^3$ | 32,205 ft$^3$<br>-- 11,340 ft$^3$<br>20,865 ft$^3$<br>storage volume |
| 50 yr. | 1.20"<br>2.76 hrs | 0.435 in<br>hr | 435,600 ft$^2$<br>or 10 acres | 0.71<br><br>0.25 | 3.087<br><br>1.087 | 30,672 ft$^3$<br><br>10,800 ft$^3$ | 30,672 ft$^3$<br>-- 10,800 ft$^3$<br>19,872 ft$^3$<br>storage volume |
| 25 yr. | 1.14"<br>2.86 hrs | 0.399 in<br>hr | 435,600 ft$^2$<br>or 10 acres | 0.71<br><br>0.25 | 2.830<br><br>0.996 | 29,138 ft$^3$<br><br>10,255 ft$^3$ | 29,138 ft$^3$<br>-- 10,255 ft$^3$<br>18,883 ft$^3$<br>storage volume |
| 10 yr. | 1.08"<br>2.97 hrs | 0.364 in<br>hr | 435,600 ft$^2$<br>or 10 acres | 0.71<br><br>0.25 | 2.582<br><br>0.909 | 27,605 ft$^3$<br><br>9,720 ft$^3$ | 27,605 ft$^3$<br>-- 9,720 ft$^3$<br>17,885 ft$^3$<br>storage volume |
| 5 yr. | 0.935"<br>3.28 hrs | 0.285 in<br>hr | 435,600 ft$^2$<br>or 10 acres | 0.71<br><br>0.25 | 2.024<br><br>0.713<br>(320 gpm) | 23,899 ft$^3$<br><br>8,415ft$^3$ | 23,899 ft$^3$<br>-- 8,415 ft$^3$<br>15,484 ft$^3$<br>storage volume |
| 2 yr. | 0.800"<br>3.64 hrs | 0.220 in<br>hr | 435,600 ft$^2$<br>or 10 acres | 0.71<br><br>0.25 | 1.560<br><br>0.549 | 20,448 ft$^3$<br><br>7,200 ft$^3$ | 20,448 ft$^3$<br>-- 7,200 ft$^3$<br>13,248 ft$^3$<br>storage volume |

**CITY OF WOODBURN**

**RUN OFF DETENTION REQUIREMENT**

1) **Construct a device that has capacity for detaining difference in run off volume received by undeveloped and developed land for a 25-year storm.**
2) **Construct a discharge orifice of a size that the quantity of run off through the orifice is equal to run off flow from a storm of 5-year or less, undeveloped land.**
3) **Construct a detention facility to have a post-development 25-year capacity with a discharge orifice (or structure) sized to limit outflow to no more than the undeveloped site peak run off for the existing (undeveloped) 5 year frequency storm. Detention volumes calculated by the following methods are acceptable:**
   **A. Santa Barbara Urban Hydrograph routing model (as prescribed by the King County Surface Water Design Manual) for the post development 25-year runoff hydrograph detained back to the existing 5 year peak site discharge.**
   **B. 18,883 CF/ 10 Acre drainage area as per City of Woodburn standard table, above, based on the rational method.**

**SAFETY REQUIREMENTS**

1) **Depth of storm water within 30 feet from the edge of detention ponds, if open to public, shall be limited to 3 feet, then gradual slope (3%) to higher depth shall be allowed. Maximum pond side slopes shall be 3' horizontal to 1' vertical, however, gentler slope is desirable.**

REV. A STRMVOLM-10/02/95 updated 08/30/96 Item #3 added 12/9/96 Safety Item revised.
REV. B. APPROVED BY CITY COUNCIL 12/9/96

design of all future detention facilities.

Portions of the existing drainageways currently function as detention sites where high water flow is backed up by road crossings such as East Lincoln Street and Hardcastle Street. These crossings were built with culverts intended to pass normal stream flows but do not pass high flows as easily. The hydraulic model simulates the high water level created during flood conditions and these levels are verified by the historical record of high water levels observed in both Mill Creek and Senecal Creek. These sites, four located in the Mill Creek drainage and one located in the Senecal Creek drainage basin will continue to function as detention areas and, with the exception of a proposed high level overflow structure at Hardcastle will not be modified to increase flows past them during storm conditions. However programs directed at improving public safeguards during periods of high flow and incorporation of storm water treatment wherever possible will be continued as a part of the Master Plan.

In addition, the City's detention policy should be made available to private developers and others who plan to alter drainage conditions or runoff volumes or rates. This policy addresses both large and small properties throughout the Mill Creek and Senecal Creek drainages. The goals of the policy should be publicized and discussed whenever the opportunity presents itself.

## Continued Planning

City staff should become familiar with and use computer-modeling techniques to assess the impacts of proposed development. With available tools such as HEC-1, the Santa Barbara Urban Hydrograph (SBUH), the Storm Water Management Model (SWMM) and similar, well-documented programs, forecasts of future runoff should be developed and reviewed periodically to document the changes in flow rates and volume of storm water. Continued use of a system model will also allow City staff to update the facilities plan portion of the Comprehensive Plan as dictated by state regulation.

The City should also plan to reanalyze the entire system and prepare new forecasts of storm water conditions every 20 years, a period typically used in major facility planning. This major effort provides opportunity to re-evaluate the entire system and incorporate changes in land use and other stormwater runoff conditions and parameters.

### Operations and Maintenance

The City has and follows a plan for ongoing operations and maintenance of the storm drainage system. This system should be continued and expanded to include regular inspection of drainage facilities including major culverts, bridges, detention areas (public and private) and major open stream segments. Periodic cleaning of debris in the stream and adjoining floodway areas should be done to prevent the buildup of flow-inhibiting materials.

Storm sewer maintenance activities which include regular inspections of inlets, catch basins, major storm sewers and outfalls should also be scheduled and findings documented.

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Storm sewer maintenance activities which include regular inspections of inlets, catch basins, major storm sewers and outfalls should also be scheduled and findings documented.

# Chapter 8

# PROJECT PHASING

Projects required to maintain the storm drainage system capacity by construction of new conveyance facilities or expansion of existing facilities must be prioritized to level the demands on the City's funds and staff. All projects will eventually be needed to maintain system capacity as the service area grows and is developed with new residential and commercial areas. This Chapter of the Study sets the methods and criteria for determining which individual projects should be funded and constructed first and which may be deferred until a later time.

## PROJECT PHASING

A detailed analysis of the runoff conditions as found in Chapter 5, Analysis of Rainfall Runoff, was used to identify problem areas in the existing system where peak hydrologic conditions reached or exceeded the facilities' ability to safely handle these flows. This data and model projections were categorized using the base assumptions and criteria outlined in Chapter 3, Study Methodology. Accordingly, budget level cost estimates for the construction of each capital project were developed and can be found in Chapter 9, Recommended Capital Improvement Projects. These cost estimates include the construction costs and other project-related costs such as design and inspection services, legal support and, where necessary, costs of land needed to implement the improvement.

Based on the severity of the problem and the cost of the improvement as determined by applying specific criteria, projects are divided into specific groups and placed into phases that will allow prudent expenditure of public funds. Each proposed project has been given a designation of high, medium or low priority. The phasing periods and the specific criteria, used to separate capital projects into a priority grouping follow.

## PHASING PERIODS

The City typically operates on a 5-year capital improvements plan (CIP) which directs planning and construction of capital public works projects. Usually, projects placed in this plan have been identified through engineering study by the City staff and citizen input. In this case, some projects were deemed suitable to be placed on the CIP due to their impact on public safety and health and are suggested herein for "high" priority for implementation within one to three years. Other projects, necessary for efficient operation of the drainage system but not threatening in terms of public health and safety are given "medium" priority and targeted for inclusion on the CIP within the succeeding 3- to 6- year portion of the plan.

Finally, some projects are driven by the rate of land development within the service area and are given "low" priority for project planning. Projects in this category should be revisited each year in preparation of the current CIP and their necessity weighed in light of local development rates upstream of the potential project. In this Study, the long-term phase is defined as the period extending beyond the CIP 5-year window.

In the case of potential development outside the jurisdiction of the City, some projects may require a change of priority to alleviate increases in runoff due to these development projects.

The phasing of drainage improvement projects was divided into the following categories which are shown with their respective time frames:

| Phase Designation | Time Frame |
|---|---|
| 3 Year | FY 2002 - FY 2005 |
| 6 Year | FY 2005 - FY 2008 |
| 10 Year | Beyond FY 2008 |

It is recognized in this Study that any of the projects identified may change priority as storm water runoff conditions upstream change. Usually, no single factor will dictate the priority of a project, however, the following discussion provides a guide for project prioritization and is intended to allow flexibility within the financial and technical ability of the City to implement them.

## CRITERIA FOR PROJECT PHASING

The improvements identified are needed to provide adequate capacity for fully developed lands within the two major watersheds. In other words, existing drainage conveyances were evaluated to determine which would need to be modified or replaced to handle increasing peak flows as development occurs. Estimates of future facility improvements were made to allow for continued proper function of the drainage system.

Additionally, more local drainage facilities will be needed within developments on large parcels. These local facilities, usually storm water inlets and piping systems are typically provided by the developer of the parcel as part of the development's infrastructure and are either kept as private drainage facilities or are constructed to City standards and turned over to the City upon acceptance by the City. It is not the intent of this Study to place restrictions on the alignment of drainage facilities within these currently undeveloped lands, except as specifically provided for in this Study.

Chapter 5, Analysis of Rainfall Runoff, provides estimates of gross subbasin runoff information and outlines the methodologies which may be used to determine design flows for these on-site improvements. Drainage facilities should be planned for these development parcels as part of the City's normal site design review process. These facilities should have capacity to handle the flows estimated by this Study and should provide for continuity of existing drainage ways.

As a first step, drainage improvement projects were identified which were needed to correct existing problems as identified by the model analysis or problems known by City staff and citizens. These improvements were checked against peak runoff flows that will be created by future development to make certain that a project would handle both current and future runoff conditions. These improvements were classified as 3-YEAR phase improvements. Within this category, the priority was rated as "High" if there was a relatively large risk associated with not completing the improvement or if the problem created by the existing situation was frequent and caused a public safety concern or significant inconvenience.

These projects were then arranged within the phasing categories by considering the following criteria:

A. Extent of a current system or facility inadequacy as judged by the relative inadequacy when compared to the peak runoff requirements for an adequate structure or drainageway feature.

B. Estimated time frame for further development within the catchment area, assuming development which will exacerbate the condition or make it untenable.

C. Relative risk from failure to make timely improvements as judged by the potential for structural failure of the facility or creation of a public hazard.

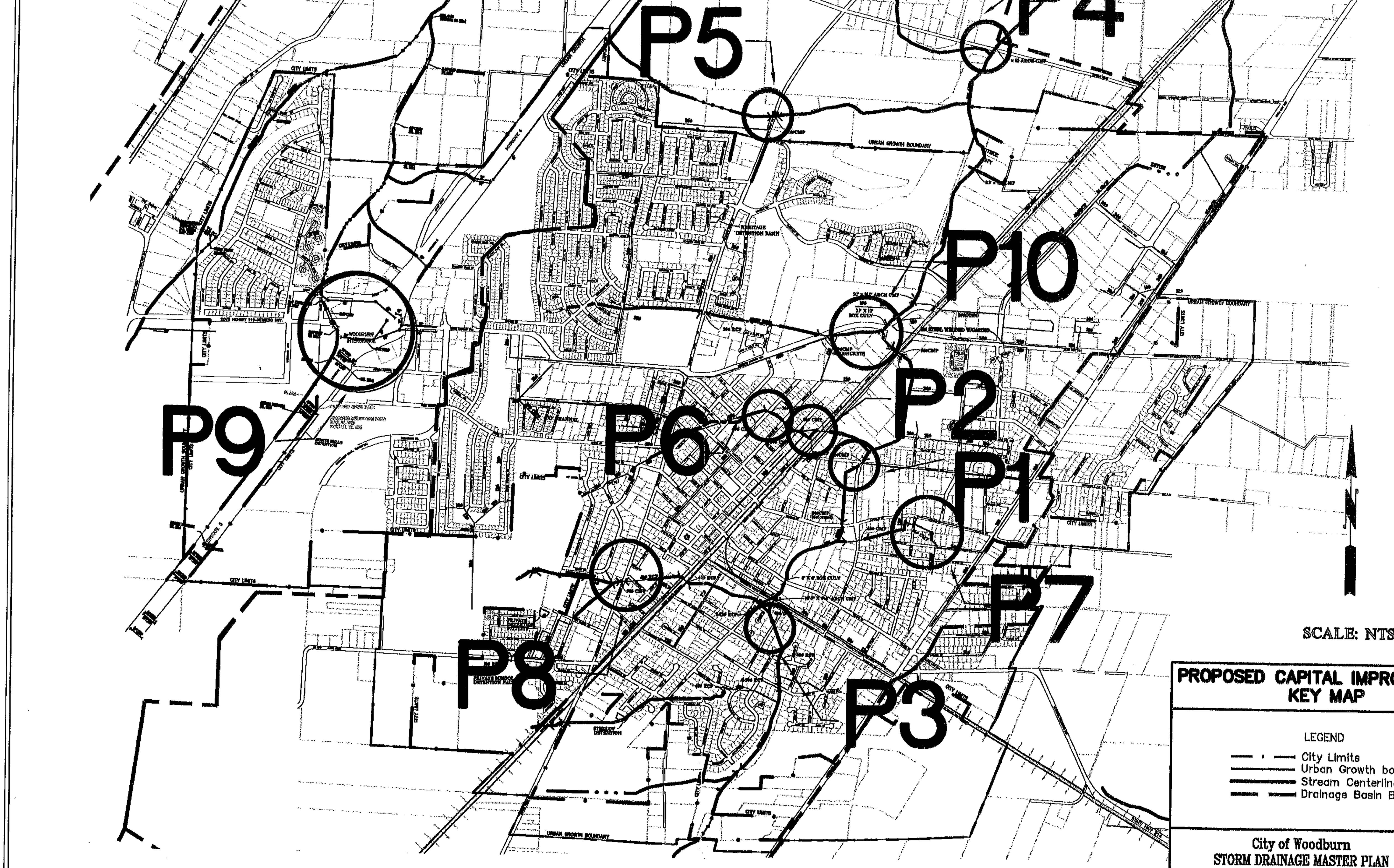
Chapter 9

# RECOMMENDED CAPITAL IMPROVEMENT PROJECTS

Capacities of existing and drainage system facilities were analyzed and the analysis presented in Chapter 5, Analysis of Rainfall Runoff. Using the criteria and methods outlined earlier in the Study, this analysis shows system deficiencies in several geographic areas. Based also on the discussion of Chapter 7, Recommendations for Drainage Improvements, the following projects are recommended for adoption by the City and should be included in the City's future Capital Improvement Program (CIP). Figure 15, Proposed Capital Improvements Key Map, shows the location of all proposed CIP projects and Table 9-1, CIP Project Summary, provides a brief summary of these proposed projects.

These projects have been defined to the extent of the information available at this time with the best information provided pertaining to their cost, phasing and implementation. Prior to actual initiation of any individual project, a detailed preliminary engineering study should be prepared and a more precise cost estimate prepared. In addition, preliminary and final designs of these projects to determine dimensions, location and facility sizes will be required. Also, detailed review of property ownership and property line location should be done during preliminary design of an individual improvement to assure new or rebuilt facilities are located on public lands or have the appropriate easements.

It should be noted here that proposed improvements are based on information available at the time of completing the modeling for this Study. The existing site information has been updated to represent the facilities as of July 2001. **Re-evaluation of these proposed improvements is recommended at the time of preliminary design.** System re-routing, upgrades and future development will affect these recommendations. Any alterations of the existing system layout or unforeseen developments should be carefully evaluated for downstream adequacy and potential conflict with the objectives of this Study.


P5
P4
P10
P2
P6
P1
P9
P7
P8
P3
URBAN GROWTH BOUNDARY
CITY LIMITS
HERITAGE DETENTION BASIN
WOODBURN INTERCHANGE
SCALE: NTS
PROPOSED CAPITAL IMPRO[V]
KEY MAP
LEGEND
City Limits
Urban Growth boun[dary]
Stream Centerline
Drainage Basin Bou[ndary]
City of Woodburn
STORM DRAINAGE MASTER PLAN

# TABLE 9-1

## CIP PROJECT SUMMARY

## Woodburn Drainage Master Plan

| Project ID | Project Name | Drainage Basin | Subbasin ID | Priority | Estimated Cost ($) |
|---|---|---|---|---|---|
| **P1** | Hardcastle Crossing | Mill Ck | M-8 | High | $ 191,729 |
| **P2** | Front Street Detention & Crossing | Mill Ck | M-7 | High | $ 151,436 |
| **P3** | Marshall Street | Mill Ck | M-10 | High | $ 78,560 |
| **P4** | Crosby Road Crossing | Mill Ck | M-1 | N/A (county) | $ 587,159 |
| **P5** | Boones Ferry Crossing | Mill Ck | M-1a | Low | $ 53,157 |
| **P6** | Old town - 2nd Street | Mill Ck | M-7 | Medium | $ 188,965 |
| **P7** | East McKinley | Mill Ck | M-9a | High | $ 953,101 |
| **P8** | Stubb Rd Detention | Mill Ck | M-11a | Medium | $ 359,571 |
| **P9** | Connect 48" at I-5 & Hwy 214 | Senecal Ck | ES-2 | High | N/A |
| **P10** | Goose Creek Re-alignment | Mill Ck | M-5 | Low | $ 224,577 |

**CIP Total= $ 2,788,255**

The following CIP projects are recommended for implementation:

Five proposed projects within the Study area have been given high priority for improvement. These are the Hardcastle Road crossing; development of a detention facility at the Front Street park and addition of a 42-inch line across Front Street and the railroad; adding capacity at Marshall Street; increasing capacity at East McKinley near Bryan Street; and consolidation of storm flows into the existing 48-inch line crossing I-5 immediately north of Hwy 214.

On Hardcastle Road, addition of an auxiliary (overflow) line in the embankment of the fill crossing Mill Creek is recommended. This should be designed as a box culvert to minimize head loss while minimizing use of the embankment. Details of this recommendation are found the outline for Project No. 1, Hardcastle Crossing.

On Front Street, flow from an open ditch in the park reenters an 18" diameter pipe before it goes under Front Street. Flows beyond the capacity of the 18" pipe are diverted to an open ditch and routed northerly to an existing 30" diameter pipe which crosses under Front Street and the Railroad. The new system would create a detention facility at the park and increase capacity of the line under Front Street and the railroad by constructing a 42-inch line in place of the existing 30" pipe. Control structures would also be constructed at the detention facility.

At the Marshall Street crossing of Mill Creek, addition of a second conduit to increase capacity of the crossing and avoid flows that overtop the street in all but the most extreme storm events is recommended for immediate development. At this location, a second, parallel pipe of 54-inch diameter should be installed to relieve flooding conditions immediately upstream. Details of this project are itemized in the write up titled Project No. 3, Marshall Street.

In the area of Blaine and East McKinley Streets, the existing storm system has inadequate capacity to carry existing high flows and the system does not meet present City standards for alignment. Complaints of flooding have been received from local residents. In order to alleviate these problems the sub-standard pipes will be abandoned and new larger diameter pipes will be constructed within the alignment of the public right-of-way.

Analysis of the flood elevation for various storm events show a significant head loss being incurred at the Crosby Road Crossing. This facility is owned by Marion County and appears to cause significant headloss in the higher flow ranges. Improvement to this facility is recommended, and the City should initiate discussions with county staff to this end. Elimination of the head losses caused at Crosby Road will lower the immediate upstream flood elevation significantly and, with that, lower flood elevations for the 100-year event as far south on Mill Creek as Highway 214.

An unused 48-inch storm sewer was constructed as part of the ODOT I-5 construction. This system can be utilized to relieve hydraulic loading to the storm system crossing under I-5 to the south of Hwy 214. This project requires further investigation to determine the best potential re-routing plan.

In addition to the five CIP projects listed above, two other locations along the main stem of Mill Creek appear to be overtopped during periods of very high flow. These are the Goose Creek confluence at Highway 214 near the Mill Creek Pump Station and the private road crossing just south of Crosby Road.

At Mill Creek at the confluence of Goose Creek just south of Highway 214 at the Mill Creek Pump Station there is significant probability of backwater build up during the 25-yr. event and overtopping appears to be possible during the 100-year storm event. To resolve this potential problem it is recommended that the Goose Creek Tributary which presently enters Mill Creek at the Pump Station south of Hwy 214 be re-aligned to cross Hwy 214 and intersect Mill Creek to the north of Hwy 214. This would include the installation of a 60" diameter culvert and is designated as Project No. 10, Goose Creek Re-alignment. This project is assigned as a lower priority, however should state or federal funds become available for a project of this nature, it could be moved higher on the list.

The private drive south of Crosby Road is within the City limits but is not a publicly-owned facility nor is it located within a public right-of-way. This site is not recommended for inclusion on the CIP list for these reasons but is mentioned in this Study because the capacity of the existing culvert (7.8'x 8.3' CMP) is inadequate to pass the existing 25-yr event. During the floods of 1996 the water surface elevation reached approximately 152.0', three feet above the existing driveway elevation of 149.0'. The type, configuration and slope of the culvert limits the capacity to less than 250 cfs. The full build-out 100-yr event flow at this location is estimated at 500 cfs. The culvert should be replaced with a more hydraulically efficient 90" or 96" pipe with increased slope on the culvert as the topography will allow. Depending on the policy of the City and discussions with the property owner, capital improvement funds should only be used to increase capacity of the crossing if such expenditure falls within the goals of the City's storm management policy and can be shown to benefit City property owners or City-owned property.

## COST DATA

The following figures and tables present the capital improvement projects recommended for implementation. Most construction costs are based on information from recent project bid tabulations and unit prices for this area as documented in a technical memorandum from *City of Portland, Environmental Services Public Facilities Plan Technical Memo, T7.C.1. Basis of Cost Estimates,* dated June 1998. These prices have been adjusted (increased by 7.6 percent) to reflect construction costs in 2001 and are now based on the July 2001 ENR CCI of **6404.** All costs should be re-evaluated at the time of preliminary and final design to refine individual items estimates. The costs provided on the following project outlines are total project costs and include design and administrative costs as well as a contingency allowance. A range of contingencies was used to allow for varied construction scenarios of each project site.

# Project No. 1

Drainage: Mill Creek

Subbasin: M-8

Project Name: Hardcastle Crossing

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**PRIORITY:** High

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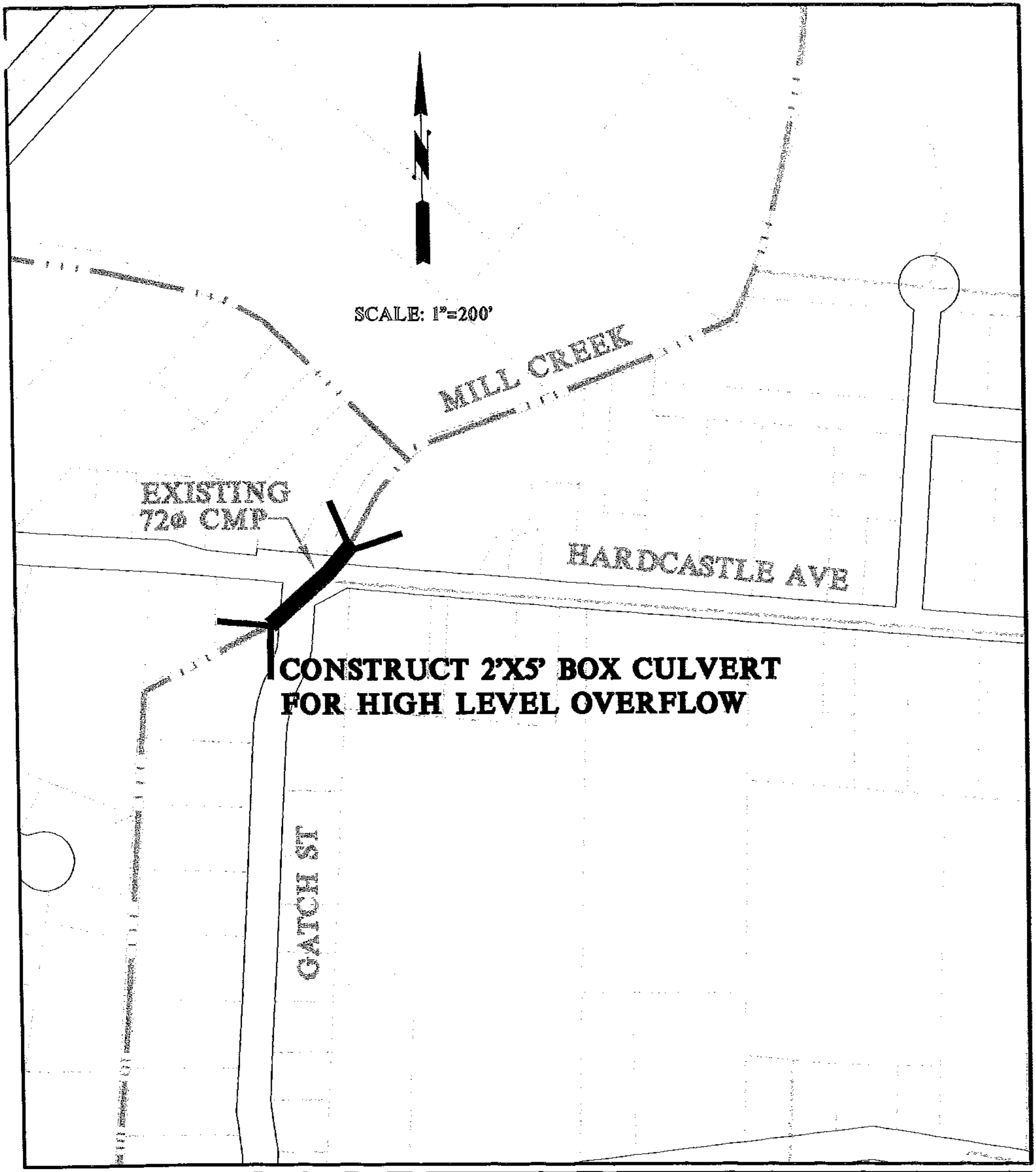
**Description:** The embankment of the Hardcastle crossing of Mill Creek restricts flows in Mill Creek during the forecast 100-year flood event. This restriction causes water to back up as far as Marshall Street and will, during a 100-year storm event, cause the water surface to come within 6 inches of overtopping the roadway. See Figure P1, Hardcastle Crossing, for the existing site layout. The existing culvert capacity is 255 CFS and future 100-year flood flows are estimated at 285 CFS.

This project proposes to create an overflow channel with an invert elevation of 160.5' which will allow continued storage to occur upstream of the embankment and allow any flows reaching the 160.5' elevation to pass. At flood flows, this high-level overflow will maintain a water surface elevation of 161.5'. Construction of this overflow will provide relief of the high backwater condition at Marshall Street and the intervening crossings and make maximum use of the detention storage in the park areas along Mill Creek.

It is most likely that use of a 2 foot high box culvert, 5 feet wide set at a slope of 0.005 (ie. in 160.5' - ie. out 160.3) will allow maximum passage of storm flows while minimizing headloss through the structure.

**Cost Estimate:** Hardcastle Crossing

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P1 | 2x5' box Culvert | lf | 190 | 409 | $ 77,709 |
| Hardcastle Crossing | Headwalls | ea | 2 | 2,153 | 4,305 |
| | Concrete Encasement | cy | 50 | 135 | 6,727 |
| | Slope Protection | cy | 20 | 108 | 2,153 |
| | Site Demo/Flow Diversion | 25% | 1 | 22723 | 22,723 |
| | Subtotal | | | | 113,617 |
| | Contingency | 35% | 1 | 39766 | 39,766 |
| | Subtotal | | | | 153,383 |
| | Eng., Legal, Admin | 25% | 1 | 38346 | 38,346 |
| | | | | **TOTAL=** | **$ 191,729** |




| DATE | | | FIGURE |
|---|---|---|---|
| JULY 2001 | CRANE & MERSETH<br>*Engineering/Surveying*<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | **PROPOSED CAPITAL IMPROVEMENT<br>HARDCASTLE CROSSING**<br>City of Woodburn<br>STORM DRAINAGE MASTER PLAN | **P1** |

# Project No. 2

Drainage: Mill Creek

Subbasin: M-7

Project Name: Front Street Detention and Crossing

---

**PRIORITY:** High

---

**Description:** West and south Front Street, drainage flows from subbasin M-7 join to enter an open ditch system in Front Street Park and eventually flow into pipe systems under Front Street and the railroad. The southerly pipe is an 18" diameter concrete pipe and low flows exit the Park via this pipe. When the hydraulic capacity of the 18" pipe is reached, overflows enter a second open ditch running north and into an existing 30" diameter concrete pipe which runs under Front Street and the Railroad. The Park can be converted into a detention facility if flows out of the Park are controlled. Also, the replacement of the 30" pipe under Front Street and the railroad with a 42" pipe will allow for high flow by-pass as needed for future development.

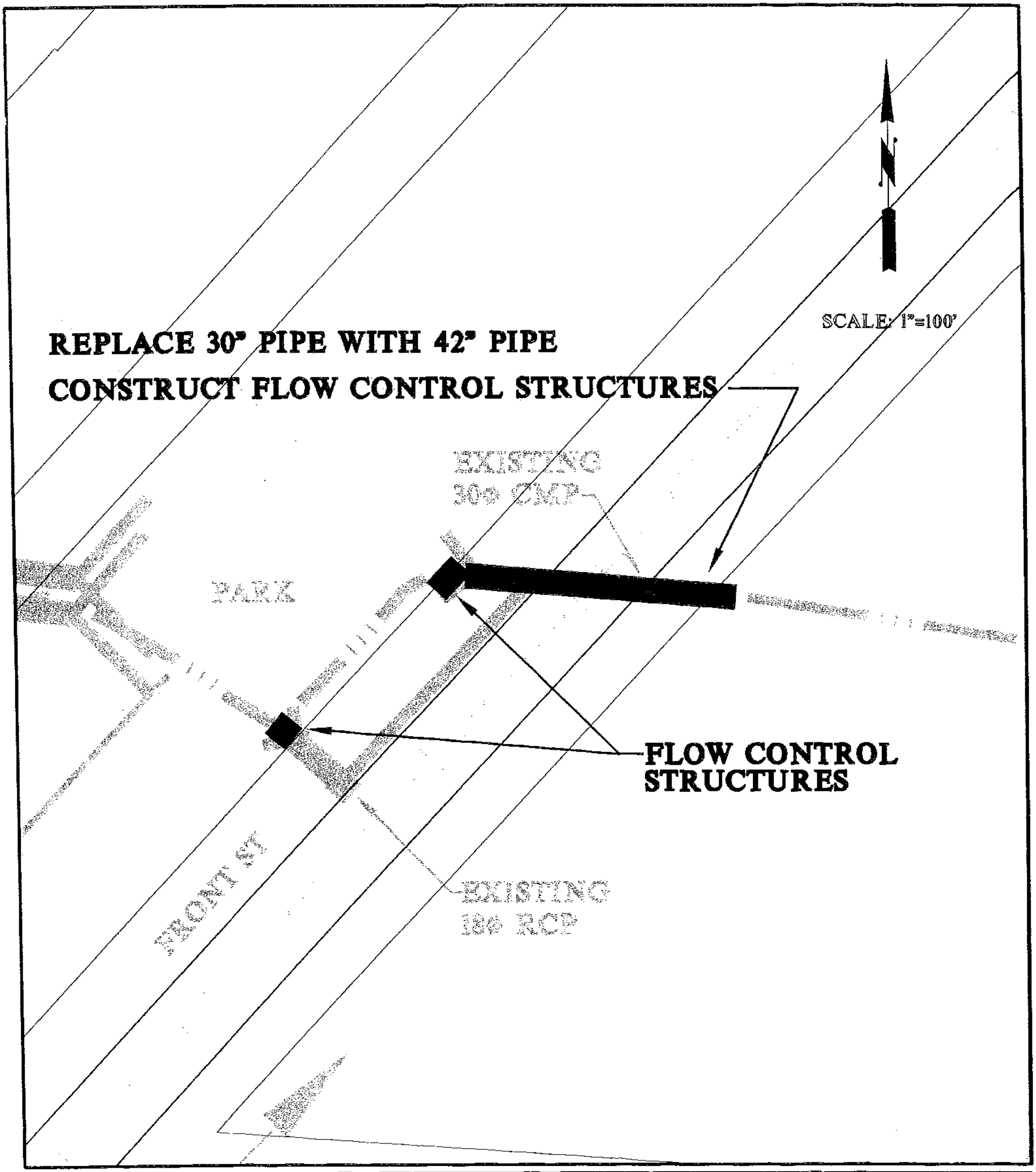
Modeling indicates that buildout conditions will increase 100-year flood flows from 65 CFS currently to 77 CFS in the future, assuming that subbasin M-11c2 will be routed to the south.

Model analysis indicates that maximum outflow capacity is 37 CFS when storage and ponding in the park area reaches 168.0', approaching the pavement on Front Street. Currently, a 25-year flood event will cause a backup of water about 1 foot deep in the park and a 100-year flood event will cause a backup of about 3 feet deep. With surrounding properties as currently developed, no damage to property or homes should occur. Future flooding conditions after complete infill in the upstream basins will create a water depth of 2 feet in the park during a 25-year event and water will overtop Front Street during a 100-year event.

To mitigate this potential for future flooding and inundation of Front Street, it is recommended that the park area be excavated to provide an additional 0.7 acre-feet of storage below elevation 167.0' and that the existing 30" CMP pipe crossing Front Street and the railroad be replaced with a 42-inch diameter reinforced concrete storm sewer line. See Figure P2, Front Street Detention & Crossing, for the existing site layout.

**Cost Estimate:** Front Street Detention & Crossing

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P2 | Excavation | cy | 1150 | 11 | $ 12,377 |
| Leaping Weir at Front St. | Seeding and Restoration | ac | 0.7 | 2153 | 1,507 |
| | 42-inch CSP w/ boring | lf | 200 | 300 | 60,058 |
| | Headwalls | ea | 2 | 2153 | 4,305 |
| | Flow control structure | ea | 2 | 5382 | 10,764 |
| | Flow control | 5% | 1 | 4181 | 4,181 |
| | Subtotal | | | | 93,192 |
| | Contingency | 30% | 1 | 26343 | 27,957 |
| | Subtotal | | | | 121,149 |
| | Eng., Legal, Admin. | 25% | 1 | 28538 | 30,287 |
| | | | | Total= | $ 151,436 |




| DATE | | | FIGURE |
|---|---|---|---|
| JULY 2001 | CRANE & MERSETH<br>Engineering/Surveying<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | PROPOSED CAPITAL IMPROVEMENT<br>FRONT ST DETENTION & CROSSING<br>City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P2 |

# Project No. 3

Drainage: Mill Creek

Subbasin: M-10

Project Name: Marshall Street

---

**PRIORITY:** High

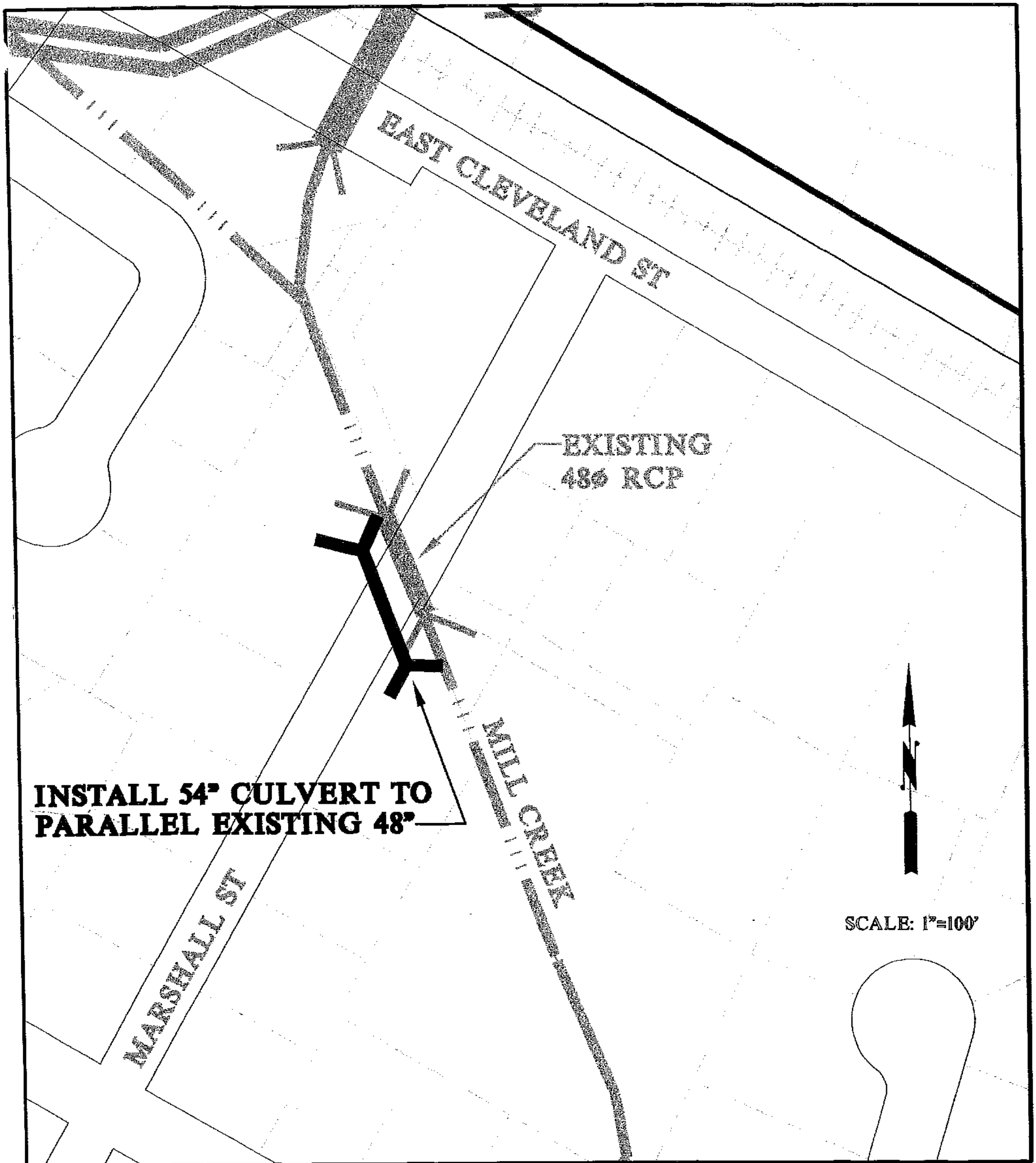
---

**Description:** The existing crossing of Marshall Street at Mill Creek is conveyed through a single 48-inch diameter line. The capacity of this line is inadequate to convey the future 5-year event and currently overtops during the existing 25-year storm event as was evidenced during the February floods in 1996. In addition, the head loss through this line during flood conditions is almost 3 feet. A future 100-year event will overtop the street by about 2.5 feet, likely causing severe damage to the pavement and embankment.

This condition will be mitigated through installation of a 54-inch diameter line paralleling the existing line. Realignment will not be necessary, however, some additional land may be required to install a second line of this diameter. See Figure P3, Marshall Street, for the existing site layout.

**Cost Estimate**: Marshall Street

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P3 | 54-inch pipe | lf | 80 | 443 | $ 35,475 |
| Marshall Street | Headwalls | ea | 2 | 4305 | 8,610 |
| | Site Demo/Flow Diversion | 10% | 1 | 4409 | 4,409 |
| | Subtotal | | | | 48,494 |
| | Contingency | 35% | 1 | 16973 | 16,973 |
| | Subtotal | | | | 65,467 |
| | Eng., Legal, Admin. | 20% | 1 | 13093 | 13,093 |
| | | | | **Total =** | **$ 78,560** |




| DATE | | | FIGURE |
|---|---|---|---|
| JULY 2001 | CRANE & MERSETH<br>*Engineering/Surveying*<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | **PROPOSED CAPITAL IMPROVEMENT<br>MARSHALL STREET**<br>City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P3 |

# Project No. 4

Drainage: Mill Creek

Subbasin: M-1

Project Name: Crosby Road Crossing (In County)

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**PRIORITY:** Subject to be discussed with Marion County

---

**Description:** A major Mill Creek crossing at Crosby Road north of the City is conveyed through a 7' x 10' arch CMP culvert. This facility has an existing capacity of 340 CFS and, during periods of unusually high water conditions, restricts the Mill Creek flows and creates a significant headloss. Model analysis indicates a 100-year flood flow of 624 CFS.

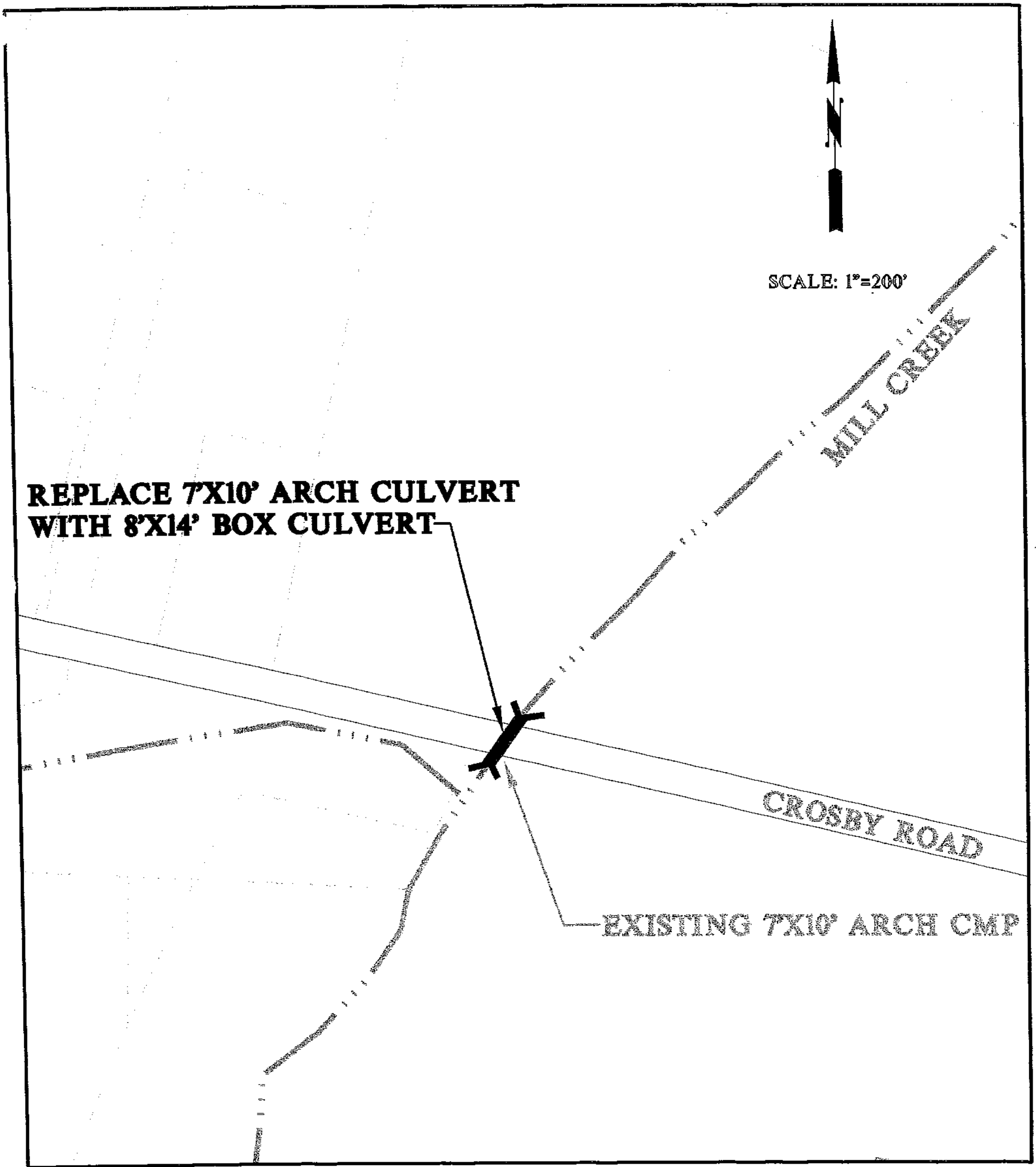
While flood profiles indicate that Crosby Road will not be overtopped by the 100-year event, the headloss and resulting backwater created by this flow raises the flood elevation in Mill Creek as far south as Highway 214 and, increases the potential for damage to structures along the stream.

In order that the flood elevation be lowered to an acceptable level, it is recommended that the existing arch culvert be replaced with 8' by 14' box culvert or structure of equivalent capacity. See Figure P4, Crosby Road Crossing, for the existing site layout. Installation of a new culvert will have the effect of lowering the flood elevation upstream of the Crosby Road structure by approximately 2 feet to a new water level of about 147.0'.

Since the Crosby Road structure is located on a roadway owned and maintained by Marion County, it is recommended that City staff notify the Marion County of this situation and together begin discussion of a mutually acceptable solution.

**Cost Estimate:** Crosby Road Crossing

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P4 | 8x14 box culvert | lf | 100 | 2368 | $ 236,787 |
| Crosby Road Crossing | Headwalls | ea | 2 | 8500 | 17,000 |
| | Site demo/Flow Diversion | 25% | 1 | 63447 | 63,447 |
| | Stream Mitigation | ls | 1 | 5382 | 5,382 |
| | Subtotal | | | | 322,615 |
| | Contingency | 40% | 1 | 129046 | 129,046 |
| | Subtotal | | | | 451,661 |
| | Eng., Legal, Admin. | 30% | 1 | 135498 | 135,498 |
| | | | | Total = | $ 587,159 |




| DATE | CRANE & MERSETH Engineering/Surveying | PROPOSED CAPITAL IMPROVEMENT CROSBY RD CROSSING | FIGURE |
|---|---|---|---|
| JULY 2001 | 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P4 |

# Project No. 5

Drainage: Mill Creek

Subbasin: M-1a

Project Name: Boones Ferry Crossing

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**PRIORITY:** Low

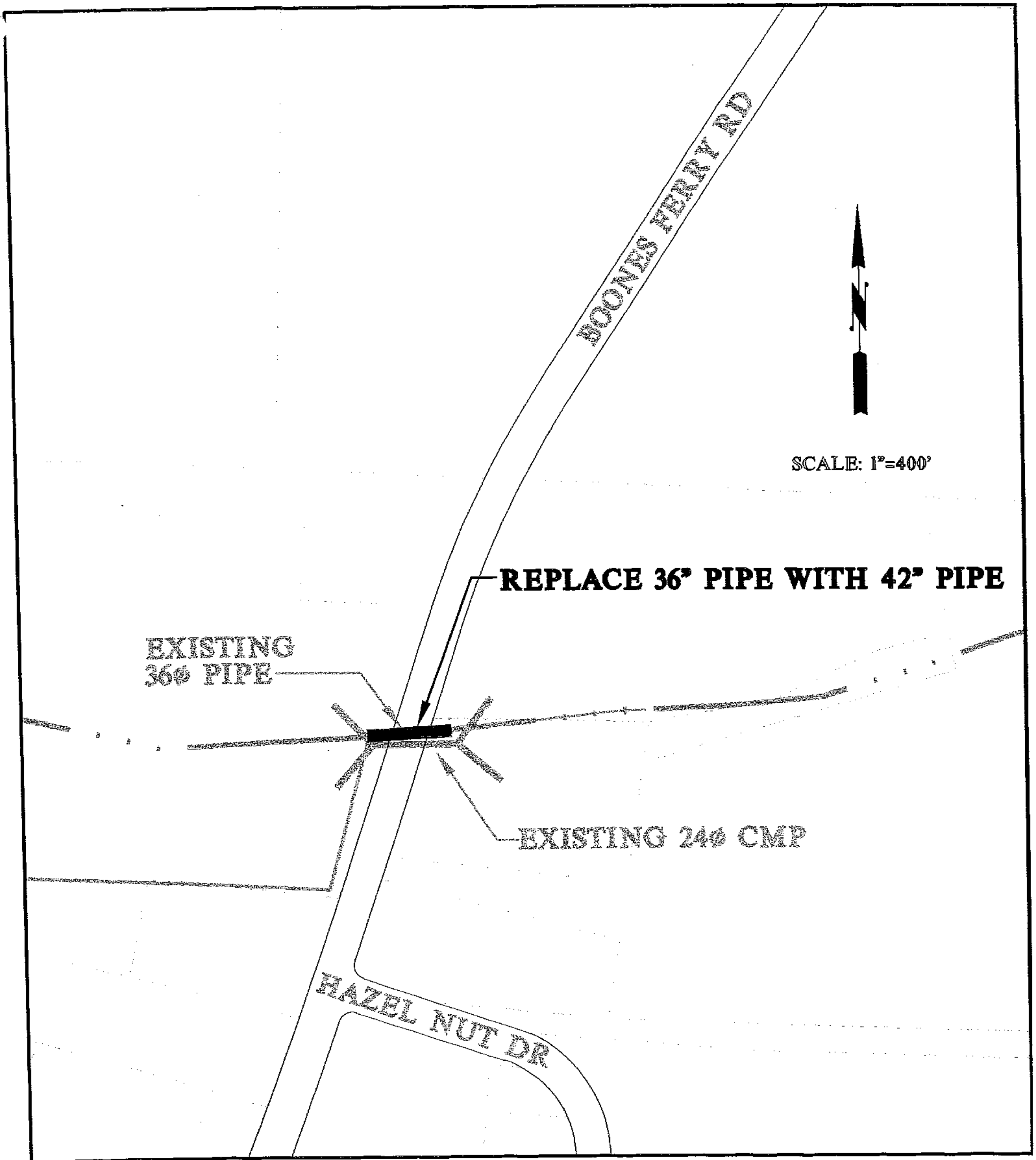
---

**Description:** This crossing of Boones Ferry Road immediately north of the current City limits carries runoff from the developed and undeveloped areas west of Boones Ferry Road including flows from subbasin M-1c. In time it will also receive runoff flows from subbasin M-1b. Subbasin M-1b is zoned for single family dwellings and will increase the runoff from this area after development. Projections show the existing 24-inch culvert and the 36" pipe crossing Boones Ferry limited at a 2-year storm flow under future (buildout) conditions. Future flows will require a 42-inch diameter line to replace the existing 36" pipe. See Figure P5, Boone's Ferry Crossing, for the existing site layout. This project proposes to only replace the section of pipe under Boones Ferry Rd. The required extent of replacement of the 36" needs to be investigated further. Further field investigations should be conducted to determine if all or portions of the 36" pipe will need to be replaced.

Since the existing crossing is outside the City limit, financial responsibility may not be the City's. Options include involving Marion County in the decision to enlarge the crossing capacity and/or strictly enforce detention policies in both subbasin M-1b and the undeveloped areas north of the swale leading to the crossing. If future flows can be limited to existing conditions, the crossing may handle a 10-year event and be suitable for some time.

**Cost Estimate:** Boones Ferry Crossing

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P5 | 42-inch pipe | lf | 85 | 300 | $ 25,525 |
| Boones Ferry Crossing | Headwalls | ea | 2 | 2153 | 4,305 |
| | Site demo/Flow Diversion | 10% | 1 | 2983 | 2,983 |
| | Subtotal | | | | 32,813 |
| | Contingency | 35% | 1 | 114844 | 11,484 |
| | Subtotal | | | | 44,297 |
| | Eng., Legal, Admin. | 20% | 1 | 88594 | 8,859 |
| | | | | **Total =** | **$ 53,157** |




| DATE | CRANE & MERSETH Engineering/Surveying | PROPOSED CAPITAL IMPROVEMENT BOONES FERRY CROSSING | FIGURE |
|---|---|---|---|
| JULY 2001 | 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | City of Woodburn STORM DRAINAGE MASTER PLAN | P5 |

# Project No. 6

Drainage: Mill Creek

Subbasin: M-7

Project Name: Old Town (1st and 2nd)

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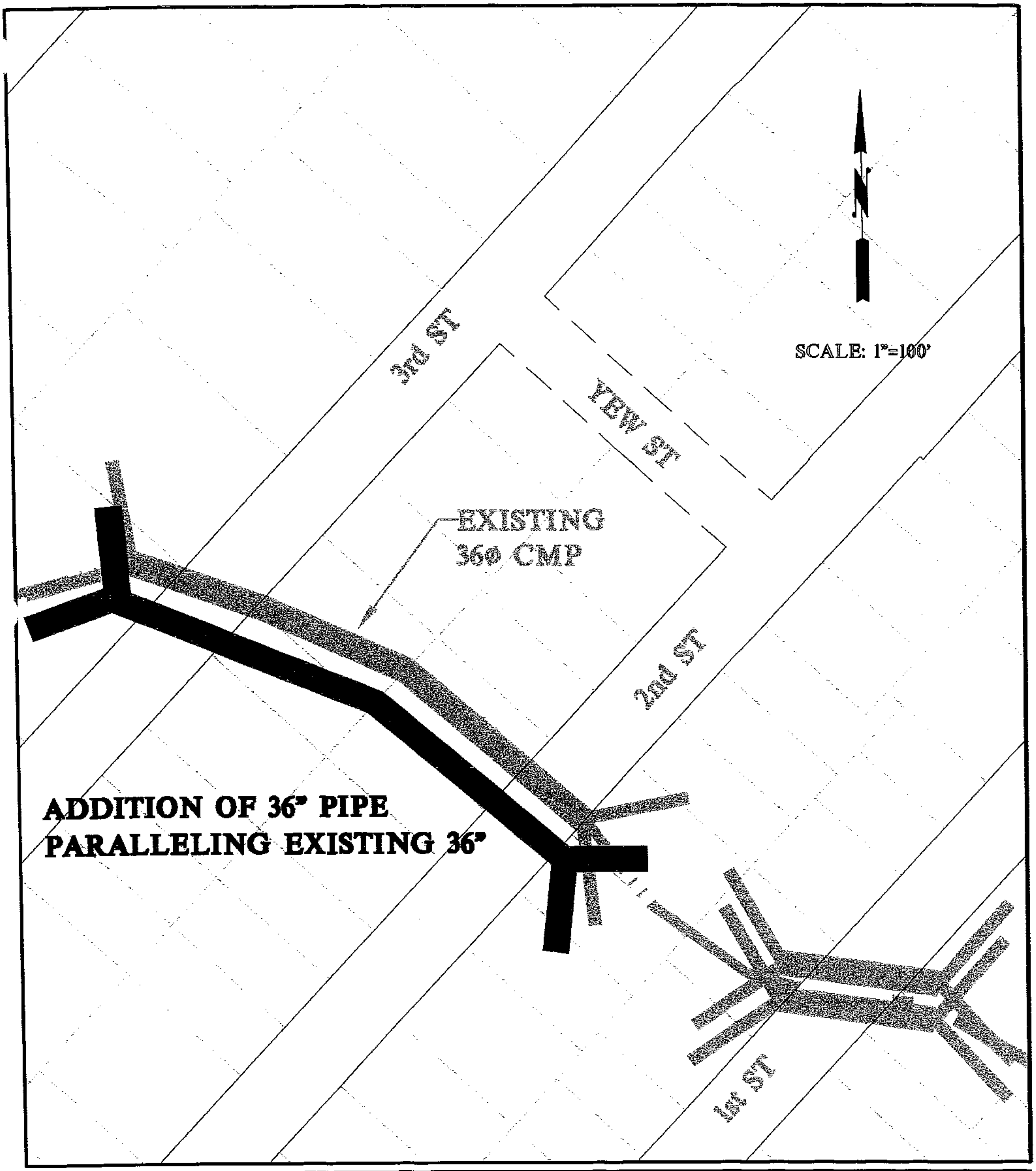
**PRIORITY:** Medium

---

**Description:** One crossing on the Old Town tributary of Mill Creek west of Front Avenue is undersized for future capacity needs. This crossing is a culvert that extends from the west side of Third Street and outlets on the east side of Second Street. The crossing is a single 36-inch diameter (2nd Street) and will be limited to a 2-year storm event at buildout conditions. Primary future flows for this tributary system will come from subbasin M-7b2, a large residentially-zoned area south of Highway 214. Replacement conduits for these lines can be either replacement lines or parallel lines depending on the existing conditions of the two crossings and the anticipated future life. See Figure P6, Old Town- 2nd St., for the existing site layout.

**Cost Estimates:** Old town – 2nd St

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P6 | 36-inch culvert | lf | 310 | 265 | $82,079 |
| Old Town-2nd St | Headwalls | ea | 2 | 2,153 | 4,305 |
| | Site Demo/Site Restoration | 25% | 1 | 21,596 | 21,596 |
| | Subtotal | | | | 107,980 |
| | Contingency | 40% | 1 | 43,192 | 43,192 |
| | Subtotal | | | | 151,172 |
| | Eng., Legal, Admin. | 25% | 1 | 37,793 | 37,793 |
| | | | | **Total=** | **$ 188,965** |




| DATE | | PROPOSED CAPITAL IMPROVEMENT OLD TOWN – 2ND ST | FIGURE |
|---|---|---|---|
| JULY 2001 | CRANE & MERSETH<br>*Engineering/Surveying*<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P6 |

# Project No. 7

Drainage: Mill Creek

Subbasin: M-9a

Project Name: East McKinley

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**PRIORITY:** High

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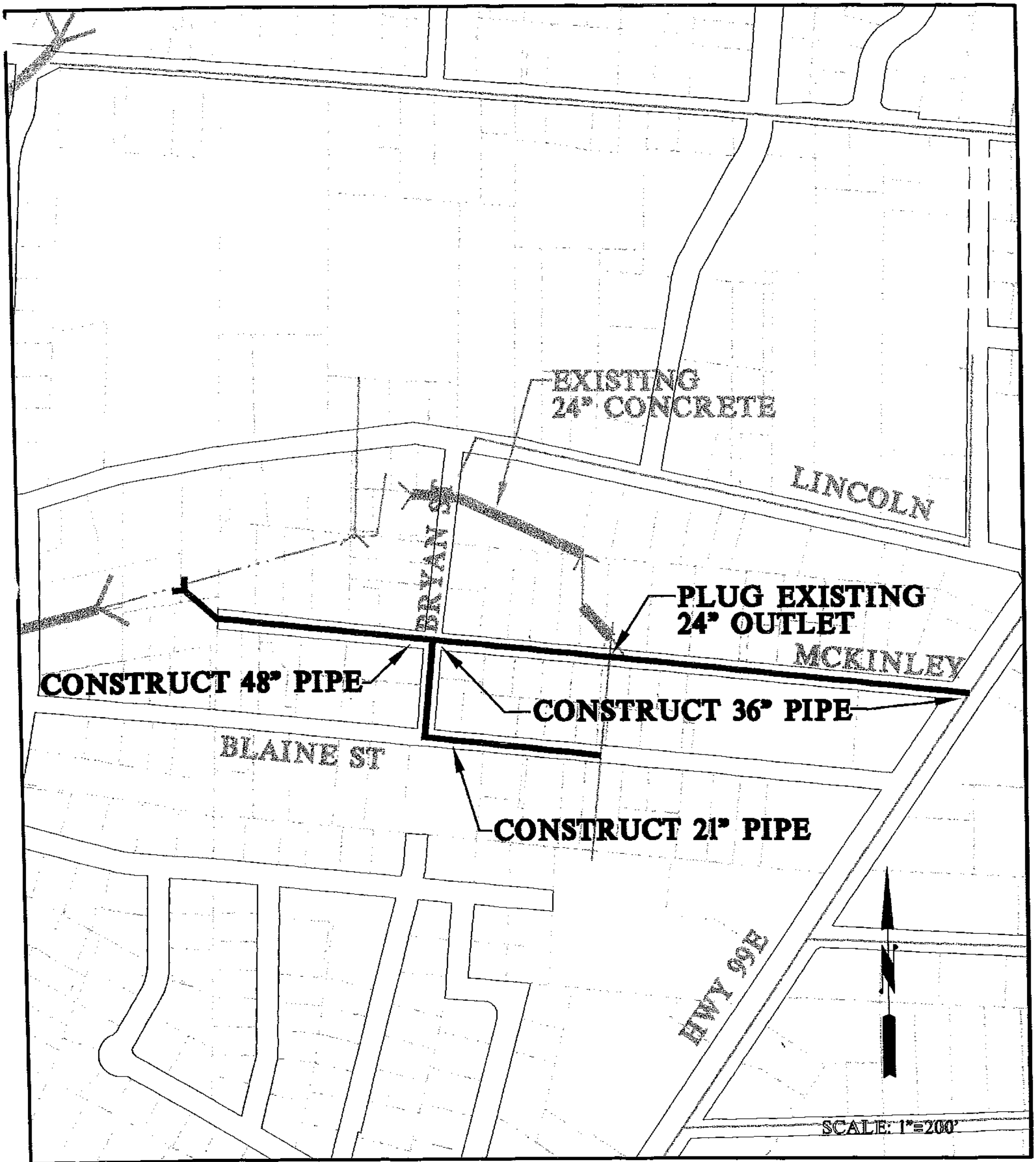
**Description:** Flows entering the 24-inch diameter line on East McKinley from the Highway 99E area cause significant headloss due to the limited capacity of the existing corrugated metal pipe. A combination of high flows from the commercial areas along the highway and use of a corrugated pipe along McKinley cause this condition to exist. The existing 24" runs along East McKinley and flows to the north through a series of private properties. The exact location of the pipes within the private properties is unknown and the alignment is non-standard. This is also the case with an existing 18" pipe located on private properties between East McKinley and Blaine Streets. Additional capacity can be provided by up-sizing the existing 24" and 18" pipes. Also, the pipes located on private property should be abandoned and newly constructed pipes within public right-of-way can service these areas.

Presently, it is proposed to replace the existing 24" pipe in East McKinley with a 36" pipe and replace the existing 18" pipe with a 21" pipe. Where the two systems converge at the intersection of McKinley and Bryan Streets a 48" diameter pipe would be constructed. The newly constructed storm sewer would follow the alignment of East McKinley and outfall to the open drainage located south of Lincoln Street and to the west of McKinley Street.

**Cost Estimate:** East McKinley

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P7 | 36-inch pipe | lf | 1250 | 265 | $ 330,963 |
| East McKinley | 48-inch pipe | lf | 525 | 357 | 187,600 |
| | 21-inch pipe | lf | 610 | 182 | 110,956 |
| | Permanent Plug | ea | 2 | 250 | 500 |
| | Headwalls | ea | 1 | 2153 | 2,153 |
| | Misc. connections | ea | 3 | 1076 | 3,229 |
| | Subtotal | | | | 635,401 |
| | Contingency | 20% | 1 | 127080 | 127,080 |
| | Subtotal | | | | 762,481 |
| | Eng., Legal, Admin. | 25% | 1 | 190620 | 190,620 |
| | | | | **Total=** | **$ 953,101** |




| DATE | CRANE & MERSETH | PROPOSED CAPITAL IMPROVEMENT | FIGURE |
|---|---|---|---|
| JULY 2001 | *Engineering/Surveying*<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | EAST MCKINLEY<br>City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P7 |

# Project No. 8

Drainage: Mill Creek

Subbasin: M-11a

Project Name: Stubb Road Detention

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**PRIORITY:** Medium

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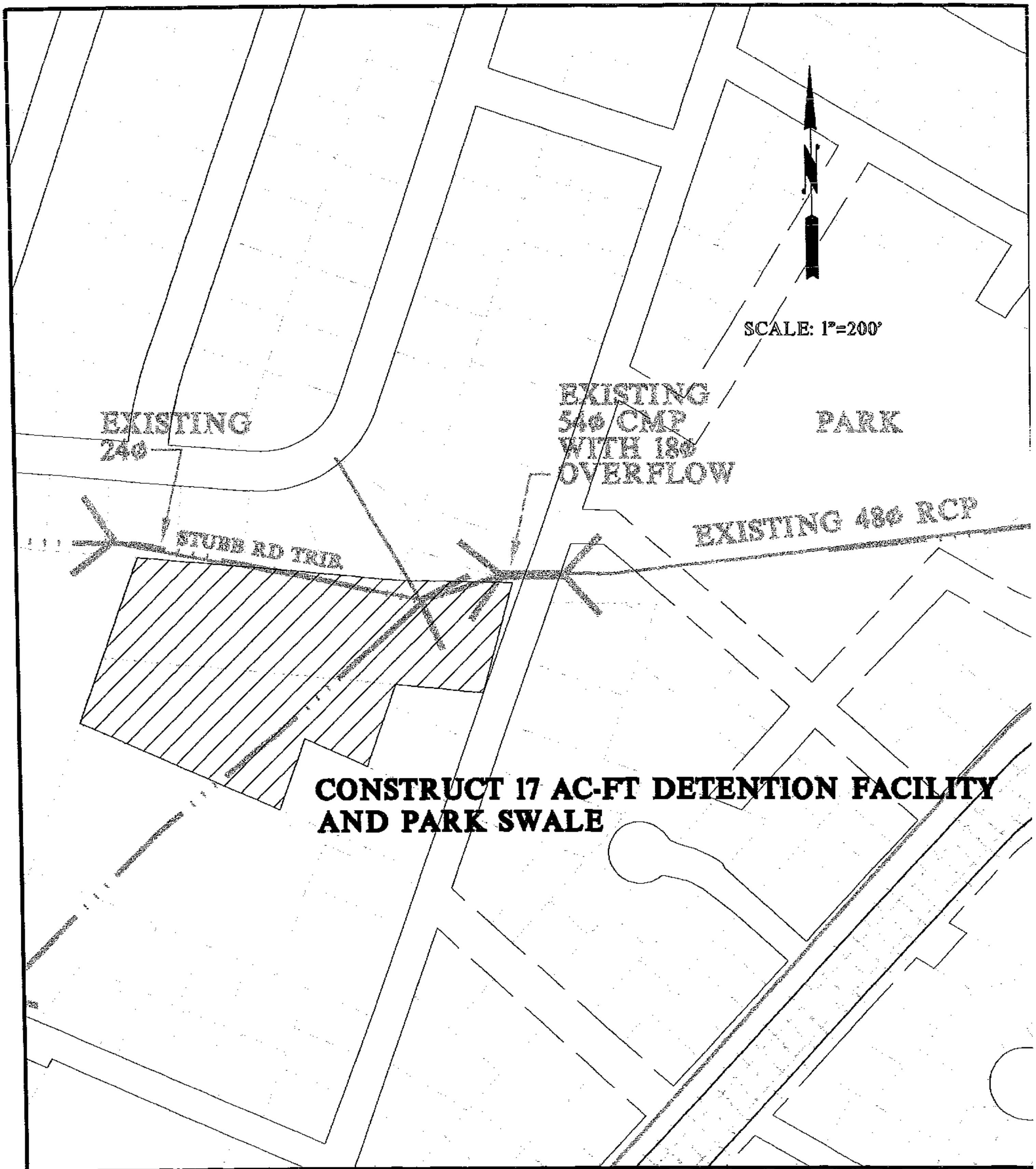
**Description:** This project is comprised of two elements; a 17 ac-ft detention lagoon west of Settlemier Avenue and a constructed swale in the park east of Settlemier. The detention above Settlemier should be 17 ac-ft below elevation 173.4'. These two facilities compliment each other and with both in place, detention of flood flows from the westerly areas of Woodburn will be provided to protect downstream properties.

Conveyance of storm runoff through the park area can be increased if the flow that exceeds the capacity of the existing pipeline is allowed to exit the pipe downstream from Settlemier Road and flow through the park area in a swale created for this purpose. This overland flow would then reenter the pipeline at the east side of the park, just upstream of where the storm sewer crosses under the railroad tracks. This park swale should be designed to convey 15 cfs with a limit on the upstream water surface of 173.0'. This will allow the Settlemier detention facility to be designed with a volume of 17 acre-feet. Without the ability to route flows through the new swale in the park, the detention pond volume will be 27 ac-ft and considerably more land and excavation will be required. Land for the detention facility is currently in private ownership and would have to be acquired by the city or granted to public ownership as part of a larger project development. See Figure P8, Stubb Road Detention, for the existing site layout.

**Cost Estimate:** Stubb Rd Detention

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P8 | Earthwork (excav/fill) | Cy | 24000 | 9 | $ 206,650 |
| Stubb Rd Detention | Erosion control | Ls | 1 | 10763 | 10,763 |
| | Structures | Ls | 1 | 10763 | 10,763 |
| | Seeding/landscaping | Ls | 1 | 21526 | 21,526 |
| | Subtotal | | | | 249,702 |
| | Contingency | 20% | 1 | 49940 | 49,940 |
| | Subtotal | | | | 299,643 |
| | Eng., Legal, Admin. | 25% | 1 | 59929 | 59,929 |
| | | | | Total= | $ 359,571 |

Land acquisition of approximately 5.7 acres that will be needed for this project is estimated at $20,000 per acre for a total land expenditure of $114,784.




| DATE | CRANE & MERSETH Engineering/Surveying<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | PROPOSED CAPITAL IMPROVEMENT<br>STUBB RD DETENTION | FIGURE |
|---|---|---|---|
| JULY 2001 | | City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P8 |

# Project No. 9

Drainage: Senecal Creek

Subbasin: M-11a

Project Name: Connecting 48" at I-5 and Hwy 214

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**PRIORITY:** High

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**Description:** A dry, existing 48-inch diameter storm sewer extends under Interstate 5 from the northeast quadrant of the intersection with Highway 214 to the northwest quadrant of the same intersection. The 48-inch diameter storm sewer was completed as part of the Interstate 5 construction project and extends westerly across the freeway. Its apparent intended use was to convey storm water from the east side of the interchange to the Senecal Creek drainage. This project would connect the existing storm sewers in the northeast quadrant of the intersection into the 48-inch line where ever possible. See Figure P9, Connect 48" at I-5 and Hwy 214, for the existing storm sewer locations and layout.

Potential connections from the existing local storm sewers into the 48-inch line have not been identified and no new connections have been made to the 48-inch line. The 48-inch line is accessible from the west end through a manhole but no manhole or other access point to the 48-inch line has been identified east of the interstate highway. Numerous attempts at locating a manhole on the east end of the line have been made, but with no success.

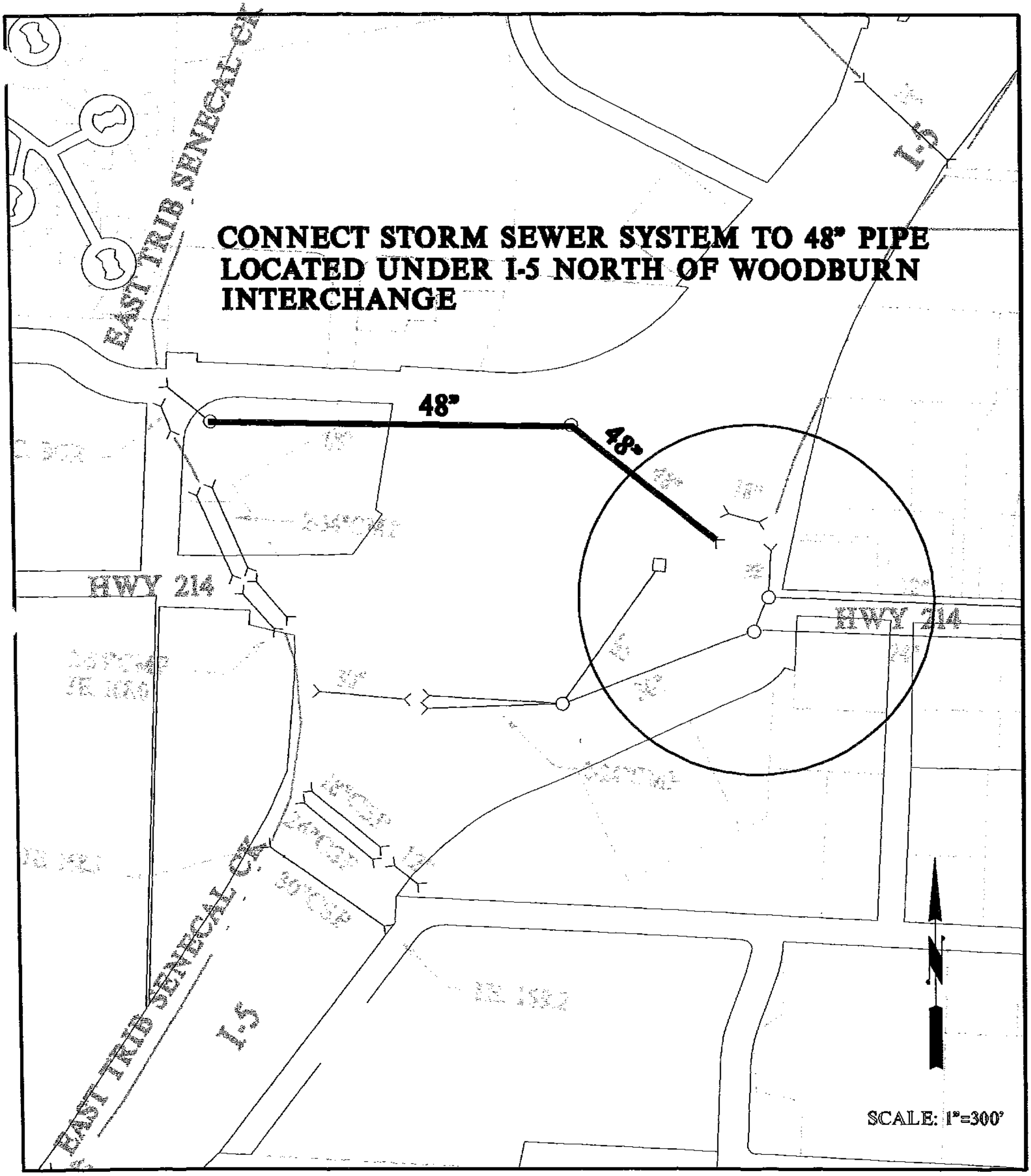
Development of the properties northeast of the interchange have increased the volume of storm runoff. These flows are now routed through an existing storm sewer system, taking a circuitous route around the intersection to the south of Hwy 214, eventually discharging to the storm sewer system west of the interchange.

At its completion, this project will allow full use of the existing 48-inch diameter line.

**Cost Estimate:** Preparation of a preliminary design report should be budgeted at $10,000.

As a minimum, the Predesign Report should:

1. Identify the potential connections between the smaller existing lines east of I-5 and the 48-inch line using field surveys of invert elevations and confirmation of pipe diameters,
2. Calculate the storm flow rates for all lines with potential of being discharged to the 48-inch line,
3. Provide hydraulic analysis necessary to forecast new flows in the pipes and open channels downstream from the project, and
4. Provide an estimate of project cost to construct the improvements.




| DATE | | | FIGURE |
|---|---|---|---|
| JULY 2001 | CRANE & MERSETH<br>Engineering/Surveying<br>6566 SE LAKE Rd., SUITE D<br>MILWAUKIE, OREGON 97222<br>BUS: (503) 654-2005<br>FAX: (503) 654-2575 | PROPOSED CAPITAL IMPROVEMENT<br>CONNECT 48 " AT I-5 & HWY 214<br>City of Woodburn<br>STORM DRAINAGE MASTER PLAN | P9 |

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# Project No. 10

Drainage: Mill Creek

Subbasin: M-5

Project Name: Goose Creek Re-alignment

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**PRIORITY:** Low

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**Description:** The flow capacity of the existing 12' x 7.7' box culvert crossing Hwy 214 immediately west of Front Street is approximately 500 cfs. The results of computer simulation modeling as shown on Figure 10A, Mill Creek Undetained Flows Structures and Flood Profiles, indicates that the water surface profile at Hwy 214 will cause backwater flooding for the 25-year event and possible overtopping of Hwy 214.

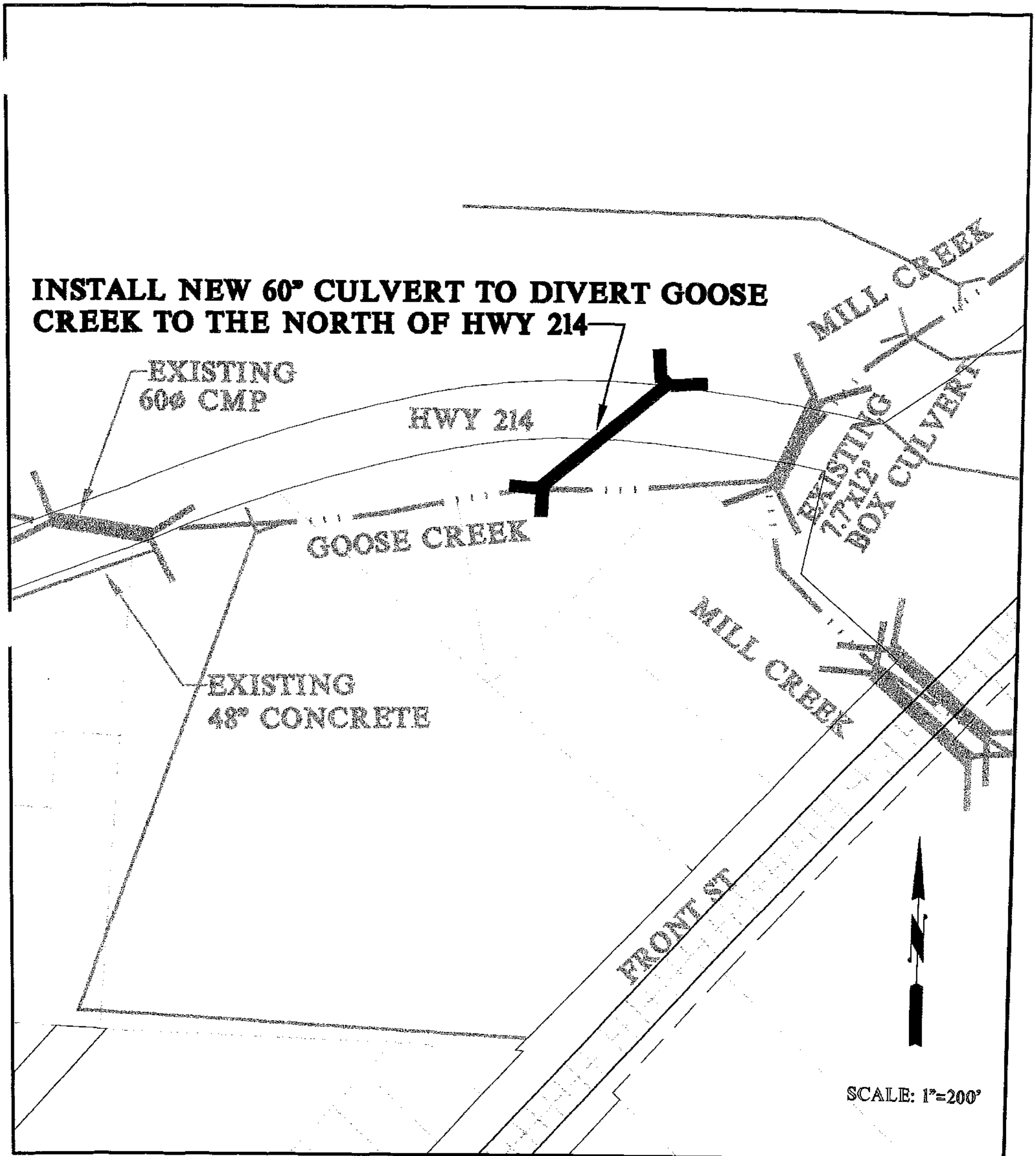
Goose Creek is a significant tributary of Mill Creek and joins Mill creek at a point just south of the box culvert. Flows for the full build out scenario in the Goose Creek subbasin when calculated at the Mill Creek confluence indicate a 25-year flow rate of 156 cfs and a100-year flow rate of 198 cfs.

In order to reduce the possibility of overtopping Hwy 214 it is recommended that flows from Goose Creek be redirected to the north, across Hwy 214 prior to flowing into Mill Creek. This would require the installation of a 60" culvert, crossing Hwy 214 located approximately 300 feet upstream of the Mill Creek confluence.

See Figure P10, Goose Creek Re-alignment, for the existing site layout.

Cost Estimate: Goose Creek Re-alignment

| Project | Item | Unit | Quantity | Unit Cost | Total |
|---|---|---|---|---|---|
| P10 | 60-inch culvert | lf | 175 | 526 | $ 92,105 |
| Goose Creek Re-alignment | Headwall | ea | 2 | 4305 | 8,610 |
| | Site Demo/Flow Diversion | 25% | 1 | 25179 | 25,179 |
| | Subtotal | | | | 125,894 |
| | Contingency | 30% | 1 | 37768 | 37,768 |
| | Subtotal | | | | 163,662 |
| | Engineering, Legal, Admin. | 25% | 1 | 40915 | 40,915 |
| | Pre-design Report/permits | ls | 1 | 20,000 | 20,000 |
| | | | | **Total=** | **$ 224,577** |




| DATE | | PROPOSED CAPITAL IMPROVEMENT GOOSE CREEK RE-ALIGNMENT | FIGURE |
|---|---|---|---|
| JULY 2001 | *CRANE & MERSETH* *Engineering/Surveying* 6566 SE LAKE Rd., SUITE D MILWAUKIE, OREGON 97222 BUS: (503) 654-2005 FAX: (503) 654-2575 | City of Woodburn STORM DRAINAGE MASTER PLAN | P10 |

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Chapter 10

# WATER QUALITY REQUIREMENTS

## Phase I Requirements

The Environmental Protection Agency (EPA) has adopted regulations pertaining to storm water quality nationwide. These regulations take a phased approach toward implementing standards leading to improving storm water runoff quality. Initial phases of the program have addressed major urban areas and cities where water quality is threatened. Rules outlined by the EPA are implemented through the National Pollutant Discharge Elimination System (NPDES) Storm Water Program. Specific areas of concern have targeted cities of over 100,000 population and major systems where sanitary and storm sewer systems are combined. In using these combined storm and sanitary sewer systems, overflows of combined sewage often occur when storm runoff exceeds the capacity of the receiving sewers. This condition causes Combined Sewer Overflows (CSO's), a major target of initial storm water quality action. Woodburn doesn't have combined sewers and has not been required to spend resources in this way.

## Phase II Requirements

Cities using municipal separate storm sewer systems (MS4s) to transport stormwater from the point of collection to the local streams without treatment will be required by EPA's Storm Water Phase II program. These regulations apply to cities or urban areas having a density greater than 1000 people per square mile or having a total size greater than 10,000 people may be regulated by Phase II requirements. Woodburn falls into this category.

The Phase II Final Rule was published in the Federal Register in December 1999. Designation of Phase II requirements by the NPDES authority of municipalities within this population category will be on a case-by-case basis. In Oregon, the DEQ is delegated the responsibility for implementing and enforcing provisions of the regulations.

As of 1997 several specific rules having direct impacts on Woodburn have been placed into effect. These requirements include:

1. That the storm runoff from the city's wastewater treatment plant site be controlled with the discharge of such stormwater falling under a permit with the DEQ. Woodburn has met this requirement and is currently in compliance with this regulation.

2. That any city-owned site which is used as a maintenance yard and where maintenance of city equipment and vehicles is done outside must be permitted through the state's WPCF program

3. That any construction site in the city exceeding 5 acres in area must be maintained during construction in a manner that mitigates storm water runoff from the site. Specific site requirements for such sites will be set by the city following DEQ guidelines.

With finalization of the Phase II requirements, Woodburn will also be required to begin implementing water quality programs in several specific areas and take measures to begin educating citizens in the area about the need and benefits of protecting storm water quality. As of June, 2002, final rules are being processed by the Department of Environmental Quality and are expected to be considered by the Environmental Quality Commission in December, 2002. If these rules are approved, Woodburn will have until July, 2003 to develop the following six MS4 elements:

1. The city must develop and begin a Public Education and Outreach Program relating to storm water quality. This could be done through distribution of educational materials and performing outreach to inform citizens about the impacts polluted storm water

discharges can have on water quality.

2. The city must develop and initiate a Public Participation/Involvement program such as effectively publicizing public hearings and encouraging citizen representatives on a storm water management panel.

3. The city must develop and implement a plan to detect and eliminate illicit discharges to the storm sewer system. This could include updating the system map and inventory, informing the community about hazards associated with illegal discharges and improper disposal of wastes.

4. The city must develop, implement, and enforce an erosion and sediment control program for construction activities that disturb 1 or more acres of land. Controls could include silt fences and temporary storm water detention ponds.

5. The city must develop, implement, and enforce a program to address discharges of post-construction storm water runoff from new development and redevelopment areas. Applicable controls could include preventative actions such as protecting sensitive areas (e.g., wetlands) or the use of structural Best Management Practices such as grassed swales or porous pavement installations.

6. The city must develop and implement a program with the goal of preventing or reducing pollutant runoff from municipal systems. This must include municipal staff training on pollution prevention measures and techniques. This should also include a program to develop and maintain the municipal system and keeping records in order to participate in a reporting system to appraise the DEQ of system conditions and changes.

# Chapter 11

# STORMWATER FLOW MANAGEMENT PROGRAM

The following policies are intended to provide clear guidance for construction of public or private drainage facilities located within the jurisdiction of the City of Woodburn. These guidelines will provide land developers, city staff and design engineers with a common set of rules to be applied when calculating stormwater runoff quantities and flow rates and determining sizes for stormwater facilities downstream from the impacting property.

## AUTHORIZATION

**Under Oregon Water Law, all water is publicly owned. The State of Oregon has adopted the civil law of drainage. Under this law, adjoining landowners are entitled to have the normal course of natural drainage maintained. Landowners with water flowing past, through, or under their property do not have the right to use or control the water without following specific state and local laws and regulations pertaining to such use. Under the provisions of the Oregon Revised Statutes, Section 536.360, all cities (public corporations) are required to provide for management and control of public waters in accord with the provisions of the statute.**

## INTRODUCTION

The City is served by two major natural drainageways -- Senecal Creek and Mill Creek. Within each of these two major drainageways, smaller drainages (denoted as subbasins) may require specific solutions to stormwater management problems including capital construction of public facilities, detention facilities and on-site detention or retention facilities. Each solution developed as a part of the City's Stormwater Master Plan has been proposed after assessing the risk of flood and high water damage to public facilities and overall risk management goals of the City. Generally, the City policy on funding major improvements to the storm water system are:

- Construction of detention, retention and some conveyance facilities may be required prior to completion of developments and are intended to be constructed with private funds.

- Areawide Capital Improvements which are designed to facilitate future property development, in addition to resolving current problems, may be funded cooperatively by the developer, the City, and the benefiting property owners.

- Areawide Capital Improvement Projects outlined in the Stormwater Master Plan for resolution of existing problems are intended to be funded primarily by the City budget and as such are subject to budget constraints.

## PROGRAM HISTORY

Recent, rapid population growth and new federal and state regulations pertaining to operation of a stormwater utility brought a new initiative to the city's public works programs. Various tasks necessary to begin this process included documentation of the system inventory, hydraulic assessment of the natural and manmade storm sewer system, review of fiscal management of utility resources and eventual provision for stormwater treatment have led to development of this program.

Historically, the City has relied on a series of stormwater ordinances, published technical guidance, and regulations issued by the Federal Emergency Management Agency (FEMA) and the Corps of Engineers (COE) to guide and regulate development within its jurisdiction. This program intends to supplement and replace existing ordinances and technical guidance.

Most communities of Woodburn's size have not developed a comprehensive approach to management of the storm water system as they have for the water and wastewater utility systems. Woodburn is no exception. Due to a lack of dedicated revenue sources, the City has relied primarily on revenue provided by the Street Fund to finance storm water system cleaning and related maintenance activities and use of System Development Charges to undertake capitol improvement projects.

## READERS GUIDE

General storm water management policies apply to all properties in the City and are presented on pages 4 through 12 of this chapter. These are supplemented with additional, specific stormwater management policies created for each of the two major drainage basins, Senecal Creek and Mill Creek. Specific policies for the Mill Creek and Senecal Creek drainage basins begin on page 13 of this chapter. Detailed hydraulic analysis of two small drainage areas generally east of Highway 99E have not been done as a part of this project since they drain to the Pudding River. However, the policies and recommendations contained in this document will apply to all land within the City.

## STORMWATER MANAGEMENT GOALS, DETENTION and DEVELOPMENT GUIDELINES

The following goals outline the City's intent to continue orderly development of the storm drainage facility system and provide for the **protection, enhancement and general public safety of life and property.**

These goals are:

- To mitigate for the hydraulic impacts on downstream properties and drainage conveyance structures resulting from increased runoff due to urbanization of parcels within the drainage basin.

- To provide for the conveyance of stormwater to established drainageways in a safe and economical manner and minimize environmental impacts.

- To establish a drainage routing plan which designates watershed limits in developing areas which do not directly abut an existing drainageway or a public storm sewer system facility.

- To establish detention performance criteria limiting stormwater discharge rates to a level which limits impacts on downstream properties and structures to acceptable levels.

- To integrate and formalize existing policies and provide developers with a known and predictable set of detention standards.

- To identify a mechanism whereby developers and property owners can be reimbursed for constructing or enlarging drainage system facilities which solely increases capacity to upstream users.

- To establish standards whereby the public or private nature of the ownership of storm water and detention facilities may be made.

- To provide for maintenance of storm water conveyance and detention facilities.

- To comply with state and federal rules and regulations pertaining to stormwater and runoff management.

- To continue preservation of existing (natural and constructed) floodwater storage volume.

## GENERAL STORMWATER POLICY PROVISIONS

### A. Responsibility to Convey Surface Runoff

It is the responsibility of any landowner, owning property which currently receives stormwater flows from parcels which are topographically upstream, to allow the continued discharge of such flows in such manner as preserves existing conveyance and provides for future upstream connections to waterways, drainageways and other routes carrying stormwater runoff.

### B. Restrictions on Runoff Discharge to an Existing Drainageway where it is:

**1. Private Property with an easement for the drainageway;**
When the receiving drainageway is located on private property and an easement for the drainageway exists, no person shall cause an increase in the volume or rate of flows onto the downstream properties by channelizing existing flows, or by increasing impervious areas which discharge onto other privately owned properties, without the approval of the City Engineer. Single family residences located in an approved subdivision are excepted from this provision.

**2. Private Property without an existing drainageway easement;**
When the receiving drainageway is located on private property and no easement for the drainageway exists, no person shall cause an increase in the volume or rate of flows onto the downstream properties by channelizing existing flows, or by increasing impervious areas which discharge onto other privately owned property, without the approval of the City Engineer and the affected property owner(s). This condition is intended to include and apply to all affected properties located between the property to be developed and the public storm water system, drainageway with a public easement, or a primary drainageway.

**3. A Public or Primary Drainageway;**
When the receiving drainageway is located on public property or is classified as a Primary Drainageway, any new stormwater sources or increases in existing stormwater flows discharged directly to City stormwater facilities shall be permitted only upon approval by the City Engineer. Such approval shall be granted only if and when the applicant demonstrates to the satisfaction of the City Engineer that such new or increased discharges, (1) will not cause the capacity of downstream structures to be significantly impacted, and (2) will not cause increased erosion of downstream drainageways. Single family residences located in an approved subdivision are excepted from this provision.

## C. Extension of Drainage Services to Upstream Parcels.

1. In general, any new development shall install closed conduit drainage conveyances which are of sufficient capacity and depth, and are suitable to serve parcels topographically upstream of the development site. Such conveyances shall be sized to receive future post-development upstream, undetained flows for a 25-year storm event in a Local Drainageway and a 50-year storm event in a Secondary Drainageway.

   In a Primary Drainageway, open channel conveyances shall be designed to convey a 100-year storm event. Open channels shall be used exclusively in Primary Drainageways except as roadway and pedestrian crossings where bridges and culverts may be used.

   These criteria also apply to all structures designed as roadway crossings.

2. All stormwater facility designs must adhere to the following basic criteria:

   a. Upstream flowrates and volumes must be calculated using the entire upstream basin area whether it is within or outside the city's Urban Growth Boundary. Calculation methods and assumptions must be provided as part of the design submittal.

   b. Any stormwater conveyance facility designated for use by upstream properties must be extended to the upstream property limit of the proposed development.

   c. The hydraulic capacity of new stormwater conveyance facilities must be calculated to convey post development runoff of the drainage basin. Runoff must be based on full buildout development created in accordance with the City's Comprehensive Plan in effect at the time of the proposal and the storm water policies and practices of the City.

3. A new development shall continue to receive upstream stormwater runoff in a manner that does not require alteration of the upstream drainage pattern unless specific written permission and appropriate easements are received from the upstream property owner. If the source of the upstream drainage flow is a public stormwater facility, then alteration of its point(s) of entry into, or exit from, the new development shall be at the discretion of the City Engineer.

4. The following provisions shall apply for all constructed stormwater conveyances:

   a. Drainage conveyances shall provide suitable invert elevations and design hydraulic grade lines for the upstream points of service.

   b. All piped (closed conduit) systems shall be designed and constructed in accordance with City of Woodburn *"Standard Design Manual"* and *the "Standard Construction Specifications"*. American Public Works Association (APWA) documents will be utilized until the Woodburn Design Manual is approved by the City Engineer. All systems shall be located within a public right-of-way or a public utility easement no less than sixteen (16) feet in width. Wider easements may be required at the discretion of the City Engineer if needed to accommodate larger or deeper pipes.

5. In unusual circumstances, or to comply with federal, state, or City rules or regulations and laws, the City Engineer may approve open channel designs that meet the following criteria:

a. All open channel systems shall be designed and constructed in accordance with the City of Woodburn *"Standard Design Manual"* and the *"Standard Construction Specifications"*. Oregon Department of Transportation (ODOT) design documents will be used in cases where City standards have not been approved by the City Engineer. All systems shall be located within a public right-of-way or a public utility easement. Such easements shall, at a minimum, extend from the top-of-bank to top-of-bank and include an additional twenty (20) feet in width outward from the top of bank along one side of the entire length of the open channel conveyance.

b. Open channel systems shall be designed using a Manning's "n" of no less than 0.080 to compensate for vegetative growth, accumulation of debris during and following storm events, and sediment accumulation between maintenance activities.

c. Open channel side slopes shall not exceed 3H:1V in inclination unless engineering and/or geotechnical analysis indicate the stability of another configuration. New open channel facilities shall allow for a maximum water depth no greater than 3 feet as measured at any point along the channel. A minimum of one (1) foot of freeboard shall be included in channel design.

d. Open channels shall be seeded or planted in order to stabilize the channel sides and shall be provided with sufficient erosion protection to minimize erosion until such seeding or planting has become mature and established.

**D. Reimbursement for Extension of Stormwater Service to Future Upstream Users.**

Developers of property located on a Primary Drainageway are required to allow the 100-year, undetained, existing stormwater flows from upstream properties to continue unimpeded to the next downstream properties and will do so at their own cost and are not eligible for reimbursement by the City. With the exception of roadway crossings, systems which are oversized may be eligible for reimbursement when oversizing is done at the request and approval of the City Engineer.

Developers of property located on a Secondary Drainageway are required to allow the 50-year, undetained, existing stormwater flows from upstream to continue unimpeded to the next downstream properties and will do so at their own cost and are not eligible for reimbursement by the City. With the exception of roadway crossings, systems which are oversized may be eligible for reimbursement when oversizing is done at the request and approval of the City Engineer.

Developers who construct public stormwater conveyances in a Local Drainageway which are intended to increase capacity to benefit upstream properties (ie. flows greater than the 25-year undetained, post-development flows) may be eligible for reimbursement according to the following conditions:

1. Facilities considered for City participation may include:

   a. Offsite lines, when the size of such lines are larger than required for the specific development under consideration as outlined above. The City's reimbursement will be limited to the incremental difference of the material cost of the oversized portion of the pipe only. Under special circumstances, consideration will be given to additional construction costs incurred as a result of increased diameter or extra depth, when required by the City

   b. Onsite lines when such lines are designed and constructed in a local drainageway and with the expressed purpose of increasing capacity beyond that required to convey the 25-year, post-development, undetained flows from upstream properties plus the additional runoff due to the proposed development. The reimbursement will be limited to the incremental difference of cost incurred as a result of increased diameter or extra depth, when required by the City.

2. To be eligible for reimbursement, the developer must:

   a. Provide a written statement to the City Engineer **within two (2) weeks of Preapplication Conference** outlining the developers intention to request reimbursement funds for the project or have **received notification by the City of specific conveyance requirement.** All reimbursements are subject to the City's budget constraints and any requests exceeding $7,500 in the aggregate will be subject to City Council approval by motion or resolution unless the cost sharing was part of an earlier council approval process for the project. The cost sharing for such improvements must meet the budget constraints of the City.

   b(1). Prepare estimates of cost for those portions of the drainage improvements which provide increased capacity for upstream properties,

   b(2). Identify the area encompassing all benefiting upstream properties,

   b(3). Propose a reasonable pro rata method for distributing a portion of the costs to benefiting upstream properties. The method shall be based on full-buildout impervious areas or number of developable lots, and degree of benefit to upstream properties relative to property being developed.

b(4). Submit a detailed reimbursement request concurrently with submittal of design drawings for review and approval.

3. The City Engineer, after review of requests from the developer, shall notify the developer in writing as to the applicability of the project for reimbursement and the acceptability of the reimbursement computation. The stormwater System Development Charges (SDC) paid by the developer for the project may be reduced by the amount of credits received for the project. However, the reimbursement amount will not exceed the total of the stormwater SDC for the project.

4. If reimbursement for oversized facilities downstream of a Secondary or Local Drainageway is approved, the following table will be used to determine the amount of reimbursement:

| Percent Diameter Increase | Reimbursement in $/LF |
|---|---|
| Up to 31 percent | $ 10 / Lineal foot |
| Between 31 % and 60 % | $ 20 / Lineal foot |
| Over 61 percent | $ 40 / Lineal foot |

5. Facilities excluded from City participation include:
   - New facilities or rehabilitated existing facilities using open channel conveyance,
   - local (non-regional) detention facilities,
   - any closed conduits of 18-inch diameter or less,
   - any structure constructed for vehicular, bikeway or pedestrian crossings (ie. bridges, culverts,etc.),
   - any facilities constructed in designated floodways or flood plains,

6. If a Reimbursement District or Local Improvement District is formed, it shall conform to City ordinances and policies.

## E. Fill within the 100-Year Floodplain

Fill may be allowed in the floodway fringe of the 100 year floodplain of any primary drainageway. Filling must be done in accordance with Section 8, "Fill Standards", City of Woodburn Floodplain Ordinance, No. 2018, and be placed as engineered fill in accordance with a plan prepared by a Registered Engineer and submitted to the City for review and approval.

## F. Fill within the Floodway of a Primary Drainageway.

No new fill, debris, or other obstructions shall be placed in the floodway of a Primary Drainageway. An exception to this requirement may be considered for purposes of constructing an essential roadway crossing of the drainageway if all the following conditions are met:

1. The road crossing is essential as determined by the City of Woodburn. Typically, approval will be limited to those cases where a roadway crossing is needed to facilitate the public transportation system.

2. All local, state and federal requirements are met.

3. The new culvert(s) or other structure must have a hydraulic capacity to pass the 100 year undetained runoff flows from the upstream watershed. Watershed runoff shall be calculated using parameters in accord with the City's current Comprehensive Land Use Plan. The culvert or other structure shall be designed to minimize the resulting head loss to no more than 0.3 feet including entrance losses. Losses greater than 0.3 feet must receive approval by the City Engineer.

4. The invert elevation of the culvert shall be set at an elevation and grade as approved by the City Engineer. In no circumstances shall the culvert invert be below the natural stream bottom.

5. The inlet and outlet of the culvert are to be sufficiently armored to prevent scour.

6. The embankment is to be constructed of compacted earth with sideslopes inclined no steeper than 3 feet horizontal to 1 foot vertical. Embankments must be seeded as specified in the city's design standards and newly-seeded areas protected from erosion until vegetation is safely established.

7. The lower chord of any bridge must be located at least 0.5 feet higher than the projected 100-year flood elevation which results from full buildout of the upstream areas as projected by the City's Comprehensive Plan. Head losses through any bridge structure must be no more than 0.3 feet, unless the City Engineer approves a higher loss design.

8. Private agricultural stream crossings are exempt from the performance standards of this section if the width and elevation of the crossing design is such that an overtopping flood causes no adverse upstream impacts. The landowner will be responsible for demonstrating this to the satisfaction of the City Engineer.

## G. Detention Requirement for Large Developments

Any new construction, or expansion of existing construction, for commercial, industrial, institutional, or multi-family development uses which creates greater than 2.5 acres of total impervious areas (not including public roads created as a part of the development) are required to provide onsite detention of storm flows. Any new single family residential development larger than 5 acres (gross area, all phases), shall also provide on-site storm water detention facilities. All onsite detention facilities shall meet the following design criteria:

1. All detention facilities shall be designed and constructed in accordance with City of Woodburn *"Standard Design Manual"* and the *"Standard Construction Specifications"*. The City's Detention Facility Sizing Table or the Santa Barbara Urban Hydrograph method as specified in the current edition of the King County (Washington) Surface Water Quality Manual, shall be used to determine the volume of the detention facility. Technical issues not addressed in the City's design guides will defer to Oregon American Public Works Association/ODOT documents until the Woodburn Design Manual is approved by the City Engineer.

**Detention facilities must be designed to contain stormwater flows resulting from a post-development 25 year storm event with a discharge orifice (or structure) sized to limit the outflow to a flow no greater than the undeveloped site peak run off for the existing 5 year frequency storm.**

The detention facility shall include provisions for a high flow bypass and maintain a 1-foot minimum freeboard at the highest water surface elevation during a bypass event. The facility designer must also provide a hydraulic analysis showing the overflow conveyance route and downstream impacts of passing the 100-year storm event.

2. Such detention shall be provided off-line from the Primary Drainageway. Off-line is defined as outside the Primary Drainageway floodway/floodplain.

3. Such detention shall also be provided off-line for Secondary Drainageways except as approved by the City Engineer. In-channel detention within Secondary Drainageways may be used if designed using dynamic hydraulic modeling performed by a qualified engineer registered in the State of Oregon. Such designs must be submitted to the City Engineer for review and must receive specific approval prior to any construction. The design must demonstrate that peak discharges are equivalent to the off-line detention specified above when calculated using both existing and future upstream development conditions.

4. The developer must provide a permanent, all weather road access for vehicular traffic to the detention facility inlet and outlet structures.

5. The developer must provide the City with a stormwater facilities easement to provide for future maintenance needs.

Derivation of the criteria for selection of the 5-acre and 2.5-acre development sizes is found in the Storm Drainage Master Plan.

## H. Detention Requirement for Small Developments

Any new construction, or expansion of existing construction, for commercial, industrial, institutional, or multi-family uses which creates less than 2.5 acres of total impervious areas (not including public roads created as a part of the development) may be required to provide on-site detention to address downstream system capacity limitations, satisfy requirements of other jurisdictions, or mitigate local conditions which preclude full discharge of stormwater. At a minimum, the following information will be required for City staff review:

1. Calculations of the volume and rate of stormwater runoff prior to and following development, done in conformance with City policy and the Storm Drainage Master Plan.

2. Identification of the closest public storm sewer or drainageway which will receive the runoff from the development.

3. Calculations showing the peak flow rate of storm water which will be discharged to the public system including any deleterious hydraulic impacts of stormwater runoff on downstream facilities (pipes, culverts, ditches, etc.)

## I. Developer Maintenance Responsibilities of Constructed Facilities

For any detention facility or open channel drainage facility which a developer constructs or causes to be constructed;

1. The developer shall provide adequate maintenance and erosion control, ensure proper performance, and re-grade, re-seed and/or re-plant as necessary to replace any eroded or failed areas within a period of two (2) years following completion and acceptance of the facility by the City.

2. Long term maintenance responsibility for all detention facilities or open channel drainage facilities must be specified prior to construction. The City may elect, but is not required, to accept responsibility for maintenance of detention facilities. If the party deemed responsible for maintenance is not the City and is other than the owner(s) of the property served by the facility, then a maintenance bond shall be posted by the developer until such time that the maintenance responsibility has been accepted by the home owners association or the City.

3. The detention facilities must be properly maintained by the responsible party. If they are not so maintained, and, following 30 days written notice from the City, the City is authorized to enter the facility, perform maintenance on the facility as needed, and to lien the property for payment of three times the cost of such services or by utilizing policies authorized by the City Council for such reimbursements.

## J. Authorization of Adoption of Watershed Management Standards

The City Engineer may establish or adopt written standards which affect individual watersheds within the City limits. Such standards may include, but are not limited to the following:

1. Technical specifications and design standards.

2. Routing of a storm drainage conveyance within, or outside of, public rights-of-way.

3. Identification of public storm sewer easements or acquisitions necessary for the provision of City storm drainage services.

4. Adoption of specific storm drainage performance standards for individual watersheds in response to requirements of other regulatory agencies such as the Oregon Department of Environmental Quality (DEQ), the Oregon State Division of State Lands (DSL) or the Corps of Engineers (COE).

## K. Final Design for Stormwater Detention Facilities

All elevations, slopes, dimensions and pipe sizes shown in the Storm Drainage Master Plan are preliminary and must be verified as part of the final design of stormwater detention facilities. Final design decisions must be based on actual field conditions and routing patterns selected by the designer at the time of final design.

## L. Variances and Referral

Interpretation of the policies in this document adopted by the City Council shall be the responsibility of the City Engineer or his designated representative.

# SPECIFIC BASIN REQUIREMENTS

## INTRODUCTION

The following sections outline specific detention requirements for individual basins contributing stormwater to either Mill Creek or Senecal Creek and within the jurisdiction of the City.

## MILL CREEK

In addition to the City's General Detention Policy Provisions, the following provisions apply to the Mill Creek drainage areas. References to specific tables and figures are taken from the City of Woodburn Storm Drainage Master Plan. Areas intended for use as Primary Drainageway detention facilities have been identified along the main stem of Mill Creek. Flood elevations referenced below are based on FEMA studies and reported flood levels. Elevations referenced in this document are based on the NGVD 29 datum. Also, in keeping with historical practice, a freeboard zone of 1.5 vertical feet is established above the FEMA flood elevation and sets the elevation below which no permanent structures may be erected in the flood plain. The following flood water storage/detention areas are shown on Figure 15 of the City's Storm Drainage Master Plan. Specifically they are:

**Goose Creek** - Goose Creek is a tributary drainage to Mill Creek generally north of and parallel to Highway 214. Goose Creek is a well defined channel whose storm water flows originate in the North Senior Estates area and the properties immediately north and south of the drainage. While not currently used for detention for storm water, the Goose Creek drainageway should be preserved by the city for use as a storm water control and treatment facility.

**Detention Area Between the Railroad Embankment and Hardcastle Avenue** - An area of approximately 4.8 acres immediately upstream of the Front Street/SPRR line along Mill Creek and extending to Hardcastle Avenue within the 100 year floodplain. With a 100-year flood event elevation projected to be 156.0 msl, a floodwater storage elevation for this area has been set at an elevation of 157.5 feet msl. Below this elevation, no permanent structures should be constructed without adequate flood protection nor should filling in the flood plain be allowed without offsetting mitigation. Posted warnings of potential flood water detention should be placed in the area, specifically at the common entrances to the area from Legion Park to the east.

**Hardcastle Avenue to Lincoln Avenue Detention** - An area of approximately 2.5 acres extending from Hardcastle Avenue to Charles Street and is currently part of the City's park system. A 100-year flood event occurring in this area is projected to reach an elevation of 160.5 feet msl. A floodwater storage elevation for this area has been set at an elevation of 162.0 feet msl. This land use will be continued and approval to construct permanent structures which would be damaged by high water levels will not be allowed if the structure is below 162.0 feet msl. Filling in the floodplain will not be allowed unless offsetting mitigation measures proposed and accepted. Posted warnings of potential flood water detention should be placed in the area, specifically at the common entrances to the park area.

**Brown Street Detention Area** - The City is preserving the Settlemier Park tributary of the Mill Creek floodplain extending from Brown Street on the east to the Southern Pacific Railroad right-of-way on the west. This area serves as a natural detention site and can be expected to flood to an elevation of about 169.5 during a 100-year storm event. The City owns most of the low-lying land in this reach of Mill Creek and will continue maintenance of the area by cutting the grass and limiting the number of trees and shrubs that grow there. Any permanent construction below elevation 171.0 will not be allowed. In the future, this area could be reshaped to maximize detention volume, provide for a controlled outlet and ensure complete drainage after high water events. Like other detention areas accessible to the public, posted warnings of potential floodwater detention should be erected in the area.

**Settlemier Park Detention Area** - The floodplain immediately downstream of Settlemier Park is 171.0 as shown on the FEMA maps. Controlling the floodplain at this elevation within this area will allow continued use of the park facilities, however this area provides for flood storage and should continue to remain an open space. Improving facilities, adding parking or constructing other, similar improvements in Settlemier Park may be done providing that no filling occurs below the flood plain elevation.

Construction of an engineered swale extending from a point near the east side of Settlemier Road and extending east toward the railroad embankment will allow excess flows to exit the 48 inch storm drain conduit and overflow to the low-lying park area. At the east side of the park, a grated entry will allow overflowing storm water to reenter the storm drain after flooding subsides. Construction of this surface overflow will mitigate the restricted capacity of the underground conduit during 100-year storm events. With it's proximity to Settlemier Park, posted warnings of potential flood levels should be installed in this flood storage area.

**Stubb Road Detention Facility** - Future storm runoff flows entering the culvert at Settlemier Road are computed to be 177 cfs for the 100 year frequency event. Flood elevations upstream of Settlemier Road must be controlled in a manner that does not allow future floodwater to exceed elevation 173.4 in the area west of Settlemier Road. Detention of storm water at this location is required for three reasons. These are:

- to prevent flood overtopping of Settlemier Road, a major thoroughfare and emergency vehicle route;

- to allow development of upstream properties to the densities specified in the City's Comprehensive Plan; and

- to limit detention system outflows at a flow rate that will allow the downstream primary drainageway to function as planned.

When constructed in a manner that will allow its operation to complement the Settlemier Park overflow swale facility, the Stubb Road detention facility will contain about 17.5 acre-feet of storage and detain storm flows coming from the developing upstream land. As development occurs upstream of this point, land owners and land developers may be required to contribute financial support to this project.

In addition to these planned detention facilities and protected areas, the following detention facilities exist or being planned for construction in the Mill Creek drainage area:

**Woodburn School Site Detention** - The Woodburn School District has constructed a 0.7 acre-foot detention facility is located at the north east corner of the Middle School property on Parr Road. This facility collects storm runoff from the school district property and discharges controlled flows to the west branch of Mill Creek through the storm drainage system serving the Parr Acres development.

**Steklov Addition Detention Site** - This 0.85 acre-foot, single pond, private detention facility is located west of Brown Street and south of Parr Road and serves the Steklov Addition development site and includes piping suitable for flows from 83 acres upstream of the development. The detention facility itself is sized to only accommodate flows from the Steklov Addition.

**North Front Street Park** - This 1-acre site is currently owned by the City and provides a small but effective detention site. It is located at the confluence of two storm water systems and will lessen downstream flooding conditions with construction of a detention facility. To mitigate this potential for future flooding and inundation of Front Street, the park area should be converted to a detention facility to provide an additional 0.7 acre-feet of storage below elevation 167.0. In addition to construction of the detention facility, the storm sewer crossing Front Street and the railroad immediately east of the park should be replaced with a 42 inch diameter reinforced concrete storm sewerline. (Refer to CIP Project No. 2).

**Young Street to Cleveland Street** - The area between Young and Cleveland Streets provides flood storage to approximately elevation 169.5 when storm conditions reach a 100 year frequency event intensity or a series of smaller intensity storms occur in a relative short time span. The City should restrict construction in this area and post flood warnings in this area as well.

**Marshall Street to Wilson Street** – The Marshall Street embankment acts as a weir when the conduit under Marshall Street becomes filled to capacity. When the basin floods, the backwater extends south from Marshall Street to Wilson Street. With construction of a second conduit (54-inch diameter) at Marshall Street, floodwaters that currently overtop Marshall Street during a 5 year event will be lessened to a frequency of a 25 year event. High water warnings should be posted in this area.

**Wilson Street to the South City Limit** - This area is currently designated as part of the City's green space areas and should be maintained as a part of the storm water control and management area. The City should acquire storm utility easements covering the flood plain at a minimum from Wilson Street to the south city limit line. If the city limit line is extended further to the south, additional flood plain easements in the new areas should also be acquired.

**Parr Acres** - A privately owned, single pond detention facility is currently in operation west of Settlemier Avenue. This 1.1 acre/foot facility controls storm runoff coming from the Parr Acres development and discharges a controlled flow to the west arm of Mill Creek.

**Heritage Park** - A privately constructed detention facility is currently in operation in the Heritage Park subdivision located west of Boones Ferry Road between Vanderbeck and Centennial Roads. Detention facilities for this development includes two ponds, one a park area which has been deeded to the City, and the second designed as underground storage giving a total detained volume of 1.5 acre feet of water. Subsequent development of the subdivision will cause a third facility to be constructed.

Storm water discharged from this facilities, crosses Boones Ferry Road, enters the storm drainage system in Hazelnut Road and is finally discharged to Mill Creek.

## SENECAL CREEK

In addition to the City's General Detention Policy Provisions, the following provisions apply to the Senecal Creek drainage areas. References to specific tables and figures are taken from the City of Woodburn Storm Drainage Master Plan. This stream has been divided into five primary subbasins (S-1, S-2, S-3, S-4 and S-5). Of these, most of the area within subbasins S-1, S-2, S-3 and S-5 is located outside the City's Urban Growth Boundary both north and south of the existing boundary. Very little development is expected in subbasins S-1, S-3 and S-5 in the near future and they have been modeled as undeveloped. Subbasin S-2 contains a portion of the West Woodburn development area. New development in this area is not anticipated at this time as it is almost completely built out according to the City's Comprehensive Plan.

Subbasin S-4 contains the East Tributary of Senecal Creek which is wholly within the UGB and in which most of the development is expected to occur in the foreseeable future. This major subbasin has been divided into 13 smaller subbasins for analysis purposes. Refer to Chapter 4 for specific information on this developing area.

Detention and management controls in subbasin S-4 are controlled by the two major thoroughfares that bisect the basin, Interstate 5 and Highway 214. In order to set specific detention and stormwater management control requirements in the East Tributary subbasin (S-4) it has been divided into four quadrants, ES-1, ES-2, ES-3 and ES-4. The management criteria for these four areas are:

**MANAGEMENT ZONE ES - 1 (North of Highway 214 and west of I - 5)**

On the East Tributary of Senecal Creek, a wetlands area west of Interstate 5 and north of Highway 214 serves to detain storm water and should be preserved as part of the City's wetlands inventory unless it is replaced by a constructed wetland approved by regulatory agencies. An area immediately east of the wetlands has recently been developed with high density commercial facilities. This development has installed on-site storm water detention facilities that discharge to a small subtributary of the East Tributary of Senecal Creek.

**MANAGEMENT ZONE ES - 2 (North of Highway 214 and east of I - 5)**

The following conditions will apply to development and storm water infrastructure changes in Zone ES-2:

1. Any new development that creates over 1 acre of impervious area shall be routed directly to the existing 48-inch diameter storm drain that crosses I - 5 immediately north of the Highway 214 interchange. The east end of this 48-inch storm sewer is not accessible from the ground surface as it was covered over during construction of the I-5 freeway. The end of this line should be excavated and a manhole access constructed at that point. Piping changes should also be made to connect this line to existing storm sewers on the east side of Interstate 5. See CIP project No. 11.

2. No new storm water flows generated in this subbasin should be allowed to discharge to the existing 18-inch culvert that crosses I - 5 north of the 48-inch culvert.

3. No additional or new detention facilities are planned for construction in this basin.

**MANAGEMENT ZONE ES - 3 (South of Highway 214 and west of I - 5)**
The following conditions will apply to development and storm water infrastructure changes in Zone ES-3:

1. The East Tributary shall be retained as an open channel drainageway. The location of the creek may be altered upstream (south) of the future connecting

culvert across I - 5, provided that all other local, state and federal requirements are met and that drainage is provided to the upstream property at Point "C" as shown on Figure ___. Point "C" is the location at which the East Tributary enters the City limits from the subbasin south of Zone ES - 3.

2. Sufficient hydraulic capacity for the East Tributary in Zone ES - 3 shall be maintained by constructing a minimum of 3-acre feet detention facility east of Interstate 5. (See ES-4 narrative). Surface water flows must be controlled to assure that the water surface at Point "B" does not exceed an elevation of 174.5 feet (NGVD 29) during the 100 year detained flow event.

3. Two (2.0) acre-feet of flood storage volume shall be retained on Parcel "A" as shown on Figure ___ of the Storm Drainage Master Plan. This flood storage volume shall be located contiguous with the East Tributary, above the normal water surface level and below an elevation of 174.0. The East Tributary floodway shall not be filled or piped through this parcel.

4. Except for Parcel "A", all new development within Zone ES - 3 shall provide on-site detention as specified in the General Policy.

5. When development occurs on parcels that abut the existing East Tributary in Zone ES - 3, creek maintenance easements specific to the East Tributary shall be deeded to the City of Woodburn.

**MANAGEMENT ZONE ES - 4 (South of Highway 214 and east of I - 5)**
Development of detention and conveyance facilities in Zone ES-4 are required to control flows and mitigate runoff resulting from development in the management zone. The following design parameters and conditions will apply to development of infrastructure facilities in Zone ES-4:

1. A culvert shall be constructed at Point "B" (Figure ___ of the Storm Drainage Master Plan) across I - 5. Design and construction shall be coordinated with the Oregon Department of Transportation and other state and federal agencies that may be involved. Design of the crossing shall comply with the following criteria:

a. The culvert shall have the capacity of a 42-inch diameter concrete sewer pipe.

b. The downstream invert shall be placed at approximately 171.0 feet and shall hydraulically connect to the East Tributary.

c. The pipe slope shall be between 0.0012 and 0.005 feet per foot if a 42-inch pipe is used.

2. A detention facility should be constructed near the upstream (east) end of the culvert described above. The facility will be located as close as possible to the Interstate 5 right-of-way and shall use the following design parameters:

a. The facility shall provide a minimum of 7 acre-feet of off-line detention and:

1. limit the upper water elevation to 177.0',
2. limit the post development release flows from a 25-year event to a rate no greater than produced by a predevelopment 5-year storm event, and
3. detain the full flows of the 100-year storm event

b. The facility shall be designed to cause the water surface in the detention facility to return to elevation 172.50 between rainfall events and the facility shall be designed to empty completely following cessation of runoff.

c. Facility side slopes shall not exceed 3H:1V.

d. The facility excavation shall be stabilized to minimize erosion and shall be seeded or planted with vegetation native to the region and suitable for the hydraulic and soils conditions of the site. Vegetation that may hinder the filling, emptying or maintenance of the facility shall not be used.

e. A 15-foot wide access road shall be provided to the facility and shall extend along one side of the facility and shall include access to the inlet and outlet structures. The access road shall be an all weather road suitable for vehicular use.

f. The detention facility including the perimeter access roadway shall be deeded to the City of Woodburn. A 20-foot wide access easement from a public street shall be granted to the City of Woodburn if the facility does not abut a public right-of-way.

# APPENDIX A

# APPENDIX B

COUNCIL BILL NO. 1138

ORDINANCE NO. 2018

AN ORDINANCE REGULATING AND CONSTRAINING DEVELOPMENT AND CONSTRUCTION WITHIN THE FLOOD-PLAIN AREAS OF WOODBURN, REPEALING ORDINANCE NO. 2010, AND DECLARING AN EMERGENCY.

THE CITY OF WOODBURN ORDAINS AS FOLLOWS:

Section 1. Purpose and Intent. It is the purpose and intent of this ordinance to promote the public health, safety and welfare, and to minimize public and private losses due to flood conditions by regulating and constraining development and construction within the flood-plain areas of Woodburn.

Section 2. Definitions.

(a) "Area of Special Flood Hazard" means the land in the flood plain within a community subject to a one percent or greater chance of flooding in any given year. Designation on the FIRM maps always include the letters A or V.

(b) "Base Flood" means the flood having a one percent chance of being equalled or exceeded in any given year. Also referred to as the "100-year flood." Designation on maps always includes the letters A or V.

(c) "Development" means any man-made changes to improved or unimproved real estate including, but not limited to, buildings or other structures, mining, dredging, filling, grading, paving, excavation, or drilling operations located within the area of special flood hazard.

(d) "Firm". An acronym for Flood Insurance Rate Map.

(e) "Flood or Flooding" means a general and temporary condition of partial or complete inundation of normally dry land areas.

(f) "Flood Insurance Study" means the official report provided by the Federal Insurance Administration that includes flood profiles, the flood boundary floodway map, and water surface elevation of a base flood.

(g) "Floodway" means the channel of a stream or other water course and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevations more than one (1) foot.

(h) "Manufactured Home" means a structure, transportable in one or more sections, which is built on a permanent chassis and is designed for use with or without a permanent foundation when connected to the required utilities. For flood plain management purposes the term "manufactured home" also includes park trailers, travel trailers, and other similar vehicles placed on a site for greater than 180 consecutive days. For insurance purposes the term "manufactured home" does not include park trailers, travel trailers, and other similar vehicles.

(i) "New Construction" means structures for which the start of construction commenced on or after the effective date of this ordinance.

(j) "Start of Construction" includes substantial improvement, and means the date the building permit was issued, provided the actual start of construction, repair, reconstruction, placement or other improvement was within 180 days of the permit date. The actual start means either the first placement of permanent construction of a structure on a site, such as the pouring of slab or footings, the installation of piles, the construction of columns, or any work beyond the stage of excavation; or the placement of a manufactured home on a foundation. Permanent construction does not include land preparation, such as clearing, grading and filling; nor does it include the installation of streets and/or walkways; nor does it include excavation for a basement, footings, piers, or foundation or the erection of temporary forms; nor does it include the installation on the property of accessory buildings, such as garages or sheds not occupied as dwelling units or not part of the main structure.

(k) "Lowest Floor" means the lowest floor of the lowest enclosed area (including basement). An unfinished or flood resistant enclosure, usable solely for parking of vehicles, building access or storage, in an area other than a basement area, is not considered a building's lowest floor, provided that such enclosure is not built so as to render the structure in violation of the applicable nonelevation design requirements of this ordinance found at Section 5.2-1(2).

(l) "Storm Water Management Plan" means the section of the City's officially adopted Comprehensive Plan which deals with storm water and flood water management.

(m) "Structure" means a walled and roofed building including a gas or liquid storage tank that is principally above ground.

(n) "Substantial Improvement" means any repair, reconstruction or improvement of the structure, the cost of which equals or exceeds fifty percent (50%) of the market value of the structure either,

(1) before the improvement or repair is started, or
(2) if the structure has been damaged and is being restored, before the damage occurred. For the purpose of this definition, substantial improvement is considered to occur when the first alteration of any wall, ceiling, floor, or other structural part of the building commences whether or not that alteration affects the external dimensions of the structure.

Section 3. General Provisions.

(a) Land to which this ordinance applies. This ordinance shall apply to all areas of special flood hazard within the jurisdiction of the City of Woodburn.

(b) Subdivision Proposals:

(1) All subdivision proposals shall be consistent with the need to minimize flood damage;

(2) All subdivision proposals shall have public utilities and facilities such as sewer, gas, electrical, and water systems located and constructed to minimize flood damage;

(3) All subdivision proposals shall have adequate drainage provided to reduce exposure to flood damage; and,

(4) Where base flood elevation data has not been provided or is not available from another authoritative source, it shall be generated for subdivision proposals and other proposed developments Which contain at least 50 lots or 5 acres (which ever is less).

(c) Review of Building Permits: Where elevation data is not available either through the Flood Insurance Study or from another authoritative source, applications for building permits shall be reviewed to assure that proposed construction will be reasonably safe from flooding. The test of reasonableness is a local judgement and includes use of historical data, high-water marks, photographs of past flooding, etc., where available. Failure to elevate at least two feet above grade in these zones may result in higher insurance rates.

(d) Basis for establishing the areas of special flood hazard. The area of special flood hazard identified by the Federal Insurance Administration in a scientific and engineering report entitled "The Flood Insurance Study for the City of Woodburn" dated March 1, 1979, with accompanying flood insurance maps is hereby adopted by reference and declared to be a part of this ordinance. In addition, the City's Storm Water Management Plan is also adopted by this ordinance and included as a part thereof.

(e) Minimum floor elevations for structures in the flood hazard area. The minimum floor elevations for structures in the Flood Hazard Areas shall be determined on a site specific basis using surveys and survey data or data found to be acceptable under the sections of this ordinance or the National Flood Insurance Standards.

(f) Floodways defined. The following floodways are hereby defined by this ordinance:

(1) For Mill Creek main drainage channel A maximum Floodway width of 150' as defined on data table #2 of the Woodburn Flood Insurance Study.

(2) For Senecal Creek main channel, a maximum floodway width of 145' as defined on Data Table #2 of the Woodburn Flood Insurance Study.

(3) For the tributary in drainage basin No. 2 as defined on the Storm Water Management Plan, a floodway of 80 feet from the confluence with tributary No. 2 with Mill Creek upstream 1,600 feet.

(4) For the tributary in drainage basin No. 3, a floodway channel of 60 feet from the confluence of tributary No. 3 with the main Mill Creek channel upstream 1,000 feet.

(5) For the tributary in drainage basin No. 5, a floodway of 80 feet from the confluence of tributary No. 5 with the Mill Creek channel upstream 1,600 feet.

(6) For the tributary in drainage basin No. 6, a floodway of 100 feet from the confluence of tributary No. 6 with the Mill Creek channel upstream 1,000 feet, a floodway of 80 feet from 1,000 feet to 1,500 feet, and a floodway of 60 feet from 1,500 feet to 2,000 feet above the confluence of Mill Creek.

(7) For the tributary in drainage basin No. 7, a floodway of 80 feet from the confluence of tributary No. 7 with Mill Creek upstream 1,800 feet.

(8) For the Senecal Creek tributary which is unnumbered on the Storm Water Management Plan but which drains the area from Interstate 5 to Woodland Avenue, a floodway of 80 feet from the point of its confluence with Senecal Creek upstream to the point at which it crosses underneath State Highway 214.

(g) In addition to the above mentioned floodways, a floodway of 40 feet shall be maintained on all open existing drainage channels within the City of Woodburn.

Section 4. Administration.

(a) Establishment of development permit. A development permit shall be obtained before construction or development begins within any area of special flood hazard established in Section 3 (b). The permits shall be for all structures including manufactured homes as set forth in the definitions and for all other developments including fill and other activities as also set forth in the definitions.

(b) Designation of the City Engineer. The City Engineer, or his designate, is hereby appointed to administer and implement this ordinance by granting or denying development applications in accordance with its provisions.

(c) Duties and responsibilities of the City Engineer. Duties of the City Engineer shall include, but are not limited to:

(1) Permit review. Review all development permits to determine whether the permit requirements of this ordinance have been satisfied.

(2) Review all development permits to determine that all necessary permits have been obtained from

Those federal, state or local governmental agencies from which prior approval is required.

(3) Review all development permits to determine if the proposed development is located in the floodway.

(4) Review all requests to fill in the flood hazard area to determine if the requests are in conformance with the criteria set forth in this ordinance.

(d) Use of other base flood data. When base flood elevation data has not been provided in accordance with Section 3 (d), basis for establishing the areas of special flood hazard, the City Engineer shall obtain, review, and reasonably utilize any base flood elevation and floodway data available from federal, state, or other sources in order to administer the provisions of this ordinance.

(e) Information to be obtained and maintained.

(1) Where base flood elevation data is provided through the Flood Insurance Study or required as in Section 4 (d), obtain and record the actual elevation in relation to mean sea level of the lowest floor (including basement) of all new or substantially improved structures and whether or not the structure contains a basement.

(2) For all new or substantially improved flood-proof structures:

(i) Obtain and record the actual elevation (in relation to mean sea level) and,

(ii) Maintain the flood proofing certifications required in Section 6 (b)(3).

(3) Elevations required above shall be provided by the owner along with a certification by an engineer or registered land surveyor of the actual elevation above mean sea level of the lowest floor of the structure.

(f) Alteration of water courses.

(1) Notify adjacent communities and the state agency responsible **(DEPARTMENT OF LAND CONSERVATION AND DEVELOPMENT)** alteration or relocation of a water course, and submit evidence of such notification to the Federal Insurance Administration.

(2) Require that maintenance is provided within the altered and relocated portion of said water course so that the flood carrying capacity is not diminished.

Section 5. General Standards. In all areas of special flood hazards the following standards are required.

(a) Anchoring.

(1) All new construction and substantial improvements to existing structures shall be anchored to prevent flotation, collapse, or lateral movement of the structure.

(2) All manufactured homes in a special flood hazard area shall be placed on fill AND elevated to the minimum elevations established in Section 3 (c) or 1.5 feet above the elevation of the base flood.

(3) All manufactured homes must likewise be anchored to prevent flotation, collapse or lateral movement, and shall be installed using methods and practices that minimize flood damage. Anchoring methods may include, but are not limited to, use of over-the-top or frame ties to ground anchors.

(b) Utilities.

(1) All new and replacement water supply systems shall be designed and constructed to minimize or elimi nate infiltration of flood waters into the system.

(2) New and replacement sanitary sewer systems shall be designed and constructed to minimize or eliminate infiltration of flood waters into the systems and discharge of the systems into the flood waters.

(3) On-site waste disposal systems shall be located to avoid impairment to them or contamination from them during flooding.

(c) Storage of materials and equipment. Materials that are buoyant, flammable, obnoxious, toxic or otherwise injurious to persons or property, if transported by flood-waters, are prohibited in the flood hazard area. Storage of materials and equipment not having these characteristics is permissible only if the materials and equipment have low-damage potential and are anchored or are readily removable from the area within the time available after forecasting and warning, however, no storage is allowed in the floodway.

Section 6. Specific Standards. In all areas of special flood hazards where base flood elevation data has been provided in this ordinance under Section 3 (c) or Section 4 (d), the following provisions are required.

(a) Electrical, heating, ventilation, plumbing, and air-conditioning equipment and other service facilities shall be designed and/or otherwise elevated or located so as to prevent water from entering or accumulating within the components during conditions of flooding.

**ALL NEW CONSTRUCTION AND SUBSTANTIAL IMPROVEMENTS SHALL BE CONSTRUCTED WITH MATERIALS AND UTILITY EQUIPMENT RESISTANT TO FLOOD DAMAGE.**

**ALL NEW CONSTRUCTION AND SUBSTANTIAL IMPROVEMENTS SHALL BE CONSTRUCTED USING METHODS AND PRACTICES THAT MINIMIZE FLOOD DAMAGE.**

(b) All manufactured homes to be placed or substantially improved within Zones A1-30, AH, and AE shall be elevated on a permanent foundation such that the lowest floor of the manufactured home is above base flood elevation and be securely anchored to an adequately anchored foundation system in accordance with the provisions of section 5 (A)(2) & 5 (A)(3).

(c) Residential construction. New construction and substantial improvement of any residential structures shall have the lowest floor, including basement, elevated to or above the elevation established in Section 3 (c), or 1.5 feet above the elevation established in Section 3 (D) & 4 (d).

(d) Fully enclosed areas below the lowest floor that are subject to flooding are prohibited, or shall be designed to automatically equalize hydrostatic flood forces on exterior walls by allowing for the entry and exit of floodwaters. Designs for meeting this requirement must either be certified by a registered professional engineer or architect or must meet or exceed the following minimum criteria:

(i) A minimum of two openings having a total of not less than one square inch for every square foot of enclosed area subject to flooding shall be provided.

(ii) The bottom of all openings shall be no higher than one foot above grade.

(iii) Openings may be equipped with screens, louvers, or other coverings or devices provided that they permit the automatic entry and exit of flood waters.

(e) Non-Residential construction. New construction and substantial improvement of any commercial, industrial or other non-residential structure shall either have the lowest floor, including basement, elevated to the level of the elevation established in Section 3 (c), or 1.5 feet above the elevation of the base flood established in Section 3 (d) & 4 (d); or, together with the attendant utility and sanitary facilities shall:

(1) Be flood proofed so that below the base flood level of the structure is water-tight with walls substantially impermeable to the passage of water.

(2) Have structural components capable of resisting hydrostatic loads and effect of buoyancy in a base flood.

(3) Be certified by a registered professional engineer or architect that the design and methods of construction are in accordance with accepted standards of practice of meeting provisions of this sub section based on their development and/or review of the structural design, specifications and plans. Such certifications shall be provided to the City Engineer.

(4) Nonresidential structures that are elevated, not floodproofed, must meet the same standards for space below the lowest floor as described in 6 (D).

(5) Applicants floodproofing nonresidential buildings shall be notified that flood insurance premiums will be based on rates that are one foot below the flood-proofed level (e.g. a building constructed to

the base flood level will be rated as one foot below that level).

Section 7. Floodways.

(A) In the Floodways as defined under Section 3 (f), no encroachments including fill, new construction, substantial improvements, and other development, within the adopted regulatory flood way that would result in any increase in flood levels, is permitted.

(B) The normal and routine maintenance of stream channels is not precluded by this ordinance provided such main tenance complies with the no rise standard in flood levels. as outlined in Section 7 (a).

Section 8. Fill Standards.

(a) All structures built in the special flood area shall be constructed on engineered fill or shall have designed footings at suitable depth, both as required by the Uniform Building Code, or in conformance with other additional standards as required by the City Engineer in accordance with good engineering practices.

(b) The slope on a fill in the special flood hazard area shall not exceed 33%. Toe of such fill shall be outside the floodways defined in Section 3 (d).

(c) The amount of fill in the special flood hazard area shall be kept to a minimum. The following standards shall apply.

(1) Only one structure per existing lot at the time of passage of this ordinance shall be allowed for areas within the special flood hazard area. The structure shall be located so that a minimum amount of fill will be necessary for the elevation of the structure above the flood level.

(2) All subdivision, partitioning, and planned unit developments which envision development of any special flood hazard area shall indicate on the preliminary plan the location of all structures proposed to be located in the flood hazard area. These structures shall be located so that a minimum amount of fill is required to develop the land.

(3) Development proposals, whether nonresidential or residential, together with public utilities and facilities attendant to them, shall be constructed to minimize flood damage, and adequate drainage shall be provided. In areas not covered by

Section 3 (b), flood elevation data shall be provided by the developer.

(4) Multiple family residential or nonresidential structures shall be located as far as practical on the existing contiguous property from the floodway.

Section 9. Density Transfer. The Planning Commission may, upon application under the variance procedure, allow a higher density of dwelling units or structures on a parcel of property which contains areas of special flood hazard if the areas of special flood hazard are left substantially without fill. The Commission shall determine the amount of fill which would practicably be allowed in the flood hazard, and the additional amount of density on land outside the special flood hazard area which should be allowed due to the loss of the developable land in the flood hazard area.

Section 10. Variances. Variances to this ordinance shall comply with the same standards and follow the same procedures for variances to the Zoning Code of the City of Woodburn.

Section 11. Enforcement.

(a) Violation of this ordinance IS A CLASS I CIVIL INFRACTION AND shall be punishable by a fine of up to $500 for the first offense (finding of violation), and by a fine of up to $500 for the second and succeeding offenses (finding of violation). A separate offense will be deemed to occur on each calendar day that the infraction continues to exist, and a separate citation may be filed for each such offense.

(b) Alternate Remedy. If a parcel of land is, or is proposed to be used, developed, or maintained in violation of this ordinance, the aforesaid use shall constitute a nuisance, and the City may, as an addition to other remedies that are legally available for enforcing this ordinance institute injunction, mandamus, abatement, or other appropriate proceedings to prevent, enjoin temporarily or permanently, abate or remove the unlawful use, development, or maintenance of the land.

Section 12. Violation as Nuisance. Violation of any provision of this ordinance is hereby declared to be a nuisance, for which remedy may be pursued by the City to the full extent of law, notwithstanding any limitation in this or any other ordinance.

Section 13. Severability. If any word, clause, phrase, section, subsection, or other portion of this ordinance is found invalid by a court of competent jurisdiction, then the remainder of the ordinance shall be given full effect.

Section 14. Ordinance Repealed. Ordinance No. 2010 is hereby repealed **AND ORDINANCE No. 1967 AND No. 1664 ARE NOT THEREBY RESURRECTED.**

Section 15. This ordinance being necessary for the immediate preservation of the public peace, health, and safety, an emergency is declared to exist and this ordinance shall take effect immediately upon passage by the City Council and approval by the Mayor.

Approved as to form: ______________________ 3/9/89
City Attorney Date

APPROVED: ______________________
NANCY A. KIRKSEY, MAYOR

| | |
|---|---|
| Passed by the Council | March 27, 1989 |
| Submitted to the Mayor | March 27, 1989 |
| Approved by the Mayor | March 27, 1989 |
| Filed in the Office of the Recorder | March 27, 1989 |

ATTEST: ______________________
BARNEY Ø. BURRIS, Recorder
City of Woodburn, Oregon

# APPENDIX C

# City of Woodburn

# SENECAL CREEK EAST TRIBUTARY CAPACITY ANALYSIS


prepared for
**GORDON L. MERSETH, P.E., CONSULTING ENGINEER**

prepared by
**OAKLEY ENGINEERING, INC.**
700 N. Hayden Island Dr., Suite 340
Portland, OR 97217
T: 289-7411; F: 289-7656

in conjunction with
**DAVID C. SMITH & ASSOCIATES, INC.**

**March 1994**

# CITY OF WOODBURN

# SENECAL CREEK EAST TRIBUTARY CAPACITY ANALYSIS

March 1994

## 1. PURPOSE AND OVERVIEW

Figure 1, "Vicinity Map" shows Senecal Creek watershed and the watershed for the East Tributary of Senecal Creek. Existing flood conditions in Main Senecal Creek were evaluated and presented in a previous report: "Senecal Creek Hydraulic Analysis: Existing Conditions," Gordon L. Merseth/Oakley Engineering, Sept. 1993. This second analysis was conducted to evaluate the existing and future conveyance capacity of the East Tributary of Senecal Creek. The capacity of the East Tributary is significant because large areas of future development land exist at the upper (southern) end of the East Tributary watershed.

The main goals of this analysis are:

1) Evaluate the existing and future 25 year and 100 year flood profile conditions for undetained flows in the East Tributary.

2) Evaluate the maximum potential hydraulic capacity of the East Tributary drainageway and its culverted crossings to convey future flows.

3) Recommend possible mitigation strategies for the East Tributary, including culvert improvements and/or detention alternatives to fully utilize the conveyance capacity of the East Tributary as a drainage corridor for the City of Woodburn.

The assumptions and results of the analysis are presented in the following sections: Watershed Parameters and Flow Calculations; Senecal Creek Flood Profiles, Future Flows; East Tributary Capacity Analysis Results; and Summary of Improvements and Recommendations.

## 2. WATERSHED PARAMETERS AND FLOW CALCULATIONS

Basin Delineation

Figure 2, "Basin and Drainageway Map," delineates the key subbasins which drain to the East Tributary. Subbasins were delineated to reflect existing drainage patterns, areas of similar land use, and potential future drainage routing.

The solid basin line in Figure 2 represents the natural watershed boundary. In addition to these natural watershed areas which currently drain to the East Tributary, the capacity analysis indicates that some additional areas outside the natural watershed could be routed to the East Tributary. These additional areas are located at the upper (southerly) end of the East Tributary and have very flat topography. Consequently, relatively large areas could be routed via storm sewers or ditches, towards the East Tributary. The parcels recommended to drain to the East Tributary can easily be routed to this drainageway. This extension of the natural watershed clarifies the decisions placed on individual land owners by specifying the direction the entire parcel can be drained.

The basin extensions shown indicate the areas which do not currently drain to the East Tributary but could be served by this drainage in the future:

- Basin S-I represents a portion of a parcel zoned for industrial use. Most of the parcel drains to the East Tributary; the S-I basin portion drains to main Senecal Creek. Rather than route part of the parcel to Main Senecal Creek and part of the parcel to East Senecal Creek, the entire S-I basin could be routed to the East Tributary.

- Basins M-C, M-LDR and M-HDR represent areas currently draining to a shallow ditch system in the Mill Creek watershed. This ditch is not sufficiently deep to serve future developments and these runoff flows may be more economically routed to the East Tributary.

<u>Existing and Future Land Use</u>

For each subbasin, runoff parameters were estimated for existing and future land use conditions. Existing land use conditions were determined using the City's Zoning Map and aerial photography (1992). (See Figure 2, "Basin and Drainageway Map.") Runoff parameters for future land use conditions were estimated assuming full buildout to the maximum densities designated by the City's Comprehensive Land Use Plan. Future land use designations are shown in Figure 3, "Comprehensive Plan Land Use Map."

Currently, only about 35% of the portion of the Woodburn Urban Growth Boundary (UGB) currently draining to the East Tributary is developed at full buildout densities. Of the remaining area in the UGB, development in the East Tributary watershed is expected to be distributed by land use as follows: 33% commercial, 23% industrial, 17% high density residential, 27% low density residential.

Runoff Parameters

The Corps of Engineers HEC-1 computer model was used to compute peak sub-basin flows. The following input parameters are required: Basin area, effective impervious area, runoff curve number and lag time. Basin areas and the approximate area of existing mapped impervious surface (MIA) were computed from the aerial photo map in Figure 2, "Basin and Drainageway Map." Future MIA values were determined by land use as follows:

| Land Use | Full Buildout MIA |
|---|---|
| Low Density Residential. | 45% |
| High Density Residential | 65% |
| Commercial | 75% |
| Industrial | 85% |

Future MIA values were determined from the City of Woodburn's maximum site coverage and on-site landscaping requirements and also account for estimated off-site (public) impervious surface.

Effective impervious area (EIA) represents the effective area over which no infiltration occurs. This area is typically less than the actual mapped impervious area (MIA) due to depression storage, runoff which flows off impervious surfaces and across pervious areas before entering the storm system and other similar factors. For input into the HEC-1 models, EIA was determined using the following regression equations (originally published by OTAK, Inc. based on an analysis of USGS data in the Willamette Valley):

Older development areas: $EIA = 0.1x(MIA)^{1.5}$
Newer development areas and future development: $EIA = 0.4x(MIA)^{1.2}$

Soil runoff characteristics for each basin were determined using the Marion County Soils Survey. A composite runoff curve number corresponding to typical soils and ground cover conditions in each sub-basin was determined using standard Soil Conservation Service (SCS) tables.

The lag time input parameter is computed as 60% of the sub-basin time of concentration. Time of concentration for each sub-basin was calculated using the methodology published in the SCS publication TR-55.

Runoff parameters for each subbasin are summarized in Table 1, "East Tributary of Senecal Creek Sub-basin Parameters."

Flow Calculations

Peak sub-basin flows as calculated by the HEC-1 models are summarized in Figure 5, "East Tributary of Senecal Creek Peak Subbasin Flows," for existing and full buildout conditions. These figures represent the peak flow at the basin outlet for free-flow, undetained conditions.

In-stream flows were calculated by combining and routing individual subbasin flow hydrographs using a combination of HEC-1 and HEC-2 computer models. HEC-1 was used to route the hydrographs using the storage/outflow method in conjunction with volumes calculated by the HEC-2 flood profile model at key cross sections.

The peak in-stream flows used for the capacity analysis are shown on the flood profiles in Figure 4 and described in Section 4, "East Tributary Capacity Analysis Results and Recommendations."

## 3. SENECAL CREEK FLOOD PROFILES, FUTURE FLOWS

The HEC-1 models for future conditions in the East Tributary were integrated with the HEC-1 models previously prepared for Senecal Creek in order to evaluate the impacts of future development on Senecal Creek. Calculations were conducted assuming full buildout within the Urban Growth Boundary for both Senecal Creek and the East Tributary basins.

Most of the land yet to be developed in the Senecal Creek watershed is located in the East Tributary basin. Since the East Tributary (and consequently most of the runoff from possible future development) enters Senecal Creek downstream of the City Limits, future development was found to have only a small impact on Senecal Creek where it is located within the City. For the 100 year event, future buildout resulted in a flow increase of 8 CFS in the portion of Senecal Creek upstream of the confluence with the East Tributary.

In Senecal Creek downstream of the East Tributary confluence, upstream, upstream development will result in a 100 year flow increase of about 36 CFS in the natural watershed, or 75 CFS with runoff from the additional basins. Both the Crosby Road bridge crossing and the existing East Tributary cross section were found to have adequate capacity to convey the entire 75 CFS flow increase with only minor (less than 0.1') increases in flood elevation.

For a 75 CFS future 100 year flow increase, a 0.4' increase in the floodplain elevation was calculated for the Agricultural Crossing located in Senecal Creek downstream of the City Limits. Backwater due to a 75 CFS future flow increase at the Agricultural Crossing was found to result in a increase of 0.2' in the 100 year floodplain elevation calculated at the City Limits. This increase does not result in flooding of any existing structures. An increase of less than a 0.1' at the City Limits was calculated for the future 25 year event.

## 4. EAST TRIBUTARY CAPACITY ANALYSIS RESULTS

Peak flood elevations in the East Tributary west of I-5 were calculated using the Corps of Engineers HEC-2 computer modelling program for the 25 year and 100 year flood events. Channel/floodplain cross sections and culvert configuration data input into the models were obtained from the DeHaas and Associates field survey data provided by the City and from the City's 1976 1"=100' orthophoto topographical maps.

Flood profiles were calculated for the 25 year and 100 year events for the following conditions:

1) Existing conditions of open and developed acreage, and,

2) Full buildout of the East Tributary watershed including the future development areas (S-I, M-C, M-LDR, and M-HDR) as shown in Figure 2, "Basin and Drainageway Map," recommended to be routed to the East Tributary. (The total East Tributary watershed plus these additional areas as identified in Figure 2, will be referred to as the "expanded area" in this report.)

Flood profile calculations are presented in Figure 4, "East Tributary of Senecal Creek Flood Profiles." The profiles shown in Figure 4 are intended to represent future conditions if no culvert improvements or detention requirements are implemented. Existing culvert configurations, channel and floodplain cross sections were used for both the existing and full buildout hydraulic profile calculations. Existing and future in-stream flows reflect routing through the existing channel sections and storage routing through the existing culvert restrictions north of the UGB.

The flood profiles do not account for any flow reductions or detention storage due to the culvert system under Highway 214, the Texaco station and Arney Road. Calculations indicate that these culverts have adequate capacity to convey existing flows without detention. However, future flows will create surcharged conditions through this system of culvert crossings. If the low lying lands, which are situated immediately upstream of the Highway 214 crossing, are used for detention, a 30 CFS reduction in flow would result. The effects of this potential detention are not shown in the flood profiles in Figure 4, but are addressed later in this discussion.

The results of the flood profile calculations are discussed below for three key sections of the East Tributary conveyance.

Mouth of the East Tributary to the Urban Growth Boundary (Station 0+00 to 24+80)

This portion of the East Tributary is outside the City's UGB. The conveyance is primarily natural drainageway with three minor culverted crossings. The culvert configuration and location of each crossing are shown in Figure 2, "Basin and Drainageway Map," and on the flood profiles in Figure 4.

The analysis indicated that no improvements to these culverts are required to protect existing houses or to facilitate development in the City. However, future flows can be expected to increase the frequency with which these crossings overtop. Upstream detention will lessen this impact but over-topping will still occur without substantial investment. The degree of over-topping is shown on Figure 4, "Flood Profiles".

UGB to Arney Road (Station 24+65 to 52+00)

This reach of the creek has adequate capacity for future flow increases, assuming that the existing creek cross section area is maintained. High flows are currently conveyed by a bottom lands open channel ranging from 140' to 250' in width. Creek embankment slopes range from 5H:1V to 10H:1V. There are two foot bridges and these are expected to have a negligible affect on the floodplain. No road crossings or other structures restrict flow in this reach.

For the expanded watershed area, undetained future flow increases result in an increase of 0.5' to 1.0' in the 100 year flood profile throughout this reach. Because of the floodplain configuration, these increases result in only a small increase in the flood width and are not expected to significantly affect future development or any existing houses or major structures.

The 140' to 250' wide existing conveyance area is not currently shown as open space or a greenway on the City Zoning and Comprehensive Plan Maps. However, in order to accommodate both existing and future flows, it is recommended that this conveyance area be maintained for future flood conveyance. Specific floodway recommendations will be made during a later phase of the overall drainage master planning study. Final recommendations will include minimum finish floor elevations, minimum floodway widths to be maintained and the maximum allowable cumulative head loss for all future road crossings within this reach.

Arney Road, Texaco and Highway 214 Culvert System

This system of culverts consists of a 6' x 4' concrete box culvert under Arney Road, two 54" CMP culverts under the Texaco station and two 54" CMP culverts under Highway 214. The capacity of these culverts affects water levels in the East Tributary upstream (south) of Highway 214. The upper reaches of the East Tributary in this area serves the adjacent freeway drainage systems and receives flow from the 30" Walmart storm drain.

Analysis of existing 1"=100' City topo maps indicates that a maximum 25 year water surface elevation of 174.0 feet could be tolerated in the East Tributary immediately upstream of Highway 214. Flood elevations of 175.0 feet begin to flood the I-5 median and adjacent low areas. This water level is not acceptable for the 25 year flood event, but is probably tolerable for the 100 year frequency. The freeway pavement itself is above elevation 178.0 feet and would not be flooded by a 100 year flood elevation of 175.0 feet. Commercial land east of the freeway typically range in elevation from 180 feet to 184 feet and would not be impacted by a 100 year flood elevation of 175.0 feet.

Hydraulic calculations indicate that a maximum 100 year flow of 155 CFS and 25 year flow of 130 CFS can be conveyed through the Highway 214 culvert system without exceeding tolerable in-stream water surface elevations. This capacity is sufficient to convey the 25 year and 100 year flows calculated for existing conditions. Routing full buildout flows from the expanded area, however, exceeds the capacity of the East Tributary's culvert system in the vicinity of Highway 214 by about 40 to 50 CFS.

The existing topography immediately upstream (south) of Highway 214 currently provides up to 3.7 acre-feet of storage if full buildout flows are routed through the existing culvert system. This volume of storage reduces the 100 year flood elevation upstream of Highway 214 to 176.0 feet for full buildout flows from the expanded watershed area.

Options to control the 100 year water surface in the East Tributary immediately upstream of Highway 214 to less than 176.0 feet include: 1) constructing a diversion to route flow to the 48" pipe which crosses the freeway north of Highway 214 and discharges into the East Tributary downstream of Arney Road or 2) requiring detention for future upstream developments.

The 48" pipe crossing the freeway north of Highway 214 has adequate capacity of about 90 CFS and, for example, will carry the total full buildout runoff from basins F, H, I, and J. While diverting flow to the 48" freeway crossing may be feasible, more information on the invert depth, pipe slope and access to the pipe is required to fully evaluate this alternative. A diversion of this nature would not provide any flow reduction benefits downstream of Arney Road.

The total detention volume needed upstream of Highway 214 in order to safely convey full buildout flows for the 100 year event from the natural watershed plus the additional sub-basins is approximately 11 acre-feet. Less detention would necessitate the eventual replacement of the Highway 214 culverts. Also, less detention would also further impact the existing agricultural crossings located downstream of Highway 214.

## 5. SUMMARY OF IMPROVEMENT RECOMMENDATIONS

The improvement alternatives discussed in the previous sections are summarized for the East Tributary:

**Culvert Crossing at the UGB**

The crossing at Station 24+65 is located at the UGB boundary. Two new 42" CSP culverts are required in addition to the existing 42" CMP to accommodate future flows and maintain the existing 10 year capacity of this crossing.

**Open Channel Reach Between the UGB and Arney Road:**

No structural or detention improvements are required. Maintain the existing 140' to 250' wide open channel bottom for flood conveyance. No fill or encroachment of this drainageway should be allowed, or, if allowed, compensating flood storage should be provided.

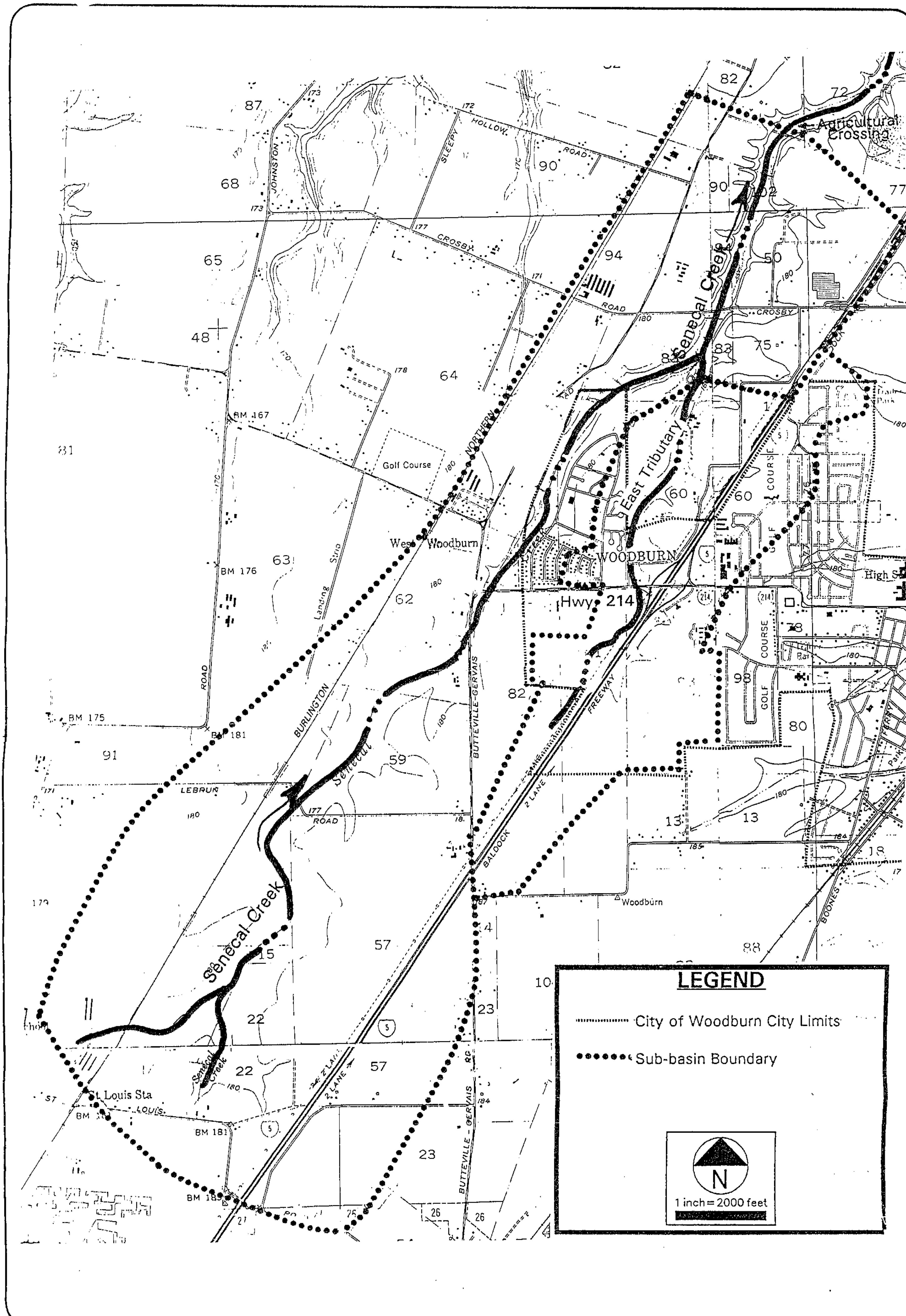
**Highway 214, Texaco and Arney Road Culvert System:**

Maintain at least 2 acre-feet of the existing 3.7 acre-feet of flood storage area immediately upstream of the Highway 214 culverts and limit the maximum water surface elevation to an elevation of 175.0 feet.

**Area Upstream of Highway 214:**

No additional detention or structural improvements are required to maintain a 100 year conveyance capacity in the East Tributary if an additional 9 acre-feet of detention is provided in this upper portion of the basin.

The above improvements specified are recommended in order to maintain optimum 25 year and 100 year water surface profiles in the East Tributary. Subsequent phases of the Drainage Master Plan study will identify specific detention improvements and policies to address these capacity issues for the East Tributary of Senecal Creek.




| DATE | VICINITY MAP | PREPARED FOR GORDON L. MERSETH, P.E. CONSULTING ENGINEER P.O. Box 82746 – Portland, Oregon 97282 Tel. (503) 233-2695 | FIGURE 1 |
|---|---|---|---|
| | City of Woodburn<br>Senecal Creek, East Tributary Capacity Analysis | PREPARED BY OAKLEY ENGINEERING, INC. DRAINAGE PLANNING & DESIGN, HYDROLOGY, CIVIL ENGINEERING, & SITE DEVELOPMENT 700 N.E. Hayden Island Drive, Suite 340 Portland, Oregon 97217 Tel. (503) 289-7411 – Fax. (503) 289-7656 | |

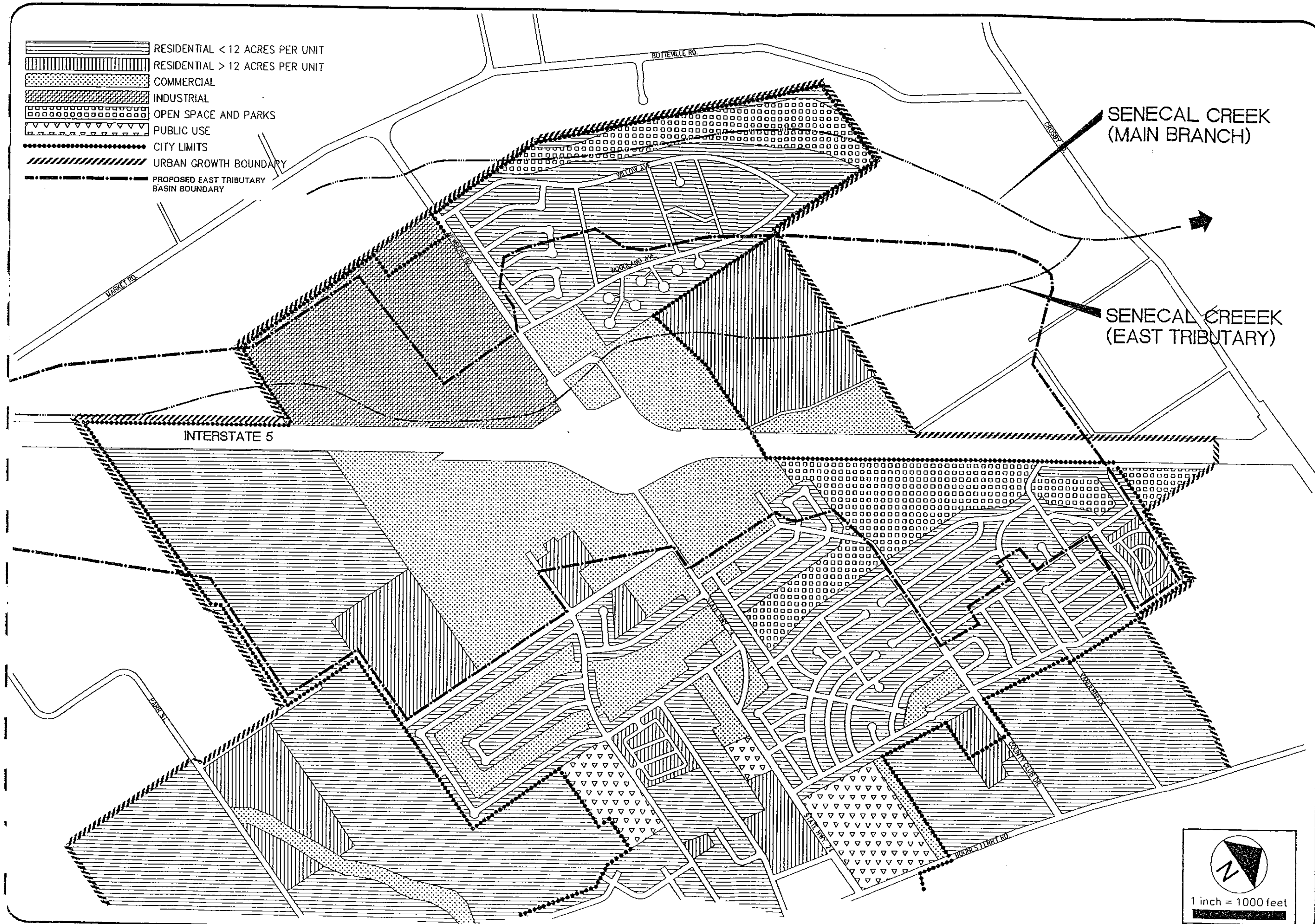



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FIGURE

3

OAKLEY ENGINEERING, INC.

DRAINAGE PLANNING & DESIGN, HYDROLOGY,
CIVIL ENGINEERING, & SITE DEVELOPMENT
700 N. Hayden Island Drive, Suite 340
Portland, Oregon 97217
Tel. (503) 289-7411 – Fax. (503) 289-7656

COMPREHENSIVE PLAN LAND USE MAP

City of Woodburn
Senecal Creek East Tributary Capacity Analysis

DATE

Mar., 1994

# TABLE 1
# East Tributary of Senecal Creek
# Subbasin Parameters

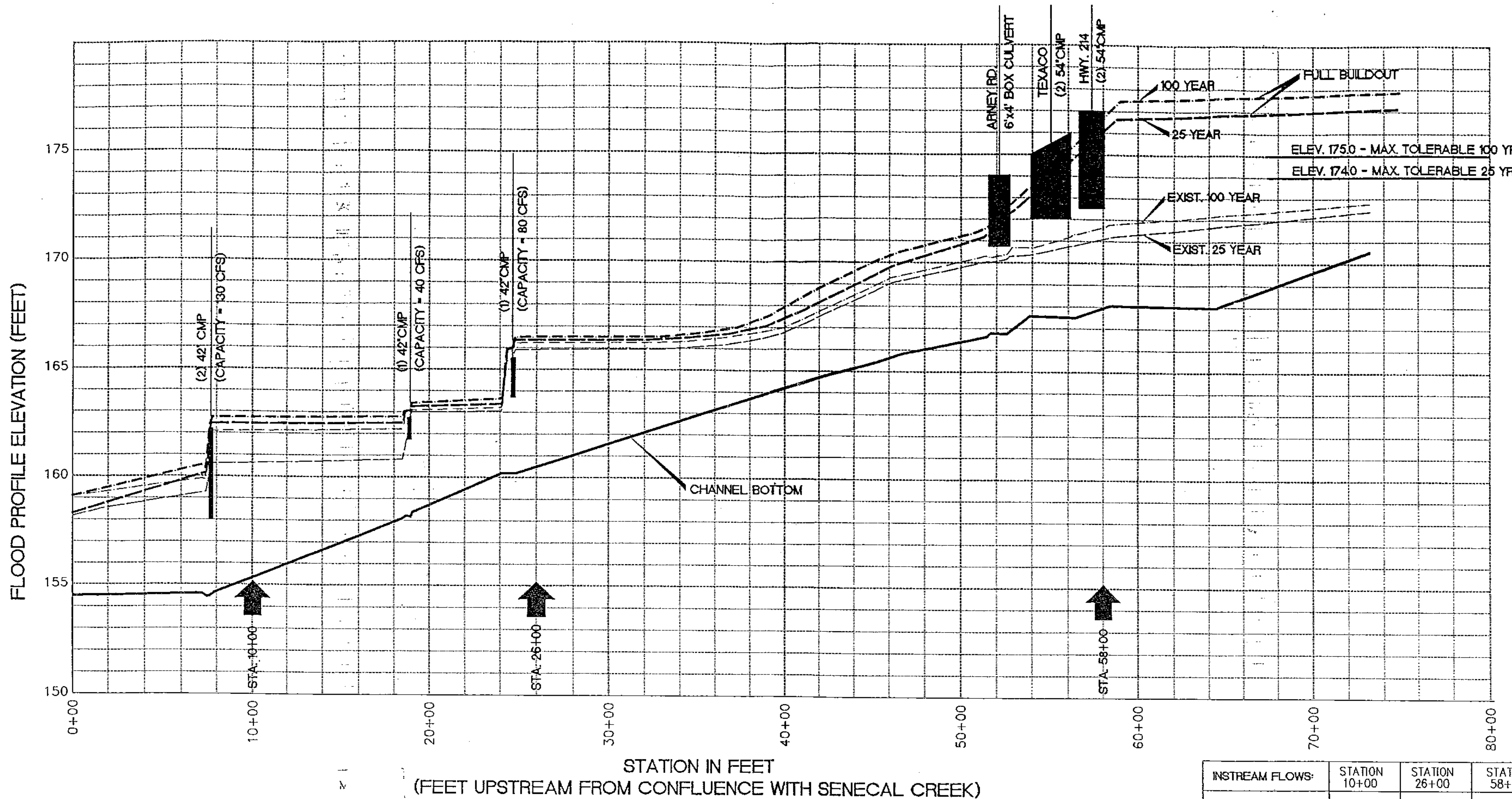
| Sub-basin | Area (Acres) | Area (Sq. Miles) | % Impervious MIA | % Impervious EIA | SCS Curve No., Pervious Areas | Time of Conc. (Minutes) | Lag Time (Hours) |
|---|---|---|---|---|---|---|---|
| **Existing 1993 Conditions** | | | | | | | |
| A | 124.5 | 0.195 | 8.0% | 1.0% | 78 | 102 | 1.02 |
| B-1 | 60.3 | 0.094 | 60.0% | 46.5% | 74 | 60 | 0.60 |
| B-2 | 49.4 | 0.077 | 2.0% | 0.5% | 74 | 31 | 0.31 |
| C | 49.0 | 0.077 | 38.0% | 23.4% | 74 | 19 | 0.19 |
| D | 78.3 | 0.122 | 3.0% | 0.5% | 80 | 35 | 0.35 |
| E | 50.2 | 0.078 | 22.0% | 10.3% | 80 | 28 | 0.28 |
| F | 30.0 | 0.047 | 35.0% | 20.7% | 80 | 20 | 0.20 |
| G | 76.6 | 0.120 | 5.0% | 0.5% | 80 | 127 | 1.27 |
| H | 26.7 | 0.042 | 60.0% | 54.4% | 77 | 20 | 0.20 |
| I | 33.6 | 0.053 | 57.0% | 51.2% | 77 | 20 | 0.20 |
| J | 40.6 | 0.063 | 2.0% | 0.5% | 80 | 130 | 1.30 |
| K | 83.1 | 0.130 | 5.0% | 0.5% | 80 | 160 | 1.60 |
| L | 121.0 | 0.189 | 3.0% | 0.5% | 81 | 190 | 1.90 |
| **Full Buildout Development Conditions** | | | | | | | |
| A | 124.5 | 0.195 | 8.0% | 1.0% | 78 | 102 | 1.02 |
| B-1 | 60.3 | 0.094 | 60.0% | 46.5% | 74 | 60 | 0.60 |
| B-2 | 49.4 | 0.077 | 2.0% | 0.5% | 74 | 31 | 0.31 |
| C | 49.0 | 0.077 | 45.0% | 30.2% | 74 | 19 | 0.19 |
| D | 78.3 | 0.122 | 70.0% | 65.5% | 77 | 22 | 0.22 |
| E | 50.2 | 0.078 | 67.0% | 62.1% | 77 | 16 | 0.16 |
| F | 30.0 | 0.047 | 66.0% | 61.0% | 77 | 18 | 0.16 |
| G | 76.6 | 0.120 | 72.5% | 68.3% | 77 | 37 | 0.37 |
| H | 26.7 | 0.042 | 68.0% | 63.3% | 77 | 16 | 0.16 |
| I | 33.6 | 0.053 | 72.0% | 67.5% | 77 | 14 | 0.14 |
| J | 40.6 | 0.063 | 75.0% | 71.1% | 77 | 18 | 0.18 |
| K | 83.1 | 0.130 | 45.0% | 38.5% | 77 | 24 | 0.24 |
| L | 121.0 | 0.189 | 3.0% | 0.5% | 81 | 190 | 1.90 |
| **New Development Areas to Be Routed (Full Buildout) *** | | | | | | | |
| M-C | 24.7 | 0.039 | 75.0% | 71.1% | 77 | 15 | 0.15 |
| S-I | 17.0 | 0.027 | 85.0% | 82.7% | 77 | 13 | 0.13 |
| M-HDR | 30.3 | 0.047 | 65.0% | 59.5% | 77 | 15 | 0.15 |
| M-LDR | 42.9 | 0.067 | 45.0% | 38.5% | 74 | 24 | 0.24 |

* NOTE: M denotes basins currently draining to Mill Creek.
S denotes basins currently draining to Main Senecal Creek.

# TABLE 2
# East Tributary of Senecal Creek
# Peak Subbasin Flows

| Sub-basin | Area (Acres) | Area (Sq. Miles) | Existing 25 YR (CFS) | 100 YR (CFS) | Full Buildout 25 YR (CFS) | 100 YR (CFS) |
|---|---|---|---|---|---|---|
| **Natural Watershed** | | | | | | |
| A | 124.5 | 0.195 | 20 | 28 | 20 | 28 |
| B-1 | 60.3 | 0.094 | 24 | 29 | 24 | 29 |
| B-2 | 49.4 | 0.077 | 7 | 10 | 7 | 10 |
| C | 49.0 | 0.077 | 15 | 20 | 17 | 22 |
| D | 78.3 | 0.122 | 20 | 27 | 42 | 50 |
| E | 42.7 | 0.067 | 16 | 20 | 28 | 33 |
| F | 31.1 | 0.049 | 11 | 15 | 17 | 20 |
| G | 76.6 | 0.120 | 13 | 17 | 38 | 45 |
| H | 26.7 | 0.042 | 13 | 16 | 15 | 18 |
| I | 33.6 | 0.053 | 16 | 20 | 20 | 24 |
| J | 40.6 | 0.063 | 7 | 9 | 24 | 28 |
| K | 83.1 | 0.130 | 13 | 17 | 33 | 41 |
| L | 121.0 | 0.189 | 18 | 24 | 18 | 24 |
| **Potential Re-routing of Future Development** | | | | | | |
| M-C | 24.7 | 0.039 | - | - | 15 | 18 |
| S-I | 17.0 | 0.027 | - | - | 12 | 14 |
| M-HDR | 30.3 | 0.047 | - | - | 16 | 19 |
| M-LDR | 42.9 | 0.067 | - | - | 11 | 14 |

* NOTE: M denotes basins currently draining to Mill Creek.
S denotes basins currently draining to Main Senecal Creek.




| INSTREAM FLOWS: | STATION 10+00 | STATION 26+00 | STATION 58+00 |
|---|---|---|---|
| EXIST. 25 YR | 115 CFS | 85 CFS | 58 CFS |
| EXIST. 100 YR | 155 CFS | 112 CFS | 76 CFS |
| FUTURE 25 YR* | 250 CFS | 230 CFS | 180 CFS |
| FUTURE 100 YR* | 350 CFS | 280 CFS | 220 CFS |

* EXPANDED AREA

NOTE:
PROFILES ARE FOR EXISTING AND FUTURE FLOWS WITH NO DETENTION, CULVERT OR CHANNEL IMPROVEMENTS IMPLEMENTED.

PREPARED FOR
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| DATE | | | | FIGURE |
|---|---|---|---|---|
| Mar., 1994 | OAKLEY ENGINEERING, INC.<br>DRAINAGE PLANNING & DESIGN, HYDROLOGY, CIVIL ENGINEERING, & SITE DEVELOPMENT<br>700 N.E. Hayden Is. Dr., Suite 340 Portland, OR 97217<br>TEL. (503) 289-7411 - FAX. (503) 289-7656 | City of Woodburn<br>Senecal Creek Hydraulic Analysis | EAST TRIBUTARY OF SENECAL CREEK<br>FLOOD PROFILES | 4 |

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