CHAPTER 8 – CONVEYANCE

Chapter Organization

8.1 INTRODUCTION

A conveyance system includes all natural or constructed components of a storm drain system that collects stormwater runoff and conveys it away from structures in a manner that adequately drains sites and roadways, 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, while taking into consideration the potential for damage to adjacent properties, should also attempt to reduce the degree of risk to traffic interruption due to flooding within the traveled roadway. The conveyance system shall also provide discharge capacity sufficient to convey the design flow at velocities that are self-cleansing without being destructive to the conveyance facilities. These objectives are achieved by designing all conveyance facilities utilizing the design storm event specified for the given facility and by adhering to various parameters such as minimum and maximum velocity, freeboard, cover, etc.

A properly designed conveyance system also attempts to maximize the hydraulic efficiency by utilizing the proper material, slope and size. Open constructed conveyance systems should emulate the natural, pre-developed conditions to the maximum extent feasible. Natural drainageways must be preserved and protected; filling them in or building on top of them is not acceptable practice. It may be necessary to set aside some drainageways or land features for regional use (see Section 8.2.4 for criteria).

Inflow to, and discharges from, the constructed system shall occur at the natural drainage points in the same manner as the existing conditions as determined by topography and existing drainage patterns. Stormwater discharge should not be diverted into irrigation canals without approval from the irrigation district and any other applicable regulating agencies. Existing drainage patterns should be discussed in the Drainage Report and reviewed in the downstream analysis. (See Section 3.4.5)

8.1.1 DESIGN FLOW

The design flow, and overflow requirements for each type of conveyance system is outlined below. Either the NRCS Hydrograph Method or the Rational Method can be used to size the conveyance system. These requirements are consistent with those listed in Chapter 2. Additional design criteria and analysis requirements are outlined in the remaining sections of this chapter.

Constructed Channels

Constructed channels shall be designed with sufficient capacity to convey and contain, at a minimum, the depth of the 50-year peak flow plus an additional 30 percent, assuming developed conditions for onsite tributary areas and existing conditions for any offsite tributary areas.

The design shall safely bypass storm events that exceed the above criteria and shall provide an overflow path, with the capacity to convey the 100-year storm event, assuming developed conditions for onsite tributaries and existing conditions for any offsite tributaries. The overflow should drain toward the natural discharge point of the contributing basin. The overflow path must be capable of conveying the 100-year storm event and should drain toward the natural discharge point of the contributing basin, away from adjacent buildings, residences, etc. (Note: Per Oregon Drainage Law, overflows leaving the project site cannot exceed natural flow rates without agreement from the downstream property owners.)

Culverts

New culverts shall be designed with sufficient capacity to convey the 50-year design storm assuming developed conditions for the onsite basin and existing conditions for any offsite contributing basins. Increase culvert size to pass the 100-year event if a safe overflow for flows above the 50-year event cannot be provided.

Storm Drain Systems and Inlets

New enclosed systems and inlets shall be designed with sufficient capacity to convey peak flow rate for the 25-year design storm event with at least six-inches of freeboard between the water surface and the proposed adjacent ground surface. Enclosed systems may surcharge or overtop drainage structures for storm events that exceed the 25-year event, so long as a safe overflow path is provided. The overflow path must be capable of conveying the 100-year storm event (assuming developed conditions for onsite tributaries and existing conditions for any offsite tributaries). The overflow should drain toward the natural discharge point of the contributing basin, away from adjacent buildings, residences, etc. (Note: Per Oregon Drainage Law, overflows leaving the project site cannot exceed natural flow rates without agreement from the downstream property owners.)

Gutters

Gutter flows in roadways shall allow for the passing of vehicular traffic during the 25-year design storm event by providing non-flooded zones. For paved roadways, the non flooding width requirement varies with the classification of the road. See Table 8-8 for non-flooded width requirements.

Drainage Inlets

Drainage inlets shall be designed with sufficient capacity to convey the 25-year design storm assuming developed conditions for the contributing area.

8.2 CONSTRUCTED CHANNELS

8.2.1 CHANNEL ANALYSIS

A channel analysis shall be performed for all constructed channels proposed for the project and for all field verified existing natural drainageways/channels present onsite. Channel analysis may be performed by hand calculations, spreadsheets, or using any of a number of widely available software programs. The following items shall be included in the drainage report and the road and drainage plans, when applicable:

- Complete channel calculations that state the design peak flow rates and design information, such as channel shape, slope, Manning's coefficient (Table 8-4). Calculations shall be provided for each distinct channel section (i.e. if the slope, shape or roughness change significantly);
- The velocity, capacity, and Froude number shall be calculated whenever the geometry of the channel changes;
- The centerline and direction of flow for all constructed drainage ditches or natural channels located within the project limits, are to be clearly shown in the construction plans and basin map. For all proposed channels, locating information shall be provided at all angle points; and,
- Channel protection shall be provided when velocities exceed permissible velocities (See Table 8-1). The calculations shall justify the protection area, thickness, material size and gradation, and filter blanket reinforcement, if needed. This information shall be included in the plans;

TABLE 8-1
PERMISSIBLE CHANNEL VELOCITIES

SOIL TYPE OF LINING (EARTH; NO VEGETATION)	MAXIMUM PERMISSIBLE VELOCITIES (ft/sec)		
	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

TABLE 8-1 CONTINUED

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. Based on uniform flow in continuously wet, aged channels with erodible linings.

- The Froude number shall be checked near the beginning and near the end of a channel that has significantly different grade changes to determine if a hydraulic jump occurs (Froude number changes 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, checkdams, or other protective measures to ensure that the jump does not erode the conveyance facility.
- When geosynthetics are used for channel stabilization, follow ODOT procedures for design and material specification. The plans shall clearly specify fabric type, placement, and anchoring requirements; and,
- The plans shall specify seed mixture and irrigation, as applicable, if grass-lined channels are proposed.

8.2.2 MINIMUM REQUIREMENTS FOR CONSTRUCTED CHANNELS

Slope

Minimum grades for constructed channels shall be the following:

- ½ percent (0.005 feet/feet) for cement concrete;
- 1 percent (0.010 feet/feet) for asphalt concrete; and,
- ½ percent (0.005 feet/feet) for graded earth or close cropped grass; and
- 1 percent (0.010 feet/feet) for rip rap lined channels.

Note: Non-structured alternatives are preferred over asphalt and concrete channels whenever possible.

Side Slopes

Ditches may be “V” shaped or trapezoidal. However, V-ditches are not recommended in easily erodible soils or where problems establishing vegetation can be anticipated.

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

Ditches or channels shall not have side slopes that exceed the natural angle of repose for a given material or per Table 8-2:

TABLE 8-2
MAXIMUM DITCH OR CHANNEL SIDE SLOPES

TYPE OF CHANNEL	SIDE SLOPE HORIZONTAL: VERTICAL
Firm Rock	Vertical to ¼:1
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

Location

Constructed channels cannot be placed within or between residential lots. Ditches and channels shall be located within a drainage tract or within a border easement. Large lot subdivisions (lots ≥ 1 acre) may be allowed to have ditches or channels traverse through the lot(s) and consideration may be given as to placement within an easement versus a tract. The local jurisdiction will review these proposals on a case by case basis.

Depth

The minimum depth of open channels shall be 1.30 times the design flow depth (ft) or one foot; whichever is greater. This provides a minimum 30 percent freeboard depth above the design flow water surface.

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 during the design flow, channel protection is required (see Section 8.2.3). In addition, the following criteria shall apply:

- Where only sparse vegetative cover can be established or maintained, velocities should not exceed 3 fps;
- Where medium density vegetation can be established by seeding, velocities in the range of 3 to 4 fps 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 fps are permitted; and,
- On well established sod of good quality, velocities in the range of 5 to 6 fps are permitted.

8.2.3 CHANNEL DESIGN

Channel Capacity

Open channels shall be sized using the following iteration of Manning's formula, equation 8-1.

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

Where: Q = rate of flow (cfs);

V = mean velocity in channel (ft/s);

A = cross-sectional area of flow in the channel (ft^2);

n = Manning's coefficient (See Table 5-4);

S = channel slope (ft/ft); and,

R = hydraulic radius (ft) = Area/wetted perimeter

Note: The Manning 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. Strictly speaking, uniform flow conditions seldom, if ever, occur in nature because channel sections change from point to point. For practical purposes, however, the Manning equation can be applied to most open channel flow problems by making judicious assumptions.

Most hydraulics handbooks contain tables that simplify the calculations, and commercially available software is often used for channel design.

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, outlet aprons, and gabion weirs.

Channel Protection

Channel velocities shall be analyzed periodically throughout the channelized route, particularly at the following locations:

- At the first point that the stormwater runoff becomes concentrated into a natural or constructed channel;
- At any significant change in channel configuration (grade, sideslopes, depth, shape, etc.); and
- Any significant change in flow (i.e. immediately downstream of an outfall)

If channel velocities are found to exceed the limits in Table 8-1, then channel protection shall be provided. Channel protection material shall be selected based on the revetment and armoring capabilities. The Manning's "n" value shall be analyzed in the given channel configuration to determine if the material proposed will reduce the velocity to a manageable speed. If the calculations reveal that common materials such as matting or riprap are not the best solution to protect the channel from erosion, stronger protection such as gabions and/or stilling pools may be necessary. Non-structural options, such as vegetation and woody debris, should also be considered.

Riprap Protection at Outlets

If the flow velocity at a conduit outlet exceeds the maximum permissible velocity for the soil or channel lining, outlet protection is required. The protection usually consists of an erosion resistant reach, such as riprap, between the outlet and the stable downstream channel to provide a stable reach at the outlet in which the exit velocity is reduced to a velocity allowable in the downstream channel. The design of such protection is based on the design storm runoff event. If protection is needed at the outlet, energy dissipation shall be provided.

The ability of riprap revetment to resist erosion is related to size, shape and weight of the stones. Most riprap lined channels require either a gravel filter blanket or filter fabric under the riprap. Where very large riprap is used it is sometime necessary to use two gravel filter layers between the subgrade and the riprap.

Apron Dimensions

The length of an apron (L_a) as shown in Figure 8-1, is determined using the following empirical relationships that were developed for the U.S. Environmental Protection Agency (1976):

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

Or

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

Where: D_o = maximum inside culvert width (ft);

Q = pipe discharge (cfs); and,

TW= tailwater depth (ft) at the outlet pipe.

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 as follows:

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

Or

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

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 one foot above the tailwater elevation.

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

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

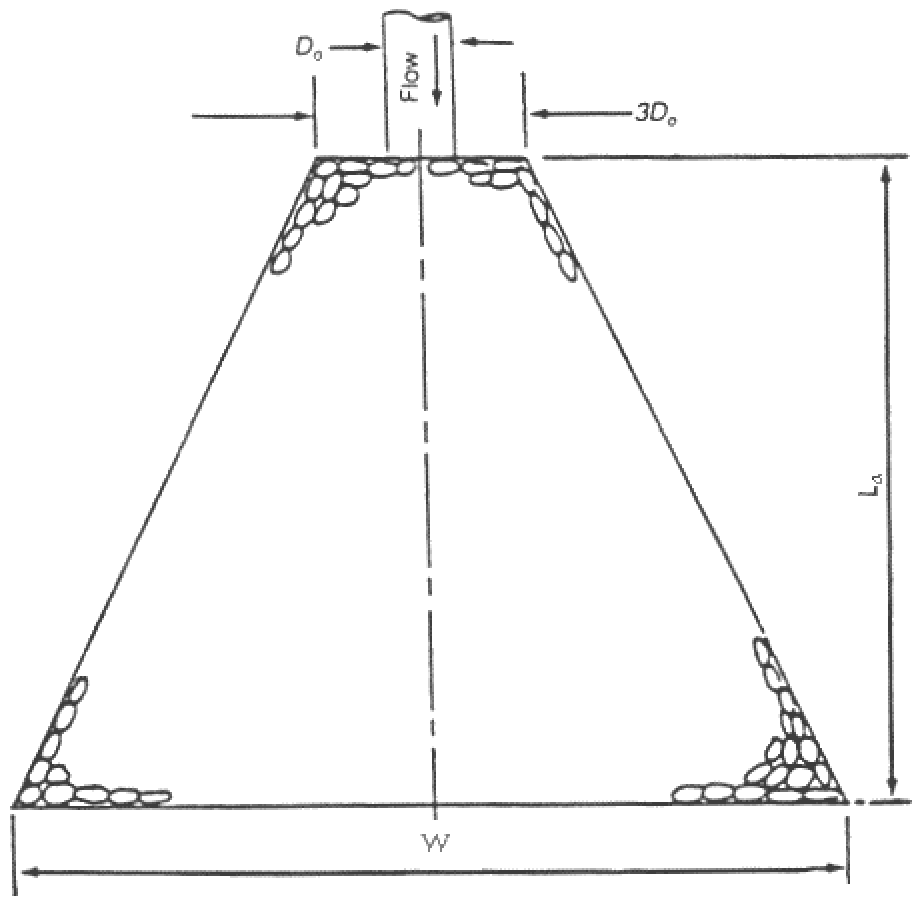


Figure 8-1 – Riprap Revetment at Outfall Schematic

Apron Materials

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

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

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

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

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

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

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

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

A filter 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. 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 sometime necessary to use two filter blanket layers between the subgrade and the riprap.

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

8.2.4 PRESERVATION OF NATURAL LOCATION OF DRAINAGE SYSTEMS

New development shall be designed to protect natural drainage features including drainageways, floodplains (Section 7.7.2), wetlands and streams (which include DSL stream channels) (Section 7.7.3), natural closed depressions (Section 7.7.4) that store and/or allow water to infiltrate into the ground, and other natural features and existing drainage/stormwater facilities. 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 opportunity and ability to utilize the more predominant systems as regional stormwater facilities.

Protection

Some portions of the NLDS fall under the jurisdiction of Oregon Department of Fish and Wildlife (ODFW) and/or the DSL and/or other state or federal agencies. The project Engineer is responsible for:

- Determining which portions of the development site are under state or federal jurisdiction, and
- Obtaining proper permits for activities in jurisdictional drainage systems.

In general, no cuts or fills are allowed in state or federally regulated NLDS except for perpendicular crossings of driveways or roads with engineering plans showing appropriately sized culverts or bridges. Such projects must meet the requirements and gain approval from Oregon Department of Fish and Wildlife (ODFW), DSL, and other local, state, or federal agencies.

Non-regulatory portions of the NLDS within developments containing lots one acre or smaller may be relocated within the development provided that it is demonstrated 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. Since some non-regulatory NLDS may be useful for managing regional stormwater, coordinate with the local jurisdiction to determine if the facility that is relocated should be increased in capacity (i.e. larger pipe or conveyance channel).

For all NLDS, the width or size of the tract or easement within which the drainageway will be contained shall be determined based upon an analysis of the proposed stormwater flows directed to these drainage systems and the extent of the resulting water surface. Maintenance and access requirements found in this Manual shall also be considered. At a minimum, the tract or easement width shall be no less than 20 feet wide.

All new development containing lots that are one 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 one acre, the drainageway may be set aside in either a tract or an easement. No part of an identified drainageway shall become part of a residential building lot.

8.3 CULVERTS

A culvert is a pipe under a roadway or embankment used to convey flow from one portion to another of a natural or constructed drainage channel. A culvert shall convey flow without causing damaging backwater flow constriction, or excessive outlet velocities.

In addition to design flows and the hydraulic performance of culverts, other factors can affect the ultimate design of a culvert and shall be taken into consideration. These factors can include the economy of alternative pipe materials and sizes, horizontal and vertical alignment, environmental concerns, flood protection, existing channel geometry, and fish passage.

8.3.1 CULVERT ANALYSIS

Culvert analysis is complex and 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.

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

- Complete culvert calculations that state the design peak flow rates, velocities at the inlet and outlet, flow control type (i.e. inlet or outlet control), flow depth, and design information for the chosen culvert such as size, slope, length, material type, Manning's coefficient (See 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 (for example, wingwall, flared end sections, etc); and,
- Wall thickness.

TABLE 8-4
MANNING'S ROUGHNESS COEFFICIENTS (n)
FOR CULVERTS OR CLOSED SYSTEMS

MATERIAL TYPE	n[1]
Concrete Pipe	0.013
Ductile Iron	0.014
HDPE	0.012
CMP	0.024
AWWA C900	0.013
ASTM D3034 PVC	0.013

[1]The "n" values presented in this table are the "Normal" values as presented in Chow (1959). For an extensive range and for additional values refer to Chow (1959) Additional or value information is often provided by pipe manufactures.

8.3.2 MINIMUM REQUIREMENTS FOR CULVERTS

Peak Flow Rate

Culverts shall be sized to handle the 50-year peak flow rates assuming developed conditions for the onsite basin and existing conditions for any offsite contributing basins. These peak flows are calculated using the methods described in Chapter 5.

In addition, the culvert shall be checked for high flow damage during the 100-year storm event.

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 and under, or 1.5 times the culvert diameter for culverts greater than 18 inches.

In addition, 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 to avoid saturation of the road base.

Velocity and Slope

To avoid siltation that reduces culvert capacity, the minimum velocity for culverts shall be 4 feet per second during the design storm event and the minimum slope shall be 0.5 percent. For grades greater than or equal to 20 percent, anchors are required unless deemed not necessary by calculations and the manufacturer's recommendations.

Diameter

The minimum culvert diameter shall be as follows:

TABLE 8-5
MINIMUM CULVERT SIZES

CULVERT LOCATION	MINIMUM SIZE (inches)
Under Public Roads	18
Under Private Roads	12
Under Driveways/Approaches	12

Material

Pipe material listed in Table 8-4 can be used for culverts. Check with the local jurisdiction for specific material restrictions or preferences.

Placement/Alignment

Culverts shall be placed on the same alignment and grade as the drainageway, whenever possible. Consider raising intakes slightly above the flow line to allow for sedimentation, but avoid cantilevered outfalls that may cause toe erosion. Drop inlets or manholes may be used to reduce exit velocities on steep terrain. Consideration should also be given to changes of conditions over time.

Angle Points

The slope and alignment 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.

If the new culvert section is to be placed at a flatter grade than the existing culvert, a manhole shall be provided at the angle point to facility the removal of debris and sediment. A manhole is not required where the new culvert section is to be placed on a steeper grade than the existing culvert.

Outfalls

Outfalls shall conform to the requirements of all federal, state, and local regulations. Erosion control shall be provided at the culvert outfall. See Section 8.2.3 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 may consist of soil deposits (i.e. silt, sand, gravel, and boulders), limbs, sticks, logs, and trees. In areas where heavy debris flow is a concern, for example dense wooded areas, debris protection shall be provided. Methods for protecting culverts from debris problems include: upsizing the culvert, debris deflectors, debris racks and debris basins.

Safety bars shall be provided for culverts with a diameter greater than 36 inches. Safety bars must be provided on both the upstream and downstream ends of the culvert. The clear space between bars shall be 4 inches maximum. An example of an outlet rack for safety or debris protection is shown in Figure 8-3.

When proposing a debris rack or safety bars, the area upstream of the culvert area shall to be evaluated to determine what would happen if debris plugged the culvert opening. 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

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 the manufacturer recommends otherwise.

The minimum cover for culverts under private driveways is 1 foot from the top of the pipe to the finish grade of the driveable surface.

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 provide 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.

The strength of the pipe depends upon its bed design, backfilling methods, and the quality of the backfill soil. In material incapable of developing adequate support, excavation and backfilling with granular material shall be performed to a width sufficient to provide the necessary support.

End Treatments

The type of end treatment used on a culvert depends on many interrelated and often conflicting considerations. The following discusses different end treatments:

- *Projecting Ends* is a treatment where 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, erosion, safety when projecting in the clear zone, and aesthetic concerns;
- *Beveled End Sections* 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 smaller 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. For larger diameter pipes (i.e. greater than 10 feet in diameter), a slope collar is recommended. A slope collar is a reinforced concrete ring that surrounds the exposed culvert end; or,
- *Wingwalls and Aprons* are common on larger reinforced concrete box culverts. Their purpose is to retain and protect the embankment, and provide a smooth transition between the culvert and the channel.

Fish Passage

Culverts located in waters in which native migratory fish are currently or were historically present must address fish passage requirements and receive an approval, waiver, or exemption from ODFW. Laws regarding fish passage may be found in ORS 509.580 through 910 and in OAR 635, Division 412. Specific design criteria are outlined in the *ODFW Fish Passage Criteria* dated October 22, 2004.

8.4 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 alignment depth, pipe strength, potential construction problems, and potential impacts to downstream properties.

8.4.1 PIPE ANALYSIS

Storm drain system analysis can be performed using commercially available computer software or spreadsheets. If hand calculations are proposed, example calculations can be found in many technical publications and design manuals. The ODOT Hydraulics Manual is a good reference that includes detailed sample calculations.

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

- A basin map showing onsite and offsite 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, full pipe capacity, and design information for each of the pipe runs (size, slope, length, material type, manning coefficient);
- Velocities at design flow for each of the pipe runs;
- The hydraulic grade line at each inlet, catch basin, manhole, angle point, and outlet; and,
- A profile of the main line stormwater system (and connections, where applicable), showing size, material type, lengths of pipes (or culverts), and invert elevations, rim/finished grade elevations for manholes, catch basin, and other structures.

Note: For lateral pipe connections to storm drain lines in existing rights-of-way, fixed invert elevations are preferred but not required. Only a minimum depth from finish grade to pipe invert and the minimum pipe slope necessary to satisfy the freeboard and self-cleansing velocity requirements must be provided on the design plans. This allowance is made to account for potential conflicts with existing utilities in the ROW.

8.4.2 MINIMUM REQUIREMENTS

Peak Flow Rate

Closed pipe systems shall be sized to handle the design peak flow rates outlined in Section 8.1.1. These peak rates can be calculated using the methods described in Chapter 5.

Hydraulic Grade Line

The Hydraulic Grade Line (HGL) represents the free water surface elevation of the flow traveling through the storm drain system. Pipes in closed systems will be sized by calculating the HGL in each catch basin or manhole. A minimum of 0.50 feet of freeboard shall be provided between the HGL in a catch basin or manhole and the proposed ground elevation during the 25-year design flow. Demonstrate that the water surface elevation will not back up into crawl spaces or flood other structures.

Pipe Velocities and Slope

Storm drains shall be designed to maintain a self-cleaning velocity of 3 feet per second (fps) or greater during the design flow. Table 8-6 provides the minimum slopes required for various pipe sizes in storm drains flowing full.

TABLE 8-6
MINIMUM PIPE SLOPES
IN STORM DRAINS FLOWING FULL

PIPE SIZE (inches)	MINIMUM SLOPE (ft/ft) *n=0.013*
8	0.0075
10	0.0056
12	0.0044
15	0.0032
18	0.0026
21	0.0021
24	0.0017
27	0.0015
30	0.0013
33	0.0011
36	0.0010

Source: FHWA, Hydraulic Engineering Circular No. 22, Second Edition

Pipe velocities should not be excessively high since high flow velocities (approaching and above 10 fps) cause abrasion of the pipes. When the design velocities are 10 fps 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 percent, then pipe anchors are required at the joints, at minimum. A pipe anchor detail is shown in Figure 8-2. Pipe anchor locations are to be defined in the plans, and a pipe anchor detail shall be referenced or provided. Table 8-7 shows minimum pipe anchor requirements.

TABLE 8-7
MINIMUM PIPE ANCHOR SPACING

PIPE SLOPE	MINIMUM ANCHOR SPACING
20 – 35 Percent	35 feet
35 – 50 Percent	25 feet
> 50 Percent	15 feet

Material

Pipe material shall meet the local jurisdiction requirements for storm sewer pipe.

Pipe Diameter and Length

The minimum pipe diameter shall be 12 inches; except for single pipe segments less than 50 feet long, which may be 8 inches in diameter. The maximum length of pipe between junctions shall be no greater than 300 feet. Pipe diameters cannot be downsized for downstream runs.

Inlets

All lines connected to storm sewers or drywells (with the exception of drywells in swales) must be designed per the local jurisdiction standard plans.

Placement and Alignment

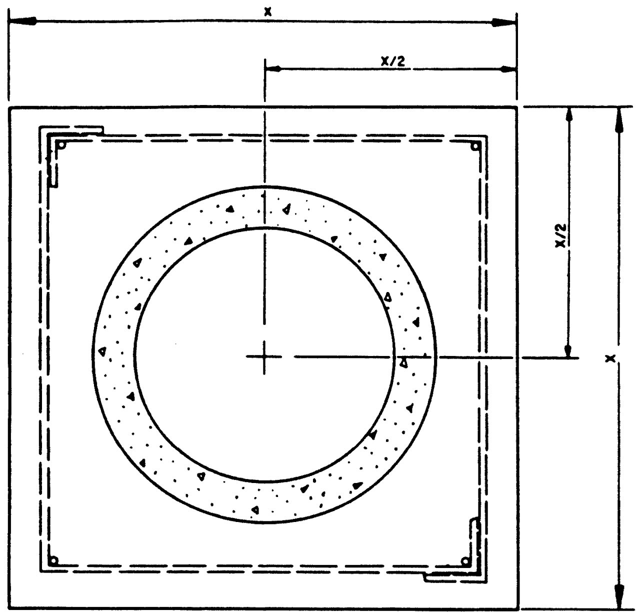
No storm drain pipe in a drainage easement shall have its centerline closer than 5 feet to a private rear or side property line or 10 feet from building foundations or other structures. Consult local jurisdiction for additional setback requirements. For a storm drain located under the road, the storm drain shall be placed in accordance with the local jurisdiction 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.

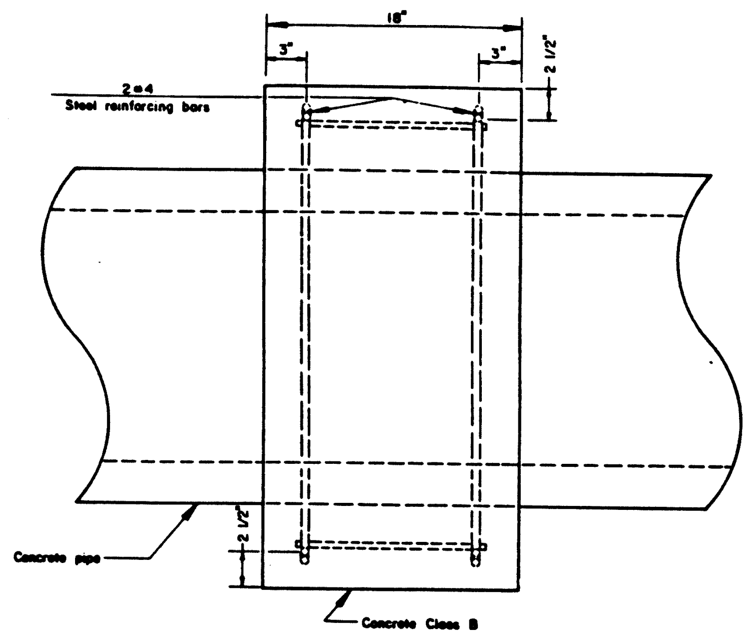
Outfalls

Pipe outfalls shall be placed on the same alignment and grade as the contributing storm drain system. Outfalls should discharge at the same elevation as the existing grade.

Outfalls shall conform to the requirements of all federal, state, and local regulations. New outfalls to the Deschutes River, Crooked River, or other water bodies designated as waters of the United States require regulatory agency approval. Erosion control shall be provided at the storm system outfall. See Section 8.2.3 for additional information regarding outfall protection.



PIPE DIAMETER	CONCRETE CLASS B APPROX. C.Y.	DIMENSION X
12"	0.3	2'-4"
18"	0.4	2'-11"
24"	0.5	3'-6"
30"	0.6	4'-1"
36"	0.7	4'-8"
42"	0.8	5'-3"
48"	0.9	5'-10"
54"	1.1	6'-5"
60"	1.3	7'-0"



NOTE
Reinforcing steel shall be Grade 40 or Grade 60.

Figure 8-2 – Pipe Anchor Typical Detail

Storm Drain Debris and Safety

The Engineer shall evaluate the site to determine whether debris protection shall be provided for storm drain systems. Debris may consist of soil deposits (i.e. silt, sand, gravel, and boulders), limbs, sticks, trash, or other landscaping materials. In areas where heavy debris flow is a concern, for example dense wooded areas, debris protection shall be provided. Methods for protecting the storm drain systems from debris problems include: debris deflectors, debris racks and debris basins.

For an enclosed storm drain system located in urban locations, safety bars shall be provided for outfalls with a diameter 18 inch or greater to protect from unauthorized individuals 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. An example of an outlet rack is shown in Figure 8-3.

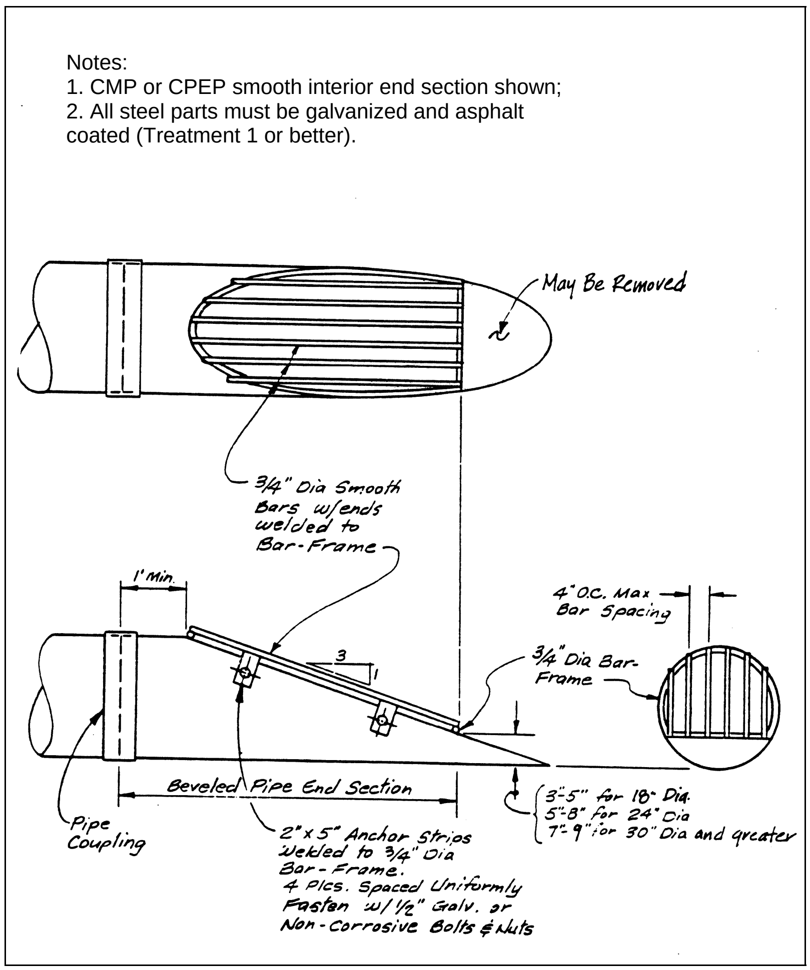


Figure 8-3 – Outlet Rack Typical Detail

Structural Design

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 the manufacturer recommends otherwise.

If the depth of cover is shallow (less than 1 feet) and truck wheel loads are present, it will be necessary to propose a design to prevent structural damage to the pipe or provide 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.

The strength of the pipe depends upon its bed design, backfilling methods, and the quality of the backfill soil. In material incapable of developing adequate support, excavation and backfilling with granular material shall be performed to a width sufficient to provide the necessary support.

Inverts at Junctions

Whenever two pipes of the same size meet at a junction, the downstream pipe shall be placed 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 would be 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.

8.4.3 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 8-4 for Manning's coefficient for different pipe materials.
2. For the trial pipe sizes chosen, determine uniform flow depth and critical flow depth;
3. Determine if upstream (accelerated flow) or downstream (retarded flow) conditions exist. Subcritical flow occurs when downstream conditions control, supercritical flow occurs when upstream conditions control. Then by comparing uniform flow depth, critical flow depth, and initial flow depth, determine what flow regime will occur. 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 complex and 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.

8.5 GUTTERS

A gutter is a section of pavement adjacent to the roadway which 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 that allows for the passing of vehicular traffic during the 25-year design storm event. The non-flooded width (L) is shown in Figure 8-4 and the minimum non-flooded widths for various road classifications are outlined in Table 8-8.

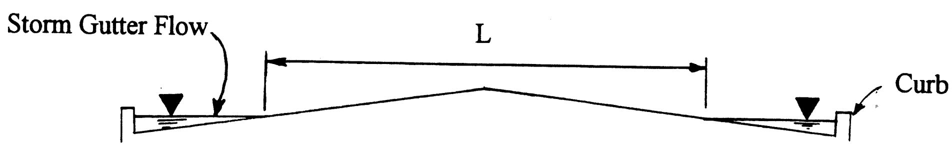


Figure 8-4 – Non-Flooded Road Width (L)

TABLE 8-8
NON-FLOODED ROAD WIDTH REQUIREMENTS

ROAD CLASSIFICATION	NON-FLOODED WIDTH (L) [1]
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

[1] During the 25-year design storm.

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 to exceed 300 feet regardless of flooded width and flow depth compliance.

8.5.1 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 (Table 8-9);
- S_L = longitudinal slope of the gutter (ft/ft);
- S_x = cross slope (ft/ft); and,
- T = spread in one gutter (ft)

TABLE 8-9
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: FHWA, Hydraulic Engineering Circular No. 22, Second Edition

8.5.2 GUTTER DESIGN

Uniform Gutter Section

Uniform gutter sections have a cross slope which is equal to the cross slope of the shoulder or travel lane adjacent to the gutter. The spread in a uniform gutter section can be calculated using equation 8-7 and solving for *T* (spread) or using the following form of that equation:

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

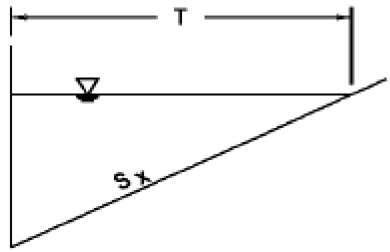


Figure 8-5 – Uniform Gutter Section

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 having composite sections are depressed in relation to the adjacent pavement slope. That is, the paved gutter has a cross slope which is steeper than that of the adjacent pavement. The design of composite gutters requires consideration of flow in the depressed segment of the gutter.

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

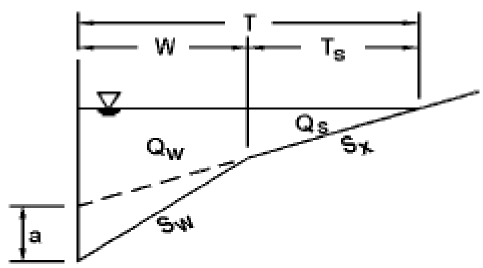


Figure 8-6 – Composite Gutter Section

1. Assume a flow rate bypassing 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 (ft/ft);
S_x = road cross slope (ft/ft);
W = gutter width (ft); and,
a = gutter depression (ft).

4. Compute E_o, the ratio of flow in a chosen width (the width of a depressed gutter or grate) to the total gutter flow, using the following equation:

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

5. Solve for T using following equation:

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

6. Compute T_S using following equation:

$$T_S = T - W \tag{8-13}$$

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.

8.6 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 Central Oregon 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 road/gutter/ditch grade increases. A major advantage of grate inlets is that they are installed along the roadway where stormwater is flowing.

Curb Inlets are vertical openings in the curb. They are most effective on flat grades, in sags, 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 sags and on grades less than 3 percent.

Combination Inlets consist of both a curb-opening and a grate inlet. They offer the advantages of both grate and curb inlets which result 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.

Appendix C of Chapter 13 in the ODOT Hydraulics Manual includes additional information guidance in selecting appropriate inlets for a given condition and includes standard dimensions for various inlet types. Local jurisdictions may also have their own design specifications for grates and inlets.

8.6.1 MINIMUM REQUIREMENTS

Peak Flow Rate

Drainage inlets shall be designed and located with capacity to carry the 25-year peak flow event. The design flow can be calculated using the methods described in Chapter 5.

Structures

Catch basins, inlets and storm manholes shall conform to the standard plans of the local jurisdiction, or ODOT's design guidelines when local jurisdiction standards are not available.

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. Structure locations shall consider maintenance access and future connection needs.

Grates

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

Grate inlets on grade shall have a minimum spacing of 20 ft to enable any bypass water to reestablish its flow against the face of curb between inlets. Also, drainage inlets shall not be located in front of pedestrian access ramps or in crosswalks.

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

Curb Inlets

Concrete curb inlets (i.e. aprons) shall be required at the entrances to all stormwater facilities and are provided in order to aid stormwater conveyance into the facility and to suppress vegetation growth at the inlet.

The maximum length of a curb inlet shall not exceed 6 feet and shall have a 2 inch depression at the curb line. The finish grade of the swale/pond sideslope, where the concrete curb inlet apron ends, shall be a minimum of 2 inches below the finished elevation of the concrete curb apron extension. The intention is to allow stormwater runoff to enter the swale/pond unobstructed, without backing up into the street and gutter due to vegetation overgrowth at the inlet.

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 curb inlet; i.e. the capacity of the inlets may exceed the design flow rate entering the facility. 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 inlet shall be no greater than 4:1.

8.6.2 GRATE INLET DESIGN – CONTINUOUS GRADES

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 following procedure is most accurate when velocities are in the range of 3 to 5 ft/s at a 2 or 3 percent longitudinal slope. For instances where the velocity is found to exceed 5 ft/s, 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 sideflow by consulting the *FHWA Hydraulic Engineering Circular No. 22 (HEC-22), 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:

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[1];
3. Analyze the most upstream inlet. The width of flow (T) is calculated using the procedure described in Section 8.5.2. Verify T is within the allowable limit

[1] ODOT lists standard inlet design parameters in Table B of Chapter 13, Appendix C of the ODOT Hydraulics Manual. Additional inlet parameters can be found in the manufactures literature.

(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);

GW = grate inlet width perpendicular to the direction of flow (ft).

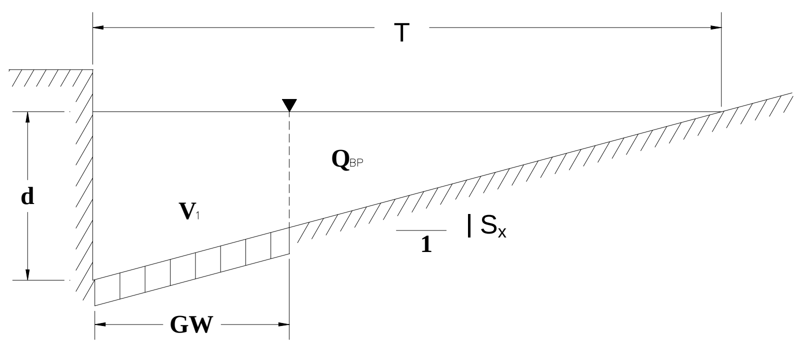


Figure 8-7 – Typical Grate Inlet Cross-Section

5. The velocity shall fall below 5 ft/s. The velocity of flow directly over the inlet is calculated as follows:

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

Where: V_1 = velocity over the inlet (ft/s);

S_x = cross slope (ft/ft); and,

d = depth of flow at the face of the curb (ft), 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 ft/sec 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 (ΣQ) to the inlet at the station being analyzed.

7. The last inlet may require an adjustment of spacing (usually smaller spacing) in order to not allow a bypass flow to the project boundaries.

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.

8.6.3 CURB INLET DESIGN – 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 capacity of a curb inlet on a continuous grade may be calculated using the procedure outlined below:

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 8.5.2. Verify 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:

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

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

S_e = equivalent cross slope (ft/ft);

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

for composite gutter sections:

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

Where: a = gutter depression (in);

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 (ft).

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.

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

here: E = efficiency; and,

L = actual curb opening length (ft).

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

$$Q_i = E * Q \qquad (8\text{-}20)$$

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.

$$Q_{BP} = Q - Q_i \qquad (8\text{-}21)$$

7. The last inlet may require an adjustment of spacing (usually smaller spacing) in order to not allow a bypass flow to the project boundaries.

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.

8.6.4 COMBINATION INLET DESIGN – SAG CONDITION

The interception capacity of the combination inlet in sag 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 sag locations.

Inlets at sag locations perform differently than inlets on a continuous grade. Inlets at sag locations operate in one of two ways: 1) they will operate as a weir at low ponding depths; or 2) they will operate 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 capacity of a combination inlet operating as a weir, in a sump or low point, may be estimated using the following procedure. There are also commercially available software programs that will analyze combination inlets in sag.

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 upon the non-flooded width requirements found in Table 8-8.

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

For composite gutter sections: $$d_{ave} = d + \frac{W}{2}\left(S_w - 2S_x\right) + y \qquad (8\text{-}23)$$

here: y = local depression (ft), 2 inches max

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

$$Q_{all} = CPd^{3/2} \qquad (8\text{-}24)$$

P = perimeter of the grate inlet;

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 repeat steps 1 through 5.

An example calculation for a combination inlet in sag is provided in Appendix 8G.

8.6.5 CURB INLET DESIGN – SAG CONDITIONS

The below procedure 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 sag condition may be calculated as follows:

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 8-8.
3. Calculate the depth of flow at the curb (*d*) using Equation 8-16.
4. Calculate the allowable flow (Q_{all}) using Equation 8-25 or 8-26:

For a depressed curb opening inlet:

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

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 repeat steps 1 through 4.

An example calculation for a curb inlet in sag is provided in Appendix 8H.

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APPENDIX 8A – EXAMPLE CALCULATION: NON-FLOODED WIDTH

Uniform Gutter Section

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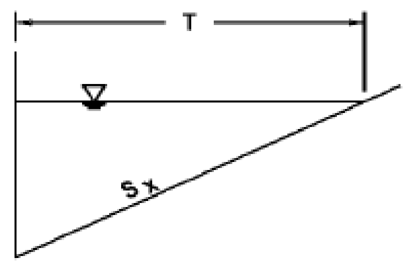
- A crowned private road with a uniform gutter section (as illustrated), assuming an equal flow rate on each side of the road.



  - Flow rate (Q) = 1.5 cfs
  - Gutter width (W) = 1.5 ft
  - Road/Gutter cross slope (S_x) = 0.02 ft/ft
  - Longitudinal slope (S_L) = 0.01 ft/ft
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet (not counting the gutters)

CALCULATIONS

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

$$T = \left(\frac{Q\ n}{0.56\ S_x^{1.67} S_L^{0.5}} \right)^{0.375} = \left(\frac{(1.5)(0.016)}{0.56\ (0.02)^{1.67}(0.01)^{0.5}} \right)^{0.375} = 8.4\,\text{ft}$$

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

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

= 2[(½)(30) + 1.5 – 8.4)] = 8.2 ft < 12 ft **OK***

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

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APPENDIX 8B – EXAMPLE CALCULATION: NON-FLOODED WIDTH

Composite Gutter Section

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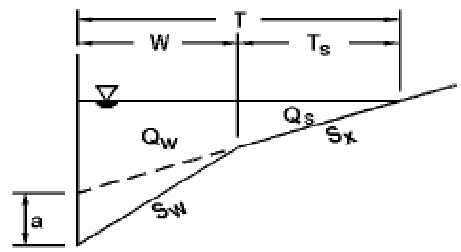
- A super-elevated local access road with a composite gutter section (as illustrated).



  - Flow rate (Q) = 4.2 cfs
  - Gutter width (W) = 1.5 ft
  - Road cross slope (S_x) = 0.02 ft/ft
  - Gutter cross slope (S_w) = .081 ft/ft
  - Longitudinal slope (S_L) = 0.01 ft/ft
  - Manning's friction coefficient, n = 0.016
  - Road width (RW) = 30 feet (not counting the gutters)

CALCULATIONS

1. The spread (T) cannot be determined by a direct solution, an iterative approach must be used. Assume a flow rate (Q_s) for that portion of the flow above the depressed gutter section.

$$\text{Assume } Q_s = 1.4 \text{ cfs}$$

2. Calculate Q_w using Equation 8-9.

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

3. Begin the first iteration to find Q_s (estimated) ≈ Q_s (computed). Calculate E_o using Equation 8-11.

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

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 T_s, 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 = 11.65 ft

T_S = 10.15 ft

Q_s = 2.48 cfs (computed)

Q_s (estimated) ≈ Q_s (computed)

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

8. Now that T has been found for the relationship: Q_s (estimated) ≈ Q_s (calculated), calculate the non-flooded width using the following relationship:

Non-flooded width = $RW + W - T$

= 30 + 1.5 - 11.65 = 19.9 feet > 12 ft **OK***

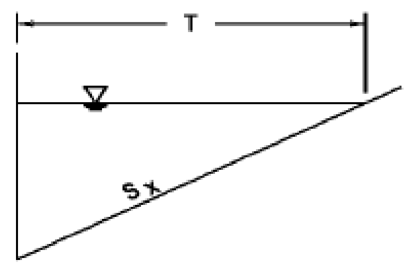
***** Table 8-7 indicates that the minimum non-flooded width is 12 feet for local access roads. Therefore, design has met the required non-flooded road width criteria.

APPENDIX 8C – EXAMPLE CALCULATION: GRATE 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) = 2.5 cfs
  - Gutter width (W) = 1.5 ft
  - Grate width (GW) = 1.67 ft
  - Road/Gutter cross slope (S_x) = 0.02 ft/ft
  - Longitudinal slope (S_L) = 0.03 ft/ft
  - 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 (GW) for later use in the calculations.

   For this example, GW = 1.67ft

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

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

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

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

= 2[(½)(30) + 1.5 – 8.31) = 16.38 ft > 12 ft **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:

$$Q_{BP} = Q\left[\frac{(T)-(GW)}{(T)}\right]^{\frac{8}{3}} = 2.5\left[\frac{8.31-1.67}{8.31}\right]^{\frac{8}{3}} = 1.37\text{cfs}$$

Therefore the capacity of the inlet = 2.5 – 1.37 = 1.13cfs

6. Verify that the velocity falls below 5 ft/s. The velocity of flow directly over the inlet is calculated using Equation 8-15 (where $d = T\,S_x$):

$$V_1 = \frac{Q-Q_{BP}}{(GW)\left[d-0.5(GW)(S_x)\right]} = \frac{2.5-1.37}{1.67\left[(8.31)(0.03)-0.5(1.67)(.03)\right]} = 3.02\text{ft/s} < 5\text{ ft/s} \quad \textbf{OK**}$$

**Reference 8.6.2 for guidance when the velocity exceeds 5 ft/s.

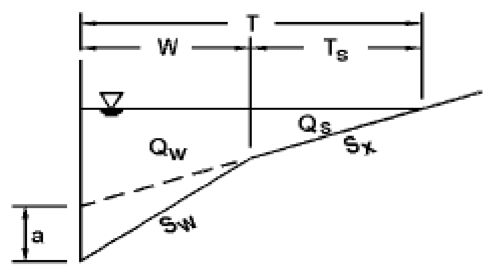
7. The analysis is then repeated with the next inlet. The bypass flow (Q_{BP}) calculated in Step 4 shall be added to the flow from the contributing basin to determine the total flow (Q) to the inlet at the station being analyzed.

APPENDIX 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) with a flow rate of 4.2 cfs.



  - Gutter Width (*W*) = 1.5 feet
  - Grate Width (*GW*) = 1.67 feet
  - Road cross slope (S_x) = 0.02 ft/ft
  - Gutter cross slope (S_w) = .081 ft/ft
  - Longitudinal slope (S_L) = 0.01 ft/ft
  - n = 0.016
  - Road width = 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 (*GW*) in the calculations.

   For this example, *GW* = 1.67ft

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 = 11.65ft (Solution from Appendix 8B)

   Non-flooded width = 19.9ft > 12ft OK (Solution from Appendix 8B)

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

$$Q_{BP} = Q\left[\frac{(T)-(GW)}{(T)}\right]^{\frac{8}{3}} = 4.2\left[\frac{11.65-1.67}{11.65}\right]^{\frac{8}{3}} = 2.78\text{cfs}$$

Therefore the capacity of the inlet = 4.2 – 2.78 = 1.42cfs

5. Verify that the velocity falls below 5 ft/s. The velocity of flow directly over the inlet is calculated using Equation 8-15:

$$V_1 = \frac{Q-Q_{BP}}{(GW)\left[d-0.5(GW)(S_x)\right]} = \frac{4.2-2.78}{1.67\left[11.65*0.02-0.5(1.67)(0.02)\right]} = 3.9\text{ft/s} < 5\text{ ft/s } \mathbf{OK}^{**}$$

**Reference 8.6.2 for guidance when the velocity exceeds 5 ft/s.

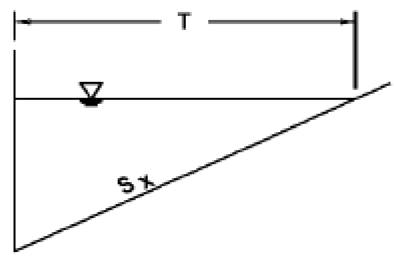
6. The analysis is then repeated with the next inlet. The bypass flow (Q_{BP}) calculated in Step 4 shall be added to the flow from the contributing basin to determine the total flow (Q) to the inlet at the station being analyzed.

APPENDIX 8E – EXAMPLE CALCULATION: CURB INLET CAPACITY – CONTINUOUS GRADE

Uniform Gutter Section

GIVEN

- A crowned private road with a uniform gutter section (as illustrated) with a flow rate of 1.5 cfs on each side of the road.



  - Curb Inlet Length (L) = 3 feet
  - Cross slope (S_x) = 0.02 ft/ft
  - Longitudinal slope (S_L) = 0.02 ft/ft
  - n = 0.016
  - Road width = 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).

$$T = \left(\frac{Q\ n}{0.56\ S_x^{1.67} S_L^{0.5}} \right)^{0.375} = \left(\frac{(1.5)(0.016)}{0.56\ (0.02)^{1.67}(0.02)^{0.5}} \right)^{0.375} = 7.41\text{ft}$$

Non-flooded width = 2*(½*Road width – T + W)

= 2*(½*30-7.41 + 1.5) = 18.2 ft > 12ft OK

3. Calculate the length of curb inlet required for total interception of gutter flow using Equation 8-17 (note that $S_e = S_x$ for uniform gutter sections):

$$L_T = 0.6Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_e} \right)^{0.6} = 0.6 * 1.5^{0.42} * 0.02^{0.3} \left(\frac{1}{0.016 * 0.02} \right)^{0.6} = 27.5\text{ft}$$

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

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} = 1 - \left(1 - \frac{3.0}{27.5}\right)^{1.8} = 0.19$$

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

$$Q_i = E * Q = 0.19 * 1.5 = 0.28\text{cfs}$$

$$Q_{BP} = Q - Q_i = 1.5 - 0.28 = 1.22\text{cfs}$$

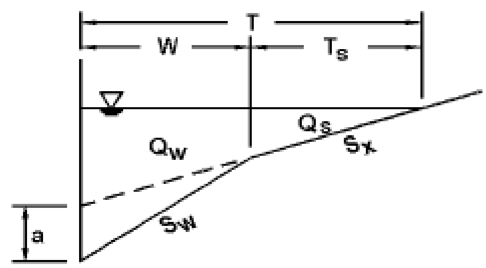
6. The analysis is then repeated with the next curb inlet. The bypass flow (Q_{BP}) calculated in Step 5 shall be added to the flow from the contributing basin to determine the total flow (Q) to the next inlet.

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) with a flow rate of 4.2 cfs.



  - Gutter Width (W) = 1.5 feet
  - Curb Inlet Length (L) = 3 feet
  - Road cross slope (S_x) = 0.02 ft/ft
  - Gutter cross slope (S_w) = .081 ft/ft
  - Longitudinal slope (S_L) = 0.01 ft/ft
  - n = 0.016
  - Road width = 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 = 11.65ft (Solution from Appendix 8B)

   Non-flooded width = 19.9ft > 12ft OK (Solution from Appendix 8B)

3. Calculate the equivalent cross slope (S_e) using Equation 8-18.

   E_o = 0.38 (Solution from Appendix 8B)

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

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

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

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

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

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

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

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

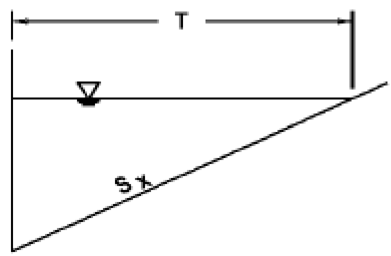
7. The analysis is then repeated with the next curb inlet. The bypass flow (Q_{BP}) calculated in Step 6 shall be added to the flow from the contributing basin to determine the total flow (Q) to the next inlet.

APPENDIX 8G – EXAMPLE CALCULATION: COMBINATION INLET CAPACITY – SAG

Uniform Gutter Section

GIVEN

- A crowned private road with a uniform gutter section (as illustrated).



  - Gutter Width (W) = 1.5 feet
  - Curb Inlet Length (L) = 3 feet
  - Grate Perimeter (P) = 6.16 feet
  - Local depression (y) = 1”
  - Cross slope (S_x) = 0.02 ft/ft
  - Road width = 30 feet
  - Upstream inlets total bypass flow rate is 0.68cfs
  - Flow rate from basins contributing flow directly is 0.82cfs

CALCULATIONS

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

$$Q = 0.68\text{cfs} + 0.82\text{cfs} = 1.5\text{cfs}$$

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:

$$T_{allow} = \frac{\text{Road Width} + 2W - \text{Non - flooded Width}}{2} = \frac{30 - 2 * 1.5 - 12}{2} = 7.5 \text{ feet}$$

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

$$d = T * S_x = 7.5 * 0.02 = 0.15 \text{ feet}$$

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

$$d_{ave} = d - S_x\left(\frac{W}{2}\right) + y = 0.15 - 0.02\left(\frac{1.5}{2}\right) + \frac{1}{12} = 0.22 \text{ feet}$$

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

$$Q_{all} = CPd^{3/2} = 3.0 * 6.16 * 0.22^{\frac{3}{2}} = 1.9 \text{ cfs}$$

6. Compare the allowable flow to the actual flow.

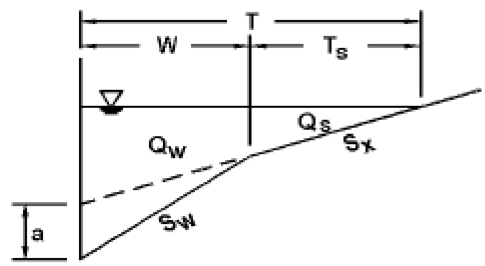
$$1.9 \text{ cfs}(Q_{all}) \succ 1.5 \text{ cfs}(Q)$$ Therefore design is **OK**.

APPENDIX 8H – EXAMPLE CALCULATION: CURB INLET CAPACITY – SAG

Composite Gutter Section

GIVEN

- A crowned private road with a composite gutter section (as illustrated).



  - Curb opening length (L) = 3.0 feet
  - Local depression (y)= 1"
  - Cross slope (S_x) = 0.02 ft/ft
  - Gutter cross slope (S_w) = 0.081 ft/ft
  - Gutter Width (local depression width) = 1.5 feet
  - Road width = 30 feet
  - Upstream inlets total bypass flow rate is 0.68cfs
  - Flow rate from basins contributing flow directly is 0.82cfs

CALCULATIONS

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

$$Q = 0.68\text{cfs} + 0.82\text{cfs} = 1.5\text{cfs}$$

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:

$$T_{all} = \frac{\text{Road Width} + 2W - \text{Non-flooded Width}}{2} = \frac{30 - 2*1.5 - 12}{2} = 7.5 \text{ feet}$$

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

$$d = T * S_x = 7.5 * 0.02 = 0.15 \text{ feet}$$

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

$$Q_{all} = 2.3(L + 1.8W)d^{3/2} = 2.3(3.0 + 1.8 * 1.5)0.15^{\frac{3}{2}} = 0.76 \text{ cfs}$$

5. Compare the allowable flow to the actual flow.

$0.76\text{cfs}(Q_{all}) \prec 1.5 \text{ cfs}\,(Q)$ Therefore design is not ok.

Consider installing a wider curb inlet or additional inlets in the vicinity of the sag point to intercept more of the flow.