CITY OF PHILOMATH
Storm Drainage System Master Plan

Section 4
HYDROLOGIC/HYDRAULIC ANALYSIS

CHAPTER 4
DRAINAGE SYSTEM ANALYSIS

4.1 Hydrology Analysis Procedure

a. Modeling Methodology

The purpose of the drainage system capacity evaluation was to identify elements of the existing drainage system that cannot accommodate current and/or projected future storm water flows. The calculation of peak flows and runoff volumes within the drainage basins is essential to any storm drainage master planning effort. Peak flows are used to size ditches, culverts and pipe systems during the design process for new facilities. The evaluation and calculation of peak flows was accomplished using a mathematical simulation computer model. The methodology used develop and model existing and future peak stormwater flow conditions was the Rational Method.

The Rational Method was selected primarily because of the relative ease with which it can be applied, its general acceptance by the engineering community, and its reliable results. For the large undeveloped basins north of the City, the methodology outlined in the "Magnitude and Frequency of Floods in Western Oregon" (reference 5) as contained in the ODOT Hydraulics Manual, was used to verify the design flows generated by the Rational Method. There are several other methods of runoff estimation, such as the unit hydrograph, the Storm Water Management Model (SWMM), and the Hydrologic Engineering Center (HEC) computer models. These methods rely upon measurable rainfall/runoff relationships and are more applicable to larger drainage areas (>1 square mile) where timing and storage of storm runoff may be of greater importance. When properly applied to drainage areas of 200 acres or less, the Rational Method provides reliable results.

The Rational Method is based on the formula: $Q=CIA$

where: Q = the runoff rate, cubic feet per second
C= the runoff coefficient, determined by land use
A = the contributing drainage area, acres
I = the rainfall intensity, inches per hour

The basic assumptions for application of the Rational Method are as outlined below, and typically result in a conservative but realistic results.

- The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the Time of Concentration (T_c) to that point.

- The maximum rate of rainfall occurs during the time of concentration, and the design rainfall depth during the time of concentration is converted to the average rainfall intensity for the time of concentration.

- The maximum runoff rate occurs when the entire area is contributing flow (ie. at the Time of Concentration).

b. Runoff Coefficients and Land Use

The runoff coefficient "C" represents the ratio of runoff to rainfall. Of all of the variables used in computation of stormwater runoff, the runoff coefficient is the most difficult to estimate because it represents the interaction of many complex factors including surface ponding, infiltration, antecedent moisture (soil saturation at beginning of storm), ground cover conditions, ground slopes, and soil type. To simplify the determination of this coefficient, the use of average values has been adopted as standard practice in the engineering profession.

As part of the evaluation process, two runoff coefficients (existing and future conditions) were determined for each drainage basin area. The assumed future conditions are based on buildout under the land use zoning as set forth in the City of Philomath and Benton County Comprehensive Plans as reflected by current zoning maps. **Table 4-1** shows the runoff coefficients used for this study based on type of development and land use zoning.

TABLE 4-1
RUNOFF COEFFICIENTS

Soil Cover or Land Use Category	Flat Terrain S<2%	Rolling Terrain 2%<S<10%	Steep Terrain S>10%
Cultivated Land	0.30	0.35	0.40
Parks & Cemeteries	0.15	0.20	0.30
Woodlands & Forests	0.10	0.15	0.20
Meadows & Pasture Land	0.25	0.30	0.35
1) Low density residential	0.40	0.45	0.50
2) Medium density residential	0.50	0.55	0.60
3) High density (multi-family) residential	0.70	0.75	0.80
4) Gravel parking lots	0.50	0.55	0.60
5) Mobile home parks	0.60	0.65	0.70
Commercial	0.50 - 0.90		
Light Industrial	0.70		
Heavy Industrial	0.80		
Highly impermeable (roofs and paved areas)	0.90		

The runoff coefficients were used to establish the stormwater runoff under future buildout conditions. Since some areas of the City have not yet been developed, a second, or existing condition runoff coefficient was determined. The existing runoff coefficient is based on recent aerial photos and field observation. Once drainage area boundaries were established, these boundaries were overlain on the aerial photos. A visual estimate of the existing land use, percentage of development and percentage of impervious area was made in order to establish existing drainage conditions.

Where a drainage area is a combination of the runoff characteristics listed, a weighted coefficient for the total drainage area is computed by dividing the summation of the products of the individual areas and their coefficients by the total area:

$$Weighted.C = \frac{\Sigma C_i A_i}{A_T}$$

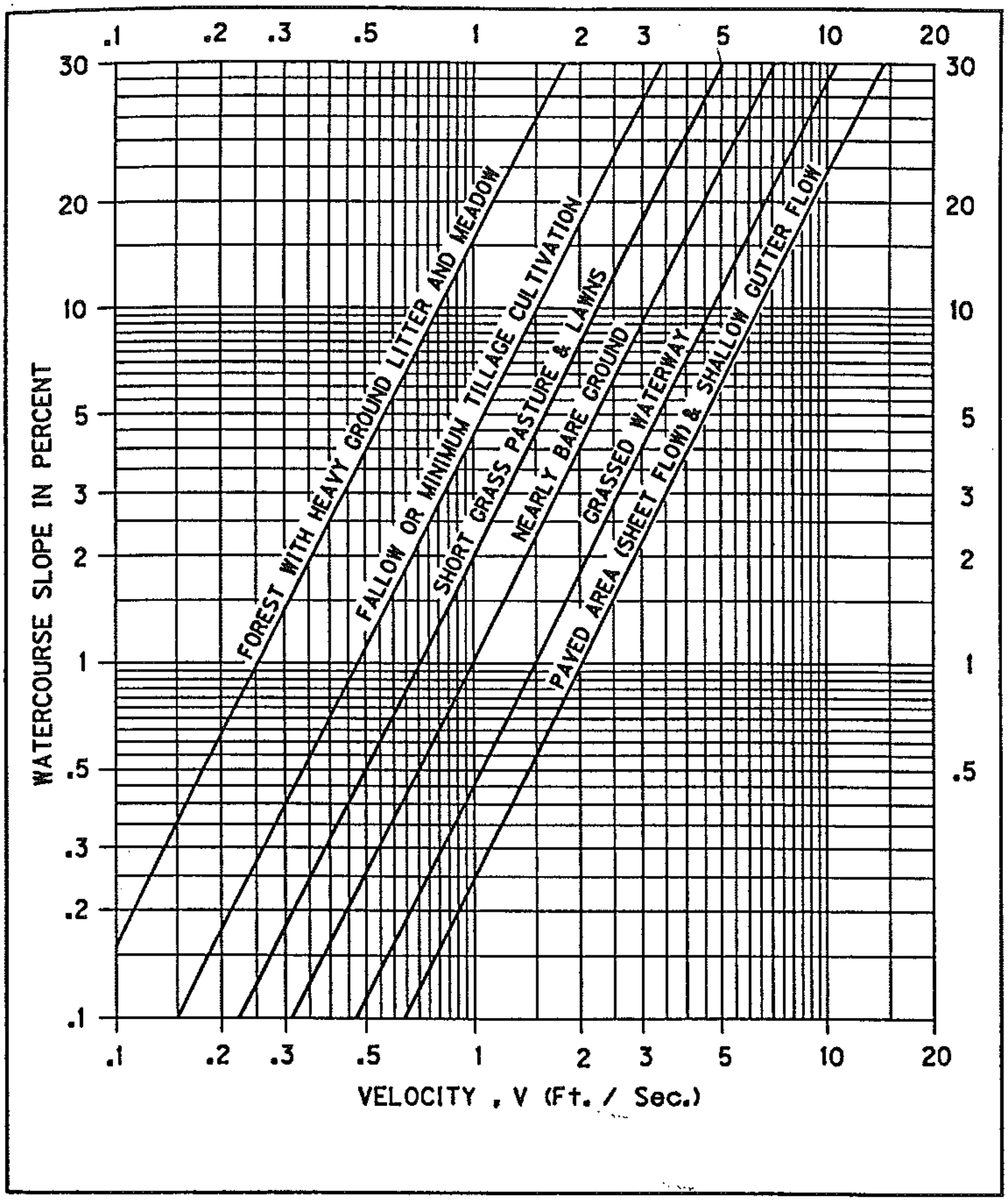
where: C_i = runoff coefficient for each sub-area
A_i = contributing drainage area for each sub-area
A_T = total area

c. Time of Concentration

In order to calculate the peak rate of runoff at any point, it is necessary to know the time of concentration to that point. The Time of Concentration, T_c, is defined as the time it takes for runoff to travel from the hydraulically most distant point in the drainage basin to the point of reference downstream. This time must be known in order to determine the rainfall intensity of a given recurrence interval storm. Most drainage basins will consist of overland flow segments as well as channel or piped flow segments. The travel time is computed for each flow segment and the time of concentration is equal to the sum of the segment travel times.

The time of concentration for the surface flow segments is a function of the surface slope, soil conditions, depression storage, surface cover, antecedent rainfall, and the distance of the surface flow. **Figure 4-1** (taken from the Oregon Department of Transportation, Highway Division, Drainage Design Manual) was used to establish surface velocities for each drainage basin of significant length. The time of concentration in a pipe system is calculated by dividing the known length by a computed velocity.

Common practice is to assume a minimum time of concentration between 10 and 30 minutes. For this planning effort a minimum time of concentration of 10 minutes has been assumed when computing the runoff contribution for each subbasin. If the computed time of concentration was less than 10 minutes, the 10 minute minimum was applied to that area. However, if the computed time of concentration was longer than 10 minutes, the actual computed time was used. This assumption is consistent with previous drainage planning for within the City and the Oregon Department of Transportation, Highway Division, Drainage Design Manual.



(Source: Hydraulics Manual, Oregon Department of Transportation, Highway Division, January 1990)

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SURFACE VELOCITY VERSUS SLOPE

FIGURE 4-1

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d. Design Storm Frequency

The selection of the design storm requires the determination of the degree of protection desired from the storm drainage system. A design storm with a low probability of being exceeded, such as the 100-year design storm (1% chance of being exceeded any given year), provides a high degree of safety in the drainage system design. However, the cost of such a system is relatively high compared to a system based on a design storm with a high exceedance probability. On the other hand, a system designed for a 2-year storm (50% chance of being exceeded any given year) will result in a lower cost drainage system whose capacity will be exceeded every few years, with possible property damage, public inconvenience and personal hazard.

For large projects involving construction in or near floodplains, the analysis can be as complex as a benefit-cost comparison where the incremental cost of protection (ie. the cost of conveying that quantity of stormwater which causes a known rise in floodwater elevation) is compared to the expected cost of damage for each additional incremental rise in floodwater elevation. Studies such as these involve determining the average home/property values at given elevations and detailed hydrologic/hydraulic analysis to determine flood elevations for each quantity of excess flow. Since the scope of this study is limited to conveying runoff through the City to the point of discharge south of the City, analysis of impacts within or reliability of the existing established flood plain are not included.

To determine a design storm for drainage planning purposes, the following factors must be considered:

- The cost of the additional level of protection (ie. sizing system to convey a larger storm)
- The size of the drainage basin
- The extent of probable property damage if the system fails
- The availability of storage within the pipe system.

The size of the drainage area has a dramatic impact on the recommended level of protection. As the size of the drainage area increases, so does the total amount of runoff. As previously noted, design standards typically require that as the storm channel or pipe gets larger, it must be designed to convey the flow from a more intense storm event due to the increased risk of property damage should the system fail.

For illustrative purposes, consider that if a small local system overflows, the likelihood of significant property damage is relatively small, while failure of the major systems can result in significant damage to property. Conversely, if the drainage facilities of a large drainage basin (such as that draining Newton Creek, with ±50 times the flow of smaller basins) is undersized by as little as 10%, those excess flows will be five times greater than the entire flow through the small basin, and may produce serious flooding damage.

Finally, in developing the computer model, gravity flow conditions were assumed for most cases. This assumption provides a conservative design since it does not take into account the ability of a pressure head to develop within a pipe under surcharged conditions and thereby increase flow, and allows for temporary storage within the pipe network.

In consideration of these factors, **Table 4-2** outlines the design storm frequencies utilized for this report. This level of protection is consistent with other Cities in the Willamette Valley.

TABLE 4-2
DESIGN STORM FREQUENCY

AREA	FREQUENCY
Residential areas	10-year storm
Commercial and high value districts	10-year storm
Trunk lines (18" pipe and larger)	25-year storm
Minor creeks and drainage ways (not shown as a flood plain on the Flood Insurance Rate Map (FIRM))	50-year storm
Major creeks (shown as a flood plain on the FIRM)	100-year storm

e. Intensity-Duration-Frequency Curve

The intensity-duration-frequency (IDF) curve is used to determine the rainfall intensity "I". Given a time of concentration and a selected design storm frequency, the rainfall intensity is found graphically or from the tabular data in the City Design Standards. The City of Philomath is located in Zone 8 per the Oregon State Highway Department Drainage Design Manual. The rainfall intensity-duration curve for Zone 8 is included in Section 3.1 of the draft Public Works Design Standards (PWDS, Appendix C).

4.2 Hydraulic Analysis

a. Open Channel Flow - Manning's Formula

Most pipes within the storm drainage system were assumed to be flowing full under open channel flow conditions. In most areas of Philomath, the storm system is flat enough that significant surcharge cannot be developed at most inlets and therefore this is a reasonable and conservative assumption. The formula used to evaluate pipes under these circumstances is the Manning Formula, which is expressed as:

$$Q = 1.49/n \times A \times R^{2/3} \times S^{1/2}$$

where: Q = flow, cubic feet per second
A = cross-sectional area, square feet
R = hydraulic radius, feet
S = slope, feet/feet
n = Manning roughness coefficient

The roughness factor for pipes varies according to the material used and the age of the pipe material. For this planning effort, a minimum "n" value of 0.013 shall be used in Manning's formula for the design of all storm pipes regardless of pipe material. In theory, new PVC sewers have manufacturer's "n" value of as low as 0.009. However, sand and grit as well as slime buildup on the pipe walls over time tend to render a true "n" value of 0.013. Hence, an "n" value of less than 0.013 for smooth wall pipe is not recommended for design purposes. An "n" value of 0.024 for corrugated pipes was used and is recommended for design purposes (PWDS 3.15).

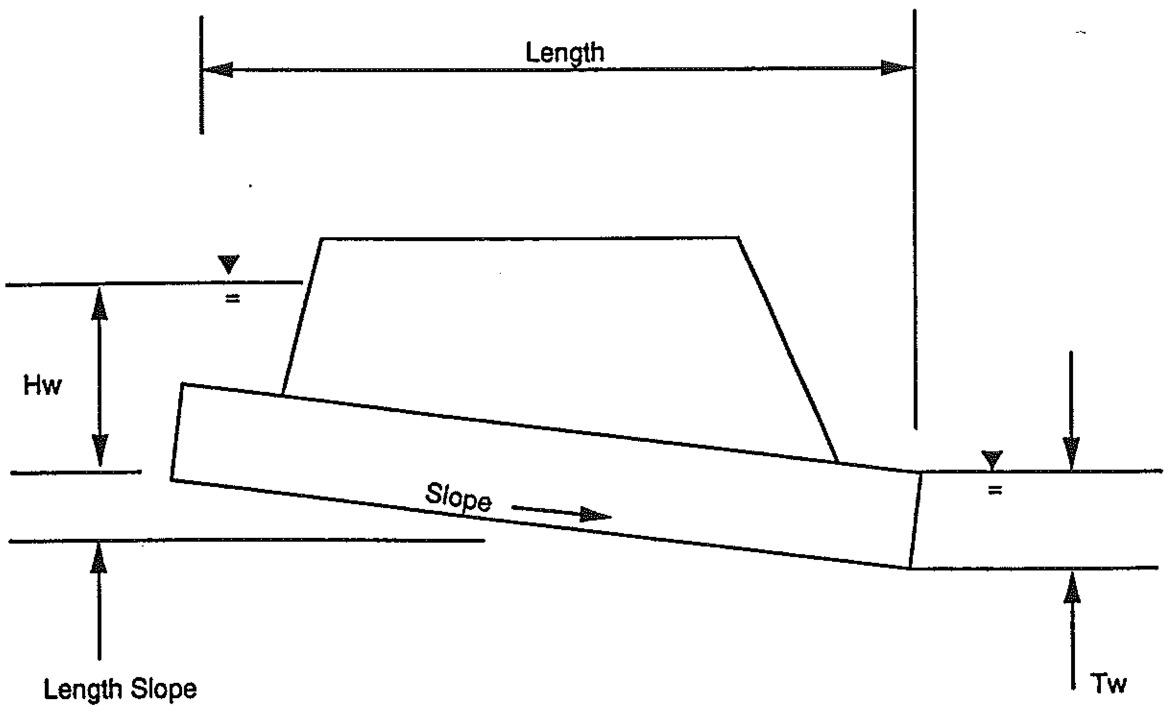
b. Surcharged Culvert Flow

Where conditions allow the build up of a surcharge, or head, and the pipe length is relatively long and the slope is gentle, the pipe is assumed to be a culvert flowing under outlet control conditions. The tailwater (TW) depth is assumed to be at the top of pipe at the outlet. Losses due to velocity, bends and junctions were ignored. This condition is presented graphically in **Figure 4-2**.

Assuming tailwater controlled flow and assuming entrance, exit, bend and junction losses are negligible, then the friction loss (H_f) through the culvert becomes:

$$H_f = HW_{elev} - Tw_{elev}$$

where: H_f = friction loss through culvert
HW_{elev} = Headwater elevation
Tw_{elev} = Tailwater elevation



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SURCHARGED CULVERT FLOW

FIGURE 4-2

JOB NUMBER 960.501.0

In cases where these elevations were not known, assumptions based on pipe size were made to facilitate completion of the study. The assumed available head is as follows:

Pipe Size	Available Head (HW_{elev} - TW_{elev})
< 18"	1.5 feet
18"-36"	2.0 feet
> 36"	4.0 feet

4.3 Computed Stormwater Flows for Future Conditions (Buildout)

Based on existing land use zoning, a spreadsheet-based computer model was developed following a field inventory of the existing drainage system. A letter designation was applied to each inlet, outlet or junction point. Each pipe segment was assigned a numeric identifier. Subdrainage basins to each individual inlet were determined using existing aerial mapping, and runoff conditions were assessed. Based on previous discussions, time of concentrations were determined to key points within each basin (typically the lower end of the basin). From this information, a rainfall intensity was found for the design storm event. Flows at junctions were summed and carried forward to the next drainage segment. Physical data describing each pipe or channel segment was input and used to calculate capacity based on the assumed flow conditions (ie. open channel or submerged inlet). The cumulative flow within each pipe was subtracted from pipe capacity and is displayed in the last column of the spreadsheet. A negative number, therefore, indicates the amount a pipe is undersized based on conditions that will exist if all areas within the UGB were developed according to zoning restrictions. These calculations are presented in Appendix D, Computations for Future Conditions.

4.4 Computed Stormwater Flows for Existing Conditions

For those areas of the City which have not yet been developed, a similar methodology was utilized to establish the peak flows in the major trunk storm lines under current conditions. These calculations are presented in Appendix E, Computations for Existing Conditions.