Chapter 9 Conveyance System Design and Hydraulic Analysis

This chapter presents acceptable methods for the analysis and design of storm and stormwater conveyance systems. Conveyance systems can be separated into the following categories:

- Pipe systems
- Culverts
- Open Channels (ditches, swales)
- Outfalls

The purpose of a conveyance system is to drain stormwater, up to a specific design flow, from properties so as to provide protection to property and the environment. This chapter contains detailed design criteria, methods of analysis and standard details for all components of a conveyance system. A complete basic understanding of hydrology and hydraulics and the principles on which the methodology of hydrologic analysis is based is essential for the proper and accurate application of methods used in designing conveyance systems.

9.1 Design Methods for Conveyance Systems

New conveyance systems shall be designed using the backwater analysis per Section 9.3.2. Existing conveyance systems shall be analyzed for capacity using either the backwater analysis per Section 9.3.2 or the Rational Method per Section 9.3.3.

Preliminary conveyance system sizing for land use planning (short plats, plats, wetland development permits, etc.) can use the Rational Method per Section 9.3.3 or the backwater analysis per Section 9.3.2. The Rational Method cannot be used for the final design of new conveyance systems.

All conveyance system analyses shall use the design criteria outlined in Section 9.1.1 and other applicable sections of the SWMM.

9.1.1 Design Criteria

- Conveyance systems shall be modeled as if no onsite detention is provided upstream.
- Conveyance systems shall be analyzed for fully developed conditions. The fully developed conditions shall be derived from the following percentages of impervious area:
  - In commercial areas, the percent impervious shall be 85%.
  - In industrial areas, the percent impervious shall be 70%.
  - In residential areas, the percent impervious shall be 60%.
- The fully developed conditions shall apply to both the offsite and onsite basins.
- Projects proposed in areas subject to tidal influence shall be analyzed at the mean high tide, which is +4.64 feet using current City datum.
9.2 Methods of Analysis

9.2.1 Onsite Analysis

All proposed onsite stormwater conveyance systems and all existing onsite stormwater conveyance systems that will be accepting additional flow shall be sized per Section 9.1.

9.2.2 Offsite Analysis

9.2.2.1 Qualitative Downstream Analysis

All project applicants shall perform and submit a qualitative downstream analysis of each upstream system entering a site (run-on) and each downstream system leaving a site (run-off).

The qualitative downstream analysis shall extend downstream for the entire flow path, from the project site to the receiving water, or up to one-quarter mile, whichever is less. The qualitative analysis may be stopped shorter than the required ¼ mile downstream if the analysis reaches a City identified trunk main. Trunk mains are defined as public stormwater drainage pipes equal to or greater than 36 inches and installed at a minimum slope of 0.5%.

The upstream qualitative analysis shall identify and describe points where water enters the site and the tributary area that contributes water to those run-on locations.

A basin map delineating the onsite and offsite basins upstream and downstream of the site shall be provided. The basin map shall be to a defined scale. Maps printed from the City’s govMe website may be used as a base for the basin map, and to obtain contours and existing stormwater facility information. Field verification of govMe information may be required as directed by Environmental Services.

Depending upon the presence of existing or predicted flooding, erosion, or water quality problems, and on the proposed design of the onsite drainage facilities, the City may require additional qualitative analysis further downstream, mitigation measures adequate to address the problems, or a quantitative analysis. See Volume 1, Chapter 4 for additional details.

9.2.2.2 Quantitative Downstream Analysis

A quantitative downstream analysis is required for all projects that meet the flow control thresholds as described in Minimum Requirement #7, except those required to provide detention per the Standard Requirement. The City may also require a quantitative downstream analysis for any project where additional downstream information is determined to be necessary. The quantitative analysis shall extend downstream for the entire flow path, from the project site to the receiving water, or up to one-quarter mile, whichever is less. The quantitative analysis may be stopped shorter than the required ¼ mile downstream if the analysis reaches a City identified trunk main. Trunk mains are defined as public stormwater drainage pipes equal to or greater than 36 inches and installed at a minimum slope of 0.5%. All existing and proposed offsite stormwater conveyance shall meet the design criteria described in the methods of analyses.

If a capacity problem or streambank erosion problem is found during the quantitative downstream analysis, the flow durations from the project will be restricted per Minimum Requirement #7 – Infrastructure Protection.

The City govMe mapping program and as-built drawings may be utilized to obtain structure information to be used in the downstream analysis. If as-built drawings are used, the engineer is responsible for verifying that all elevations are in the same datum. Environmental Services may require a field survey of the existing storm drainage system downstream from the project for a
minimum of ¼ mile from the point of connection to the existing public drainage system, or may require portions of the system to be field surveyed.

Include the following as part of the quantitative downstream analysis:

- Capacity and percent full in each reach.
- Description of design flows used in analysis.
- Velocity in each reach.
- Upstream and downstream basin maps showing the flow route for both onsite and offsite stormwater.
- Include all model assumptions, outputs, and equations used in the analysis. If model parameters are used that are different than typical standards of practice, justification of the parameters is required.
- Clearly describe headwater and tailwater assumptions.
- The 25-year and 100-year hydraulic gradelines must be shown.

9.3 Hydrologic Modeling

Single event modeling shall be used for designing conveyance systems. Continuous simulation modeling is not accepted.

9.3.1 Computer Models for Conveyance Systems

All components of a conveyance system can be modeled using readily available computer models. There are several acceptable computer models available for these analyses. The project engineer is responsible for providing information describing how the model was used, assumptions the model makes and descriptions of all variables, columns, rows, summary tables, and graphs. The project engineer shall use the most current version of any model proposed within 1 year of the version’s release. Tacoma may determine that specific models are not acceptable for use in design. Please check with Environmental Services to confirm the applicability of a particular model prior to starting the analysis.

9.3.2 Backwater Analysis

The backwater analysis is used to compute a simple backwater profile (hydraulic grade line) for a proposed or existing conveyance system for the purposes of verifying capacity.

The backwater analysis begins at the downstream end of the conveyance system, when the analysis reaches a trunk main, or at the last downstream manhole of a ¼ mile analysis and is computed back through each upstream pipe segment and structure.

For discharges to tidally influenced areas, the tailwater elevation shall be the mean high tide which is 4.64 feet using current City of Tacoma datum.

9.3.2.1 Pipe Systems

All new conveyance systems shall be designed using the backwater analysis. The design events for pipe systems are as follows.

- For the 25-year, 24-hour design storm, there shall be a minimum of 0.5 feet of freeboard between the water surface and the top of any manhole or catch basin.
For the 100-year, 24-hour design storm, overtopping of the pipe conveyance system may occur, however, the additional flow shall not extend beyond half the lane width of the outside lane of the traveled way and shall not exceed 4 inches in depth at its deepest point.

For the 100-year, 24-hour design storm, off-channel storage on private property is allowed with recording of the proper easements. When this occurs, the additional flow over the ground surface is analyzed using the methods for open channels described in Section 9.3.2.3. Environmental Services will evaluate and determine the acceptability of this type of localized flooding.

The starting tailwater elevation to be used in the backwater analysis for pipe systems is the water surface elevation of the next downstream pipe at an assumed depth of 90% full.

For discharges to tidally influenced areas, the tailwater elevation shall be the mean high tide which is 4.64 feet using current City of Tacoma datum.

### 9.3.2.2 Culverts

All new conveyance systems shall be designed using the backwater analysis. The design events for culverts systems are as follows:

- Culverts shall convey the 25-year, 24-hour peak flow rate without submerging the culvert inlet.
- For culverts 18-inch diameter or less, the maximum allowable headwater elevation for the 100-year, 24-hour design storm (measured from the inlet invert) shall not exceed 2 times the pipe diameter or arch-culvert-rise.
- For culverts larger than 18-inch diameter, the maximum allowable headwater elevation for the 100-year, 24-hour design storm (measured from the inlet invert) shall not exceed 1.5 times the pipe diameter or arch-culvert-rise.
- The maximum headwater elevation at the 100-year, 24-hour design flow shall be below any road or parking lot subgrade except as allowed per Chapter 8.

The starting tailwater elevation to be used in the backwater analysis for culverts systems is the water surface elevation of the next downstream culvert at an assumed depth of 100% full.

For discharges to tidally influenced areas, the tailwater elevation shall be the mean high tide which is 4.64 feet using current City of Tacoma datum.

### 9.3.2.3 Open Channels

All new conveyance systems shall be designed using the backwater analysis. The design event for open channels is as follows:

- Constructed and natural channels shall contain the 100-year, 24-hour storm event.
- A minimum of 6 inches of freeboard is required for all open channels.

The Direct Step Backwater Method can be used to compute backwater profiles on prismatic channel reaches (i.e. reaches having uniform cross section and slope) where a backwater condition or restriction to normal flow is known to exist. The method can be applied to a series of prismatic channel reaches in succession beginning at the downstream end of the channel and computing the profile upstream.

Calculating the coordinates of the water surface profile using the method is an iterative process achieved by choosing a range of flow depths, beginning at the downstream end, and proceeding incrementally up to the point of interest or to the point of normal flow depth. This is best
accomplished by the use of a table or computer programs. Provide documentation of the equations and assumptions used in the analysis.

The Standard Step Backwater Method is a variation of the Direct Step Backwater Method and can be used to compute backwater profiles on both prismatic and non-prismatic channels. In this method, stations are established along the channel where cross section data is known or has been determined through field survey. The computation is carried out in steps from station to station rather than throughout a given channel reach as is done in the Direct Step method. As a result, the analysis involves significantly more trial-and-error calculation in order to determine the flow depth at each station.

9.3.3 Rational Method

The Rational Method is a simple, conservative method for analyzing and sizing conveyance elements serving small drainage sub-basins, subject to the following specific limitations:

- Only for use in predicting peak flow rates for sizing conveyance elements (not for use in sizing flow control or treatment facilities)
- Drainage sub-basin area, \( A \), cannot exceed 10 acres for a single peak flow calculation
- The time of concentration, \( T_c \), must be computed using the method described below and cannot exceed 100 minutes. A minimum \( T_c \) of 6.3 minutes shall be used.
- Unlike other methods of computing times of concentration, the 6.3 minutes is not an initial collection time to be added to the total computed time of concentration.

9.3.3.1 Rational Method Design Events

Pipe Systems

The design events for pipe systems using the rational method shall be:

- All public pipe systems shall be designed to convey the 25-year, 24-hour peak flow rates at 90% full.
- All private pipe systems less than 24 inches in diameter shall be designed to convey the 10-year, 24-hour peak flow rates at 90% full except as allowed per Chapter 8.
- All private pipe systems greater than or equal to 24 inches in diameter shall be designed to convey the 25-year, 24-hour peak flow rates at 90% full except as allowed per Chapter 8.
- For projects in areas subject to tidal influence, the 100-year, 24-hour storm event shall be analyzed and if flooding occurs, the flooding must be mapped. Flooding on private property may be allowed with recording of proper easements. Environmental Services will evaluate and determine the acceptability of this type of localized flooding.

Culverts

- The design events for culverts using the rational method shall be:
- Culverts shall convey the 25-year, 24-hour peak flow rate without submerging the culvert inlet.
- For culverts 18-inch diameter or less, the maximum allowable headwater elevation for the 100-year, 24-hour design storm (measured from the inlet invert) shall not exceed 2 times the pipe diameter or arch-culvert-rise.
• For culverts larger than 18-inch diameter, the maximum allowable headwater elevation for the 100-year, 24-hour design storm (measured from the inlet invert) shall not exceed 1.5 times the pipe diameter or arch-culvert-rise.

• The maximum headwater elevation at the 100-year, 24-hour design flow shall be below any road or parking lot subgrade except as allowed per Chapter 8.

**Open Channels**

The design event for open channels using the rational method shall be:

• Constructed and natural channels shall contain the 100-year, 24-hour storm event.

• A minimum of 6 inches of freeboard is required for all open channels at the 100-year, 24-hour storm event.

**9.3.3.2 Rational Method Equation**

The following is the traditional Rational Method equation:

\[ Q_R = C I_R A \]  

Where:

- \( Q_R \) = peak flow (cfs) for a storm of return frequency \( R \)
- \( C \) = estimated runoff coefficient (ratio of rainfall that becomes runoff)
- \( I_R \) = peak rainfall intensity (inches/hour) for a storm of return frequency \( R \)
- \( A \) = drainage sub-basin area (acres)

When the composite runoff coefficient, \( C_C \) (see equation 2) of a drainage basin exceeds 0.60, the \( T_C \) and peak flow rate from the impervious area should be computed separately. The computed peak rate of flow for the impervious surface alone may exceed that for the entire drainage basin using the value at \( T_C \) for the total drainage basin. The higher of the two peak flow rates shall then be used to size the conveyance element.

**“C” Values**

The allowable runoff coefficients to be used in this method are shown in Table 3 - 13 by type of land cover. These values were selected following a review of the values previously accepted by Tacoma for use in the Rational Method and as described in several engineering handbooks. The value for single family residential areas were computed as composite values (as illustrated in the following equation) based on the estimated percentage of coverage by roads, roofs, yards, and unimproved areas for each density. For drainage basins containing several land cover types, the following formula may be used to compute a composite runoff coefficient, \( C_C \):

\[ C_C = (C_1A_1+C_2A_2+\ldots+C_nA_n)/A_t \]  

Where:

- \( A_t \) = total area (acres)
- \( A_{1,2\ldots n} \) = areas of land cover types (acres)
C_{1,2,...n} = runoff coefficients for each area land cover type

### Table 3 - 13: Runoff Coefficients - “C” Values for the Rational Method

<table>
<thead>
<tr>
<th>Land Cover</th>
<th>C</th>
<th>Land Cover</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dense forest</td>
<td>0.10</td>
<td>Gravel areas</td>
<td>0.80</td>
</tr>
<tr>
<td>Light forest</td>
<td>0.15</td>
<td>Pavement and roofs</td>
<td>0.90</td>
</tr>
<tr>
<td>Pasture</td>
<td>0.20</td>
<td>Open water (pond, lakes, wetlands)</td>
<td>1.00</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### SINGLE FAMILY RESIDENTIAL AREAS

[Density is in dwelling units per gross acreage (DU/GA)]

<table>
<thead>
<tr>
<th>Land Cover Density</th>
<th>C</th>
<th>Land Cover Density</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20 DU/GA (1 unit per 5 ac.)</td>
<td>0.17</td>
<td>3.00 DU/GA</td>
<td>0.42</td>
</tr>
<tr>
<td>0.40 DU/GA (1 unit per 2.5 ac.)</td>
<td>0.20</td>
<td>3.50 DU/GA</td>
<td>0.45</td>
</tr>
<tr>
<td>0.80 DU/GA (1 unit per 1.25 ac.)</td>
<td>0.27</td>
<td>4.00 DU/GA</td>
<td>0.48</td>
</tr>
<tr>
<td>1.00 DU/GA</td>
<td>0.30</td>
<td>4.50 DU/GA</td>
<td>0.51</td>
</tr>
<tr>
<td>1.50 DU/GA</td>
<td>0.33</td>
<td>5.00 DU/GA</td>
<td>0.54</td>
</tr>
<tr>
<td>2.00 DU/GA</td>
<td>0.36</td>
<td>5.50 DU/GA</td>
<td>0.57</td>
</tr>
<tr>
<td>2.50 DU/GA</td>
<td>0.39</td>
<td>6.00 DU/GA</td>
<td>0.60</td>
</tr>
</tbody>
</table>

**“I_R” Peak Rainfall Intensity**

The peak rainfall intensity, \( I_R \), for the specified design storm of return frequency \( R \) is determined using a unit peak rainfall intensity factor, \( i_R \), in the following equation:

\[
I_R = (P_R)(i_R) \quad \text{(equation 3)}
\]

Where:
- \( P_R \) = the total precipitation at the project site for the 24-hour duration storm event for the given return frequency. Refer to Table 3 - 14 for \( P_R \) values. Total precipitation can also be found in Chapter 1 of Volume 3.
- \( i_R \) = the unit peak rainfall intensity factor

The unit peak rainfall intensity factor, \( i_R \), is determined by the following equation:

\[
i_R = (a_R)(T_c)^{-b_R} \quad \text{(equation 4)}
\]

Where:
- \( T_c \) = time of concentration (minutes), calculated using the method described below and subject to equation limitations (6.3 < \( T_c \) < 100)
- \( a_R, b_R \) = coefficients from Table 3 - 14 used to adjust the equation for the design storm return frequency \( R \)
Table 3 - 15 includes a table of rainfall intensity as a function of time of concentration, calculated using the coefficients from Table 3 - 14.

**Table 3 - 14: Coefficients for the Rational Method**

<table>
<thead>
<tr>
<th>Design Storm Frequency</th>
<th>P_R (inches)</th>
<th>a_R</th>
<th>b_R</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 years</td>
<td>2.0</td>
<td>1.58</td>
<td>0.58</td>
</tr>
<tr>
<td>5 years</td>
<td>2.5</td>
<td>2.33</td>
<td>0.63</td>
</tr>
<tr>
<td>10 years</td>
<td>3.0</td>
<td>2.44</td>
<td>0.64</td>
</tr>
<tr>
<td>25 years</td>
<td>3.5</td>
<td>2.66</td>
<td>0.65</td>
</tr>
<tr>
<td>50 years</td>
<td>3.5</td>
<td>2.75</td>
<td>0.65</td>
</tr>
<tr>
<td>100 years</td>
<td>4.1</td>
<td>2.61</td>
<td>0.63</td>
</tr>
</tbody>
</table>
Table 3 - 15: Rainfall Intensities ($I_P$) (inches per hour) for the City of Tacoma

<table>
<thead>
<tr>
<th>Time of Concentration (min)</th>
<th>Design Storm Recurrence Interval (Probability)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.3</td>
<td>2-year (50%) 1.09 5-year (20%) 1.83 10-year (10%) 2.25 25-year (4%) 2.81 50-year (2%) 2.91 100-year (1%) 3.36</td>
</tr>
<tr>
<td>7</td>
<td>0.95 1.02 1.71 2.11 2.63 2.72 3.14</td>
</tr>
<tr>
<td>8</td>
<td>0.88 0.95 1.46 1.79 2.23 2.49 2.89</td>
</tr>
<tr>
<td>9</td>
<td>0.83 0.88 1.37 1.68 2.08 2.31 2.68</td>
</tr>
<tr>
<td>10</td>
<td>0.79 0.83 1.29 1.58 1.96 2.03 2.36</td>
</tr>
<tr>
<td>11</td>
<td>0.75 0.79 1.22 1.49 1.85 1.91 2.24</td>
</tr>
<tr>
<td>12</td>
<td>0.71 0.75 1.16 1.42 1.76 1.82 2.13</td>
</tr>
<tr>
<td>13</td>
<td>0.68 0.71 1.10 1.35 1.67 1.73 2.03</td>
</tr>
<tr>
<td>14</td>
<td>0.66 0.68 1.06 1.29 1.60 1.66 1.94</td>
</tr>
<tr>
<td>15</td>
<td>0.63 0.66 1.02 1.24 1.54 1.59 1.87</td>
</tr>
<tr>
<td>16</td>
<td>0.61 0.63 0.98 1.19 1.48 1.53 1.80</td>
</tr>
<tr>
<td>17</td>
<td>0.59 0.61 0.94 1.15 1.42 1.47 1.73</td>
</tr>
<tr>
<td>18</td>
<td>0.57 0.59 0.91 1.11 1.37 1.42 1.67</td>
</tr>
<tr>
<td>19</td>
<td>0.56 0.57 0.88 1.08 1.33 1.37 1.62</td>
</tr>
<tr>
<td>20</td>
<td>0.49 0.56 0.77 0.93 1.15 1.19 1.41</td>
</tr>
<tr>
<td>25</td>
<td>0.44 0.49 0.68 0.83 1.02 1.06 1.26</td>
</tr>
<tr>
<td>30</td>
<td>0.40 0.44 0.62 0.75 0.92 0.95 1.14</td>
</tr>
<tr>
<td>35</td>
<td>0.37 0.40 0.57 0.69 0.85 0.88 1.05</td>
</tr>
<tr>
<td>40</td>
<td>0.35 0.37 0.53 0.64 0.78 0.81 0.97</td>
</tr>
<tr>
<td>45</td>
<td>0.33 0.35 0.50 0.60 0.73 0.76 0.91</td>
</tr>
<tr>
<td>50</td>
<td>0.31 0.33 0.47 0.56 0.69 0.71 0.86</td>
</tr>
<tr>
<td>55</td>
<td>0.29 0.31 0.44 0.53 0.65 0.67 0.81</td>
</tr>
<tr>
<td>60</td>
<td>0.27 0.29 0.40 0.48 0.59 0.61 0.74</td>
</tr>
<tr>
<td>70</td>
<td>0.25 0.27 0.37 0.44 0.54 0.56 0.68</td>
</tr>
<tr>
<td>80</td>
<td>0.23 0.25 0.34 0.41 0.50 0.52 0.63</td>
</tr>
<tr>
<td>90</td>
<td>0.22 0.23 0.32 0.38 0.47 0.48 0.59</td>
</tr>
<tr>
<td>100</td>
<td>0.22 0.22 0.32 0.38 0.47 0.48 0.59</td>
</tr>
</tbody>
</table>
"$T_c$” Time of Concentration

The time of concentration is defined as the time it takes runoff to travel overland (from the onset of precipitation) from the most hydraulically distant location in the drainage basin to the point of discharge.

Due to the mathematical limits of the equation coefficients, values of $T_c$ less than 6.3 minutes or greater than 100 minutes cannot be used. Therefore, real values of $T_c$ less than 6.3 minutes must be assumed to be equal to 6.3 minutes, and values greater than 100 minutes must be assumed to be equal to 100 minutes.

$T_c$ is computed by summation of the travel times $T_t$ of overland flow across separate flowpath segments. The equation for time of concentration is:

$$T_c = T_1 + T_2 + \ldots + T_n$$  \hspace{1cm} (equation 5)

Where

$T_{1,2,\ldots,n} =$ travel time for consecutive flowpath segments with different categories or flowpath slope

Travel time for each segment, $t$, is computed using the following equation:

$$T_t = \frac{L}{60V}$$  \hspace{1cm} (equation 6)

Where

$T_t =$ travel time (minutes)

$L =$ the distance of flow across a given segment (feet)

$V =$ average velocity (ft/s) across the land cover

$$V = (K_R)(\sqrt{S_0})$$

Where

$k_R =$ time of concentration velocity factor; see Table 3 - 16.

$s_0 =$ slope of flowpath (feet/feet)
### Table 3 - 16: “n” and “k” Values Used in Time Calculations for Hydrographs

<table>
<thead>
<tr>
<th>“n”&lt;sub&gt;s&lt;/sub&gt; Sheet Flow Equation Manning’s Values (for the initial 300 ft. of travel)</th>
<th>n&lt;sub&gt;s&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth surfaces (concrete, asphalt, gravel, or bare hand packed soil)</td>
<td>0.011</td>
</tr>
<tr>
<td>Fallow fields or loose soil surface (no residue)</td>
<td>0.05</td>
</tr>
<tr>
<td>Cultivated soil with residue cover ≤20%</td>
<td>0.06</td>
</tr>
<tr>
<td>Cultivated soil with residue cover &gt;20%</td>
<td>0.17</td>
</tr>
<tr>
<td>Short prairie grass and lawns</td>
<td>0.15</td>
</tr>
<tr>
<td>Dense grasses</td>
<td>0.24</td>
</tr>
<tr>
<td>Bermuda grass</td>
<td>0.41</td>
</tr>
<tr>
<td>Range (natural)</td>
<td>0.13</td>
</tr>
<tr>
<td>Woods or forest with light underbrush</td>
<td>0.40</td>
</tr>
<tr>
<td>Woods or forest with dense underbrush</td>
<td>0.80</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>“k” Values Used in Travel Time/Time of Concentration Calculations&lt;sup&gt;b&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sheet Flow</strong></td>
</tr>
<tr>
<td>Forest with heavy ground litter and meadow</td>
</tr>
<tr>
<td>Fallow or minimum tillage cultivation</td>
</tr>
<tr>
<td>Short grass pasture and lawns</td>
</tr>
<tr>
<td>Nearly bare ground</td>
</tr>
<tr>
<td>Grasses waterway</td>
</tr>
<tr>
<td>Paved area (sheet flow) and shallow gutter flow</td>
</tr>
</tbody>
</table>

**Shallow Concentrated Flow (After the initial 300 ft. of sheet flow, R = 0.1)** | k<sub>s</sub> |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Forest with heavy ground litter and meadows (n = 0.10)</td>
<td>3</td>
</tr>
<tr>
<td>2. Brushy ground with some trees (n = 0.060)</td>
<td>5</td>
</tr>
<tr>
<td>3. Fallow or minimum tillage cultivation (n = 0.040)</td>
<td>8</td>
</tr>
<tr>
<td>4. High grass (n = 0.035)</td>
<td>9</td>
</tr>
<tr>
<td>5. Short grass, pasture and lawns (n = 0.030)</td>
<td>11</td>
</tr>
<tr>
<td>6. Nearly bare ground (n = 0.025)</td>
<td>13</td>
</tr>
<tr>
<td>7. Paved and gravel areas (n = 0.012)</td>
<td>27</td>
</tr>
</tbody>
</table>

**Channel Flow (intermittent) (At the beginning of visible channels R = 0.2)** | k<sub>c</sub> |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Forested swale with heavy ground litter (n = 0.10)</td>
<td>5</td>
</tr>
<tr>
<td>2. Forested drainage course/ravine with defined channel bed (n = 0.050)</td>
<td>10</td>
</tr>
<tr>
<td>3. Rock-lined waterway (n = 0.035)</td>
<td>15</td>
</tr>
<tr>
<td>4. Grassed waterway (n = 0.030)</td>
<td>17</td>
</tr>
<tr>
<td>5. Earth-lined waterway (n = 0.025)</td>
<td>20</td>
</tr>
<tr>
<td>6. CMP pipe, uniform flow (n = 0.024)</td>
<td>21</td>
</tr>
<tr>
<td>7. Concrete pipe, uniform flow (0.012)</td>
<td>42</td>
</tr>
<tr>
<td>8. Other waterways and pipe</td>
<td>0.508/n</td>
</tr>
</tbody>
</table>

**Channel Flow (Continuous stream, R = 0.4)** | k<sub>c</sub> |
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>9. Meandering stream with some pools (n = 0.040)</td>
<td>20</td>
</tr>
<tr>
<td>10. Rock-lined stream (n = 0.035)</td>
<td>23</td>
</tr>
<tr>
<td>11. Grass-lined stream (n = 0.030)</td>
<td>27</td>
</tr>
<tr>
<td>12. Other streams, man-made channels and pipe</td>
<td>0.807/n</td>
</tr>
</tbody>
</table>

---

a. See TR-55, 1986  
9.4 Inlet Grate Capacity

This section has been divided into three areas: inlets on a continuous grade, side flow interception and sag analysis. Properties of typical grate are summarized in Table 3 - 17 and further discussed below.

Table 3 - 17: Properties of Grate Inlets

<table>
<thead>
<tr>
<th>WSDOT Standard Plan</th>
<th>Description</th>
<th>Continuous Grade&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Sump Condition&lt;sup&gt;b&lt;/sup&gt; Perimeter Flows as Weir</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Grate Width</td>
<td>Grate Length</td>
</tr>
<tr>
<td>B-30.50&lt;sup&gt;c&lt;/sup&gt;</td>
<td>Rectangular herringbone grate</td>
<td>1.67 ft (0.50 m)</td>
<td>2.0 ft (0.61 m)</td>
</tr>
<tr>
<td>B-30.30 or 30.40&lt;sup&gt;d&lt;/sup&gt;</td>
<td>Vaned grate for catch basin and inlet</td>
<td>1.67 ft (0.50 m)</td>
<td>2.0 ft (0.61 m)</td>
</tr>
<tr>
<td>B-25.20&lt;sup&gt;b&lt;/sup&gt;</td>
<td>Combination inlet</td>
<td>1.67 ft (0.50 m)</td>
<td>2.0 ft (0.61 m)</td>
</tr>
<tr>
<td>B-40.20</td>
<td>Grate Inlet Type 1 (Grate A or B&lt;sup&gt;e&lt;/sup&gt;)</td>
<td>2.01 ft (0.62 m)</td>
<td>3.89 ft (0.62 m)</td>
</tr>
<tr>
<td>B-30.80</td>
<td>Circular Grate&lt;sup&gt;g&lt;/sup&gt;</td>
<td>1.52 ft (0.47 m)</td>
<td>2.01 ft (1.20 m)</td>
</tr>
<tr>
<td>B-40.40</td>
<td>Frame and Vaned Grates for Grate Inlet Type 2</td>
<td>1.75 ft (0.52 m)</td>
<td>3.52 ft (1.05 m)</td>
</tr>
</tbody>
</table>

---

9.4.1 Inlet Types

The characteristics of the most commonly used inlets are summarized below. For inlet additional specifications including dimensions see the inlet specific WSDOT Standard Plan.

9.4.1.1 Herringbone Pattern (WSDOT Standard Plan B-30.50)

Herringbone grates are not approved for installation within the City Right of Way. Herringbone pattern grate information is included for analyzing existing conditions and new construction on private property.
9.4.1.2 Vaned Grate (WSDOT Standard Plan B-30.30 or 30.40)

Catch basin grate installation within the City ROW shall use vaned type grates. The vaned grate is bicycle safe, and as described further in this section is hydraulically superior under most conditions. Installation of the vaned grate is critical as the grate is directional. If installed backwards the interception capacity is severely limited. At low velocities the vaned grate and herringbone grate are equally efficient. At higher velocities, greater than 5 ft/s (1.5 m/s), a portion of the flow tends to skip over the herringbone whereas the vaned grate will capture a greater portion of this flow. The vaned grate also has a higher capacity for passing debris and should be used for high debris areas.

9.4.1.3 Combination Inlets (WSDOT Standard Plan B-25.20)

The combination inlet is a vaned grate on a catch basin with a hooded curb cut area. Its vaned grate is very debris efficient, and if the grate does become clogged, the overflow goes into the hooded opening. These inlets are extremely useful for sag condition installations, although they can also be effective on continuous grades. The interception capacity of a combination inlet is only slightly greater than with a grate alone. Therefore the capacity is computed neglecting the curb opening and designers should follow the same analysis as for a vaned grate alone. See Section 9.4.4 for design guidance in a sag condition.

9.4.1.4 Grate Inlets Type 1 or 2 (WSDOT Standard Plans B-35.20, B-35.40, and B-40.20)

Both Types 1 or 2 grate inlets have large openings that can compensate for debris problems, however, there are limitations in their usage. A Type 1 grate inlet is a non-reinforced, cast-in-place concrete inlet, which cannot support traffic loads. Type 2 grate inlets are pre-cast and can withstand traffic loading. These inlets are installed with a Grate A or Grate B (see WSDOT Standard Plan B-40.20-00) or a frame and vaned grate (see the next paragraph and WSDOT Standard Plan B-40.40-01 for more information on frame and vaned grates). Due to structural failure of both Grates A or B, neither of these grates can be installed in heavy traffic areas where wheel loads will pass directly over. Grate B has very large openings and is useful in ditches or non-paved median locations, in areas where there is no pedestrian or bicycle traffic. Grate A can be used anywhere Grate B is used as well as at the curb line of a wide interstate shoulder. Grate A may occasionally be hit by low-speed traffic or parked on but it cannot withstand repeated interstate loading or turning vehicles.

9.4.1.5 Frame and Vaned Grates (WSDOT Standard Plan B-40.40)

WSDOT Standard Plan B-40.40 has been tested in H-25 loading and was determined compatible with heavy traffic installations. This frame and double vaned grate should be installed in a Unit H on top of a grate inlet Type 2. The frame and vaned grates may be used in either new construction or retrofit situations. When used in areas of highway speeds, lock down grates should be specified.

9.4.1.6 Circular Grates

See WSDOT Standard Plans B-20.20 and WSDOT B-20.60 for details. Install with circular frames (rings) as detailed in WSDOT Standard Plan B-30.70. Circular grates are only allowed for use on private property and as approved by Environmental Services.
9.4.2 Capacity of Inlets on a Continuous Grade

The interception capacity of an inlet on a continuous grade depends on the amount of water flowing over the grate, the size and configuration of the grate, the velocity of the flow in the gutter, and the longitudinal slope of the roadway.

- For longitudinal slopes between 2 to 3 percent and for velocities in the range of 3 to 5 ft/s the interception capacity of an inlet is based mainly on frontal flow. Frontal flow is water that travels through the gutter and enters through the front side (width) of the inlet.

- For longitudinal slopes less than 2 percent and velocities less than 3 ft/s side flow interception shall also be considered as described in Section 9.4.3. An inlet will intercept essentially all frontal flow passing over the width of the inlet as long as the velocity is less than 5 ft/s.

- When velocities exceed 5 ft/s water will “splash-over” the inlets reducing the portion of the flow that will be intercepted and increase the bypass flow. When this occurs, consult with Environmental Services to determine a design appropriate for the location. Additional guidance can be found in the FHWA HEC No. 22, Section 4-3 at: www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=22&id=47.

\[ \text{Figure 3 - 29. Section at Inlet} \]

The flow that is not intercepted by the first grate inlet is considered bypass flow and should be added to the flow traveling toward the next grate located downstream. This carry-over process continues to the bottom of the grade or the end of the inlet system.

The last inlet on the system is allowed to bypass 0.10 cfs (0.003 m$^3$/s) during the 25-year, 24-hour storm event. If discharges exceed 0.10 cfs, additional catch basins or structures will be required to handle the flow.
The amount of flow bypassing the inlet on a continuous grade is computed as follows:

\[ Q_{BP} = Q \left[ \frac{(Z_d) - (GW)}{(Z_d)} \right]^{8/3} \]  
(equation 7)

Where:
- \( Q_{B} \) = portion of flow outside the width of the grate, cfs
- \( Q \) = total flow of gutter approaching the inlet, cfs
- \( Z_d \) = top width of the flow prism, feet
- \( GW \) = width of the grate inlet perpendicular to the direction of flow in feet, assuming that grates areas are 80% free of debris and vaned grates are 95% free of debris.

The flow that is intercepted by the inlet is calculated as follows:

\[ Q_I = \Delta Q - Q_{BP} \]  
(equation 8)

The velocity of flow directly over the inlet is calculated as follows:

\[ V_{continuous} = \frac{Q - Q_{BP}}{(GW)[d - 0.5(GW)(S_t)]} \]  
(equation 9)

Where:
- \( V_{continuous} \) = velocity over the inlet in ft/s (m/s)
- \( S_t \) = transverse slope or superelevation in ft/ft (m/m)
- \( D \) = depth of flow at the face of the curb in ft (m)
9.4.3 **Side Flow Interception**

For longitudinal slopes less than 2 percent and when equation 9 yields velocities less than 3 ft/s, side flow interception begins to make an appreciable contribution to the inlet capacity analysis and should be considered.

The velocity of flow entering the side of an inlet is shown in equation 10.

\[
V_{\text{side}} = \left( \frac{1.11}{n} \right) (S_L^{0.5} S_t^{0.67} Z_d^{0.67}) \\
\text{ (equation 10)}
\]

Where:

- \( V_{\text{side}} \) = velocity in triangular channel in ft/s
- \( n = 0.015 \) (Manning’s value for concrete pavement)
- \( S_L \) = longitudinal slope
- \( S_t \) = transverse slope or superelevation in ft/ft (m/m)
- \( Z_d \) = top width of the flow prism, ft

The ratio of frontal flow to total gutter flow is shown in equation 11.

\[
E_o = 1 - \left( 1 - \frac{GW}{Z_d} \right)^{2.67} \\
\text{ (equation 11)}
\]

Where:

- \( GW \) = width of depressed grate in ft
- \( Z_d \) = top width of the flow prism in ft

The ratio of side flow intercepted to total side flow is shown in equation 12.

\[
R_s = \frac{1}{1 + \left( \frac{0.15 V_{\text{side}}^{1.8}}{S_t G L^{2.3}} \right)} \\
\text{ (equation 12)}
\]

Where:

- \( GL \) = gutter length

The efficiency of the grate is expressed in equation 13.

\[
E = R_f E_o + R_s (1 - E_o) \\
\text{ (equation 13)}
\]

Where:

- \( R_f \) = Ratio of front flow intercepted to total frontal flow
The amount of flow intercepted by an inlet when side flow is considered is expressed in equation 14.

\[ Q_i = Q(R_f E_o + R_s (1 - E_o)) \]  

\[(equation \ 13)\]

9.4.3.1 Inlet Analysis Spreadsheet

A Microsoft Excel spreadsheet has been developed by the Washington State Department of Transportation that follows the procedure to calculate roadway runoff and inlet interception for a roadway on a longitudinal slope. When velocities are less than 3 ft/s and the longitudinal slope is less than 2 percent, the spreadsheet will automatically consider side flow in the analysis. Also, when velocities exceed 5 ft/s or the bypass flow at the last inlet exceeds 0.1 cfs, the spreadsheet will warn the designer. The spreadsheet is located at www.wsdot.wa.gov/Design/Hydraulics/Programdownloads.htm.

9.4.4 Capacity of Inlets in Sag Locations

By definition, a sag is any portion of the roadway where the profile changes from a negative grade to a positive grade. Inlets at sag locations perform differently than inlets on a continuous grade and therefore require a different design criterion. Theoretically, inlets at sag locations may operate in one of two ways: (1) at low ponding depths, the inlet will operate as a weir; and (2) high ponding depths (5” depth above the grated inlet and 1.4 times the grate opening height for combination inlets), the inlet will operate as an orifice. It is very rare that ponding on a roadway will become deep enough to force the inlet to operate as an orifice. As a result, this section will focus on inlets operating as a weir with flow spilling in from the three sides of the inlet that are exposed to the ponding.

Inlets at sag locations can easily become plugged with debris and therefore, it is good engineering practice to provide some type of relief. This relief can be accomplished by locating flanking inlets, on either side of the sag inlet, so they will operate before water exceeds the allowable spread into the travel lane at the sag. This manual recommends flanking inlets be located so the depth of water at the flanking inlet ponds to half the allowable depth at the sag (or \( \frac{1}{2}d_B \)). With that said, flanking inlets are only required when the sag is located in a depressed area and water has no outlet except through the system. However, if runoff is capable of overtopping the curb and flowing away from the roadway before exceeding the allowable limits noted in Table 3 - 18, flanking inlets are not required. With this situation there is a low potential for danger to drivers if the inlets do not function as designed. Before flanking inlets are removed in this situation, designers should consider the potential damage of water going over the curb. Designers should use the guidelines provided in this section for locating flanking inlets. If a designer suspects flanking inlets are unnecessary, consult Environmental Services early in the design for approval.

Any section of roadway located in a sag should be designed according to the criteria described below. To aid the designer in sag analysis, a copy of the sag worksheet is located on the WSDOT Hydraulic web page at http://www.wsdot.wa.gov/publications/fulltext/Hydraulics/Programs/SagWorksheetud.xls.
Once an inlet has been placed in a sag location, the total actual flow to the inlet can be
determined as shown below. $Q_{\text{Total}}$ must be less than $Q_{\text{allowable}}$ as described in equation 14.

$$Q_{\text{Total}} = Q_{BP1} + Q_{BP2} + \Delta Q_1 + \Delta Q_2$$  \hspace{1cm} (equation 14)

Where:

$Q_{BP1&2} = \text{bypass flow from the last inlet on either side of a continuous grade calculated using equation 7.}$

$\Delta Q_{1&2} = \text{runoff that is generated from the last inlet on either side of the continuous grades, see WSDOT Standard Plan B-25.20-00.}$

The effective perimeter of the flanking and sag inlets can be determined using the length and
widths for various grates given in Figure 3 - 29. This would be the sum of the three sides of the
inlet where flow spills in and where ponding would occur. The grates shall be assumed to be 50
percent plugged, the vaned grates should be assumed to be 75% free, and the Combination Inlet
B-25.20, which should be considered 100 percent free. This adjustment is in addition to reducing
the perimeter to account for the obstruction caused by the bars in the grate. Figure 3 - 29 lists
perimeters for various grates with reductions already made for bars.

$$P_n = 0.5[L + 2W]$$  \hspace{1cm} (equation 15)
Where:

\[ P = \text{effective perimeter of the flanking and sag inlet} \]
\[ L = \text{length of the inlet from Table 3 - 17.} \]
\[ W = \text{width of inlet from Table 3 - 17.} \]

The allowable capacity of an inlet operating as a weir, that is the maximum \( Q_{allowable} \), can be found depending on the inlet layout as described below:

When there is only a single inlet at the sag (no flanking inlets) the following equation should be used:

\[ Q_{allowable} = C_w \times P \times d_{Ballowable}^{1.8} \]  
\[ \text{(equation 16)} \]

Where:

\[ C_w = \text{weir coefficient; 3.0 for English Units, 1.66 for Metric Units} \]
\[ P = \text{effective perimeter of the grate in feet} \]
\[ d_{Ballowable} = \text{maximum depth of water at the sag inlet in feet} \]

As noted previously it is recommended that flanking inlets be located laterally from the sag inlet at a distance equal to 0.5\( d_{Ballowable} \). When this recommendation is followed, \( Q_{allowable} \) can be simplified as shown below. If the inlets are not all the same size, the following equation will need to be modified to account for different perimeters:

\[ \Sigma Q = C_w \times P \times [2(0.5d_B)^{1.5} + (d_B)^{1.5}] \]  
\[ \text{(equation 17)} \]

Where \( d_B = \text{depth of water at the sag inlet (ft)} \)

In some applications, locating inlets so water ponds to 0.5\( d_{Ballowable} \) is too far (generally in cases with long flat slopes). Designers should instead ensure that the spread of stormwater does not exceed those noted in Table 3 - 18 and use equation 18.

\[ Q_{allowable} = C_wP[d_A^{1.5} + d_B^{1.5} + d_C^{1.5}] \]  
\[ \text{(equation 18)} \]

Where:

\[ d_N = \text{depth of water at the flanking inlets and the sag (ft)} \]
The actual depth of water over the sag inlet can be found with equation 19 below and must be less than $d_B$ allowable which can be found using equation 21. If however, the inlets are or are not located at $0.5d_B$ allowable, Equation 19 will need to be modified to reflect this.

$$d_B = \left[ \frac{Q_{total}}{(C_{WA}P_A^{0.3536} + C_{WB}P_B^3 + C_{WC}P_C^{0.3536})} \right]^{\frac{2}{3}} \quad \text{(equation 19)}$$

Where:

- $Q_{total} =$ actual flow into the inlet in cfs (cms)
- $C_W =$ weir coefficient, 3.0 (1.66 for metric)
- $P_N =$ effective grate perimeter, in feet (m), see Figure 3 - 29
- $d_B =$ actual depth of ponded water at the inlet in feet (m)

Verify the allowable depth and flow have not been exceeded. That is verify $Q_{allowable} > Q_{Total}$ and $d_B$ allowable $> d_B$. If the allowable flow and depth are greater than the actual, the maximum allowable spread will not be exceeded and the design is acceptable. If the actual depth or flow is greater than the allowable, then the runoff will spread beyond the maximum limits and the design is not acceptable. In this case, the designer should add flanking inlets or replace the three original inlets with inlets that have larger openings. If additional flanking inlets are used they should be placed close to the sag inlet to increase the flow interception and reduce the flow into the sag.

### 9.5 Gutter Capacity Analysis

When stormwater is collected and carried along the roadside in a gutter, the allowable top width of the flow prism ($Z_d$) is dependent on the Road Classification as noted in Table 3 - 18.

**Table 3 - 18: Design Frequency and Spread**

<table>
<thead>
<tr>
<th>Road Classification</th>
<th>Design Frequency</th>
<th>Design Spread ($Z_d$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interstate, Principal, Minor Arterial, or Divided</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 45 mph (70 km/hr) Sag Pt.</td>
<td></td>
<td>Shoulder + 2 ft (0.67 m)$^a$</td>
</tr>
<tr>
<td>≥ 45 mph (70 km/hr)</td>
<td>25-year</td>
<td>Shoulder</td>
</tr>
<tr>
<td>≥ 45 mph (70 km/hr)</td>
<td>50-year</td>
<td>Shoulder + 2 ft (0.67 m)$^a$</td>
</tr>
<tr>
<td>Collector and Local Streets</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 45 mph (70 km/hr) Sag Pt.</td>
<td>25-year</td>
<td>Shoulder + ½ Driving Lane$^b$</td>
</tr>
<tr>
<td>≥ 45 mph (70 km/hr)</td>
<td>25-year</td>
<td>Shoulder</td>
</tr>
<tr>
<td>≥ 45 mph (70 km/hr)</td>
<td>50-year</td>
<td>½ Driving Lane$^b$</td>
</tr>
</tbody>
</table>

---

$^a$ The travel way shall have at least 10 feet that is free of water.

$^b$ In addition to the design spread requirement, the depth of flow shall not exceed 0.12 feet at the edge of the shoulder.

In addition to the requirements above, areas where a superelevation transition causes a crossover of gutter flow, the amount of flow calculated at the point of zero superelevation shall be limited to 0.10 cfs (0.003 m$^3$/s). The designer will find, by the time the roadway approaches the
zero point, the $Z_d$ will become very wide. The flow width criteria will be exceeded at the crossover point even when the flow is less than 0.10 cfs (0.003 m$^3$/s).

The equation for calculating the gutter flow capacity is a modified version of Manning’s Equation. It is based on a roughness coefficient of 0.015, which assumes a rough, concrete or asphalt pavement gutter. Equation 20 and Equation 21 assume a uniform gutter section as shown in Figure 3 - 31. If the gutter section is different, designers should consult the *Hydraulic Engineering Circular No. 22*, Chapter 4, for further guidance found at: www.fhwa.dot.gov/bridge/hydpub.htm.

![Figure 3 - 31. Typical Gutter Section](image)

\[
\begin{align*}
d &= \left[ \frac{\Delta O S_t}{37(S_L)^{0.5}} \right]^{3/8} \quad \text{(equation 20)} \\
Z_d &= \frac{d}{S_t} \quad \text{(equation 21)}
\end{align*}
\]

Where:

- $d =$ depth of flow at the face of curb (ft)
- $\Delta O =$ gutter discharge (cfs); this is the design event per Table 3 - 18
- $S_L =$ longitudinal slope of the gutter (ft/ft); assume 0.4% for cement curb and gutter and 1% for asphalt unless survey of the gutter line is conducted.
- $S_t =$ transverse slope or superelevation (ft/ft); assume 2% unless survey of the slope is conducted.
- $Z_d =$ top width of the flow prism (ft)
9.6 Capacity Analysis

A capacity analysis of the road gutter, conveyance piping, catch basin grate, and catch basin leads will be required to ensure that adequate capacity exists in the downstream system before a connection to the City stormwater system will be allowed for projects that create greater than 5,000 square feet of impervious surface and cannot infiltrate or disperse stormwater onsite in accordance with Minimum Requirement #5, and are not required to provide onsite detention or complete a quantatitive downstream analysis per Minimum Requirement #7. The capacity analysis shall meet the requirements of Section 9.1 except that the analysis only needs to extend to the first City of Tacoma stormwater conveyance main (either City owned piping or City owned ditch). For the purposes of this analysis,a stormwater main can be a City owned conveyance system of any size. Environmental Services shall make the final determination of the extent of the capacity analysis.

9.7 Basis of Design

Applicants may be required to submit a basis for pipe and culvert design which includes pipe type selection, suitability for depth of bury and loading, deflection calculations for above-ground pipes, anchoring, armoring, analysis for above ground pipes or outfalls, etc. Environmental Services will make the final determination as to when the basis of design is required.