CHAPTER 8 – CONVEYANCE

Chapter Organization

8.1 INTRODUCTION

A conveyance system includes all natural or constructed components of a storm drain system that collect stormwater runoff and convey it away from structures, minimizing the potential for flooding and erosion.

Conveyance facilities consist of curbs and gutters, inlets, storm drains, catch basins, channels, ditches, pipes and culverts. The placement and hydraulic capacities of storm drain structures and conveyance systems shall consider the potential for damage to adjacent properties and minimize flooding within traveled roadways. The conveyance system shall also provide discharge capacity sufficient to convey the design flow at velocities that are self-cleaning without being destructive to the conveyance facilities. These objectives are achieved by designing all conveyance facilities using the design storm event specified for the given facility and by adhering to requirements such as minimum velocity, freeboard, cover, etc.

A properly designed conveyance system maximizes hydraulic efficiency by using the proper material, slope and size. Constructed conveyance systems should emulate natural, pre-developed conditions to the maximum extent feasible. Field-verified defined natural drainageways must be preserved and protected; filling them in and building on top of them is not an acceptable practice. In addition, some drainageways may be required for regional use (refer to Section 8.3.4 for criteria).

Inflow and discharge from the system shall occur at the natural drainage points in the same manner as the pre-developed condition as determined by topography and existing drainage patterns.

8.2 APPLICABILITY

All projects shall comply with this Basic Requirement regardless of whether the project they meet the regulatory threshold.

8.3 NATURAL AND CONSTRUCTED CHANNELS

8.3.1 CHANNEL ANALYSIS

A channel analysis shall be performed for all constructed channels proposed for a project and for all field-verified existing natural drainageways/channels present on-site (refer to Section 8.3.4 for details). The following requirements apply to the Drainage Report and the road and drainage plans, when applicable:

- Complete channel calculations shall be provided, indicating the design peak flow rates and assumptions, such as channel shape, slope and Manning's coefficient (see Table 5-4);
- Calculations, including the velocity, capacity, and Froude number shall be provided for each distinct channel segment whenever the geometry of the channel changes (i.e. if the slope, shape or roughness changes significantly);
- The centerline and direction of flow for all constructed drainage ditches or natural channels within the project limits are to be clearly shown on the construction plans and basin map. For all proposed channels, locating information shall be provided at all angle points;
- Calculations shall support the riprap area, thickness, riprap size and gradation, and filter blanket reinforcement for all channel protection, which shall be provided when permissible velocities are exceeded (see Table 8-1). This information shall be included in the plans;

TABLE 8-1
PERMISSIBLE VELOCITIES FOR CHANNELS WITH ERODIBLE LININGS, BASED ON UNIFORM FLOW IN CONTINUOUSLY WET, AGED CHANNELS

Soil Type Of Lining (Earth; No Vegetation)	Maximum Permissible Velocities (feet/second)		
	Clear Water	Water Carrying Fine Silts	Water Carrying Sand & Gravel
Fine sand (non-colloidal)	1.5	2.5	1.5
Sandy loam (non-colloidal)	1.7	2.5	2.0
Silt loam (non-colloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (non-colloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (non-colloidal)	2.0	3.5	2.0
Alluvial silts (colloidal)	3.7	5.0	3.0
Coarse gravel (non-colloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

Source: Special Committee on Irrigation Research, American Society of Civil Engineers, 1926.

- The Froude number shall be checked near the beginning and near the end of a channel that has significant grade changes to determine if a hydraulic jump occurs (as indicated by the Froude number changing from <1 to >1, or vice versa). Since it is difficult to correlate the location of a hydraulic jump to the actual location in the field, the engineer shall propose evenly spaced riprap berms, check dams, or other protective measures to ensure that the jump does not erode the conveyance facility;
- When geosynthetics are used for channel protection, the plans shall clearly specify fabric type, placement, and anchoring requirements. Installation shall be per the manufacturer's recommendation; and,
- Plans for grass-lined channels shall specify seed mixture and irrigation requirements, as applicable.

8.3.2 MINIMUM REQUIREMENTS

Slope

Minimum grades for constructed channels shall be as follows:

- 1.0% for asphalt concrete; and,
- 0.5% for cement concrete, graded earth or close-cropped grass.

Side Slopes

Ditch cross-sections may be V-shaped or trapezoidal. However, V-ditches are not recommended in easily erodible soils or where problems establishing vegetation are anticipated.

The side slope of roadside ditches shall conform to the requirements for clear zone of the local jurisdiction and WSDOT design standards.

No ditches or channels shall have side slopes that exceed the natural angle of repose for a given material or per Table 8-2.

Location

Constructed channels shall not be placed within or between residential lots. Ditches and channels shall be located within a drainage tract or within a border easement. Ditches or channels may be allowed to traverse through lots in large-lot subdivisions (lots of 1 acre or more) and consideration may be given to placement within an easement versus a tract. The local jurisdiction will review these proposals on a case-by-case basis.

TABLE 8-2
MAXIMUM DITCH OR CHANNEL SIDE SLOPES

Type of Channel	Side Slope (Horizontal: Vertical)
Firm rock	¼:1 to Vertical
Concrete-lined stiff clay	½:1
Fissured rock	½:1
Firm earth with stone lining	1½:1
Firm earth, large channels	1½:1
Firm earth, small channels	2:1
Loose, sandy earth	2:1
Sandy, porous loam	3:1

Source: Civil Engineering Reference Manual, 8th Edition

Depth

The minimum depth of open channels shall be 1.3 times the flow depth or 1 foot; whichever is greater.

Velocity

Table 8-1 lists the maximum permissible mean channel velocities for various types of soil and ground cover. If mean channel velocities exceed these values, channel protection is required (refer to Section 8.3.3). In addition, the following criteria shall apply:

- Where only sparse vegetative cover can be established or maintained, velocities should not exceed 3 feet/second;
- Where the vegetation is established by seeding, velocities in the range of 3 to 4 feet/second are permitted;
- Where dense sod can be developed quickly or where the normal flow in the channel can be diverted until a vegetative cover is established, velocities of 4 to 5 feet/second are permitted; and,
- On well established sod of good quality, velocities in the range of 5 to 6 feet/second are permitted.

8.3.3 CHANNEL DESIGN

Channel Capacity

Open channels shall be sized using the following variation of Manning's formula.

$$Q = VA = \frac{1.486\ A\ R^{2/3}\ \ S^{1/2}}{n} \tag{8-1}$$

Where:	Q	=	rate of flow (cfs);
	V	=	mean velocity in channel (feet/second);
	A	=	cross-sectional area of flow in the channel (square feet);
	R	=	hydraulic radius (feet); where R = A/P, and
			P = wetted perimeter (feet)
	S	=	channel slope (feet/foot);
	n	=	Manning's roughness coefficient (Table 5-4); and,

<u>Note:</u> Manning's equation will give a reliable estimate of velocity only if the discharge, channel cross-section, roughness, and slope are constant over a sufficient distance to establish uniform flow conditions. Uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes, however, Manning's equation can be applied to most open channel flow problems by making judicious assumptions.

Energy Dissipation Design

An energy dissipater is useful in reducing excess velocity, as a means of preventing erosion below an outfall or spillway. Common types of energy dissipaters for small hydraulic works are: hydraulic jumps, stilling wells, riprap outfall pads, and gabion weirs.

Channel Protection

Channel velocities shall be analyzed at the following locations, and if they are found to be erosive, channel protection shall be provided:

- At the top of a watershed, at the point where the stormwater runoff becomes concentrated into a natural or constructed channel;
- At all changes in channel configuration (grade, side slopes, depth, shape, etc.), if an erosive velocity is determined at a change in channel

configuration, the velocity shall be evaluated up the channel until the point at which the velocity is determined not to be erosive; and,

- At periodic locations along the entire channelized route.

A material shall be selected that has revetment and armoring capabilities, and the channel shall be analyzed using the Manning's "n" value for that material to determine if the material will reduce the velocity in the channel. In some cases, vegetative cover (natural grasses, etc.) may provide excellent protection without changing the flow characteristics and should be evaluated. If the calculations reveal that common materials such as riprap are not adequate, stronger protection such as gabions and/or stilling pools may be necessary.

Riprap Protection at Outlets

If the velocity at a channel or culvert outlet exceeds the maximum permissible velocity for the soil or channel lining, channel protection is required. The protection usually consists of a reach between the outlet and the stable downstream channel lined with an erosion-resistant material such as riprap.

The ability of riprap revetment to resist erosion is related to the size, shape and weight of the stones. Most riprap-lined channels require either a gravel filter blanket or filter fabric under the riprap.

Riprap material shall be blocky in shape rather than elongated. The riprap stone shall have sharp, angular, clean edges. Riprap stone shall be reasonably well-graded.

<u>Apron Dimensions:</u> The length of an apron (L_a) is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency (ASCE, 1992):

$$L_a = \left(\frac{1.8Q}{D_O^{3/2}}\right) + \left(7D_O\right) \text{ for } \quad TW < \frac{D_O}{2} \tag{8-2}$$

Or

$$L_a = \left(\frac{3Q}{D_O^{3/2}}\right) + \left(7D_O\right) \text{ for } \quad TW \geq \frac{D_O}{2} \tag{8-3}$$

Where:

D_o = maximum inside culvert width (feet);

Q = pipe discharge (cfs); and,

TW = tailwater depth (feet).

When there is no well-defined channel downstream of the apron, the width, W, of the apron outlet as shown in Figure 8-1, shall be calculated using Equation 8-4 or 8-5:

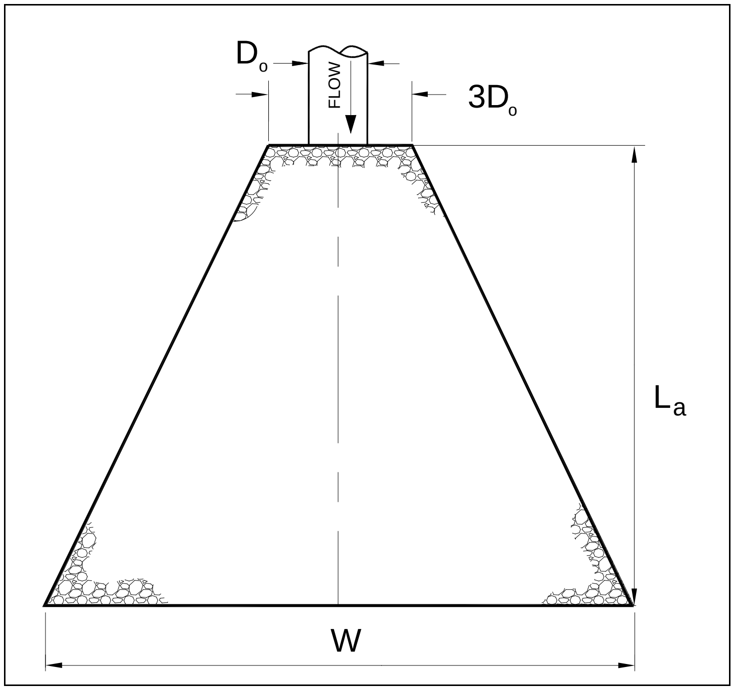


Figure 8-1 – Riprap Revetment at Outfall Schematic

$$W = 3D_O + 0.4L_a \text{ for } \qquad TW \geq \frac{D_O}{2} \tag{8-4}$$

$$W = 3D_O + L_a \text{ for } \qquad TW < \frac{D_O}{2} \tag{8-5}$$

When there is a well-defined channel downstream of the apron, the bottom width of the apron should be at least equal to the bottom width of the channel and the lining should extend at least 1 foot above the tailwater elevation.

The width of the apron at a culvert outlet should be at least 3 times the culvert width.

Apron Materials: The median stone diameter, D_{50} is determined from the following equation:

$$D_{50} = \frac{0.02Q^{4/3}}{TW(D_O)} \tag{8-6}$$

Where: D_{50} = the diameter of rock, for which 50% of the particles are finer.

The riprap should be reasonably well graded, within the following gradation parameters:

$$1.25 \leq \frac{D_{max}}{D_{50}} \leq 1.50 \text{ and } \frac{D_{15}}{D_{50}} = 0.50 \text{ and } \frac{D_{min}}{D_{50}} = 0.25$$

Where: D_{max} = the maximum particle size;

D_{min} = the minimum particle size; and,

D_{15} = the diameter of rock, for which 15% of the particles are finer.

Minimum Thickness: The minimum thickness of the riprap layer shall be 12 inches, D_{max} or $1.5D_{50}$, whichever is greater.

Filter Blanket: A filter fabric blanket under the riprap is normally needed. If a gravel or sand filter blanket is used, then it shall conform to the gradation parameters listed in Table 8-3.

TABLE 8-3
CRITERIA FOR GRAVEL OR
SAND FILTER BLANKET GRADATION

Primary Criterion	$D_{15} < 5d_{85}$
Recommended Secondary Criteria	$5d_{15} < D_{15} < 40d_{15}$ $D_{50}/d_{50} < 50$

Guidelines for Stormwater Management, Spokane County, February 1998

The size of the filter blanket material is designated d_{xx}, the size of the riprap is designated D_{xx}, and the size of the subgrade is designated d'_{xx}. The thickness of each filter blanket should be one-half that of the riprap layer. If it is found that $D_{15}/d'_{85} < 2$ then no filter blanket is needed. Where very large riprap is used, it is sometimes necessary to use two filter blanket layers between the sub-grade and the riprap.

8.3.4 PRESERVATION OF NATURAL LOCATION OF DRAINAGE SYSTEMS (NLDS)

New development shall be designed to protect certain natural drainage features that convey or store water or allow it to infiltrate into the ground in its natural location, including drainageways, floodplains (Section 7.9.2), wetlands and streams (including classified streams) (Section 7.9.3), and natural closed depressions (Section 7.9.4). These features are collectively referred to as the Natural Location of Drainage Systems (NLDS). Preserving the NLDS will help ensure that stormwater runoff can continue to be conveyed and disposed of at its natural location. Preservation will also increase the ability to use the predominant systems as regional stormwater facilities. A regional stormwater facility is typically defined as a system designed and built by a local jurisdiction to receive an agreed-upon rate and volume of stormwater from a defined contributing drainage area, but it can also refer to a private system that serves multiple developments.

Projects located within the City of Spokane shall refer to the City of Spokane's Stormwater Ordinance for specific requirements with regard to the Natural Location of Drainage Systems that may differ from the information found in this section.

Definitions

Some of the drainageways that need to be evaluated for preservation purposes or for potential use as part of a regional facility have been mapped. These drainageways are generally defined as Type A and Type B:

- <u>Type A</u> drainageways are predominant systems that are considered a significant part of a larger existing natural conveyance system.
- <u>Type B</u> drainageways are systems that are generally less prominent, but are deemed to perform important functions in the existing management of stormwater runoff and may be necessary for managing stormwater as part of a larger regional or natural system.

Because every site is unique, the local jurisdiction shall make interpretations, as necessary, based on site visits and technical information as to the exact location and type of drainageways or any NLDS on a project site. The local jurisdiction may also require the project proponent to provide engineering information to assist in this determination.

The maps denoting these drainageways are not definitive; a computer program was used to generate the contours and identify the drainageways. The Type A/B designations are not concrete labels nor are they all inclusive. The maps are only one tool that may be used to identify existing natural drainageways; field verification will typically be required to fully identify the existence of a drainageway and its significance with regard to a natural conveyance system. The Spokane County Stormwater Utility Section maintains maps of drainageways identified within the Spokane County Stormwater Service Areas. The criteria for analysis and preservation of all other NLDS (floodplains, wetlands, closed depressions and wetlands/streams) are covered in Chapter 7.

Protection

No cuts or fills shall be allowed in predominant drainageways except for perpendicular driveway or road crossings with engineering plans showing appropriately sized culverts or bridges. Predominant drainageways shall be preserved for stormwater conveyance in their existing location and state, and shall also be considered for use as regional facilities.

Less prominent drainageways in a non-residential development and in a residential development containing lots 1 acre or smaller may be realigned within the development provided that the drainageway will enter and exit the site at the pre-developed location and that discharge will occur in the same manner as prior to development.

Realignment of a less prominent drainageway shall be defined as still following the “basic” flow path of the original drainageway. An acceptable example would be if the drainageway is proposed to be realigned such that it will follow a new road within the proposed development, and will be left in its existing state or utilized as part of the project’s on-site stormwater system.

Stormwater leaving the site in the same manner shall be defined as replicating the way the stormwater left the site in its existing condition. If the drainageway is preserved in its existing location and is left undisturbed, this goal should be met. If the local jurisdiction accepts the proposal to allow a less predominant drainageway to be routed through the site via a pipe, the following additional criteria shall be met:

- Where the less prominent drainageway enters the site, the design shall ensure that the entire drainageway is “captured” as it enters the site; i.e. the surrounding property shall not be regraded to “neck-down” the drainageway so that it fits into a drainage easement or tract or structure intended to capture and reroute the off-site stormwater runoff.
- Where the less prominent drainageway exits the site, the design shall ensure that the stormwater leaves the pipe, pond or structure a significant distance from the edge of the adjacent property so that by the time the stormwater reaches the property boundary, its dispersal shall mimic that of the pre-developed condition.

Since some of the less prominent drainageways may also be useful for managing regional stormwater, if identified as a significant drainageway (i.e. necessary conveyance for flood control, or being considered as a connection to a planned regional facility or conveyance route), then the drainageway may be subject to the same limitations and criteria as a predominant drainageway.

The size of the tract or easement containing the drainageway shall be determined based on an analysis of the existing and proposed stormwater flows directed to these drainage systems and any access and maintenance requirements found in this Manual. This analysis shall be performed as per the criteria found in Basic Requirement No. 5, Section 2.2.5.

All new development containing lots that are 1 acre or smaller shall be required to set aside the drainageway as open space in a separate tract. For new development containing lots that are greater than 1 acre, the drainageway may be set aside in either a tract or an easement.

All projects shall be reviewed for the presence of any NLDS and a determination will be made as to their significance with respect to preservation for continued natural conveyance and for potential use as part of a regional system.

8.4 CULVERTS

A culvert is a short pipe used to convey flow under a roadway or embankment. A culvert shall convey flow without causing damaging backwater flow constriction, or excessive outlet velocities. Factors to be taken into consideration in culvert design include design flows, the culvert's hydraulic performance, the economy of alternative pipe materials and sizes, horizontal and vertical alignment, and environmental concerns.

8.4.1 CULVERT ANALYSIS

When applicable, the following items shall be included in the Drainage Report, or on road and drainage plans:

- Complete culvert calculations that state the design peak flow rates, velocities at the inlet and outlet, flow control type, and design information for the culvert such as size, slope, length, material type, and Manning's coefficient (refer to Table 8-4);
- Headwater depths and water surface elevations for the design flow rate;
- Roadway cross-section and roadway profile;
- Location information for each of the culvert inverts and invert elevations;
- Type of end treatment (wingwall, flared end sections, etc); and,
- Wall thickness.

8.4.2 MINIMUM REQUIREMENTS FOR CULVERTS

Peak Flow Rate

Culverts shall be sized to handle the design peak flow rates calculated using the methods described in Chapter 5 and the design criteria specified in Chapter 2.

To avoid saturation of the road base, culverts shall be designed such that the water surface elevation for the design storm event does not exceed the elevation of the base course of the roadway.

Culverts shall be designed to withstand the 100-year storm event without damage.

TABLE 8-4
MANNING'S ROUGHNESS COEFFICIENT (n)
FOR CULVERTS

Material Type	n
Concrete pipe	0.013
Ductile iron	0.013
HDPE (only allowed in private roads)	0.013
CMP	0.024

HDPE = high-density polyethylene; CMP = corrugated metal pipe; PVC = polyvinyl chloride

Allowable Headwater Elevation

Headwater is the depth of water at the culvert entrance at a given design flow. Headwater depth is measured from the invert of the culvert to the water surface.

Culverts shall be designed to carry the design runoff with a headwater depth less than 2 times the culvert diameter for culverts 18 inches or less in diameter, and less than 1.5 times the culvert diameter for culverts more than 18 inches in diameter.

Velocity and Slope

To avoid silting, the minimum velocity of flow through culverts shall be 4 feet/second and the minimum slope shall be 0.5%.

Diameter

Table 8-5 lists required minimum culvert diameters.

TABLE 8-5
MINIMUM CULVERT SIZES

Culvert Location	Minimum Size (inches)
Under public roads	18
Under private roads	12
Under driveways/approaches	12

Material and Anchoring

Corrugated metal pipe, ductile iron, or concrete boxes can be used for all culverts. High-density polyethylene (HDPE) is only allowed in private roads. For grades greater than or equal to 20%, anchors are required unless calculations or the manufacturer's recommendations show that they are not necessary.

Placement/Alignment

Generally, culverts shall be placed on the same alignment and grade as the drainageway. Consideration should also be given to changes of conditions over time by using design measures such as:

- Cambering or crowning under high tapered fill zones;
- Raising intakes slightly above the flow line to allow for sedimentation;
- Using cantilevered outfalls away from road banks to allow for toe erosion; and,
- Using drop inlets or manholes to reduce exit velocities on steep terrain.

Angle Points

The slope of a culvert shall remain constant throughout the entire length of the culvert. However, in situations where existing roadways are to be widened, it may be necessary to extend an existing culvert at a different slope; the location where the slope changes is referred to as the angle point. The change in slope tends to create a location in the culvert that catches debris and sediment. If an extension of a culvert is to be placed at a different grade than the existing culvert, a manhole shall be provided at the angle point to facilitate culvert maintenance.

Outfalls

Outfalls shall conform to the requirements of all federal, state, and local regulations. Erosion control shall be provided at the culvert outfall. Refer to Section 8.3.3 for additional information regarding outfall protection.

Culvert Debris and Safety

The engineer shall evaluate the site to determine whether debris protection shall be provided for culverts. Debris protection shall be provided in areas where heavy debris flow is a concern, for example, in densely wooded areas. Methods for protecting culverts from debris problems include: upsizing the culvert and installing debris deflectors, trash racks or debris basins. Section 3.4.8 of the *WSDOT Hydraulic Manual* has additional information on debris protection.

Safety bars to prevent unauthorized individuals from entering the culvert shall be provided for culverts with a diameter greater than 36 inch (see WSDOT standard drawings).

When a trash rack is proposed, the effects of plugging shall be evaluated. Consideration should be given to the potential degree of damage to the roadway and adjacent property, potential hazard and inconvenience to the public, and the number of users of the roadway.

Structural Design

The *WSDOT Hydraulics Manual*, Tables 8-11.1 through 8-11.18, shows the maximum cover for different pipe materials and sizes.

For culverts under roadways, the amount of cover over the culvert is defined as the distance from the top of the pipe to the bottom of the pavement. It does not include asphalt or concrete paving above the base. The minimum amount of cover is 2 feet for culverts, unless proposing ductile iron pipe. The minimum cover for ductile iron pipe is 1 foot.

The minimum cover for culverts under private driveways is 1 foot from the top of the pipe to the finish grade of the drivable surface. Driveway culverts shall be a minimum of 12” CMP or ductile iron pipe.

If the depth of cover is shallow (less than 1 foot) and truck wheel loads are present, it will be necessary to propose a design to prevent structural damage to the pipe or to implement the manufacturer’s recommendations. Also, extreme fill heights (20 feet or greater) may cause structural damage to pipes and will require a special design or adherence to the manufacturer’s recommendations.

End Treatments

The type of end treatment used on a culvert depends on many interrelated and often conflicting considerations:

- <u>Projecting Ends</u> is a treatment in which the culvert is simply allowed to protrude out of the embankment. This is the simplest and most economical. There are several disadvantages such as susceptibility to flotation and erosion, safety when projecting into a roadway clear zone (an area beyond the traveled roadway provided for recovery of errant vehicles), and aesthetic concerns;
- <u>Beveled End Sections</u> consist of cutting the end of the culvert at an angle to match the embankment slope surrounding the culvert. Beveled ends should be considered for culverts 6 feet in diameter or less. Structural problems may be encountered for larger culverts not reinforced with a headwall or slope collar;

- Flared End Sections are manufactured culvert ends that provide a simple transition from culvert to a drainage way. Flared end sections are typically only used on circular pipe or pipe arches. This end treatment is typically the most feasible option in pipes up to 48 inches in diameter. Safety concerns generally prohibit their use in the clear zone for all but the smallest diameters;
- Headwalls are concrete frames poured around a beveled or projecting culvert. They provide structural support and eliminate the tendency for buoyancy. They are considered feasible for metal culverts that range from 6 to 10 feet in diameter. For larger diameters, a slope collar is recommended. A slope collar is a reinforced concrete ring that surrounds the exposed culvert end; or,
- Wingwalls and Aprons are intended for use on reinforced concrete box culverts. Their purpose is to retain and protect the embankment, and provide a smooth transition between the culvert and the channel.

8.4.3 CULVERT DESIGN

Culvert analysis is typically performed using commercially available computer software. If hand calculations are proposed, example calculations can be found in several technical publications and open channel hydraulics manuals.

8.5 STORM DRAIN SYSTEMS

A storm drain system is a network of pipes that convey surface drainage from catch basins or other surface inlets, through manholes, to an outfall.

The design of storm drain systems shall take into consideration runoff rates, pipe flow capacity, hydraulic grade line, soil characteristics, pipe strength, potential construction problems, and potential impacts on down-gradient properties.

8.5.1 PIPE ANALYSIS

The following items shall be included in the Drainage Report, or on road and drainage plans:

- A basin map showing on-site and off-site basins contributing runoff to each inlet, which includes a plan view of the location of the conveyance system;
- Complete pipe calculations that state the design peak flow rates and design information for each pipe run, such as size, slope, length, material type, and Manning's coefficient (see Table 8-6);

- Velocities at design flow for each pipe run;
- The hydraulic grade line at each inlet, angle point, and outlet; and,

TABLE 8-6
MANNING'S ROUGHNESS COEFFICIENTS (n)
FOR CLOSED SYSTEMS

Material Type	n
Concrete pipe	0.013
Ductile iron	0.013
HDPE[1]	0.013
PVC (only allowed in closed system)	0.013

[1] Contact the local jurisdiction for additional requirements when using HDPE pipe.

For lateral pipe connections to storm drain lines in existing rights-of-way (i.e. from a catch basin to a drywell, a main line stormwater system, a pond or a swale), fixed invert elevations are preferred but not required. The minimum depth from finish grade to pipe invert and the minimum pipe slope necessary to satisfy the freeboard and self-cleaning velocity requirements shall be provided. If necessary, invert elevations may be adjusted during construction to avoid potential conflicts with existing utilities in the right of way.

8.5.2 MINIMUM REQUIREMENTS

Peak Flow Rate

Closed pipe systems shall be sized to handle the design peak flow rates. These peak rates can be calculated using the methods described in Chapter 5 and the design criteria specified in Chapter 2.

Hydraulic Grade Line

The hydraulic grade line (HGL) represents the free water surface elevation of the flow traveling through a storm drain system. Pipes in closed systems will be sized by calculating the HGL in each catch basin or manhole. A minimum of 0.5 feet of freeboard shall be provided between the HGL in a catch basin or manhole and the top of grate or cover.

Pipe Velocities and Slope

In Spokane County and the City of Spokane Valley pipe systems shall be designed to have a self-cleaning velocity of 2.5 feet/second at design flow. In the City of Spokane, pipe systems shall be designed to have a self-cleaning velocity of 3 feet/second or greater calculated under full flow conditions even if the pipe is only flowing partially full during the design storm.

Pipe velocities should not be excessively high since high flow velocities (approaching and above 10 feet/second) cause abrasion of the pipes. When the design velocities are 10 feet/second or greater, manufacturer's recommendations demonstrating that the pipe material can sustain the proposed velocities shall be provided.

When the grade of a storm pipe is greater than or equal to 20%, then pipe anchors are required at the joints, at a minimum, unless calculations and manufacturer's recommendations demonstrate that pipe anchors are not needed. Pipe anchor locations are to be defined on the plans, and a pipe anchor detail shall be referenced or provided.

Pipe material shall meet the WSDOT standards for storm sewer pipe. All pipe segments shall be pressure tested, according to WSDOT testing procedures and standards

Pipe Diameter and Length

The minimum pipe diameter shall be 12 inches, except that single pipe segments less than 50 feet long may be 8 inches in diameter. The maximum length of pipe between junctions shall be no greater than 300 feet. No pipe segment shall have a diameter smaller than the upstream segments.

Placement and Alignment

No storm drain pipe in a drainage easement shall have its centerline closer than 5 feet to a private rear or side property line. A storm drain located under a road shall be placed in accordance with the local jurisdiction's requirements or standard plans.

If it is anticipated that a storm drain system may be expanded in the future, provisions for the expansion shall be incorporated into the current design.

Outfalls

Pipe outfalls shall be placed on the same alignment and grade as the drainage way. Outfalls shall conform to the requirements of all federal, state, and local regulations. Erosion control is required at the storm system outfalls. Refer to Section 8.3.3 for additional information regarding outfall protection.

Storm Drain Debris and Safety

The engineer shall evaluate the site to determine whether debris protection shall be provided for storm drain systems. Debris protection shall be provided in areas where heavy debris flow is a concern, for example, in densely wooded areas. Methods for protecting storm drain systems from debris problems include debris deflectors, trash racks and debris basins. The WSDOT Hydraulic Manual has additional information on debris protection.

For enclosed storm drain systems in urban locations, safety bars shall be provided for outfalls with a diameter 18 inches or greater, in order to prevent unauthorized individuals from entering the storm drain system. Outfalls within a fenced area are not required to have safety bars. The clear space between bars shall be 4 inches maximum.

Structural Design

The *WSDOT Hydraulics Manual*, Tables 8-11.1 through 8-11.18, shows the maximum cover for different pipe materials and sizes.

In unincorporated Spokane County and the City of Spokane Valley, the amount of cover over the pipe is defined as the distance from the top of the pipe to the bottom of the pavement. It does not include asphalt or concrete paving above the base. The minimum amount of cover is 2 feet, unless proposing ductile iron. The minimum cover for ductile iron pipe is 1 foot.

In the City of Spokane, cover is measured from the top of pipe to the top of the pavement. The minimum amount of cover is 3 feet, unless proposing ductile iron. The minimum cover for ductile iron pipe is 1 foot.

If the depth of cover is shallow (less than 1 foot) and truck wheel loads are present, it will be necessary to propose a design to prevent structural damage to the pipe or to implement manufacturer’s recommendations. Extreme fill heights (20 feet or greater) may also cause structural damage to pipes and will thus require a special design or adherence to the manufacturer’s recommendations.

Inverts at Junctions

Whenever two pipes of the same size meet at a junction, the downstream pipe shall be placed with its invert 0.1 feet below the upstream pipe invert. When two different sizes of pipes are joined, pipe crowns shall be placed at the same elevation. The exception to this rule is at drop manholes. Exceptions may be allowed by the local jurisdiction when topographic conditions will significantly impact the depth of the disposal location.

Combined Systems

Combined sanitary and stormwater sewer systems are prohibited.

8.5.3 PIPE DESIGN

To analyze the conveyance capacity of a closed pipe system, the following general steps may be followed when steady flow conditions exist, or conditions can be accurately approximated assuming steady flow conditions:

1. Estimate the size of the pipes assuming a uniform flow condition, using Equation 8-1. Refer to Table 8-6 for Manning's coefficient values.
2. For the pipe sizes chosen, determine uniform and critical flow depth;
3. Determine if upstream (accelerated) flow conditions or downstream (retarded) flow conditions exist. Subcritical flow occurs when downstream conditions control, supercritical flow occurs when upstream conditions control. Determine what flow regime will occur by comparing uniform flow depth, critical flow depth, and initial flow depth. Identify hydraulic jump locations, and where any other discontinuity of flow depth will occur.
4. Conduct a more detailed analysis by computing the hydraulic grade line. The direct step method or standard step method is often used to calculate the hydraulic grade line. For supercritical flow, begin at the upstream end and compute flow sections in consecutive order heading downstream. For sub-critical flow, begin at the downstream end and compute flow sections in consecutive order heading upstream.

The analysis of closed pipe systems is typically done using commercially available computer software packages. If hand calculations are proposed, example calculations can be found in several technical publications on open channel hydraulics, such as: "Handbook of Hydraulics", by Brater and King; and "Open-Channel Hydraulics" by French.

8.6 GUTTERS

A gutter is a section of pavement adjacent to a roadway that conveys water during a storm runoff event. Gutter flow calculations are necessary to establish the spread of water onto the shoulder, parking lane, or travel lane. Roadways shall have an adequate non-flooded width to allow for the passing of vehicular traffic during the design storm event. The non-flooded width (L) is shown in Figure 8-2 and the minimum non-flooded widths for various road classifications are outlined in Table 8-7.

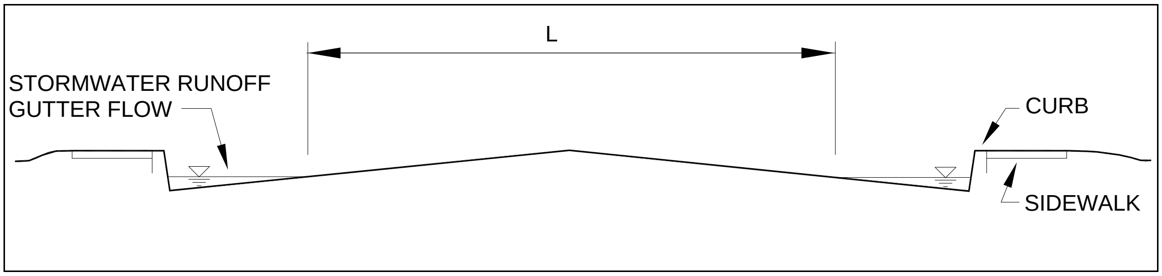


Figure 8-2 – Non-Flooded Road Width (L)

TABLE 8-7
NON-FLOODED ROAD WIDTH REQUIREMENTS

Road Classification	Non-Flooded Width (L)
Private Road	12 feet
Local Access	12 feet
Collector Arterial, 2 Lane	16 feet
Minor Arterial, 2 Lane	24 feet
Other road types	Per local jurisdiction

The non-flooded width shall be evaluated at low points and at proposed inlet locations. The non-flooded width shall also be evaluated at intersections. Bypass flow shall be limited to 0.1 cfs at intersections and at the project boundary.

Non-flooded width and flow depth at the curb are often used as criteria for spacing pavement drainage inlets (curb or grate inlets). Drainage inlets shall be spaced so that the non-flooded width requirements are met and stormwater does not flow over the back of the curb. Spacing shall not exceed 300 feet regardless of flooded width and flow depth compliance.

Generally, inlets shall be placed in the uphill side of the curb return. Additionally, the first inlet shall not be located more 500 feet from the point where the gutter flow path originates.

8.6.1 GUTTER ANALYSIS

When applicable, the drainage report shall include complete gutter calculations that state the design peak flow rates, design flow depth, road cross slope, road grade, and non-flooded width.

The equation for calculating gutter flow is a modified version of Manning's equation.

$$Q = \frac{0.56 \;\; S_x^{1.67} S_L^{0.5} T^{2.67}}{n} \tag{8-7}$$

Where:
- Q = flow rate (cfs);
- n = Manning's coefficient (from Table 8-8);
- S_L = longitudinal slope of the gutter (feet/foot);
- S_x = cross slope (feet/foot); and,
- T = spread (feet)

TABLE 8-8
MANNING'S ROUGHNESS COEFFICIENTS (*N*)
FOR STREET & PAVEMENT GUTTERS

Type of Gutter or pavement	n
Concrete gutter, troweled finish	0.012
Asphalt Pavement	
Smooth Texture	0.013
Rough Texture	0.016
Concrete pavement	
Float finish	0.014
Broom finish	0.016
Source: Federal Highway Administration (FHWA), Hydraulic Engineering Circular No. 22, Second Edition	

8.6.2 GUTTER DESIGN

Uniform Gutter Section

Uniform gutter sections have a cross slope that is equal to the cross slope of the shoulder or travel lane adjacent to the gutter (see Figure 8-3). The spread (T) in a uniform gutter section can be calculated using Equation 8-7 and solving for T (spread) as follows:

$$T = \left(\frac{Q \;\; n}{0.56 \;\; S_x^{1.67} S_L^{0.5}} \right)^{0.375} \tag{8-8}$$

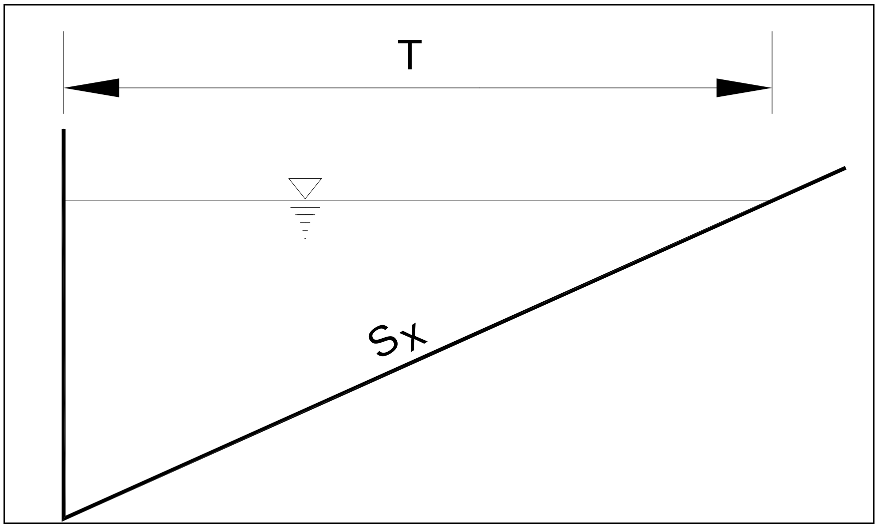


Figure 8-3 – Uniform Gutter Section

An example calculation for determining the non-flooded width and the depth of flow for a uniform gutter section is provided in Appendix 8A.

Composite Gutter Section

Gutters with composite sections have has a cross slope that is steeper than that of the adjacent pavement (see Figure 8-4). The design of composite gutters requires consideration of flow in the depressed segment of the gutter.

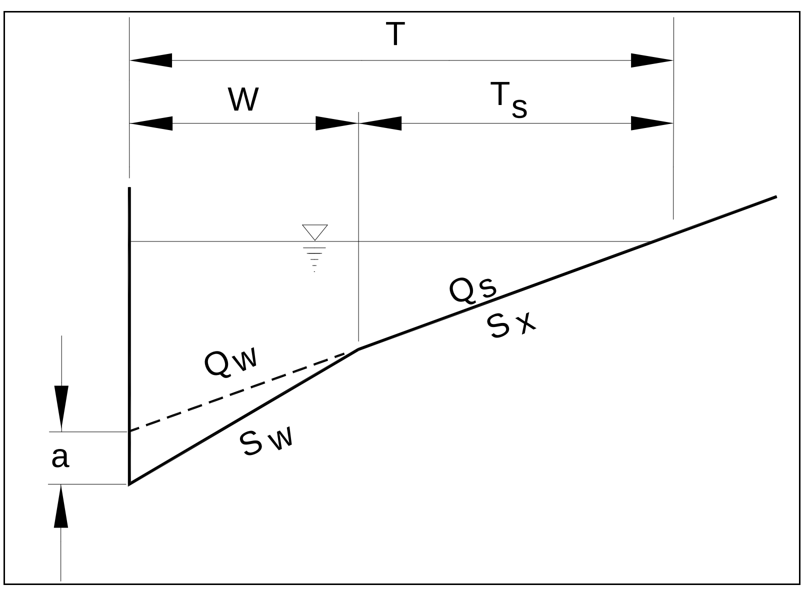


Figure 8-4 – Composite Gutter Section

The spread (T) in composite gutter sections cannot be determined by a direct solution; an iterative approach following the procedure outlined below must be used. An example calculation for determining the spread for a composite gutter section is included in Appendix 8B.

1. Assume a flow rate above the depressed gutter section, Q_s.
2. Compute Q_w using the following:

$$Q_w = Q - Q_s \tag{8-9}$$

Where: Q_w = flow rate in the depressed section of the gutter (cfs);
Q = design flow rate (cfs);
Q_s = flow rate in the gutter section beyond the depressed section (cfs);

3. Compute the gutter cross slope (if it is not given), S_w, using following equation:

$$S_w = S_x + a/W \tag{8-10}$$

Where: S_w = cross slope of the depressed gutter (feet/foot);
S_x = road cross slope (feet/foot);
W = gutter width (feet); and,
a = gutter depression (feet).

4. Compute E_o using the following equation:

$$E_o = \frac{Q - Q_s}{Q} = \frac{Q_w}{Q} \tag{8-11}$$

Where: E_o = ratio of flow in a chosen width (the width of a depressed gutter or grate) to the total gutter flow.

5. Solve for T using following equation:

$$T = W\left\{1 + \left(\frac{\frac{S_w}{S_x}}{\left[\frac{S_w}{S_x}\left(\frac{E_o}{1-E_o}\right)+1\right]^{3/8} - 1}\right)\right\} \tag{8-12}$$

6. Compute T_S using following equation:

$$T_S = T - W \quad (8\text{-}13)$$

Where: T_S = the width of the spread from the junction of the gutter with the edge of pavement to the edge of the spread (feet).

7. Use Equation 8-7 to determine Q_s for T_S and compare to estimated Q_s from Step 1. Steps 1 through 6 shall be repeated until the estimated and computed Q_s are approximately the same.

8.7 DRAINAGE INLETS

Drainage inlets are used to collect runoff and discharge it to a storm drainage system. They are typically located in gutter sections, paved medians, and roadside and median ditches. Inlets most commonly used in the Spokane Region are as follows:

- Grate Inlets consist of an opening in the gutter or ditch covered by a grate. They perform satisfactorily over a wide range of longitudinal slopes. Grate inlets generally lose capacity as the grade of the road, gutter or ditch increases.
- Curb Inlets are vertical openings in the curb. They are most effective on flat grades, in sumps, and where flows are found to carry significant amounts of floating debris. Curb inlets lose interception capacity as the gutter grade increases; therefore, the use of curb inlets is recommended in sumps and on grades less than 3%.
- Combination Inlets consist of both a curb-opening and a grate inlet. They offer the advantages of both grate and curb inlets, resulting in a high capacity inlet.

There are many variables involved in designing the number and placement of inlets, and in determining the hydraulic capacity of an inlet. The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

8.7.1 MINIMUM REQUIREMENTS

Peak Flow Rate

The capacity of drainage inlets shall be determined using the design peak flow rates. These rates can be calculated using the methods described in Chapter 5 and the design storm criteria specified in Chapter 2.

Bypass flow shall be limited to 0.1 cfs at intersections and at the project boundary.

Structures

Catch basins, inlets and storm manholes shall conform to the standard plans of the local jurisdiction, or the standard plans jointly published by WSDOT and APWA (M21-01).

Catch basins shall be used in all public and private roads unless utility conflicts prohibit their use.

WSDOT/County Type 1 Catch Basins shall not be used where invert elevation depths are more than 5 feet below lid elevations. Manholes shall be used in these situations.

Catch basins, inlets, and storm manholes shall be placed at all breaks in grade and horizontal alignments. Pipe runs shall not exceed 300 feet for all pipe sizes.

Horizontal and vertical angle points shall not be allowed in a storm system unless a manhole is provided for cleaning.

Grates

Herringbone grates are no longer accepted in roadway applications.

All grate inlets constructed at low points shall be combination inlets. The most commonly used combination inlet is a vaned grate with a hooded curb cut area.

Grate inlets on grade shall have a minimum spacing of 20 feet to enable the bypass water to reestablish its flow against the face of curb. Drainage inlets shall not be located on the curved portion of a curb return.

Grates shall be depressed to ensure satisfactory operation; the maximum depression is 2 inches.

Inlets with larger openings may be used for additional capacity, such as WSDOT Grate Inlet Type 2 (WSDOT Standard Plan B-40.35-00) with frame and vaned grate (WSDOT Standard Plan B-40.40-00). WSDOT Grate Inlet Type 1 and Grates A and B shall not be used in areas of pedestrian or vehicular traffic. Refer to WSDOT Manual and Standard Plans if any of the WSDOT inlets are proposed.

Curb Inlets

Concrete curb inlets (i.e. aprons) shall be used at the entrances to all stormwater facilities to aid stormwater conveyance into the facility and to suppress grass growth at the inlet.

The curb inlet shall have a 2-inch depression at the curb line and a maximum length of 6 feet.

At a minimum (where space constraints allow), curb inlets shall be placed at the most upstream and downstream point along the road adjacent to the treatment or disposal facility, regardless of the flow directed to the curb inlet. In many cases, when a long drainage facility is proposed, and the engineering calculations support it, additional intermediate curb inlets may be required.

Overflow structures, such as drywells or catch basins, shall be located away from the point or points where runoff flows into the facility. When the overflow structure is located within the facility, slopes around the structure shall be no greater than 4:1 (horizontal to vertical).

8.7.2 DRAINAGE INLET DESIGN

Grate Inlets, Continuous Grade

The capacity of an inlet on a continuous grade can be found by determining the portion of the gutter discharge directly over the width of the inlet. On continuous grades (assuming that the grate has the capacity to intercept the entire flow rate directed toward it), the amount of stormwater intercepted by a grate is equal to the amount of stormwater runoff flowing directly over the grate plus the amount that flows in over the side of the grate through the slats/bars. The analysis shall include a 35% clogging factor. The use of formulas for side flow interception for grate inlets found in *FHWA Hydraulic Engineering Circular No. 22* (HEC-22) will be accepted.

The following procedure is most accurate when velocities are in the range of 3 to 5 feet/second at a 2% or 3% longitudinal slope. For instances where the velocity is found to exceed 5 feet/second, additional intermediate inlets can be added, contributing basins redefined, and the associated velocities recalculated. While adding inlets is one solution to reducing the velocity, more information may be found regarding the affect of side flow by consulting the HEC-22 Circular, Section 4.4 Drainage Inlet Design. Note that commercially available software may be used to determine grate inlet capacity.

The capacity of a grate inlet on a continuous grade may be calculated using the procedure outlined below. Figure 8-5 identifies key parameters. Example calculations for grate inlets on a continuous grade for a uniform gutter section and a composite gutter section are provided in Appendices 8C and 8D.

1. Determine the runoff from the contributing basin at the high point to the first inlet. This is the amount of runoff that could be intercepted by the first inlet.
2. Select an inlet and note the grate width (GW) in the calculations (refer to Table 8-9).

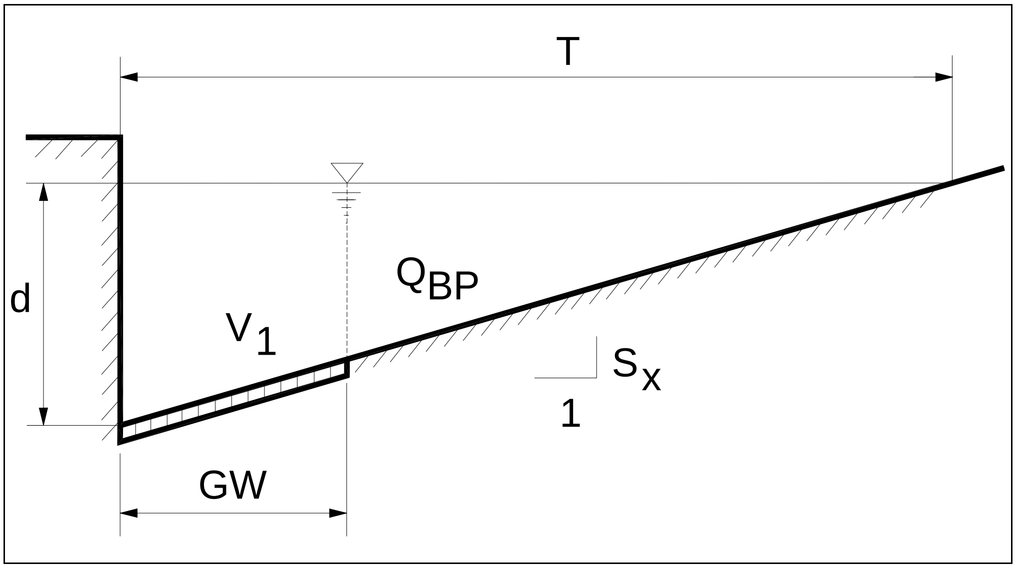


Figure 8-5 – Typical Grate Inlet Cross-Section

TABLE 8-9
ALLOWABLE WIDTH AND PERIMETER
FOR GRATE CAPACITY ANALYSIS

Structure Type	Allowable Width on a Continuous Grade (feet)	Allowable Perimeter in a Sump Condition (feet)
Vaned Grate for Catch Basin and Inlet	1.67	—
Metal Frame and Grate for Catch Basin and Inlet (Herringbone Pattern)[1]	1.67	—
Metal Frame with Hood and Bi-Directional Vaned Grate	1.67	3.13[4,5]
Frame and Vaned Grates for Grate Inlet Type 2 (WSDOT B-40.40-00)	1.75[2] 3.50[3]	2.96[4,5]

[1] Not recommended for new construction. Values are presented for evaluation of existing conditions.
[2] Normal Installation – see Figure 5-5.5 of WSDOT Hydraulics Manual
[3] Rotated Installation – see Figure 5-5.5 of WSDOT Hydraulics Manual
[4] This perimeter value has already been reduced by 50% for clogging.
[5] This perimeter value has also been reduced for bar area.

Note: Readers should review the most current versions of the local jurisdiction's standard plans for any revisions that may have been made to values provided in this table.

3. Analyze the most upstream inlet. The width of flow (T) is calculated using the procedure described in Section 8.6.2. Verify that T is within the allowable limit (see Table 8-7), then determine the amount of flow intercepted by the grate (basin flow – bypass flow).

4. The inlet bypass flow on a continuous grade is computed as follows:

$$Q_{BP} = Q\left[\frac{(T - GW)}{(T)}\right]^{\frac{8}{3}} \qquad (8\text{-}14)$$

Where: Q_{BP} = portion of flow outside the grate width (cfs);

Q = total flow of gutter approaching the inlet (cfs);

T = spread, calculated from the gutter section upstream of the inlet (feet); and

GW = grate inlet width perpendicular to the direction of flow (feet), see Table 8-9.

5. The velocity shall not exceed 5 feet/second. The velocity of flow directly over the inlet is calculated as follows:

$$V_1 = \frac{Q - Q_{BP}}{(GW)\left[d - 0.5(GW)(S_x)\right]} \qquad (8\text{-}15)$$

Where: V_1 = velocity over the inlet (feet/second);

S_x = cross slope (feet/foot); and,

d = depth of flow at the face of the curb (feet), given by:

$$d = (T)(S_x) \qquad (8\text{-}16)$$

If the non-flooded road width does not meet the minimum criteria, an additional inlet should be placed at an intermediate location and the procedure repeated. If the velocity exceeds 5 feet/second then side flow shall be considered using the method outlined in HEC-22.

6. The analysis is then repeated with the next inlet. The bypass flow (Q_{BP}) from the previous inlet shall be added to the flow from the contributing basin to determine the total flow (to the inlet at the station being analyzed.

7. The last inlet may require an adjustment of spacing (usually smaller spacing) in order to prevent a bypass flow to the project boundaries.

Curb Inlets, Continuous Grade

The capacity of a curb inlet on a continuous grade depends upon the length of opening and the depth of flow at the opening. This depth in turn depends upon the amount of depression of the flow line at the inlet, the cross slope, the longitudinal slope, and the roughness of the gutter. The analysis shall include a 35% clogging factor.

The capacity of a curb inlet on a continuous grade may be calculated using the procedure outlined below. Example calculations for curb inlets on a continuous grade for a uniform gutter section and a composite gutter section are provided in Appendices 8E and 8F.

1. Determine the runoff from the contributing basin at the high point to the first curb inlet. This is the amount of runoff that could be intercepted by the first curb inlet.
2. Analyze the most upstream inlet. The width of flow (T) is calculated using the procedure described in Section 8.6.2. Verify that T is within the allowable limit (Table 8-7).
3. The length of the curb-opening inlet required for total interception of gutter flow is calculated as follows:

$$L_T = 0.6Q^{0.42}S_L^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \tag{8-17}$$

Where: L_T = curb opening length required to intercept 100% of the flow (feet);

S_e = equivalent cross slope (feet/foot);

for uniform gutter sections: $S_e = S_x$; and,

for composite gutter sections:

$$S_e = S_x + E_o\left(S_w - S_x\right) = S_x + \left(\frac{E_o a}{12W}\right) \tag{8-18}$$

where: a = gutter depression (inches);

E_o = ratio of flow in the depressed section to total gutter flow, calculated in the gutter configuration upstream of the inlet; and,

W = gutter width (feet).

4. When the actual curb inlet is shorter than the length required for total interception, calculate the efficiency of the curb inlet using Equation 8-19.

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \tag{8-19}$$

Where: E = efficiency; and,

L = actual curb opening length (feet).

5. Compute the interception capacity of the curb inlet using the following relationship:

$$Q_i = (E)(Q) \tag{8-20}$$

Where: Q_i = curb inlet capacity (cfs),

6. The analysis is then repeated with the next inlet. The bypass flow (Q_{BP}) from the previous inlet shall be added to the flow from the contributing basin to determine the total flow (Q) to the inlet at the station being analyzed.

$$Q_{BP} = Q - Q_i \tag{8-21}$$

7. The last inlet may require an adjustment of spacing (usually smaller spacing) in order to prevent a bypass flow to the project boundaries.

Combination Inlets, Sump Condition

Inlets in sump locations perform differently than inlets on a continuous grade. Inlets in sump locations operate in one of two ways: 1) as a weir, at low ponding depths; or 2) as an orifice, at high ponding depths (1.4 times the grate opening length). It is very rare that ponding on a roadway will become deep enough to force the inlet to operate as an orifice; therefore, this section will focus on the inlet operating as a weir.

The interception capacity of a combination inlet in a sump is equal to that of a grate inlet alone in weir flow. Design procedures presented here are a conservative approach to estimating the capacity of inlets in sump locations. All inlets in a sump condition shall be evaluated using a 50% clogging factor.

The analysis shall include an evaluation of the inlet and the surrounding street, gutter, curb and adjacent properties for storm events exceeding the required level of service. An emergency overflow path shall be provided.

The capacity of a combination inlet operating in a sump as a weir may be estimated using the following procedure. There are also commercially available software programs that will analyze combination inlets in a sump location. An example calculation for a combination inlet in a sump location is provided in Appendix 8G.

1. Determine the runoff contributing to the combination inlet. This is the sum of the bypassed flows from all upstream inlets and the runoff generated from the basin contributing directly to the combination inlet.
2. Determine the allowable spread (T_{all}) based on the non-flooded width requirements in Table 8-7.
3. Calculate the depth of flow at the curb (d) using Equation 8-16.

4. Determine the average depth of flow over the grate using one of the following relationships:

   For uniform gutter sections:

$$d_{ave} = d - S_x\left(\frac{W}{2}\right) + y \tag{8-22}$$

   For composite gutter sections:

$$d_{ave} = d + \frac{W}{2}\left(S_w - 2S_x\right) + y \tag{8-23}$$

   Where: y = local depression (feet), Spokane County Standard Plans B-7 and B-18 show a 1-inch local depression at the grate.

5. Calculate the allowable flow (Q_{all}) using the following relationship:

$$Q_{all} = CPd^{3/2} \tag{8-24}$$

   Where: Q_{all} = allowable flow based upon the maximum allowable spread (cfs);

   P = perimeter of the grate inlet (refer to Table 8-9 for projects in Spokane County and the City of Spokane Valley);

   d = average depth of water across the grate (feet); and,

   C = may be taken as 3.0.

6. Compare the allowable flow to the actual flow. If the actual flow is less than the allowable flow then the combination inlet capacity is adequate. Otherwise, changes shall be made to the design and steps 1 through 5 repeated.

Curb Inlets, Sump Condition

The procedure below assumes that the curb inlet is operating as a weir and the depth of flow is less than the height of the curb opening.

The capacity of a concrete curb inlet (no grate) in a sump condition may be calculated by the method described below. An example calculation for a curb inlet in a sump location is provided in Appendix 8H.

1. Determine the runoff contributing to the curb inlet. This is the sum of the bypassed flows from all upstream inlets and the runoff generated from the basin contributing directly to the combination inlet.

2. Determine the allowable spread (T_{all}) based upon the non-flooded width requirements found in Table 8-7.
3. Calculate the depth of flow at the curb (d).
4. Calculate the allowable flow (Q_{all}) using one of the following relationships:

   For a depressed curb opening inlet:

$$Q_{all} = 2.3(L + 1.8W)d^{3/2} \tag{8-25}$$

Where: Q_{all} = allowable flow based upon the maximum allowable spread (cfs);

W = lateral width of depression (feet);

L = length of curb opening (feet); and,

d = depth of flow at the curb (feet).

For a curb opening inlet without a depression:

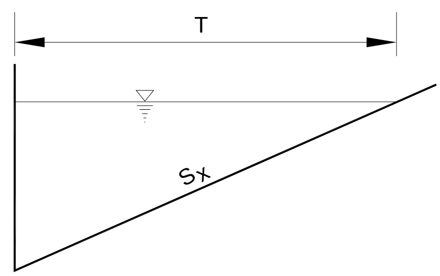
$$Q_{all} = 3.0Ld^{3/2} \tag{8-26}$$

5. Compare the allowable flow to the actual flow. If the actual flow is less than the allowable flow then the curb inlet capacity is adequate. Otherwise, changes shall be made to the design and steps 1 through 4 repeated.

APPENDIX 8A – EXAMPLE CALCULATION: NON-FLOODED WIDTH (UNIFORM GUTTER SECTION)

GIVEN

- A crowned private road with a uniform gutter section (as illustrated), assuming an equal flow rate on each side of the road.
  - Flow rate (Q) = 4.2 cfs
  - Gutter width (W) = 1.5 feet
  - Road/Gutter cross slope (S_X) = 0.02 feet/foot
  - Longitudinal slope (S_L) = 0.01 feet/ft
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet



CALCULATIONS

1. Calculate the spread (T) for half of the roadway using Equation 8-8.

$$T = \left(\frac{Q\ \ n}{0.56\ \ S_x^{1.67} S_L^{0.5}} \right)^{0.375} = \left(\frac{(4.2)(0.016)}{0.56\ \ (0.02)^{1.67} (0.01)^{0.5}} \right)^{0.375} = 12.4\ \text{ feet}$$

2. Calculate the non-flooded width using the following relationship for crowned roadways, and then verify that the non-flooded width is within the allowable limit (refer to Table 8-7):

Non-flooded width	=	2[(½)(RW) + W – T)]
	=	2[(½)(30) + 1.5 – 12.4)]
	=	8.2 feet < 12 feet **FAIL***

* Table 8-7 indicates that the minimum non-flooded width is 12 feet for private roads. Therefore, the design fails to meet the required non-flooded road width criteria. The design will need to be altered (i.e. try an additional inlet placed at an intermediate location, contributing basins redefined, new flow rates calculated, and the above steps repeated).

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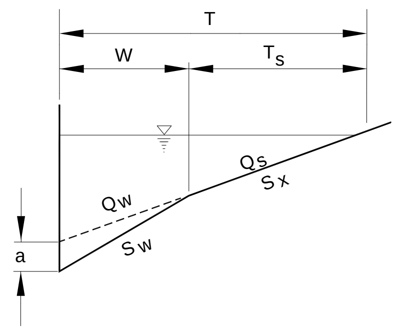
APPENDIX 8B – EXAMPLE CALCULATION: NON-FLOODED WIDTH (COMPOSITE GUTTER SECTION)

GIVEN

- A super-elevated local access road with a composite gutter section (as illustrated).
  - Flow rate (Q) = 4.2 cfs
  - Gutter width (W) = 1.5 feet
  - Road cross slope (S_X) = 0.02 feet/foot
  - Gutter cross slope (S_W) = .081 feet/foot
  - Longitudinal slope (S_L) = 0.01 feet/foot
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet



CALCULATIONS

1. Assume a flow rate (Q_s) for that portion of the flow above the depressed gutter section.

   Assume Q_s = 1.4 cfs

2. Calculate Q_w using Equation 8-9.

$$Q_w = Q - Q_s = 4.2 - 1.4 = 2.8\text{cfs}$$

3. Calculate E_o using Equation 8-11.

$$E_o = \frac{Q - Q_s}{Q} = \frac{Q_w}{Q} = \frac{2.8}{4.2} = 0.67$$

4. Calculate the spread (T) using Equation 8-12.

$$T = W\left\{1 + \left(\frac{\frac{S_w}{S_x}}{\left[\frac{S_w}{S_x}\left(\frac{E_o}{1-E_o}\right)+1\right]^{3/8} - 1}\right)\right\} = 1.5\left\{1 + \left(\frac{\frac{0.081}{0.02}}{\left[\left(\frac{0.081}{0.02}\right)\left(\frac{0.67}{1-0.67}\right)+1\right]^{3/8} - 1}\right)\right\} = 6.17 \text{ ft}$$

5. Calculate T_S using Equation 8-13.

$$T_S = T - W = 6.17 - 1.5 = 4.67\text{ft}$$

6. Use Equation 8-7 to compute Q_s for the calculated Ts, then compare to the estimated Q_s from Step 1.

$$Q_S\,(\text{computed}) = \frac{0.56\ S_x^{1.67} S_L^{0.5} T_S^{2.67}}{n} = \frac{0.56\ (0.020)^{1.67}(0.01)^{0.5}(4.67)^{2.67}}{0.016} = 0.31\ \text{cfs} < 1.4\ \text{cfs}$$

Since Q_s (estimated) and Q_s (computed) are not approximately equal, repeat Steps 1 through 6 until the estimated and computed Q_s are numerically closer in value.

7. Assume a new Q_s and repeat steps 2 through 6. The following parameters are calculated using Q_s = 2.6 cfs.

Q_w = 1.6 cfs

E_o = 0.38

T = 11.68 feet

T_S = 10.18 feet

Q_s = 2.5 cfs (computed)

Q_s (estimated) ≈ Q_s (computed)

Note that a spreadsheet can be set up to perform the above calculations, and commercially available software can calculate spread in composite gutters.

8. Now that T has been found for the relationship: Qs (estimated) ≈ Qs (calculated), calculate the non-flooded width using the following relationship for super-elevated roadways, and then verify that the non-flooded width is within the allowable limit (refer to Table 8-7):

Non-flooded width = RW + 2W - T

= 30 + 2(1.5) - 11.68

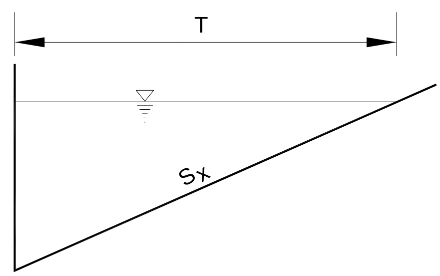
= 21.3 feet > 12 feet **OK***

* Table 8-7 indicates that the minimum non-flooded width is 12 feet for local access roads. Therefore, the design has met the required non-flooded road width criteria.

APPENDIX 8C – EXAMPLE CALCULATION: GRATE INLET CAPACITY (UNIFORM GUTTER SECTION)

GIVEN

- A crowned private road with a uniform gutter section (as illustrated), assuming an equal flow rate on each side of the road.
  - Flow rate (Q) = 2.5 cfs
  - Gutter width (W) = 1.5 ft
  - Spokane County Type 1 Grate (Standard Plan B-12) Grate width (GW) = 1.67 feet
  - Road/Gutter cross slope (S_X) = 0.02 feet/foot
  - Longitudinal slope (S_L) = 0.03 feet/foot
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet



CALCULATIONS

1. Determine the runoff from the contributing basin at the high point to the first inlet;

   For this example, the design flow rate (Q) is given as 2.5 cfs

2. Select an inlet and note the grate width.

   For this example, the grate width (GW) is given as 1.67 ft

3. Calculate the spread (T) for half of the roadway using Equation 8-8.

$$T = \left(\frac{Q\ n}{0.56\ S_x^{1.67} S_L^{0.5}} \right)^{0.375} = \left(\frac{(2.5)(0.016)}{0.56\ (0.02)^{1.67} (0.03)^{0.5}} \right)^{0.375} = 8.31 \text{feet}$$

4. Calculate the non-flooded width using the following relationship, and then verify that the non-flooded width is within the allowable limit (refer to Table 8-7):

Non-flooded width	=	2[(½)(RW) + W – T)]
	=	2[(½)(30) + 1.5 – 8.31)]
	=	16.38 feet > 12 feet **OK***

* Table 8-7 indicates that the minimum non-flooded width is 12 feet for private roads. Therefore, design has met the required non-flooded road width criteria.

5. Calculate the inlet bypass flow using Equation 8-14:

   With 35% clogging factor, grate width (GW) = 1.67(1 - 0.35) = 1.09’

$$Q_{BP} = Q\left[\frac{(T)-(GW)}{(T)}\right]^{\frac{8}{3}} = 2.5\left[\frac{8.31-1.09}{8.31}\right]^{\frac{8}{3}} = 1.72\text{cfs}$$

   Therefore the capacity of the inlet = 2.5 – 1.72 = 0.78 cfs

6. Verify that the velocity does not exceed 5 feet/second. The velocity of flow directly over the inlet is calculated using Equation 8-15 (where d = T S_x):

$$V_1 = \frac{Q-Q_{BP}}{(GW)\left[d-0.5(GW)(S_x)\right]} = \frac{2.5-1.72}{1.09\left[(8.31)(0.02)-0.5(1.09)(.02)\right]} = 4.61\text{ft/s} < 5 \text{ feet/second } \textbf{OK**}$$

   **Refer to Section 8.7.2 for guidance when the velocity exceeds 5 feet/second.

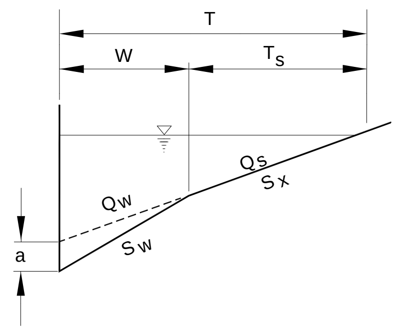
7. The analysis is then repeated with the next inlet. The bypass flow (Q_{BP}) from the previous inlet shall be added to the flow from the contributing basin to determine the total flow (Q) to the inlet at the station being analyzed.

Note that the City of Spokane requires the analysis to include a 50% clogging factor.

APPENDIX 8D – EXAMPLE CALCULATION: GRATE INLET CAPACITY, CONTINUOUS GRADE (COMPOSITE GUTTER SECTION)

GIVEN

- A super-elevated local access road with a composite gutter section (as illustrated)
  - Flow rate (Q) = 4.2 cfs
  - Gutter width (W) = 1.5 feet
  - Spokane County Type 1 Grate (Standard Plan B-12) Grate Width (GW) = 1.67 feet
  - Road cross slope (S_X) = 0.02 feet/foot
  - Gutter cross slope (S_W) = .081 feet/foot
  - Longitudinal slope (S_L) = 0.01 feet/foot
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet



CALCULATIONS

1. Determine the runoff from the contributing basin at the high point to the first inlet;

   For this example, the design flow rate is given as 4.2 cfs

2. Select an inlet and note the grate width.

   For this example, the grate width (GW) is given as 1.67 feet

3. Calculate the spread (T) for half of the roadway using the method outlined in Appendix 8B and verify that the non-flooded width is within the allowable limit (Table 8-7).

   T = 11.68 feet
   (Solution from Appendix 8B)

   Non-flooded width = 21.3 feet > 12 feet **OK***
   (Solution from Appendix 8B)

   * Table 8-7 indicates that the minimum non-flooded width is 12 feet for private roads. Therefore, design has met the required non-flooded road width criteria.

4. Calculate the inlet bypass flow using Equation 8-14:

   With 35% clogging factor, grate width (GW) = 1.67(1 - 0.35) = 1.09'

$$Q_{BP} = Q\left[\frac{(T)-(GW)}{(T)}\right]^{\frac{8}{3}} = 4.2\left[\frac{11.68-1.09}{11.68}\right]^{\frac{8}{3}} = 3.23\text{cfs}$$

Therefore the capacity of the inlet = 4.2 – 3.23 = 0.97 cfs

5. Verify that the velocity does not exceed 5 feet/second. The velocity of flow directly over the inlet is calculated using Equation 8-15:

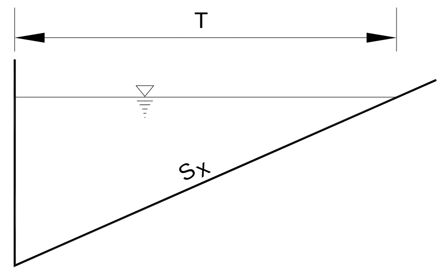
$$V_1 = \frac{Q-Q_{BP}}{(GW)\left[d-0.5(GW)(S_x)\right]} = \frac{4.2-3.23}{1.09\left[(11.68)(0.02)-0.5(1.09)(0.02)\right]} = 4.00\text{ft/s} < 5 \text{ feet/second } \textbf{OK}$$

6. The analysis is then repeated with the next inlet. The bypass flow (Q_{BP}) from the previous inlet shall be added to the flow from the contributing basin to determine the total flow (Q) to the inlet at the station being analyzed.

APPENDIX 8E – EXAMPLE CALCULATION: CURB INLET CAPACITY, CONTINUOUS GRADE (UNIFORM GUTTER SECTION)

GIVEN

- A crowned private road with a uniform gutter section (as illustrated), assuming an equal flow rate on each side of the road.
  - Flow rate (Q) = 1.5 cfs
  - Gutter width (W) = 1.5 feet
  - Curb Inlet Length (L) = 3 feet
  - Road/Gutter cross slope (S_x) = 0.02 feet/foot
  - Longitudinal slope (S_L) = 0.03 feet/foot
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet



CALCULATIONS

1. Determine the runoff from the contributing basin at the high point to the first inlet;

   For this example, the design flow rate is given as 1.5 cfs

2. Calculate the spread (T) for half of the roadway using Equation 8-8 and verify that the non-flooded width is within the allowable limit (Table 8-7).

$$T = \left(\frac{Q \ n}{0.56 \ S_x^{1.67} S_L^{0.5}} \right)^{0.375} = \left(\frac{(1.5)(0.016)}{0.56 \ (0.02)^{1.67} (0.03)^{0.5}} \right)^{0.375} = 6.86 \text{feet}$$

Non-flooded width = 2[(½)(RW) + W – T)]

= 2[(½)(30) + 1.5 – 6.86)]

= 19.3 feet > 12 feet **OK***

* Table 8-7 indicates that the minimum non-flooded width is 12 feet for private roads. Therefore, design has met the required non-flooded road width criteria.

3. Calculate the length of curb inlet required for total interception of gutter flow using Equation 8-17:

$$L_T = 0.6Q^{0.42}S_L^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} = (0.6)(1.5^{0.42})(0.03^{0.3})\left(\frac{1}{0.016*0.02}\right)^{0.6} = 31.1\text{feet}$$

4. Calculate the efficiency of the curb inlet using Equation 8-19.

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{3.0}{31.1}\right)^{1.8} = 0.167$$

5. Compute the interception capacity and the bypass flow of the curb inlet using Equations 8-20 and 8-21.

$$Q_i = (E)(Q) = (0.167)(1.5) = 0.25\text{cfs}$$

$$Q_{BP} = Q - Q_i = 1.5 - 0.25 = 1.25\text{cfs}$$

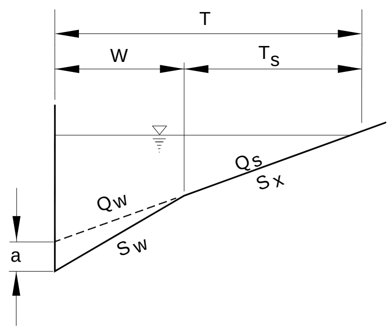
6. The analysis is then repeated with the next curb inlet. The bypass flow (Q_{BP}) from the previous inlet shall be added to the flow from the contributing basin to determine the total flow (Q) to the next inlet.

Note that the City of Spokane requires the analysis to include a 50% clogging factor.

APPENDIX 8F – EXAMPLE CALCULATION: CURB INLET CAPACITY, CONTINUOUS GRADE (COMPOSITE GUTTER SECTION)

GIVEN

- A super-elevated local access road with a composite gutter section (as illustrated)
    - Flow rate (Q) = 4.2 cfs
    - Gutter width (W) = 1.5 feet
    - Curb Inlet Width (GW) = 3 feet
    - Road cross slope (S_x) = 0.02 feet/foot
    - Gutter cross slope (S_w) = .081 feet/foot
    - Longitudinal slope (S_L) = 0.01 feet/foot
    - Manning's friction coefficient, n = 0.016
    - Road width (RW) = 30 feet



CALCULATIONS

1. Determine the runoff from the contributing basin at the high point to the first inlet;

    For this example, the design flow rate is given as 4.2 cfs

2. Calculate the spread (T) for half of the roadway using the method outlined in Appendix 8B and verify that the non-flooded width is within the allowable limit (Table 8-7).

    T = 11.68 feet (Solution from Appendix 8B)

    Non-flooded width = 21.3 feet > 12 feet **OK*** (Solution from Appendix 8B)

    * Table 8-7 indicates that the minimum non-flooded width is 12 feet for private roads. Therefore, design has met the required non-flooded road width criteria.

3. Calculate the equivalent cross slope (S_e) using Equation 8-18 and the length of curb inlet required for total interception of gutter flow (L_T) using Equation 8-17.

$$S_e = S_x + E_o\left(S_w - S_x\right) = 0.02 + 0.38(0.081 - 0.02) = 0.043$$

Where, E_o = 0.38 (Solution from Appendix 8B)

$$L_T = 0.6Q^{0.42}S_L^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} = (0.6)(4.2^{0.42})(0.01^{0.3})\left(\frac{1}{(0.016)(0.043)}\right)^{0.6} = 21.8\text{feet}$$

4. Calculate the efficiency of the curb inlet using Equation 8-19.

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{3.0}{21.8}\right)^{1.8} = 0.23$$

5. Compute the interception capacity and the bypass flow of the curb inlet using Equations 8-20 and 8-21.

$$Q_i = (E)(Q) = (0.23)(4.2) = 0.97\text{cfs}$$

$$Q_{BP} = Q - Q_i = 4.2 - 0.97 = 3.23\text{cfs}$$

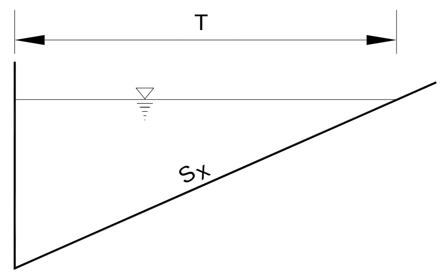
6. The analysis is then repeated with the next curb inlet. The bypass flow (Q_{BP}) from the previous inlet shall be added to the flow from the contributing basin to determine the total flow (Q) to the next inlet.

Note that the City of Spokane requires the analysis to include a 50% clogging factor.

APPENDIX 8G – EXAMPLE CALCULATION: COMBINATION INLET CAPACITY, SUMP

GIVEN

- A crowned private road with a uniform gutter section (as illustrated).
  - Inlet: Metal Frame with Hood, Type 2 and Bi-Directional Vaned Grate, Type 3 – Spokane County Standard Plans B-11 and B-14
  - Gutter Width (W) = 1.5 feet
  - Local depression = 1 inch
  - Cross slope (S_x) = 0.02 feet/foot
  - Road width (RW) = 30 feet
  - Q_{BP} = 0.68 cfs = Upstream inlets total bypass flow rate
  - Q_{BASIN} = 0.82 cfs = Contributing drainage basin direct flow rate



CALCULATIONS

1. Determine the total runoff contributing and bypassed to the combination inlet.

$$Q_{TOTAL} = Q_{BP} + Q_{BASIN} = 0.68\text{cfs} + 0.82\text{cfs} = 1.5\text{cfs}$$

2. From Table 8-7, the non-flooded width for a private road is 12 feet minimum. Determine the allowable spread (T) for the roadway using the following relationship for a crowned roadway:

$$T_{all} = \frac{\text{R}W + 2W - \text{Non - flooded Width}}{2} = \frac{30 + (2)(1.5) - 12}{2} = 10.5\,\text{feet}$$

3. Calculate the depth of flow at the curb (d) using Equation 8-16.

$$d = (T)(S_x) = (10.5)(0.02) = 0.21\,\text{feet}$$

4. Determine the average depth of flow over the grate using Equation 8-22.

$$d_{ave} = d - S_x\left(\frac{W}{2}\right) + y = 0.21 - 0.02\left(\frac{1.5}{2}\right) + \frac{1}{12} = 0.28\,\text{feet}$$

5. Calculate the allowable flow (Q_{all}) using Equation 8-24.

$$Q_{all} = CPd^{3/2} = (3.0)(3.13)(0.28)^{3/2} = 1.38\,\text{cfs}$$

6. Compare the allowable flow to the actual flow.

$$1.38\,\text{cfs}(Q_{all}) \prec 1.5\,\text{cfs}\,(Q) \qquad \textbf{FAIL*}$$

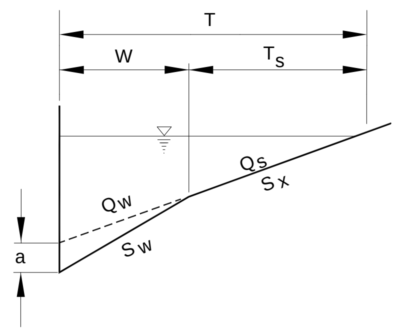
   * The actual flow rate directed at the given metal frame and grate inlet combination exceeds the calculated allowable flow capacity of the structure. The design will need to be altered (i.e. try an additional inlet placed at an intermediate location, contributing basins redefined, new flow rates calculated, and the above steps repeated).

Note that grate perimeter used in this example includes a 50% clogging factor (refer to Table 8-9).

APPENDIX 8H – EXAMPLE CALCULATION: CURB INLET CAPACITY, SUMP

GIVEN

- A crowned private road with a composite gutter section (as illustrated).
  - Curb opening length (L) = 3.0 feet (reduce by half – clogging safety factor)
  - Local depression = 1 inch
  - Cross slope (S_X) = 0.02 feet/foot
  - Gutter cross slope (S_W) = 0.081 feet/foot
  - Gutter Width = 1.5 feet
  - Road width (RW) = 30 feet
  - Q_{BP} = 0.68 cfs = Upstream inlets total bypass
  - Q_{BASIN} = 0.82 cfs = Contributing drainage basin direct flow rate



CALCULATIONS

1. Determine the total runoff contributing and bypassed to the curb inlet.

$$Q_{TOTAL} = Q_{BP} + Q_{BASIN} = 0.68\text{cfs} + 0.82\text{cfs} = 1.5\text{cfs}$$

2. From Table 8-7, the non-flooded width for a private road is 12 feet minimum. Determine the allowable spread (T) for the roadway using the following relationship for crowned roadways:

$$T_{all} = \frac{\text{R}W + 2W - \text{Non-flooded Width}}{2} = \frac{30 + (2)(1.5) - 12}{2} = 10.5\text{ feet}$$

3. Calculate the depth of flow at the curb (d).

$$d = (1.5)(0.081) + (10.5 - 1.5)(0.02) = 0.30\text{ feet}$$

4. Calculate the allowable flow (Q_{all}) using Equation 8-25.

$$Q_{all} = 2.3(L + 1.8W)d^{3/2} = 2.3[(1.5 + (1.8)(1.5)](0.30)^{3/2} = 1.59\text{ cfs}$$

5. Compare the allowable flow to the actual flow.

$$1.59\text{cfs}(Q_{all}) > 1.5\,\text{cfs}\,(Q) \qquad \textbf{OK*}$$

* The actual flow rate directed at the curb inlet is less than the calculated allowable flow capacity of the structure. The design is adequate.