SECTION 3

Methodology

This section describes the analytical techniques and outlines the conceptual methodology that this plan will use to analyze the existing and future drainage facility requirements. First, several basic assumptions are presented. Next, analytical techniques used for hydrologic and hydraulic analyses are described. Finally, this section briefly describes how the City's drainage facilities will be evaluated.

3.1 Basic Assumptions

The following basic assumptions for the hydrologic and hydraulic analyses were made:

- Land use zoning within the study area in the year 2023 will be those designated in the McMinnville Comprehensive Plan Land Use Map.
- Percent impervious area for future development scenarios will be estimated based on an assumed percent impervious area that is typical for each of the various land uses.
- Full build-out assumes complete development within the UGB.
- The National Oceanic and Atmospheric Administration's (NOAA) Precipitation-Frequency Atlas of the Western United states, Volume X—Oregon, 1973, were used as the basis to select a design rainfall event. See Section 5, Rainfall Analysis, for further details.
- Rainfall is assumed to be uniform throughout the watershed during a design storm event. That is, the design rain event occurs at the same time and uniformly across the entire City.
- Parameters, such as hydrologic soil types, will be estimated for drainage sub-basins using parameter values weighted by area within each sub-basin.
- Yamhill River starting elevations were based on the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for the City of McMinnville (1982). The average annual high water in the South Yamhill River was assumed to be the starting water surface elevation for the 10-year event for creeks within the study area.
- The 24-hour Type 1A storm is assumed to be the design storm. Recurrence intervals of 10, 25, 50, and 100 years will be considered as appropriate. The relationship between recurrence interval or frequencies and annual probability of occurrence is shown in Table 3-1.

TABLE 3-1
Recurrence Interval and Annual Probability of Occurrence
City of McMinnville Storm Drainage Master Plan

Recurrence Interval (or) Frequency	Annual Probability of Occurrence
2-year	0.50
5-year	0.20
10-year	0.10
25-year	0.04
50-year	0.02
100-year	0.01

Larger storm events occur less frequently, i.e., at longer return intervals. If the potential for inconvenience or damage is large, it will be assumed that a relatively large recurrence interval (low probability) event will be an appropriate standard for that particular drainageway.

3.2 Hydrologic Analysis

A hydrologic computer model was constructed as a tool for estimating rates of stormwater runoff at the sub-basin scale for both the existing (2005) and ultimate build-out conditions. Modeling was performed using the U.S. Army Corps of Engineers HEC-HMS (v3.1.0) hydrologic modeling software.

The HEC-HMS model is designed to simulate the stormwater runoff response in a basin for a given precipitation event. The rainfall hyetograph is translated into a runoff hydrograph and the hydrograph from each sub-basin is routed by the model to the point of confluence with other sub-basins. When combined with the hydrograph from another sub-basin, a composite hydrograph is computed by the model that accounts for any differences in time of concentration between the two hydrographs. The kinematic wave method was used for routing the runoff hydrographs through the sub-basins and major drainageways. The kinematic wave equation is a differential equation that models the behavior of the hydrograph as a function of channel cross-sectional area and flow. The runoff hydrograph was determined by using the NRCS unit hydrograph method. This method uses lag time as the single parameter in a set of empirical equations to determine the shape of the runoff hydrograph for each sub-basin. The HEC-HMS model requires five input parameters for calculating the runoff hydrograph in each sub-basin: Total Area, Impervious Area, Curve Number, Lag Time, and Precipitation.

3.2.1 Sub-Basin Area (Acres)

Sub-basin area is the watershed within which runoff can be assumed to flow to a single discharge point.

3.2.2 Design Precipitation Hyetograph

The hyetograph defines the temporal distribution of rainfall intensity during a storm event. There are three parameters that define any hyetograph: depth, duration, and distribution or shape. For this analysis the NRCS Type-1A, 24-hour storm was selected (see Section 5 for a more detailed discussion).

3.2.3 Percent Effective Impervious Area

The effective impervious area (EIA) is computed for existing development conditions based on estimated mapped impervious area (MIA). EIA for future development conditions is based on typical percentages of impervious areas for the land uses planned for the sub-basin. See discussion of EIA in Section 4.5.

3.2.4 Composite Soil Conservation Service's Soil Curve Number

The NRCS soil curve number is used to determine the initial abstraction and runoff potential based on the composite hydrologic group of the soils and land use within each sub-basin. The greater the soil curve number, the more impervious the soil is to infiltration and the greater the percentage runoff.

3.2.5 Lag Time (Hours)

Lag time is a function of time of concentration and is the time difference between the peak precipitation intensity and the peak runoff rate from the sub-basin in question.

The result of the HEC-HMS modeling process is the computation of sub-basin runoff hydrographs (runoff from each individual sub-basin versus time) and stream flow hydrographs (stream flow rates from all upstream sub-basins versus time). The free flowing peak flows at desired locations in the drainageways were estimated for both existing and ultimate (full build-out) development conditions. Peak flows from individual sub-basins (Sub-Basin Flows) for existing development and future conditions for the 10-year recurrence intervals are described in Section 6, Runoff Analysis. Composite peak flows (peak drainageway flows) are shown for the 10-, 25-, 50-, and 100-year events.

Estimated peak flows are used to input into the hydraulic models described below. The HEC-HMS 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.

3.2.6 HEC-HMS Model Calibration

To calibrate the hydrologic runoff model, peak flows were monitored during March 2008 at three locations and the resulting observations were then compared to the model predicted flows. Table 3-2 describes the location and stream used in the monitoring.

TABLE 3-2
Hydrologic Flow Monitoring Locations for Model Calibration
City of McMinnville Storm Drainage Master Plan

Basin	Location	Gaging Method	Number of Measurements
Midtown	Dancer Outfall	Crest-stage	7
North Cozine	Fleishauer	Crest-stage	5
West Cozine	Michelbook	Crest-stage	5

Lag time and effective impervious area were adjusted to calibrate the hydrologic model results to the measured data.

3.3 Hydraulic Analysis

Hydraulic analysis of the stormwater system has two components: an evaluation of the constructed drainage collection system and an open channel analysis of the major drainageways. Though these two systems operate in conjunction, the methods and tools for evaluating their performance is quite different. The sections below describe the computational tools used to evaluate the hydraulic performance of various components of the system under existing and future conditions.

3.3.1 HEC-RAS

Analysis of the open channel drainageways involves computing several water surface profiles for a range of storms of various recurrence intervals. Open channel hydraulic models were created for West Cozine Creek, North Cozine Creek, and Cozine Creek.

Hydraulic modeling was performed using the U.S. Army Corps of Engineers HEC-RAS hydraulic modeling software (v.3.1.3). The primary inputs to the hydraulic model are as follows: (1) peak flows from the HEC-HMS hydrologic models, (2) cross-sectional geometry at regular intervals along the channel, (3) Manning’s roughness coefficients, and (4) downstream boundary conditions such as starting water surface elevations. Cross sections were defined using 2-foot contour data provided by the City. Hydraulic structures such as stream crossings and bridges were defined based on information gathered during field investigations and supplemented with information provided in the previous (1991) master plan. Manning’s roughness coefficients were defined from aerial photographs and field reconnaissance and engineering judgment. The downstream boundary conditions were set to match the FEMA FIS water surface profiles at the confluence with the South Yamhill River.

The accuracy of the data used to construct these models is rather coarse and in some areas outdated due to recent land development. The resulting water surface profiles are therefore approximate and should be updated as new and better data become available.

HEC-RAS is a one-dimensional model that computes water surface profiles for steady, gradually varied flow. The model is capable of computing profiles for both sub- and supercritical flow regimes. The basic computational procedure is based on solving the one-dimensional energy equation, where friction losses are computed using the Manning’s equation and energy losses due to contraction and expansion are computed as a function of the velocity head between cross sections. The momentum equation is applied in situations where the water surface profile varies rapidly such as occurs at stream crossings and at the confluence with other waterways.

3.3.2 Dynamic Modeling of Cozine Creek

Hydrodynamic modeling was performed on Cozine Creek to evaluate the capacity of seven existing culvert crossing in the lower reach. A dynamic (changing with time) model was used to account for the effects of storage and variable timing of peak flows, which are neglected in steady-state analysis. Storage has a significant influence on maximum water surface elevations in the lower basin due to the presence of large roadway embankments at each crossing that behave similarly to a reservoir and attenuate peak flows. The effects of timing are also important because the drainage area of contributing basins varies considerably which causes the timing of peak flows to be offset.

To perform the dynamic modeling analysis, stormwater runoff was computed using the calibrated HEC-HMS model. Two flow scenarios were selected as the basis for evaluating hydraulic capacity of the culvert crossings; the 10-year (24-hour, NRCS Type 1A) storm event was used to evaluate the potential for overtopping to occur at any location, and the 50-year (24-hour, NRCS Type 1A) storm was checked to ensure that emergency routes (such as Highway 99) were not overtopped during extreme flood events.

Dynamic modeling was performed using HEC-RAS v4.0,

3.3.3 HY8

The Federal Highway Administration's (FHWA) culvert analysis program HY8 (v7) was used to evaluate the hydraulic performance of several large culverts that are not included as part of the HEC-RAS models. The HY8 model incorporates analytical methods described in FHWA's Hydraulic Design Series Number 5. The program essentially computes hydraulic conditions at the inlet and outlet of the culvert over a range of flows and produces rating curves for inlet and outlet control conditions. The capacity of a culvert is typically evaluated based on the lower performance of those two curves. This ensures adequate performance under the least favorable hydraulic conditions. Appendix C contains the HY-8 Culvert Analysis reports.

3.3.4 Pipe Hydraulics

An in-house spreadsheet model was developed to evaluate the capacity of select areas of the piped collections system. The analysis was limited to pipes that were previously analyzed in the 1991 master plan. The intent of the analysis was to reevaluate the capacity of those pipes using updated flows that account for development that has occurred since the previous study.

Hydraulic deficiency is defined by two criteria, the flow ratio (design flow/full-pipe capacity), and unit head loss. The flow ratio is often the primary criteria for establishing hydraulic deficiency, however, alone, it does not account for hydraulic efficiency gained as a function of increasing pipe diameter. Pipes of larger diameter will surcharge to a lesser extent compared to a smaller pipe with the same flow ratio. For that reason, unit head loss was introduced as a secondary criterion to account for the severity of surcharge in addition to the flow ratio.

Each pipe is given a deficiency rating based on its flow ratio and unit head loss. All pipes with a flow ratio greater than 1 surcharge during the design event. These pipes were classified as low, medium, or high deficiency based on the severity of surcharge. The flow ratio limits were set based on experience and criteria used by other municipalities. Pipes with a flow ratio between 1 and 1.2 are considered low hydraulic deficiency, flow ratios from 1.2 to 1.4 are classified as medium hydraulic deficiency, and pipes with a flow ratio greater than 1.4 are considered high hydraulic deficiency. The classification limits for unit head loss were established by calculating allowable surcharge over a “typical” length of pipe. The upper limit was established by computing the unit head loss that would produce street flooding for a pipe with an average length and average freeboard conditions. The average length of pipe was found to be 220 feet and the average freeboard to be 4.2 feet. The lower limit to define the break point between low and medium deficiency was set to half of the surcharge level of the high classification. These unit head loss limits correspond to 0.009 and 0.01.

The design flow in each pipe was approximated using a graph (shown in Figure 3-1 and developed for the 1991 plan) that relates partial sub-basin area to peak flow. The hydraulic performance of each pipe was evaluated based on the unit head loss per foot of pipe. The unit head loss is an approximation of the friction slope in each pipe. Friction losses in each pipe were computed using the Darcy-Weisbach equation in conjunction with the Colebrook-White formula to calculate friction factors. This approach assumes that all pipes are flowing full and ignores all minor head losses.

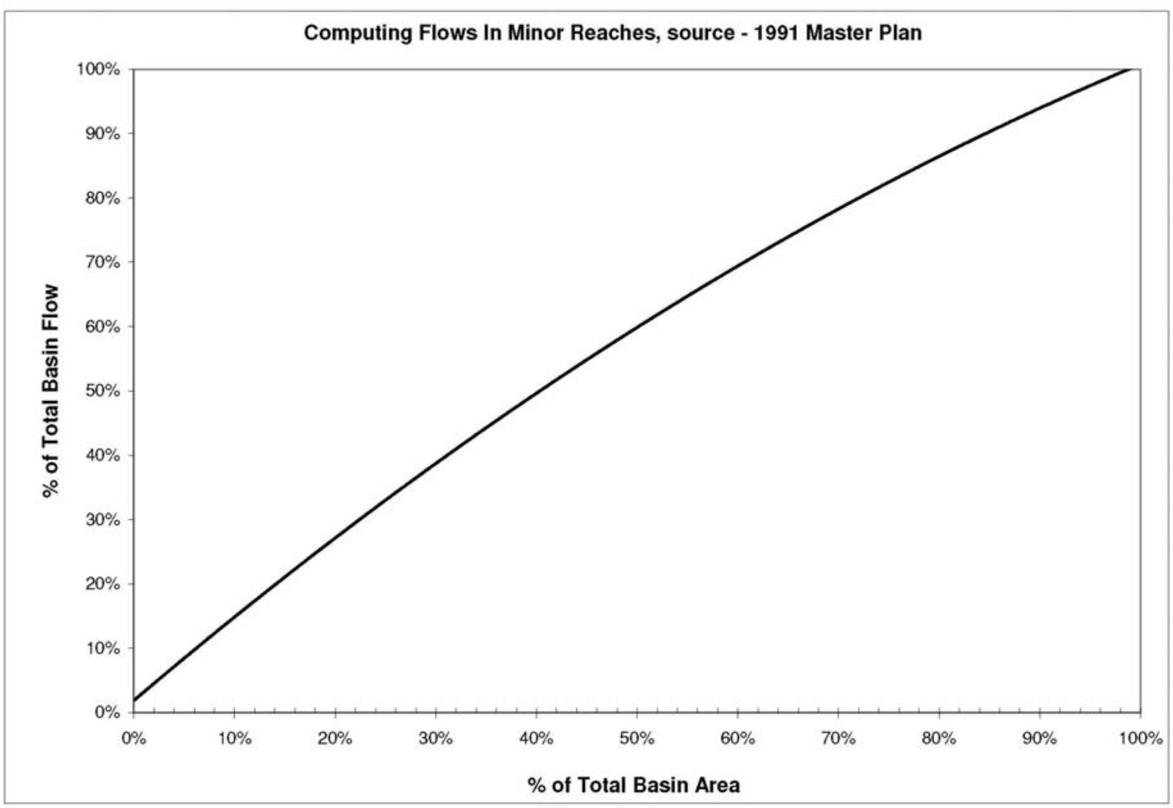

FIGURE 3-1
Computing Flows in Minor Reaches
Source: 1991 City of McMinnville Storm Drainage Master Plan
City of McMinnville Storm Drainage Master Plan

Figure 3-2 shows a plot of unit head loss and the corresponding flow ratio for each pipe evaluated in the system.

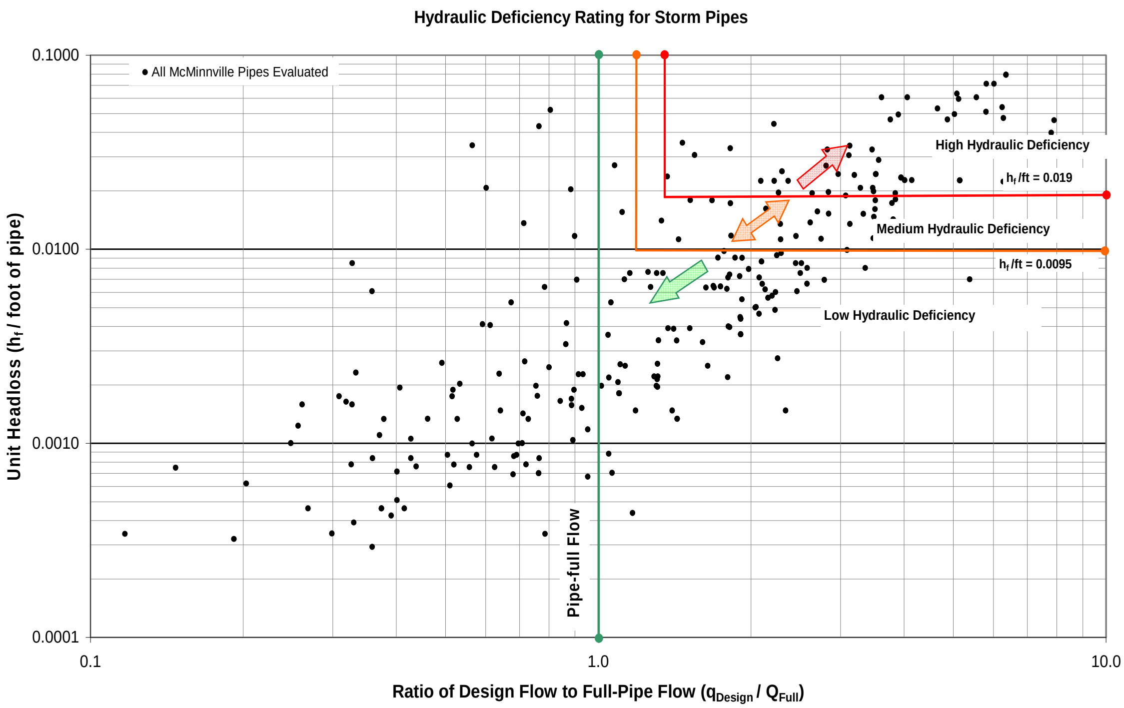

FIGURE 3-2
Hydraulic Deficiency Rating for Storm Pipes
City of McMinnville Storm Drainage Master Plan

3.4 Analysis Approach

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 in the 1991 plan based on the major tributary creeks. It was assumed that these boundaries have not changed for purposes of this update. Some areas of the current UGB were not considered within the study area in the 1991 plan. Areas on the west side of the City were included in offsite drainage basins in the 1991 plan, and are included in the updated basins as well. Areas on the eastern fringe of the City are assumed to drain directly to the Yamhill River, and were not further investigated for impacts to the existing City system. Drainage sub-basins were delineated during the 1991 master planning effort by identifying areas that could be characterized as draining to one discharge point and that were relatively uniform as to slope and land use. Sub-basins were delineated into relatively small areas (30 to 60 acres) in densely developed or unique areas and were larger in predominantly undeveloped areas. Sub-basin boundaries are approximate and may be outdated in areas where recent land development has occurred. As development projects occur, there may be cases in which a land development grading or drainage plan alters the sub-basin boundaries, diverting significant flows into another sub-basin, or, less likely, a major basin. Site designers should review current drainage patterns and update the basin boundaries and other parameters to reflect the most current conditions.

The design rainfall event for input into the HEC-HMS model was determined using the NOAA Atlas and the NRCS Type-1A storm distribution. (See Section 5, Rainfall Analysis.)

The calibrated HEC-HMS model was run with the NRCS Type-1A storm to determine the "free-flowing" peak flows for both individual sub-basin flow and for cumulative drainageway flows. These flows were estimated for a variety of return intervals and for both existing and future levels of development. (See Section 6, Runoff Analysis.)

The peak drainageway flows estimated in Section 6 were input to the hydraulic models created for North Cozine, West Cozine, and Cozine Creek. Return frequencies from 10 to 100 years were considered for the major culverted crossings. The water surface profiles produced by the hydraulic model were then analyzed to assess whether the existing culverts are undersized and leading to potential flooding problems. A dynamic evaluation of Cozine Creek was also performed to evaluate the effects of storage on the expected peak water surface elevations (see Section 7, Hydraulic Analysis Results and Drainage Improvements).

Updated flows from the HEC-HMS model were input to the pipe hydraulics spreadsheet. The design return interval for piped systems was 10 years.

A hydraulic deficiency rating was assigned to each pipe based on the unit head loss criteria shown in Figure 3-2.

An approach was developed for initially screening pipes based on hydraulic criteria and then further evaluating those pipes base on other risk factors. The approach was to identify all the evaluated pipes that may have hydraulic capacity problems, classify them by severity, and review other potential risk factors to determine the CIP priority ranking for each pipe and determine if a CIP project is warranted.